## 5.5 Seismic Response of Curved Bridges

# 1) Structural Response of Curved Bridges

Curved bridges are susceptible to damage from strong motion excitation. For example, the 5/14 South Connector Overcrossing of the Golden State freeway and the Antelope Valley freeway interchange collapsed in the San Fernando earthquake in 1971 (Tseng and Penzien 1973). Because the response in the longitudinal and the transverse directions are coupled in curved bridges, the piers are subjected to multi-directional deformation with torsion. This tends to cause complex flexural and shear failure in the piers. Another problem is due to the nonlinearity of deck at expansion joints; Fig. 5.38 shows a typical model of the expansion joints. The expansion joints are generally of rubber pads, shear key and tie bars, and the nonlinearity is associated with slippage and collisions which take place between girders.



Fig. 5.38 Typical Expansion Joint

Fig. 5.39 shows the force vs. relative displacement relation of the expansion joint in which a positive relative displacement corresponds to an opening of the joint gap while a negative relative displacement corresponds to a closing. When the expansion joint undergoes an increasing relative movement in the positive direction, the rubber pad first deforms under shear action providing resistance to the motion. Then, slippage takes place when the applied force reaches the maximum friction force which can be developed on the contact plane of the expansion joint (Fig. 5.39 (a)). When the positive relative movement reaches the tie gap  $\Delta_T$ , the pair of tie bars begin to resist further opening of the joint gap. This resistance builds up linearly with joint separation until the yield strength of the tie bars is reached; then, yielding under constant force takes place (Fig. 5.39 (b)). When the expansion joint undergoes a relative movement in the negative direction, the rubber pad deforms and resists the motion in the same manner described above for the positive direction. However the tie bars do not resist motion in this direction. If the expansion joint undergoes further negative movement reaching the seat gap  $\Delta_G$ , collisions take place between the girders.

To exhibit such effect of the nonlinearity of expansion joints, a series of model excitation tests was made (Williams and Godden 1979). Fig. 5.40 shows the model bridge. This was originally based on a symmetrical simplified version of the east-half of the 5/14 South Connector Overcrossing, and was composed of three subassemblages, a center girder/column system and two side girder /column /abutment systems. These assemblages were tied together by two expansion joints of the type shown in Fig. 5.38 placed in symmetrical position. The fundamental natural frequency of the model was measured as 5 Hz, 6.6 Hz and 9-11 Hz in longitudinal, transverse and vertical directions, respectively.



Fig. 5.39 Force vs. Relative Displacement Relation of Expansion Joint



Fig. 5.40 Experimental Model

Fig. 5.41 shows the response of the model when it was subjected to a low intensity artificially generated ground motion with a peak acceleration of 0.11 g in the transverse horizontal direction. Analytical responses which will be described later are also presented for comparison. During the test, collisions of the girder did not occur since the relative response displacements at the expansion joints remained below the initial joint gap. The joint restrainer tie bars however resisted joint separation. Since the tie bars resisted only joint separation, the displacement response was small in the outward or positive direction but large for the inward or negative direction.

Fig. 5.42 shows the response of the model when it was subjected to high intensity horizontal and vertical excitations with peak accelerations of 0.47 g and 0.27 g, respectively. Analytical responses which will be described later are also presented for comparison. The same horizontal acceleration used for the low intensity excitation was adopted for this test by increasing the intensity. The vertical excitation was prescribed by artificially generated accelerograms with peak acceleration approximately one half of the peak horizontal excitation. Multiple collisions of the girders together with yielding of the joint restrainer bars took place at the expansion joints. As with the low intensity excitation, the responses of displacement, especially at the expansion joints, were unsymmetrical. However, they were of a type opposite to that developed in the low intensity excitation. This type of response was caused by closure of the expansion joints during inward motion causing stiff arch action to take place

between the abutments, while motion in the outward direction was resisted mainly by the more flexible tie bars.



Fig. 5.41 Experimental and Analytical Reponses of Model Bridge Subjected to a Low-intensity Excitation



Fig. 5.42 Experimental and Analytical Reponses of Model Bridge Subjected to a High-intensity Excitation

## 2) Analytical Model of Expansion Joints

Because the characteristics of expansion joints have a major influence on the seismic response of curved bridges, they must be correctly modeled. A nonlinear analytical model as shown in Fig. 5.43 well expresses the nonlinear behavior of the expansion joints. This model includes relative translational and rotational degree of freedom, elastoplastic joint restrainer tie bars, acting in tension, impact and Coulomb type friction with slippage (Tseng and Penzien 1973, Kawashima and Penzien 1976).



Fig. 5.43 Analytical Model of Expansion Joint

Longitudinal collisions are defined as taking place at points A and B (Fig. 5.43) when the relative displacement between the two end diaphragms  $u_{Ax}$  and  $u_{Bx}$  close the joint gap  $\Delta_G$  with a non-zero velocity. At the instant collision takes place, the longitudinal impact springs with large stiffness  $k_I$ , which are attached to one end diaphragm leaving a small gap  $\Delta_G$  with the other end diaphragm, start to resist the motion. A collision is completed when rebound occurs and the relative displacement between the two diaphragms becomes equal to the joint gap  $\Delta_G$ . The contact force acting at points A and B can be written from Eq. (5.22) as

$$P_{AI} = k_I \langle u_{Ax} + \Delta_G \rangle (u_{Ax} + \Delta_G); \quad P_{BI} = k_I \langle u_{Bx} + \Delta_G \rangle (u_{Bx} + \Delta_G)$$
(5.25)

where

$$\left\langle u_{Ax} + \Delta_G \right\rangle = \begin{cases} 1 \cdots u_{Ax} + \Delta_G < 0\\ 0 \cdots u_{Ax} + \Delta_G \ge 0 \end{cases}$$

$$\left\langle u_{Bx} + \Delta_G \right\rangle = \begin{cases} 1 \cdots u_{Bx} + \Delta_G < 0\\ 0 \cdots u_{Bx} + \Delta_G \ge 0 \end{cases}$$

$$(5.26)$$

If it is assumed that  $u_{Ax} + \Delta_G$  and  $u_{Bx} + \Delta_G$  do not change sign during a time interval  $\Delta_t$ , the change of contact force during a time interval can be expressed as

$$\Delta P_{AI} = k_I \langle u_{Ax} + \Delta_G \rangle \Delta u_{Ax}; \quad \Delta P_{BI} = k_I \langle u_{Bx} + \Delta_G \rangle \Delta u_{Bx}$$
(5.27)

Coulomb friction forces are developed at contact points A and B when the expansion joint undergoes longitudinal relative displacement and when the vertical contact forces are compressive. The friction force at each point A and B always acts in the direction opposite to the relative velocity as a pair of self-equilibrating forces. When the expansion joint does not undergo longitudinal relative displacement, each friction force can have a magnitude anywhere between its maximum and minimum values. The magnitude of a friction force depends on the magnitude of other forces acting on the expansion joint. Such characteristics can be represented by a rigid-plastic hysteretic force-relative displacement model. To avoid numerical instability caused by a sudden change in the Coulomb friction force at zero relative velocity, the force is modeled by the elastoplastic hysteretic force-relative displacement model. The change of Coulomb force  $\Delta C_{Ax}$  and  $\Delta C_{Bx}$  acting at points A and B during the time increment  $\Delta t$  can be expressed when the vertical compressive contact force  $F_{Az}$  and  $F_{Bz}$  are constant during a time interval, i.e.

$$\Delta C_{Ax} = \begin{cases} k^{C} \langle F_{Az} \rangle \Delta u_{Ax} \cdots u_{Ax}^{S} - u_{Ax}^{E} < u_{Ax} < u_{Ax}^{S} - u_{Ax}^{E} \\ 0 \cdots \cdots u_{Ax} \le u_{Ax}^{S} - u_{Ax}^{E} \\ 0 \cdots \cdots u_{Ax} \ge u_{Ax}^{S} + u_{Ax}^{E} \\ 0 \cdots \cdots u_{Ax} \ge u_{Bx}^{S} + u_{Bx}^{E} \end{cases}$$

$$\Delta C_{Bx} = \begin{cases} k^{C} \langle F_{Bz} \rangle \Delta u_{Bx} \cdots u_{Bx}^{S} - u_{Bx}^{E} < u_{Bx} < u_{Bx}^{S} - u_{Bx}^{E} \\ 0 \cdots \cdots u_{Bx} \le u_{Bx}^{S} - u_{Bx}^{E} \\ 0 \cdots \cdots u_{Bx} \ge u_{Bx}^{S} + u_{Bx}^{E} \end{cases}$$
(5.28)

where  $u_{Ax}^E$  and  $u_{Bx}^E$  are the current slippage at points A and B, respectively, and  $u_{Ax}^S$  and  $u_{Bx}^S$  are the elastic deformations at points A and B, respectively, as given by  $u_{Ax}^E = v |F_{Az}| / k_C$ and  $u_{Bx}^E = v |F_{Bz}| / k_C$  where v is a constant coefficient of Coulomb friction, and

$$\left\langle F_{Az} \right\rangle = \begin{cases} 1 \cdots F_{Az} < 0\\ 0 \cdots F_{Az} \ge 0 \end{cases}; \quad \left\langle F_{Bz} \right\rangle = \begin{cases} 1 \cdots F_{Bz} < 0\\ 0 \cdots F_{Bz} \ge 0 \end{cases}$$
(5.29)

Eqs. (5.27) and (5.28), after transforming the relative expansion joint displacement from local to global co-ordinate, can be used to assemble the total equilibrium equation of motion in incremental form. The equation of motion for an n degree of freedom system representing dynamic equilibrium can be solved by Eq. (2.47). Numerical integration can be used by Eq. (2.63) if required.

## 3) Analytical Prediction of Seismic Response

The seismic response of the model bridge was computed by the analytical idealization of the expansion joint shown above. Figs. 5.41 and 5.42 show the comparison of analytical and experimental responses for the low and high intensity excitations, respectively. In the low intensity excitation, the effect of restrainer bars are well represented in the analysis. Because the force induced in the tie bars is less than the yield strength, yielding did not develop.

On the other hand, the effect of multiple collisions and constraints of the tie bars are realistically represented in the high intensity excitation. Several yielding of the tie bars developed maximum ductility factor, defined as the ratio of the maximum tie elongation to its yield elongation, equal to approximately 4 and 12 for expansion joints No. 1 and No. 2, respectively. The maximum contact force of approximately 4000 lb was induced at the expansion joint No. 2.

It is interesting to see how accurately the dynamic response of model bridge subjected to high intensity excitation can be predicted by linear analysis in which the expansion joint is idealized only by a set of linear springs, disregarding the effect of impacts, slippage and yielding of restrainers. Fig. 5.44 compares the computed response by linear analysis and the test results. It is apparent that the predicted response is significantly different from the measured response both in the magnitude of peak amplitude and in the frequency characteristics. Several collisions within the joints and the yielding of restrainers do not allow a good correlation by linear analysis.

It is therefore important to correctly idealize the structural integrity of bridges including nonlinear behavior at expansion joints to realistically predict the seismic response of curved bridges.



Fig. 5.44 Linear Correlation for Model Bridge Subjected to a High-Intensity Excitation

### 5.6 Seismic Response of Skewed Bridges

Skewed bridges exhibit unique seismic response during an earthquake due to the strut action (Chen and Penzien 1975, Liu, Rieles, Imbsen, Priestley and Seible 1990, Priestley, Seible and Calvi 1996, Watanabe and Kawashima 2001). When a single skewed bridge collide with an abutment as shown in Fig. 5.55, a moment M is induced at the center of gravity as

$$M = I_A \cdot e_A + I_B \cdot e_B \tag{5.30}$$

where  $I_A$  and  $I_B$  represent impact forces at the acute and obtuse edges, respectively, and  $e_A$  and  $e_B$  represent the distance of eccentricity for  $I_A$  and  $I_B$ , respectively, given as

$$e_A = (l \cdot \cos\theta + \frac{d}{\tan\theta})/2$$
$$e_B = (l \cdot \cos\theta - \frac{d}{\tan\theta})/2$$
(5.31)

where  $\theta$  represents the skew edge, and l and d represent the length and width in the longitudinal and transverse directions, respectively. The moment M yields a rotation of the skewed bridge  $\varphi$  (positive for clockwise rotation) as

$$\varphi = \frac{v_j - v_i}{l} \tag{5.32}$$

where  $v_i$  and  $v_j$  represent displacement of the bridge at *i* and *j*, respectively, in the transverse direction. In the following, the direction along the two ends of skewed bridge and the direction perpendicular to the direction along the two ends are referred as y direction (skewed transverse direction) and x direction (skewed longitudinal direction).



Fig. 5.55 Rotation of Skewed Bridge Resulting from Collision with Abutment

Rotation of a skewed bridge also occurs when stiffness of the substructures is different as shown in Fig. 5.56. Different flexural displacements of two substructures resulting from different stiffness develop different displacement in the transverse direction. Representing the displacements at the both ends of the deck in the y axis as  $u_{p1}$  and  $u_{p2}$ , the rotation of the deck is written as



Fig. 5.56 Rotation of Skewed Bridge Resulting from Different Stiffnesses of Substructures

Watanabe and Kawashima (2001) analyzed the effectiveness of cable restrainers which are provided in three ways as shown in Fig. 5.57; (1) two restrainers are provided along x direction at both sides (Type 1), (2) two restrainers are provided along the longitudinal direction at both sides (Type 2), and (3) two restrainers each (four restrainers in total) are provided along y direction at both sides (Type 3). The Type 3 restrainers intends to prevent the deck movements along y direction at both sides, but the movements in x direction is allowed to take place.



(c) Type 3

Fig. 5.58 Rotation of Skewed Bridge Resulting from Tension Forces by Restrainers

The moments which are induced around the center of gravity of the skewed bridge by the Types 1, 2 and 3 restrainers,  $M_{R1}$ ,  $M_{R2}$ , and  $M_{R3}$ , respectively, become as shown in Fig. 5.58, and they are written as

$$M_{R1} = -(R_A \cdot e_A + R_B \cdot e_B) \tag{5.34}$$

$$M_{R2} = -(R_A - R_B) \cdot d/2 \tag{5.35}$$

$$M_{R3} = \begin{cases} (R_A + R_B) \cdot d_C & \text{for counterclockwise rotation} \\ -(R_A' + R_B') \cdot d_C & \text{for clockwise rotation} \end{cases}$$
(5.36)

where  $R_A$  and  $R_{B'}$  are forces pulled by two restrainers when the bridge rotates counterclockwise,  $R_A'$  and  $R_B'$  are forces pulled by two restrainers when the bridge rotates clockwise, and

$$d_C = \frac{l}{2}\sin\theta \tag{5.37}$$

If  $R_A = R_B = R$ ,  $M_{R1}$ ,  $M_{R2}$  and  $M_{R3}$  by Eqs.(5.34), (5.35) and (5.36) become as

$$M_{R1} = RI\cos\theta$$

$$M_{R2} = 0$$

$$M_{R3} = RI\sin\theta$$
(5.38)

Consequently, in a skewed bridge with a skewed edge  $\theta = 50$  degree,  $M_{R1} = 0.64 RI$ ,  $M_{R2} = 0$ , and  $M_{R3} = 0.76 RI$ . Since  $R_A$  is not necessarily equal to  $R_B$  in reality, Eq. (5.38) provides only an estimate for the magnitude of the moments induced in skewed bridges by restrainers.

An analytical result on the effectiveness of the above three restrainers are described below for a 200 m long skewed bridge with a skewed edge  $\theta$  of 50 degrees as shown in Fig. 5.59. The bridge consists of a 3 span continuous decks (deck 1), 2 simply supported decks (decks 2 and 3), and substructures. Fig. 5.60 shows a three dimensional discrete model in which poundings between the abutments and the end of the decks (decks 1 and 3) are idealized by impact springs using Eqs. (5.25) and (5.26). A restrainer resists only tension and its force vs. relative displacement relation is idealized as shown in Fig. 5.61 (a). Since a pair of restrainers is set at both sides in the type 3, a pair of restrainers is idealized as shown in Fig. 5.61 (b). The model is subjected to the NS and EW components of the JMA Kobe Observatory in the 1995 Kobe earthquake (refer to Fig. 1.2) as shown in Fig. 5.62.



Fig. 5.59 Skewed Bridge Analyzed for the Effectiveness of Restrainers



Fig. 5.60 Analytical Model of Skewed Bridge



(a) Types 1 and 2

Fig. 5.61 Modeling of Cable Restrainers

(b) Type 3



(c) Response Accelerations with  $\xi = 0.05$ 

Fig. 5.62 Ground Motion (JMA Kobe Observatory during the 1995 Kobe Earthquake)

Fig. 5.63 shows how three decks with the type 1 restrainers move, rotate and collide among the three decks and the two abutments during the first 1.5 s (between 1.5 s and 3 s). The gap of the restrainers  $\Delta_G$  in Eq. (5.26) is 50 mm. Fig. 5.64 shows an example of relative displacements, impact forces and restrainers force between decks and the rotation of deck 1 between the 1.5 s. The numbers in Fig. 5.64 correspond to the numbers in Fig. 5.63. The deck 1 first collides with the abutment 1 at 1.99 s (No. 1, refer to Fig. 5.63). This collision continues until 3.13 s with the peak impact force of 6MN, which corresponds to 45% of the weight of deck 1 of 13.55 MN. This collision results in an anticlockwise rotation of the deck 1 (refer to Fig. 5.64 (3)) based on the mechanism shown in Fig. 5.55. The anticlockwise rotation then results in separation of the deck 1 from the abutment 1. When the separation reaches the gap  $\Delta_G$ , the restrainer at the acute edge first starts to resist further separation at 2.28 s (No. 6) and the restrainer at the obtuse edge follows this at 2.34 s (No. 7). The restrainers at the obtuse and acute edges continue to pull the deck 1 until 2.39 s and 2.59 s with the maxim restrainer force of 3.8 MN and 7.5 MN, respectively. As shown in Fig. 5.64 (3), this action of restrainers changes the anticlockwise rotation to a clockwise rotation (No.7). At the instance of 2.34 s (No. 7), the restrainer between the acute edge of deck 1 and the obtuse edge of deck 2 starts to work with the maximum force of 2.3 MN. Furthermore, the obtuse edge of the deck 1 collides with the acute edge of the deck 2 at 2.52 s (No. 9), which results in the maximum impact force of 2.5 MN. Those two actions accelerate the clockwise rotation of the deck 1. I this manner, very complex responses occur resulting from actions of restrainers and collisions. An action results in another action, thus responses occur progressively.



Fig. 5.63 Response of Skewed Bridges with Type 1 Restrainers



(b) Relative Displacement between Abutment a and Deck 1 in y Direction

(1) Left End (1) Left End  $u_{RG}$  Left Direction (7) (9)  $u_{RG}$  Right Direction (8)  $u_{G}$  Right Direction (8) Time (sec) 2.5 3

(a) Restrainers Force between Decks 1 and 2



(b) elative Displacement between Decks 1 and 2 in y Direction



(3) Rotation

Fig. 5.64 Response of Deck 1 with Type 1 Restrainers

Based on the similar analyses, Fig. 5.65 shows the displacements of the three decks at typical instances. Since relative displacements along y direction between two adjacent decks are not restricted in the type 2 restrainers, each deck rotates in a similar edge in the bridges with the type 2 restrainers. On the other hand, relative displacements along y direction between two adjacent decks are restricted in the types 1 and 3 restrainers, the magnitude of rotation of the three decks change smoothly. In particular, the rotations of the decks with the type 3 restrainers are small. Fig. 5.66 shows the dependence of rotation modes of three decks on the type of restrainers.

It is important in skewed bridges to restrict rotations since the rotations result in the movement of the decks in longitudinal direction as shown in Fig. 5.67, which dislodges the decks from their supports.



Fig. 5.67 Longitudinal Movement of Decks Resulting from Rotations

## 5.7 Seismic Response of Bridges Supported by Pile Foundations

#### 1) Significance of Pile Foundations

Pile foundations are commonly used in bridges at soft soil sites. Since piles are flexible than other structural members such as deck and columns, the soil-structure interaction effect is predominant. Pile foundations are often idealized by a set of springs which represents the stiffness and restoring force of piles in an analysis with emphasis on the response of columns and superstructures. However an analytical model including piles and surrounding ground is required for the analysis of pile foundations.

There exist various idealizations for pile foundations with different levels of sophisticateness. Failure of pile foundations occurs in three ways, i.e., (1) failure of lateral bearing capacity of the surrounding ground, (2) punching failure of the ground at the bottom of piles and pull-out of piles from the ground, and (3) failure of piles due to shear and/or flexure. In the (1) above, the lateral stiffness and the lateral bearing capacity of the ground around piles are important in an idealization of pile foundations. The vertical stiffness and the bearing capacity of the ground are important in the (2). There are many uncertainties in the evaluation of the properties of the ground. A pile foundation is generally idealized as shown in Fig. 5.68. Piles are supported by lateral and vertical soil springs which represent the confinement by the surrounding soils.



Fig. 5.68 Analytical Model of Pile Foundations

### 2) Stiffness and Bearing Capacities of the Ground

In-situ loadings tests have been conducted at many sites. For example, Kimura, Kosa and others conducted a lateral loading test for a single pile and a pile foundation consisting of two piles at Umeda bridge (Kimura, Kosa, Ito and Sakamoto 1998). The piles are about 20 m long cast-in-place reinforced concrete piles with a diameter of 1m. Fig. 5.69 shows lateral force vs. lateral displacement hystereses for a single pile and a pile foundation. Based on various loading test results, the lateral force vs. lateral displacement hystereses are provided as a bilinear hysteresis shown in Fig. 5.70 (1). The bearing capacity  $P_{Hy}$  and the lateral stiffness

 $K_o$  are given as

$$P_{Hy} = \eta_{p} \alpha_{p} \cdot P_{u} \cdot D \cdot \Delta l \tag{5.39}$$

$$K_0 = \eta_k \cdot \alpha_k \cdot D \cdot \Delta l \cdot k_h \tag{5.40}$$

where,  $\mu_p$  and  $\eta_k$ : modification factors for the group pile effects for the bearing capacity and the lateral stiffness, respectively,  $\alpha_p$  and  $\alpha_k$ : modification factors for bearing capacity and the lateral stiffness, respectively, of a single pile,  $P_u$ : Coulomb's passive soil pressure,  $k_h$ : subgrade reaction of the ground, D: diameter of pile, and  $\Delta I$ : distance between soil springs.



(a) Single Pile
 (b) Pile Foundation consisting of Two Piles
 Fig. 5.69 Lateral Force vs. Lateral Displacement Hysteresis based on In-Situ Loading Test at Umeda Bridge (Okahara, Kimura, Takagi and Ohori 1993)



Fig.5.70 Hysteresis Models of Soil Springs

The parameters  $\alpha_k$  and  $\alpha_k$  have been studied based on loading test results as shown in Table 5.2. The parameters depend on the group pile effect. Consequently, it is recommended in the design as (Japan Road Association 2002)

$$\eta_{p} \cdot \alpha_{p} = \begin{cases} 1.5 & \text{clayey sites} \\ \frac{I_{d}}{D} \le 3.0 & \text{sandy sites} \\ \eta_{p} = 2/3 & \text{and} & \alpha_{k} = 1.5 \end{cases}$$
(5.41)

where  $l_d$  represents a distance between adjacent two piles in group piles.

Table 5.2 Parameters         o	lk	and	$\alpha_p$	based on In-Situ Loading Test of Pile Foundations				
(Yabe 2000)								
			$(\mathbf{n})$	Dilas in Sandy Sail				

(a) Thes in Sandy Son							
No.	Diameter and	Pile Length	(1) Max. Loaded Lateral	(1)/Pile	$\alpha_p$	$\alpha_k$	
	Thickness (mm)	(m)	Displacement (mm)	Diameter(%)	1		
1	φ 190.7x5.3	4.5	25.5	13.3	2	1.5	
2	ф 318.5x6.9	15.0	38.6	12.1	6	6	
3	ф 609.6x9.5	21.0	70.0	11.5	3.5	1.5	
4	φ 812.8x15.0	17.0	186.4	22.9	4.5	1.5	
5	φ 800x16.0	46.0	84.9	10.6	4	4	

6	φ 600x12.7	45.0	46.0	7.7	3.5	1.5
7	φ 812.8x12.7	15.0	68.0	8.4	4	2.5
8	φ 600.0x16.0	39.0	87.8	14.6	4	2
9	φ 2000x22	36.5	165.7	8.2	3	1.5

No.	Diameter and	Pile Length	(1) Max. Loaded Lateral	(1)/Pile	$\alpha_p$	$\alpha_k$
	Thickness (mm)	(m)	Displacement (mm)	Diameter(%)	•	
1	ф 812.8x14.0	30.7	70.0	8.6	2.5	1.5
2	ф 1000x27.2	20.0	96.4	9.6	1	2.5
3	φ 1500x13.0	33.1	231.6	15.4	1	1.5
4	ф 2000x22.0	51.5	255.9	12.8	2	6
5	ф 812.8x9.0	40.0	50.0	8.3	1.5	1.5
6	φ 609x12.7	18.0	134.6	22.1	3	0.5
7	ф 508.8x9.5	23.0	56.0	11.0	1	1

#### (b) Piles in Clayey Soil

The vertical stiffness of soil springs consists of the deformation under the piles and the deformation along the piles (skin friction). They have not yet clarified independently, and as a result they are evaluated in the combined manner including the stiffness of piles as

$$K_V = \alpha \, \frac{EA}{L} \tag{5.43}$$

where,

	$\left(\frac{0.014L}{D} + 0.78\right)$	Driven Steel Piles	
	$\frac{0.013L}{D} + 0.61$	Driven Prestressed Hume Piles	
$\alpha = $	$\frac{0.0031L}{D} - 0.15$	Cast - in - place Piles	(5.44)
	$\frac{0.009L}{D} + 0.39$	Center Bored Steel Piles	
	$\left(\frac{0.011L}{D} + 0.36\right)$	Center Bored Prestressed Hume iles	

where EA: axial rigidity of pile, L and D: length and diameter of pile, and  $\alpha$ : empirical modification factor to fit to the field loading test results.

Eq. (6.44) was derived based on the yielding stiffness of piles determined from 20-40 in-situ loading tests for each type of piles. Since Eq. (5.43) includes the stiffness of soils along piles, vertical springs with the stiffness by Eq. (5.43) are provided at the bottom of piles.

The compression and tension capacities,  $P_{yC}$  and  $P_{yT}$ , respectively, are given as

$$P_{yC} = \min\{N_{pC}, P_C\}$$

$$(5.45)$$

$$P_{yT} = \min\{N_{pT}, P_T\}$$
(5.46)

where,

$$P_C = q_d A + U \sum L_i f_i \tag{5.47}$$

$$P_T = U \sum L_l f_i \tag{5.48}$$

where  $N_{pC}$  and  $N_{pT}$ : capacity of pile for compression and tension, respectively,  $P_C$  and  $P_T$ : capacity of bearing capacity of ground for compression and tension, respectively,  $U = \pi D$ ,  $q_d \cdot A$ : bearing capacity at the bottom of the ground,  $f_i$ : skin friction between i-th and (i+1)th soil springs, and  $L_i$ : distance between i-th and (i+1) th soil springs.

The capacities of a pile for tension and compression,  $N_{pT}$  and  $N_{pC}$ , respectively, are evaluated as

$$N_{pT} = \sigma_y A_s \tag{5.49}$$

$$N_{pC} = 0.85\sigma_{ck}A_c + \sigma_{y}A_s \tag{5.50}$$

Soils springs for footings are idealized by elastic perfect plastic, and the lateral stiffness  $K_0$  and bearing capacity  $P_{H_V}$  of a footing are given as

$$K_0 = B_e \cdot \Delta l \cdot k_h \tag{5.51}$$

$$P_{Hy} = \alpha_{pF} \cdot P_{u} \cdot D \cdot \Delta l \tag{5.52}$$

where

$$\alpha_{pF} = 1.0 + 0.5 \frac{z}{B_e} \le 3.0 \tag{5.53}$$

where  $B_e$ : width of footing, and z: depth from the ground surface.

#### 3) Ultimate Mode of Pile Foundations

Under a seismic action, the ultimate modes of a pile foundation are (1) failure of piles, (2) the demand for lateral force becomes larger than the capacity of the surrounding ground resulting in excessive lateral displacement of the pile foundation, (3) the demand for compression becomes larger than the capacity in compression resulting in an excessive rotation of the pile foundation, (4) the demand for tension becomes larger than the capacity in tension resulting in pulling-out of piles. Which modes are predominant depends on the design procedures and determination of the capacities.

The yield of a pile foundation is defined at the stage when either of the following first occurs; (1) all piles at one pile line yield or (2) the compression at the bottom of a pile exceeds the capacity for compression  $q_d \cdot A$  because lateral displacement of the pile foundation sharply increases after this stage based on a number of in-situ loading tests (Japan Road association 2002).

#### 4) Seismic Response of a Bridge supported by Pile Foundations

Based on the above idealization, an example of seismic response of two bridges supported by pile foundations are presented here. Analyzed is two plate girder bridges (A-bridge and B-bridge) supported by 10 m tall reinforced concrete columns and pile foundations as shown in Fig. 5.71. Five elastomeric bearings are used to support the deck per column. The superstructure and the column are identical between the A-bridge and B-bridge. The ground consists of alternatives of sandy and clayey soils at A-bridge, while it consists of thick clayey soils at the B-bridge. The natural period of the ground is 0.38 s and 1.17 s at the A-bridge and B-bridge, respectively. The pile foundation consists of 3x3 cast-in-place reinforced concrete piles with a diameter of 1.2 m. The piles are 14.9 m and 30.4 m long at the A-bridge and

B-bridge respectively.



Fig. 5.71 Bridge supported by Pile Foundations

A push over analysis is conducted to evaluate the capacity of the pile foundation. The pile foundation is idealized as shown in Fig. 5.72 based on the above analytical modeling. The column hysteretic behavior is idealized by the Takeda degrading model (Takeda, Sozen and Nielsen 1970). A static lateral force  $F_L$  and a bending moment  $M = F_L \cdot (H_P + H_F)$ , in which  $H_P$  and  $H_F$  represent the height of the column and the footing, respectively, are applied at the bottom of the footing of the analytical model. As a consequence, the lateral force vs. lateral displacement hystereses at the gravity center of the deck are obtained as shown in Fig. 5.73, in which lateral coefficient  $k_F$  defined as

$$k_F = \frac{F_L}{W_e} \tag{5.54}$$

Fig. 5.72 Analytical Model of Pile Foundation for Push Over Analysis

where,

$$W_e = W_U + 0.5W_P \tag{5.55}$$

where  $W_U$  represents the tributary weight of the deck and  $W_P$  represents the weight of the column.

The deck displacement consists of the following contributions

$$\boldsymbol{u} = \boldsymbol{u}_{Ft} + \boldsymbol{\theta}_F \cdot \boldsymbol{H} + \boldsymbol{u}_C + \boldsymbol{u}_B \tag{5.56}$$

where  $u_{Ft}$  and  $\theta_F$ : translation and rotation of the pile foundation, respectively,  $u_C$ : deck response displacement resulted by the column response, and  $u_B$ : deck response displacement resulted from bearing deformation. Based on Fig. 5.73, the pile foundations undergo inelastic hystereses after experiencing the pull-out, yield, and push-in of the piles. Based on the definition presented in 3), the pile foundations yield at the lateral coefficient of 0.85 and 0.79 at the A-bridge and B-bridge, respectively.



Fig. 5.73 Lateral Force vs. Lateral displacement Hystereses at Gravity Center of Deck

For dynamic response analysis, the pile foundations are idealized as shown in Fig. 5.74. This model is essentially the same to the model shown in Fig. 5.72, but the deck, the column, the footing and the piles are idealized as a model. The response of surrounding ground is idealized by a one-dimensional model with the Hardin-Drnevich type nonlinear hysteresis (Hardin and Drnevich 1972). Two design spectrum compatible ground accelerations are used as the input ground motions at the ground surface. Bedrock ground motions are computed from the ground surface accelerations by SHAKE (Schnabel, Lysmer and Seed 1972), and those computed bedrock ground motions are used as input ground motions for the one-dimensional soil column models. The computed response of the ground is then used for the multiple excitation of the deck, column, footing and pile system.



Fig. 5.74 Analytical Model for Dynamic Response Analysis

Fig. 5.75 shows the response displacements of A-bridge. The response displacement is mostly developed by the column deformation  $u_C$  in Eq. (5.56). The deck response displacement resulted from of the foundation responses  $u_{Ft}$  and  $\theta_F \cdot H$  is 10 mm and 27 mm, respectively. Fig. 5.76 shows the restoring forces of a pile on the left line and the surrounding ground. The pile stays in elastic, and the vertical pull-in force does not reach its yielding force by Eq. (5.45). The lateral soil springs surrounding the piles has a hysteretic behavior. On the other hand, Fig. 5.77 shows the response displacements of B-bridge. Since the soil is weaker than Bridge-A, the response displacement contributed by the foundation responses  $u_{Ft}$  and  $\theta_F \cdot H$  is 37 mm and 31 mm, respectively, which are larger than those values of A-bridge.







(a) Moment vs. Curvature Hysteresis at the Top of a Pile on the Left Line

(b) Vertical Force vs. Vertical Displacement at the Bottom of a Pile on the Right Line



(c) Lateral Force vs. Lateral Displacement Hysteresis of a Soil Spring Surrounding a Plie Fig. 5.76 Hysteretic Behavior of Piles and Ground



Fig. 5.77 Response Displacements of B-bridge

## 5.8 Seismic Response of Bridges supported by Spread Foundations

Spread foundations (direct foundations) are well used for bridges at the sites with high bearing capacity. They are first designed for dead weights of superstructures and substructures and then the performance for seismic effects are checked as shown in Fig. 5.78. The seismic effect is critical for sizing of the foundations as the seismic force increases. Under the static loads W, the footing with width 1 settles with the magnitude of  $v_{SF}$  as

$$v_{SF} = \frac{W}{k_{SV} \cdot l} \tag{5.57}$$

where  $k_{sv}$  represents the stiffness of vertical soil springs per unit width. Winkler type soil springs are generally assumed in design purpose.



(c) Foundations with Uplift Fig. 5.78 Reactions under Spread Foundation

Under a seismic lateral force, the footing starts to rotate from the displaced position. Safety for sliding, overturning and bearing capacity of soils under the footing is generally checked. If one disregards the lateral displacement, as the seismic lateral force increases, the rotation of footing  $\theta_F$  increases and an upward displacement at the left edge reaches the static settlement, an uplift starts to occurs. If the foundation rotates around central axis, the stiffness of rotating spring  $K_{F\theta}$  may be written as

$$K_{F\Theta} = \begin{cases} \int_{-l/2}^{l/2} k_{sv}(x) \cdot x^2 dx & l \cdot \Theta_F / 2 < v_{FS} \\ \int_{x}^{l/2} k_{sv}(x) \cdot x^2 dx & l \cdot \Theta_F / 2 \ge v_{SF} \end{cases}$$
(5.58)

In the evaluation of seismic response of spread foundations, it is common that the spring stiffness of rotation is obtained assuming that uplift does not occur. However, this assumption is not valid under a strong ground motion excitation.

An example of this effect is shown here for a 10 m tall reinforced concrete column

supported by a 7 m long and 6.5 m wide spread foundation as shown in Fig. 5.79 (Kawashima and Hosoiri 2002). This is a column which support 200 m long 5 span continuous decks. Since soil condition and column heights are nearly uniform along the entire bridge axis, only a column and a part of the superstructure which is supported by the column is analyzed here.

The column and the foundation are idealized by a discrete model as shown in Fig. 5.80, in which the footing is assumed to be supported by soil springs. When the footing separates from the rebounded surface of the underlying ground as a result of rotation, the soil springs where the separations occur do not resist tension as shown in Fig. 5.81. When soil springs resist tension even after separations occur between the footing and the rebounded surface of underlying ground, a soil spring which resist rotation determined by Eq. (5.58) is used to idealize rotation of the footing. The plastic hinge of the column is idealized by Takeda degrading nonlinear element. The JMA Kobe Observatory ground motion during the 1995 Kobe earthquake (refer to Fig. 1.1 (a)) is used as an input ground motion.





Fig. 5.81 Soil Springs which do not resist Tension

Fig. 5.82 shows deck acceleration and displacement, and footing rotation when the standard soil spring for rotation is used. The displacement of the deck u in Fig. 5.82 consists of four sources as

$$\boldsymbol{u} = \boldsymbol{u}_{Ft} + \boldsymbol{\theta}_F \cdot \boldsymbol{h}_0 + \boldsymbol{u}_{Pp} + \boldsymbol{u}_{Pf} \tag{5.59}$$

where  $u_{Ft}$ : translation of the footing,  $\theta_F$ : rotation of the footing,  $h_0$ : distance between the gravity centers of the deck and the footing,  $u_{Pp}$ : displacement of the deck resulted from the plastic deformation of the column, and  $u_{Pf}$ : displacement of the deck resulted from elastic flexural deformation of the column. In the deck displacement shown in Fig. 5.82 (b), the displacement response corresponding to $\theta_F \cdot h_0$  is also presented for evaluate of the effect of the footing rotation. Since the soil springs underlying the footing resist not only compression but also tension, rotation of the footing  $\theta_F$  is only 0.003 radian, resulting that the deck displacement by the footing rotation  $\theta_F \cdot h_0$  is only 0.04 m, which is less than 20% of the total deck displacement u of 0.22 m. The peak deck acceleration is 6.66 m/s<sup>2</sup>. Fig. 5.83 shows the moment vs. curvature hysteresis at the plastic hinge of the column. The column yields, and the peak curvature is 0.016/m resulting in the peak curvature ductility factor of 14.



Fig. 5.82 Responses of the Bridge when Standard Rotation Soil Spring is Used



Fig. 5.83 Moment vs. Curvature Hysteresis at the Plastic Hinge of the Column

On the other hand, Fig. 5.84 shows responses of bridge when the soil springs underlying the footing do not resist tension. The peak deck displacement is 0.28 m which is nearly the same with the above analysis. The peak footing rotation  $\theta_F$  is 0.019 radian which is about 6 times larger than that in the above analysis. Since the footing rotation is large, the deck displacement resulting from the rotation of the footing  $\theta_F \cdot h_0$  is 0.22 m which is 80% of the total deck displacement u. Fig. 5.85 shows the moment vs. curvature hysteresis at the plastic hinge of the column. The peak curvature is 0.003/m which is only about 20% of the value presented above. The larger rotation of the footing results in smaller plastic deformation at the plastic hinge of the column when tension of the soil springs under the footing is ignored. It is interesting to note that the footing rotation has an isolation effect to the column.



(c) Footing Rotation Fig. 5.84 Responses of the Bridge when Soil Springs do not resist Tension



Fig. 5.85 Moment vs. Curvature Hysteresis at the Plastic Hinge of the Column

Fig. 5.86 shows how the footing uplifts. An uplift of the footing starts to occur from an edge followed by next uplift from the other edge, thus uplifts repeat alternatively from one another during strong excitation. Uplift does not occur at the entire zone under the footing at any time during excitation.

Since rocking response has an isolation effect to the column, size of the footing is intentionally reduced from the original size of 7m by 6.5 m to 5m by 4.5 m. Fig. 5.87 shows responses of the bridge. About 95% of the total deck displacement results from the footing rotation. The column stays in elastic. Fig. 5.88 shows how the footing rotation increases and the plastic curvature of the column decreases as the footing size decreases.

Although the rotation of spread foundations with uplifts from the underlying ground is effective to reduce plastic deformation of columns, the uplift results in an increase of reaction at the edge. If bearing capacity of the underlying ground is insufficient, plastic deformation may occur in the soils. Fig. 5.89 shows such an example of rotation of a 7 m by 6.5 m footing when the bearing capacity of the underlying ground has a yield at 2 MPa. Reaction of soils saturates at 2 MPa, which results in the increase of footing rotation from 0.039 radian (without yield) to 0.043 radian (with yield). It is noted however that protection of underlying ground (Priestley, Seible and Calvi 1996).



(a) Uplift at the Left and Right Edges of Footing(b) Time History of Uplift of HootingFig. 5.86 Uplift of Footing



(c) Moment vs. Curvature Hysteresis at Plastic Hinge of Column Fig. 5.87 Responses when Bridge is supported by 4.5 m by 5 m footing



Fig. 5.88 Variation of Footing Rotation and Plastic Curvature of Column



(b) Footing Rotation when Underlying Ground does not Yield  $_{0.05\,\square}$ 



(c) Footing Rotation when Underlying Ground Yields Fig. 5.89 Effect of Yielding of Underlying Ground at 2 MPa