DYNAMIC BEHAVIORS OF BRIDGES UNDER SEISMIC EXCITATIONS WITH POUNDING BETWEEN ADJACENT GIRDERS

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SUMMARY

An idealized mechanical system is proposed to examine the response behaviors of the bridge system consisting of several simple spans. The system is modeled as the multiple oscillators, and individual oscillating units are composed of 3 degree-of-freedom system, which are translational motion of superstructure, and translational and rotational motions of foundations. The corresponding equations of motion are then derived and the effects of pounding and restrainers are analyzed.

The pounding is found to affect the global motion of the bridge. It is found that the pounding may increase or decrease the relative motions between adjacent units according to the given conditions. The maximum relative displacements occur between the abutment and nearby girder. The restrainers are found to reduce the relative displacements efficiently lowering the probability of span failures.

INTRODUCTION

Among many structural damages of bridges due to earthquakes, the span collapse is one of the most unwanted results since the bridge stops its major function producing the bigger problems. The span collapses can take a place due to many components. From recent research, it is found that pounding may cause the severe local damages to girder ends and furthermore play the primary role in the span collapses [Tanabe *et al*., 1998]. It is also known that the pounding actions may change the longitudinal motion of the bridge [Malhotra, 1998; Jankowski *et al*., 1998]. It has been reported that this pounding action caused the actual span collapses in Kobe earthquake. Consequently these pounding phenomena have recently attracted attentions. Therefore, the analysis tool is desired to predict the dynamic behavior of the bridge system properly, which can unveil the effects of pounding. These pounding actions can occur from the bridge system consisting of several simple spans, and can be modeled using the multiple units of oscillators. Using direct integration, the pounding effects can be examined [Watanabe *et al*., 1998].

In this study, the idealized mechanical model for the bridge system is proposed, which can represent the bridge motions including pounding, as well as the inelastic behavior of pier, and influence of foundation and abutment. The whole system is modeled as a multi degree-of-freedom system, which is composed with individual mass-spring-damper system connected to each other by the impact elements. To simulate the bridge motions, the corresponding equations of motion are derived by adopting Lagrange equations, and a direct numerical integration technique is used to simulate the bridge motions under seismic excitations with various intensities. To prevent span collapse from the piers due to excessive relative displacement, the connecting cable is widely used as restrainer. Using proposed model, the effects of restrainers are examined with various clearance distances.

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POUNDING

The pounding action can be described by a spring-damper element or by applying the impact laws of mechanics for particles. It is known that the former approach can provide the better approximation of practical problems, under the condition that appropriate values of the spring-damper element properties are used [Anagnostopoulos, 1995]. The pounding is modeled by using spring-damper elements (impact elements) between the masses of adjacent units in this study (Fig. 1). Poundings only occur when the masses are contacted, and the pounding condition can be defined as follows:

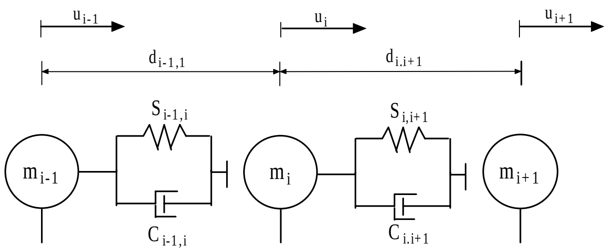


Figure 1. Idealization of pounding between adjacent structures

$$\delta_i = u_i - u_{i+1} - d_{i,i+1} \geq 0 \tag{1}$$

where, m_i and m_{i+1} = masses of ith units, u_i, u_{i+1} =the displacement of mass m_i and m_{i+1}, respectively, and $d_{i,i+1}$ =the gap distance between m_i and m_{i+1}. The pounding force between m_i and m_{i+1} can be defined as follows:

$$\begin{aligned} F_{i,i+1} &= S_{i,i+1}\delta_i + C_{i,i+1}\dot{\delta}_i \quad for\ \delta_i > 0, \\ F_{i,i+1} &= 0 \qquad \text{otherwise} \end{aligned} \tag{2}$$

where $S_{i,i+1}$ and $C_{i,i+1}$ are the stiffness of spring and damping constant of impact elements, respectively. The stiffness of spring of impact is typically large and highly uncertain due to the unknown geometry of the impact surfaces, uncertain material properties under impact loadings, and variable impact velocities, etc. Based on a limited sensitivity studies, it is known that the system responses are not very sensitive to changes in the stiffness of spring [Anagnostopoulos, 1988; Davis, 1988; Maison and Kasai, 1992]. Damping constant which determines the amount of energy dissipation can be obtained by the following relations [Anagnostopoulos, 1988].

$$C_{i,i+1} = 2\xi_i \sqrt{S_{i,i+1} \times m_i m_{i+1} / (m_i + m_{i+1})} \quad , \qquad \xi_i = -\ln r / \sqrt{\pi^2 + (\ln r)^2}$$
(3)

where, r = coefficient of restitution.

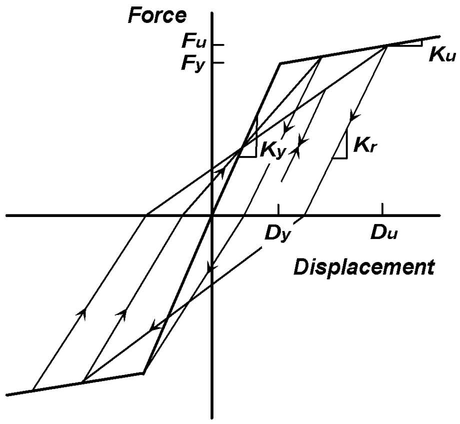
NONLINEAR PIER, FOUNDATION, AND ABUTMENT

Pier Motion

The material nonlinearity of the RC pier can be modeled by adopting the hysteresis model obtained from the force-displacement relationship from the moment-curvature curve based on the constitutive laws of the material of piers. The hysteresis model used in this study is shown in Fig. 2. In Fig. 2, F_y, F_u = yielding force and ultimate force of the pier, D_y, D_u = yielding displacement and ultimate displacement of pier, and K_y, K_u, K_r = elastic stiffness, strain-hardening stiffness, and unloading stiffness of pier. Because slenderness ratios of the object piers are about 7~9, the influence of shear strain may be neglected and only the flexural behaviors of piers are assumed to control the displacement of piers [Ghavamian and Mazars, 1996]. The geometric nonlinearity of RC pier due to the $P-\Delta$ effects is also considered [Macrae, 1994].

Foundations and Abutments

Foundations and abutments of the bridge are modeled by using translational and rotational springs and dampers to consider the ground conditions. The stiffness of foundation and abutment are determined according to the Korean Standard Specification for Highway Bridge [1996]: Seismic Design. The simplified model for the foundation and abutment is shown in Fig. 3.



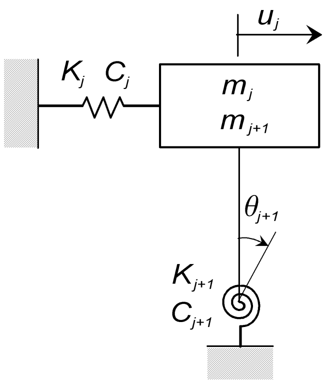


Figure 2. Hysteresis model **Figure 3. Model for the foundation and abutment**

MODELING OF BRIDGE CONSIDERED

Input Ground Motions

Artificial seismic excitations are used as the input ground motions. By using the well known SIMQKE code [Gasparini and Vanmarcke, 1976], the seismic excitations compatible to the design response spectra specified in the Korean Highway Standard Specification [1996] are generated.

Bridge Considered

The bridge considered is a six span simple steel girder bridge with 35m span length as shown Fig. 4. Piers of Π type and shallow foundations are used, and the pier heights are 12m. As mentioned before, the total brige system can be described as the combination of several oscillators. Each oscillator is composed of foundation, pier and the span with fixed supports.

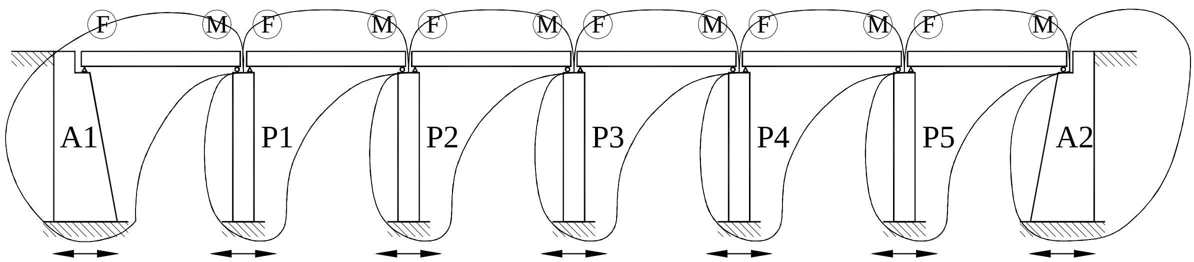


Figure 4. Bridge considered

Mechanical Model

The idealized mechanical model corresponding to the bridge system above is depicted in Fig. 5. $m_1 \sim m_5$ are masses of the superstructures with contribution of pier mass, $m_6 \sim m_{10}$ are masses of foundations with contribution of pier mass, $m_{11} \sim m_{15}$ are rotational mass moments of inertia of foundations, m_{16} is mass of left abutment and connected superstructure, m_{17}, m_{19} are rotational mass moments of inertia of abutment, and m_{18} is mass of right abutment. $K_1 \sim K_5$ and $C_1 \sim C_5$ are stiffness and damping constants of piers, $K_6 \sim K_{10}$ and $C_6 \sim C_{10}$ are translational stiffness and damping constants of foundations, $K_{11} \sim K_{15}$ and $C_{11} \sim C_{15}$ are rotational stiffness and damping constants of foundations, respectively. K_{16}, K_{18} and C_{16}, C_{18} are translational stiffness and damping constants of abutments. K_{17}, K_{19} and C_{17}, C_{19} are rotational stiffness and damping constants of abutments. $l_1 \sim l_4$ are heights of piers and $u_1 \sim u_5$ are displacementss of superstructures. l_5, l_6 are heights of abutments and u_{16}, u_{18} are displacements of abutments. u_g is the ground displacement. Restrainer cable is modeled as a linear spring which only acts when relative displacement exceeds the specified clearance. CS is the stiffness of the connecting cable between adjacent superstructures and between abutment and nearby superstructure. The restrainer cable is assumed to be undestructible during the earthquakes. Using the mechanical model, the governing equations of motion can be derived by solving the corresponding Lagrange equations [Kim *et al*, 1999].

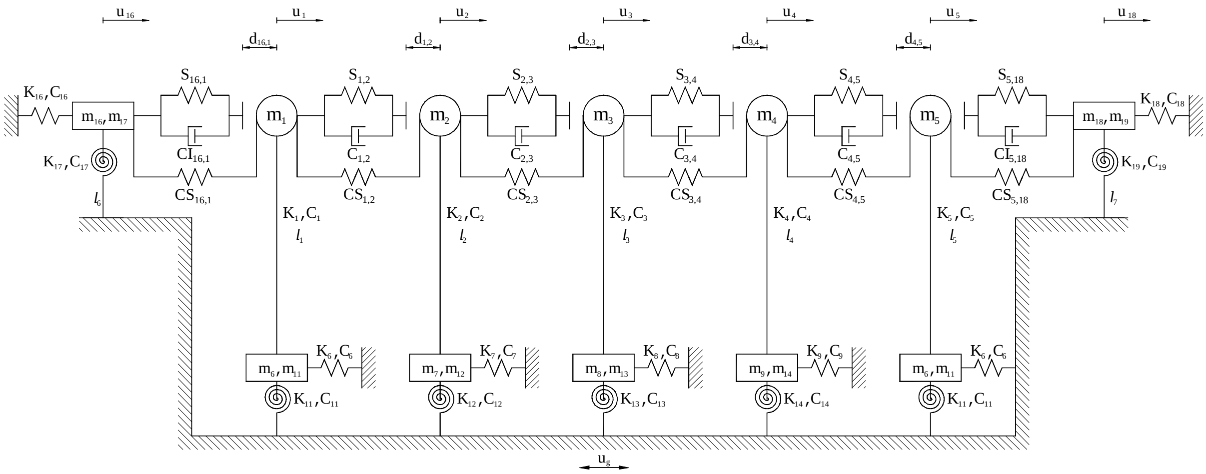


Figure 5. Simplified mechanical model of the bridge

RESULTS

It is assumed that the propagating speed of the seismic wave is fast enough so that the differences of the arriving times of the seismic excitations to each foundation of the piers be negligible. The middle oscillating units, which consist of foundation, pier and superstructure, are intentionally identical by applying the same pier heights and foundation conditions. By these assumptions, the pounding phenomena occur due to the differences of the natural frequencies of the abutments and the middle oscillators. Consequently, both effects of the pounding and abutments can be determined at the same time.

Analysis of the Effects of Poundings

The response behaviors are examined to see the effects of the pounding for the bridge system with and without pounding actions. Both actual and relative system displacements are compared under seismic excitations with various intensities. Two gap sizes are examined, and one is 5cm and the other is 10cm. First, the maximum displacements are investigated and the results are tabulated in Table. 1. 10 individual results are used for each case to obtain the average values. Without pounding, all the responses of the middle units are the same since they have the same properties, and only the differences are between the abutment and the nearby units

With pounding, the results show the totally different responses of the systems. The pounding produces the interaction between the oscillating units, and this is due to the fact that the abutment has the relatively high stiffness compared to those of middle units. The first pounding occurs at the position where the abutment and nearby unit are, and are transferred to the next unit by another pounding.

As the ground acceleration increases, the number of occurrences and the forces of the pounding increase, and this trend is more significant for the bridge with shorter gaps. To see this trend clearly, the time histories of the pounding forces are obtained (Fig. 6). From the figures, the pounding with 5cm gap occurs more frequently with higher intensities. Hence, it can be said that more care should be taken when the shorter gap is applied between girders.

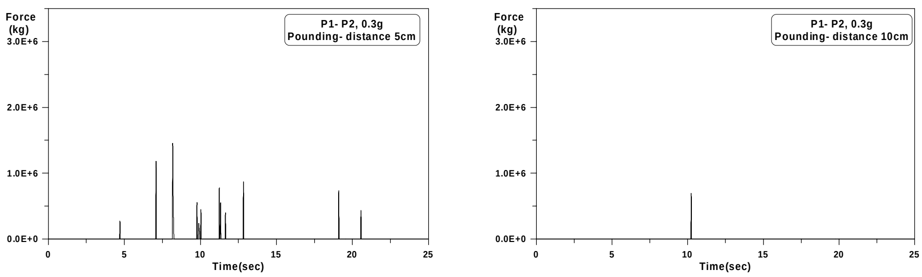
The pounding effects can be more clearly observed when the relative displacements are compared. The results of maximum relative displacements (MRD) are tabulated in Table. 2. Number of the sample size 10 is also used here for each case. The MRD between piers are none since they have the same natural frequencies when pounding motions are not included. Between abutment and pier are only None zero values of MRD.

For the systems with pounding, the various values of MRDs can be obtained, and this is due to the pounding occurring near the abutment. First, the relative displacements between piers are compared. For the earthquakes with small peak ground accelerations (0.1g~0.3g), the systems with shorter gap (5cm) are found to have the larger MRDs. For stronger earthquakes (0.4g~0.6g), the system with longer gap are found to have the larger MRDs. Under weak earthquakes, the pounding can hardly occur with 10 cm gap since the individual responses are to small to make contacts, and consequently, the relative displacements in the middle portion of the bridge with shorter gap become larger than those of the system with longer gap. This trend is outstanding for the units between P1-P3, and P3-P4, which are at the center of the bridge. The opposite trend can be found for the systems under strong earthquakes.

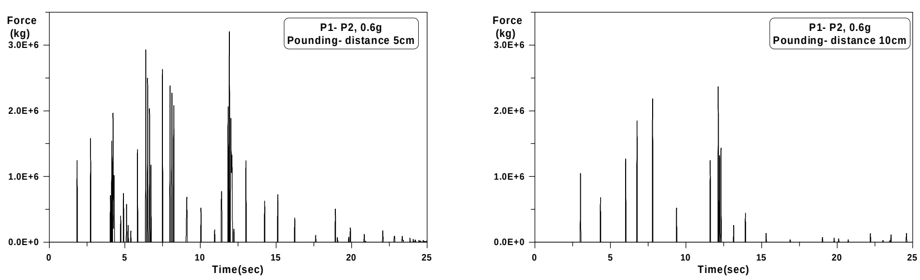
When the relative displacements between abutments and nearby units are considered, the MRDs for both systems with 5cm and 10cm gaps are almost identical under earthquakes with 0.1g~0.3g. For the earthquakes with 0.4g~0.6g, the system with 10cm gap shows the slightly larger MRDs.

Table 1. Maximum displacement of superstructure with and without considering pounding (cm)

CASE	PGA	A1	P1	P2	P3	P4	P5	A2
No Pounding	0.1g	0.65	6.16	6.16	6.16	6.16	6.16	0.25
	0.2g	1.30	11.90	11.90	11.90	11.90	11.90	0.50
	0.3g	1.95	16.39	16.39	16.39	16.39	16.39	0.75
	0.4g	2.60	24.45	24.45	24.45	24.45	24.45	1.01
	0.5g	3.31	33.10	33.10	33.10	33.10	33.10	1.28
	0.6g	3.98	37.83	37.83	37.83	37.83	37.83	1.53
Pounding ;gap5cm	0.1g	0.71	5.80	6.16	6.16	6.26	5.79	0.37
	0.2g	1.64	8.13	11.25	12.73	10.71	8.45	1.07
	0.3g	2.78	12.81	14.81	14.96	13.57	12.13	2.13
	0.4g	3.64	18.84	18.93	16.64	16.91	18.84	3.18
	0.5g	5.19	24.62	22.02	19.02	21.75	24.40	4.96
	0.6g	7.25	28.43	24.84	21.78	25.43	28.96	6.28
Pounding ; gap 10cm	0.1g	0.65	6.16	6.16	6.16	6.16	6.16	0.25
	0.2g	1.36	11.27	11.90	11.90	11.90	11.34	0.60
	0.3g	2.07	15.60	16.59	16.39	17.37	13.58	1.19
	0.4g	2.77	22.24	24.69	24.13	20.66	18.00	1.84
	0.5g	3.63	30.78	31.67	29.73	25.13	27.25	2.40
	0.6g	4.69	37.39	34.48	30.52	31.26	31.49	3.66



(a)PGA=0.3g, gap= 5cm **(b)PGA=0.3g, gap= 10cm**



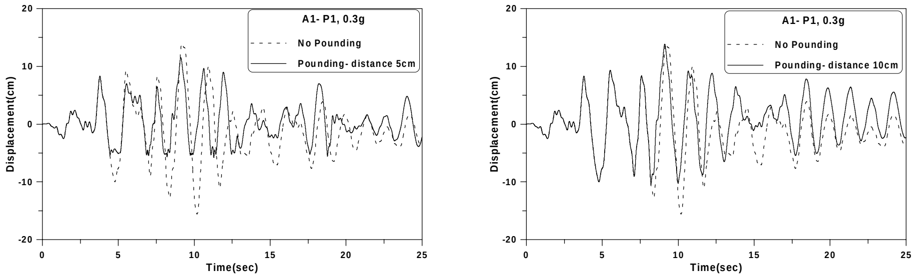
(c)PGA=0.6g, gap= 5cm **(d)PGA=0.6g, gap= 10cm**

Figure 6. The time histories of pounding force between P1 and P2

The time histories of relative displacements of the system with and without pounding between the abutment (A1) and nearby unit (P1) are prepared and shown in Fig. 7. From the figure, it can be seen that the pounding occurs earlier for the system with 5 cm gap (Fig. 7a) than for the system with 10 cm gap (Fig. 7b). Hence, it can be said that the pounding action dominates the response behaviors more in the system with shorter gap. It should be noticed that the relative displacements in the negative direction become larger than the given gap when the pounding is not considered. This means that two girders are overlapped to each other, and it is a practically impossible response. Therefore, the pounding should be implied in modeling the bridge system with multiple spans. The relative displacements of the system at the positions between piers away from the abutments are found to become significantly different from those of the system without pounding (Fig. 8). Without pounding, there is no relative displacement, since the motions of adjacent units are synchronized, but the relative motions start to oscillate for the system with pounding from the moment of the first occurrence of pounding.

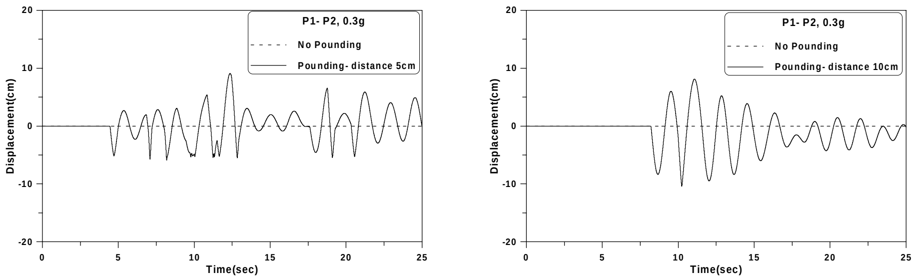
Table 2. Maximum relative displacement of superstructure with and without considering pounding (cm)

CASE	PGA	A1 - P1	P1 - P2	P2 - P3	P3 - P4	P4 - P5	P5 – A2
No Pounding	0.1g	6.05	0.00	0.00	0.00	0.00	6.08
	0.2g	10.88	0.00	0.00	0.00	0.00	10.75
	0.3g	15.94	0.00	0.00	0.00	0.00	13.18
	0.4g	23.13	0.00	0.00	0.00	0.00	14.96
	0.5g	29.38	0.00	0.00	0.00	0.00	18.73
	0.6g	34.16	0.00	0.00	0.00	0.00	24.61
Pounding ; gap 5cm	0.1g	5.88	3.21	0.18	0.26	2.52	5.73
	0.2g	8.54	6.65	5.38	6.17	5.39	8.52
	0.3g	13.90	7.35	6.14	6.32	5.82	12.35
	0.4g	19.84	6.65	6.82	5.48	5.95	19.03
	0.5g	26.03	9.19	4.76	4.35	6.85	25.70
	0.6g	30.37	8.15	7.30	7.25	7.09	30.13
Pounding ; gap 10cm	0.1g	6.05	0.00	0.00	0.00	0.00	6.08
	0.2g	11.24	2.14	0.22	0.15	2.40	10.95
	0.3g	16.12	5.17	3.06	3.45	6.96	13.45
	0.4g	22.24	9.32	7.25	6.54	5.33	18.30
	0.5g	30.77	11.26	7.26	6.65	3.22	27.43
	0.6g	37.71	8.38	5.62	5.87	8.39	31.68



(a)PGA=0.3g, gap= 5cm (b)PGA=0.3g, gap= 10cm

Figure 7. Relative displacement time histories of superstructure between A1 and P1



(a)PGA=0.3g, gap= 5cm (b)PGA=0.3g, gap= 10cm

Figure 8. Relative displacement time histories of superstructure between P1 and P2

Effects of restrainers

For analysis of the restrainer effects upon the bridge motions, the responses of the system with 10 cm gap are examined, and the restrainer cables with 5cm and 10 cm clearances are selected. The MRDs for each system are tabulated in Table. 3.

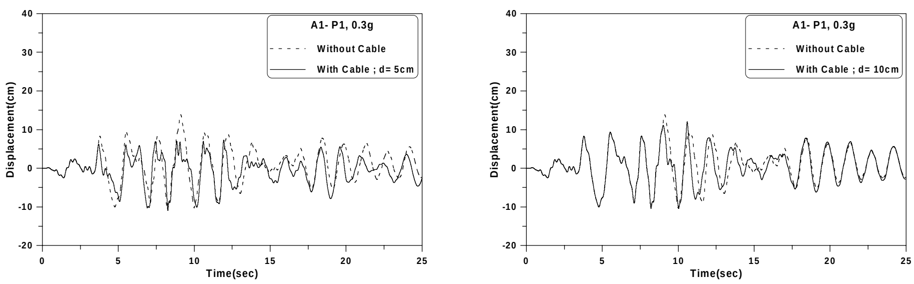
From Table. 2, the relative displacements between abutment and nearby units are found to be large enough to cause the span collapse. The results in Table. 3 show that the relative displacements are significantly reduced when the restrainers are utilized at these positions. This trend is more remarkable for the system under strong earthquakes. The MRDs are decreased by nearly 60% for the system with 10 cm clearance distance. The decreasing rates are even bigger for the system with 5 cm clearance by about 70%. The time histories of the relative displacements are shown in Fig. 9 for the system with the restrainers of both 5 cm and 10 cm clearances.

It clearly shows the reducing effect of the restrainers upon the relative displacements, and it also shows that the shorter cable affects the responses earlier than the longer cable. From results, it can be said that the shorter restrainers are more efficient to reduce the relative displacements. The relative displacements in the middle portion of the bridge, which are between piers, tend to become larger than those of the system without restrainers. The increments of MRDs increase as the PGAs of the given earthquakes increase. This tendency is more apparent, at the center of the bridge, which is furthest from the abutment. However, these magnitudes of the relative displacements between the oscillating units away from the abutment, even though they increase due to the existence of the restrainers. Therefore, it can be said that the utilization of restrainers is helpful to reduce the probability of failure, which may result from the excessive relative displacement due to pounding actions.

It should be noted that the fracture and inelastic property of the restrainer cables is not considered in this study. In the near future, these characteristics of the restrainer cable will be regarded for the more practical analysis.

Table 3. Maximum relative displacement of superstructure with cable (cm)

CASE	PGA	A1 - P1	P1 - P2	P2 - P3	P3 - P4	P4 - P5	P5 - A2
Pounding ; gap 10cm Cable ; clearance 5cm	0.1g	5.44	3.38	0.47	0.06	3.19	5.36
	0.2g	6.44	6.58	6.32	6.20	6.59	6.08
	0.3g	7.47	7.70	7.41	7.25	7.60	6.84
	0.4g	8.23	8.76	8.24	8.45	8.45	7.63
	0.5g	9.09	9.37	8.82	9.13	9.22	8.05
	0.6g	10.25	10.15	9.77	9.92	9.89	9.01
Pounding ; gap 10cm Cable ; clearance 10cm	0.1g	6.05	0.00	0.00	0.00	0.00	6.08
	0.2g	10.41	7.16	0.76	0.15	6.07	10.13
	0.3g	11.51	11.58	8.89	7.19	9.85	10.56
	0.4g	12.26	12.62	11.35	9.89	11.12	11.22
	0.5g	13.01	13.56	11.58	11.14	13.14	12.08
	0.6g	13.90	14.71	12.73	12.63	13.99	12.62



(a)PGA=0.3g, cable clearance=5 cm (b)PGA=0.3g, cable clearance= 10cm
Figure 9. Relative displacement time histories of superstructure between P1 and P2 with and without cable

CONCLUSION

It is found that the proposed analysis model using the simplified multiple oscillators is appropriate to evaluate the response behaviors of the several simple span bridge system under seismic excitations revealing the effects of both pounding and restrainers. Using the proposed system, it is found that the pounding phenomena can occur even for the bridge system consisting of the same oscillating units, which have the same natural frequencies. The pounding occurs due to the higher stiffness of the abutment, and the interactions are transferred to the whole system. It is found that the relative displacements in the middle portion of the bridge can be obtained, which cannot be accessible without considering pounding phenomena. The biggest relative displacements are found to occur between the abutment and nearby girder. The utilization of the restrainer is found to be effective to prevent the spans from collapsing by reducing the relative displacement properly.

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