Effect of slack length on seismic response of curved viaduct equipped with cable restrainers under great earthquakes

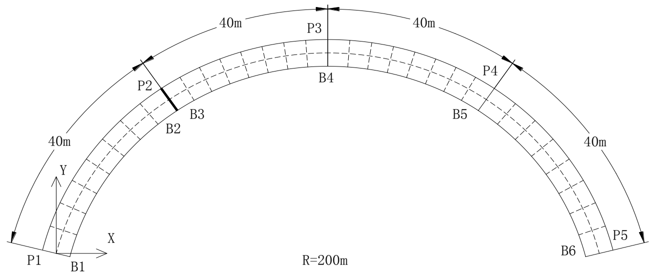
Hokkaido University　Student member　○ Qin TIAN
Hokkaido University　Fellow member　Toshiro HAYASHIKAWA
Hokkaido University　Member　Takashi MATSUMOTO
Hokkaido University　Member　Xingwen HE

1. INTRODUCTION

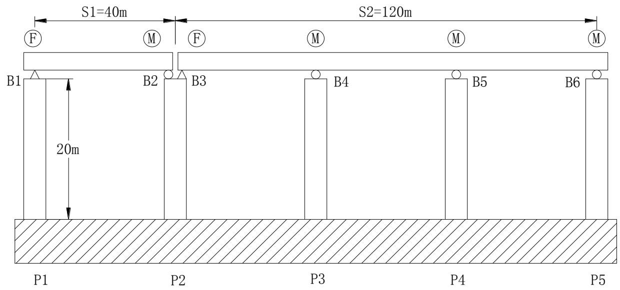
Horizontally curved viaducts have become an important component in modern highway systems in past decades. They represent a viable option at complicated interchanges or river crossings. In addition, curved alignments result in better aesthetic, an increase in traffic sight distances and economically competitive construction costs compared with straight bridges. On the other hand, bridges with curved configurations may sustain severe damage owing to rotation of the superstructure or displacement towards the outside of the curve due to the complex vibrations that occur during an earthquake[1].

In earthquake, failure of steel bearings resulted in collapse of highway viaducts during the 1995 Kobe earthquake[2]. So roller bearings equipped with stopper are installed on top of piers to mitigate viaduct damage. In addition, another commonly adopted earthquake protection technology, cable restrainers, provides connection between adjacent spans. The purpose is to prevent the unseating of decks from top of the piers at expansion joints by limiting the relative movements of adjacent bridge superstructures. Moreover, cable restrainer provides a fail-safe function by supporting a fallen girder unseated in the event of severe earthquake[1]. Although the cost of cable restrainer installation is relative low, their performance has been generally satisfactory during recent moderate earthquakes.

Slack value is one of very important parameters of cable restrainers. So the paper's purpose is to analyze the overall performance of highway viaducts with cable restrainers of different slack values. The effect of cable restrainer's slack values on deck unseating damage, tangential and radial joint residual opening damage, pier damage and ratio of energy dissipation is analyzed. The relationship between viaduct damage and cable restrainer's slack values will be found. According to the relationship, the seismic design of curved viaduct based on the cable restrainer with appropriate slack value will be put forward. The study combines non-linear dynamic analysis with a three-dimensional bridge model in order to evaluate the seismic response accurately.



(a) Plan view of curved highway viaduct model



(b) Elevation view of curved highway viaduct model

Fig. 1 Analytical model of viaduct

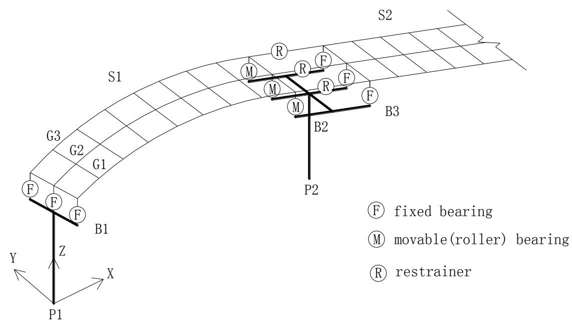


Fig. 2 Curved viaduct finite element model

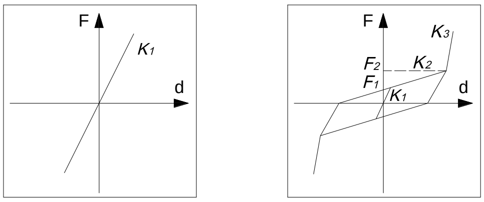
2. ANALYTICAL MODEL OF VIADUCT

The highway viaduct considered in the analysis is composed by a three-span continuous span connected to a single simply-supported span. The overall viaduct length of 160 m is divided in equal spans of 40 m, as represented in **Fig. 1**. The bridge alignment is horizontally curved in a circular arc with a radius of curvature of 200 m, measured from the origin of the circular arc to the centre-line of the deck superstructure. Piers and bearing supports adopt a tangential configuration with respect to the global coordinate system, in which the X and Y-axes lie in the horizontal plane while the Z-axis is vertical.

2.1 Deck superstructure and piers

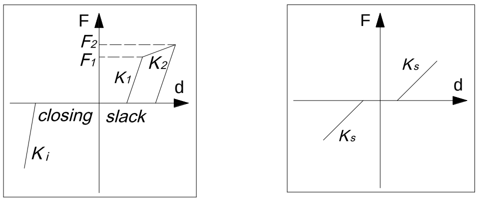
The bridge superstructure consists of a concrete deck slab that rests on three I-section steel girders (G1, G2 and G3) equally spaced at a distance of 2.1 m. The girders are inter-connected by end-span diaphragms as well as intermediate diaphragms at a uniform spacing of 5.0 m. Full composite action between the slab and the girders is assumed for the linear elastic elements of the superstructure model, which is represented by the three dimensional grillage beam system shown in **Fig. 2**. The deck weight is supported on four hollow box section steel piers 20 m high designed according to the Japanese seismic code[1]. The cross-sectional properties of the deck and bridge piers are summarized in **Table 1**. Steel and concrete densities are 7850 kg/m^3 and 2500 kg/m^3 respectively.

Characterization of structural pier elements is based on fiber element modeling where the inelasticity of the flexure element is accounted for by the division of the cross-section into a discrete number of longitudinal and transverse fiber regions with the constitutive model based on uniaxial stress-strain relationship for each zone[3]. The pier top is modeled in the transverse as a massless rigid bar to simulate the interactions between deck and pier motions.



(a) Fixed bearing　　(b) Roller bearing

Fig. 3 Analytical model of steel bearing supports



(a) Tangential deformation　　(b) Radial deformation

Fig. 4 Analytical model of cable restrainers

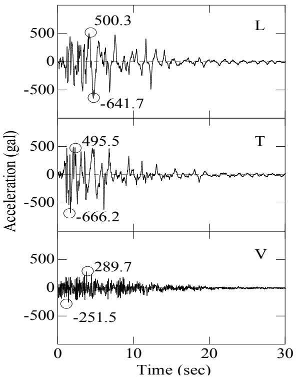


Fig. 5 JR Takatori St. record 1995 Kobe earthquake

Table 1 Cross-sectional properties of deck and piers

Pier or Girder	A (m^2)	I_x (m^4)	I_y (m^4)[a]
P1	0.4500	0.3798	0.3798
P2	0.4700	0.4329	0.4329
P3	0.4700	0.4329	0.4329
P4	0.4700	0.4329	0.4329
P5	0.4500	0.3798	0.3798
G1	0.2100	0.1005	0.0994
G2	0.4200	0.1609	0.2182
G3	0.2100	0.1005	0.0994

[a] I_z in case of G1, G2 and G3.

Table 2 Structural properties of steel bearing supports

Bearing type	Com-ponent	K_1 (MN/m)	K_2 (MN/m)	K_3 (MN/m)	F_1 (MN)	F_2 (MN)
Fixed	Longi-tudinal	980.0	–	–	–	–
	Trans-verse	980.0	–	–	–	–
Roller	Longi-tudinal	49.0	0.0098	980.0	0.0735	0.0743
	Trans-verse	980.0	–	–	–	–

Table 3 Structural properties of cable restrainer

Cable restrainer	K_1 (MN/m)	K_2 (MN/m)	F_1 (MN)	F_2 (MN)
Restrainer 4 (R4)	204.058	10.203	2.584	3.040

2.2 Bearing supports

For continuous span, fixed bearings (**Fig. 3a**) are rested on pier 2. Roller bearings are rested on the other piers. For simple supported span, fixed bearings are installed at the left end (P1), and roller bearings are installed at the right end. **Table 2** shows the structural properties of the steel bearings. Roller bearings are represented by using a trilinear element shown in **Fig. 3b**. Coulomb friction force is taken into account in numerical analysis. The frictional force of a roller bearing is obtained by multiplying the vertical reaction due to the dead load acting on the support by the coefficient of friction assumed to be 0.05. In addition, lateral steel stoppers are provided at each side of roller bearings in order to prevent rollers to be dislodged from the bearing assembly. The effect of stoppers is introduced in the analytical model by the high third stiffness slope K_3. Besides the stopper value, 8 cm, is used in this analysis.

2.3 Expansion joint

The continuous span and single simply-supported span of viaduct are separated, introducing a gap of 10 cm that could close resulting in collision between deck superstructures. The pounding phenomenon, defined as taking place at the three girder ends, is modeled using impact spring whose stiffness (K_i) is 980.0 MN/m as shown in **Fig. 4a**.

Cable restrainers units are anchored to the three girder ends. Seismic cable restrainers, illustrated in **Fig. 4a**, have been tangentially modeled as tension-only spring elements. Initially, cable restrainers behave elastically with stiffness K_1, while their plasticity is introduced by the yield force (F_1) and the post-yielding stiffness ($K_2 = 0.05 \times K_1$). Finally, the failure statement is taken into account for ultimate strength F_2, and since then, adjacent spans can separate freely without any action of the unseating prevention device[4]. The expansion joint which is constrained in the relative vertical movement, allows for both, tangential and radial, horizontal displacements. The effect of restricted radial displacements due to cable–girder interaction in the expansion is considered by the activation of a shear stiffness K_s = 49.0 MN/m once the gap of 0.05 m is exceeded as shown in **Fig. 4b**. Structural properties of cable restrainer are shown in **Table 3**. The calculation cases are based on the different slack values of cable restrainers.

3. METHOD OF ANALYSIS

The analysis on the viaduct model is conducted using an analytical method based on the elasto-plastic finite displacement dynamic response analysis. The tangent stiffness matrix, considering both geometric and material nonlinearities is adopted in this study, being the cross sectional properties of the nonlinear elements prescribed by using fiber elements. The stress-strain relationship of beam-column element is modeled as a bilinear type. The yield stress is 235.4 MPa, the elastic modulus is 200 GPa and the strain hardening in plastic area is 0.01. The implicit time integration Newmark scheme is formulated and used to directly calculate the responses, while the Newton-Raphson iteration method is used to achieve the acceptable accuracy in the response calculations. The damping of the structure is supposed a Rayleigh's type, assuming a damping coefficient of the first two natural modes of 2%. The non-linear bridge model was subjected to the strong ground motion records (**Fig. 5**) measured by the Takatori (TAK) stations during the 1995 Kobe earthquake.

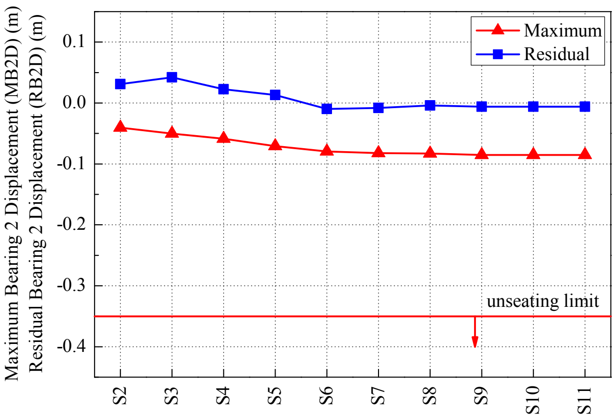


Fig. 6 Evaluation of deck unseating damage

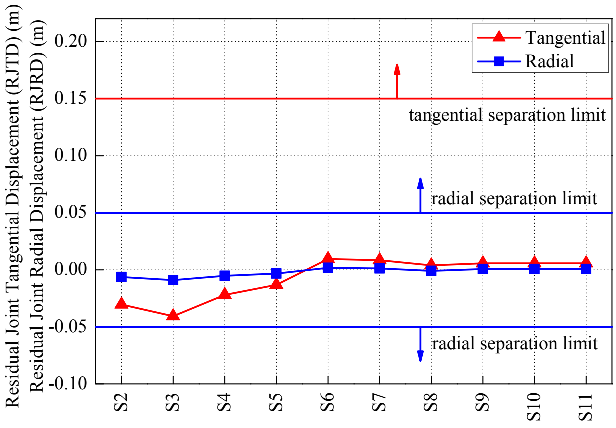


Fig. 7 Tangential and radial joint residual opening damage

4. NUMERICAL RESULTS

For an easy identification of the different calculation cases, a specific nomenclature is adopted in this research: “S” refers to the slack of cable restrainer. So “S4” indicates that the slack value is 4 cm.

4.1 Deck unseating damage

One of the most catastrophic damages to bridge structures is the failure due to deck unseating. In simply-supported spans, the induced relative displacements to steel roller bearings can exceed the seat width at the pier top, causing superstructure unseating. The maximum roller bearing (B2) relative displacement in the negative tangential direction has been established as the damage index to evaluate the potential possibility of deck unseating. A limit of 0.35 m is considered to indicate the high unseating probability for the viaduct. Collapse due to deck unseating does not happen for all cases as shown in **Fig. 6**. Maximum bearing (B2) displacements for negative direction increase with cable restrainers slack value increasing.

As stopper values are from 2 cm to 5 cm which are much smaller than stopper value, 8 cm, the maximum negative displacement of roller bearing (B2) is same with slack value respectively. Since cable restrainer extension force instead of negative pounding force between roller bearing and stopper that resists the negative displacements of simple supported span. In other words, negative pounding phenomena between roller bearing (B2) and stopper do not happen. So the slack values determine the maximum roller bearing negative displacement.

As slack values are from 6 cm to 8 cm which are a little smaller than stopper value, the maximum negative displacement of roller bearing is 8 cm, same with stopper value. At the beginning, cable restrainers start to resist negative displacement of simple supported span. As negative displacement of simple supported span becomes large, the negative pounding force between roller bearing (B2) and stopper also join the work. Then cable restrainer extension force and negative pounding force between roller bearing and stopper work together to resist the negative displacements of simple supported span. So the maximum displacements of roller bearing (B2) are determined by both of them.

As slack values are larger than 8 cm, the maximum negative displacement of roller bearing is also 8 cm, same with stopper value. It is stopper that resists negative displacements of simple supported span. Cable restrainers do not work in these cases.

4.2 Tangential and radial joint residual opening damage

Permanent tangential offsets of expansion joints have substantially interfered with the post earthquake serviceability. The possibility for vehicles to pass over the tangential gap length, measured as the contact length of a truck tire (0.15 m), is suggested as the limit of the damage index. According to the experiences obtained from previous earthquake damages, residual joint radial displacement (RJRD) values greater than 0.05 m reduce the available deck width, being able to eliminate the usability of one traffic lane. RJTD and RJRD are all under the limit line for all cases as shown in **Fig. 7**, so there is no damage for RJTD and RJRD in earthquakes.

As slack values are from 2 cm to 5 cm which are much smaller than stopper value, 8 cm, residual joint tangential displacements (RJTD) are minus values which present the closing trend. Since slack values are much smaller, cable restrainers work earlier and more intensively. Moreover, since radical direction of all bearings is restraint, residual joint radical displacement (RJRD) is near 0.

4.3 Pier damage

Bridges supported on piers with large residual inclination may lose their serviceability, becoming largely unsafe and probably irreparable. Due to this fact, the 1996 Japanese specifications imposed the limitation of 1% maximum inclination of the bridge pier height in post-earthquake pier demolition for important bridges[1]. Cable restrainer slack values have influence on residual pier inclination as shown in **Fig. 8**. Viaducts with smaller cable restrainer slack values have less residual pier inclination.

In the case of slack value (from 2 cm to 8 cm) is smaller than stopper value (8 cm), as slack value of cable restrainer decreases, residual pier inclination decreases. Viaduct with slack value, 2 cm and 3 cm, has no pier inclination damage. At the beginning, top of piers has positive direction trend for TAK input. Maximum positive pounding force between roller bearing and stopper can results in large positive displacement on top of piers. However, cable restrainer extension can reduce maximum positive pounding force between roller bearing and stopper for pier 3, pier 4, and pier 5. As slack value of cable restrainer decreases, cable restrainer begins to work earlier. Then it can results in less positive pounding force between roller bearing and stopper on top of piers (P3, P4 and P5).

As slack value is large than 8 cm, residual pier inclination presents same value. Cable restrainers do not work, because stopper value, 8 cm, is smaller than slack value. So, slack values do not affect on the calculation results. In addition, residual pier inclination is larger than limit line value. Since there is no cable restrainer effect which can reduces residual pier inclination by reducing positive pounding force between roller bearing and stopper on top of piers (P3, P4 and P5).

Moreover, viaducts with cable restrainers have more centralized residual pier inclination for all piers than those without cable restrainers as shown in **Fig. 8**. In other words, cable restrainers can reduce the degree of difference for the

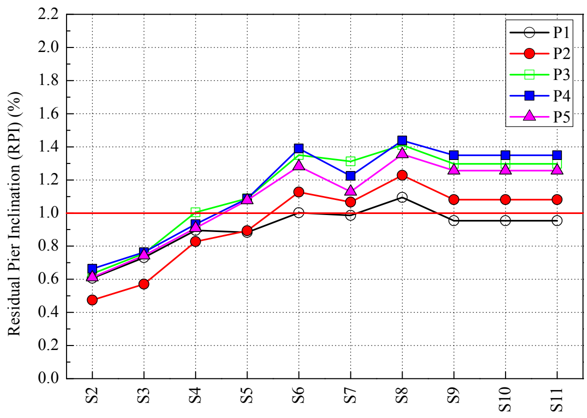


Fig. 8 Evaluation of pier inclination damage

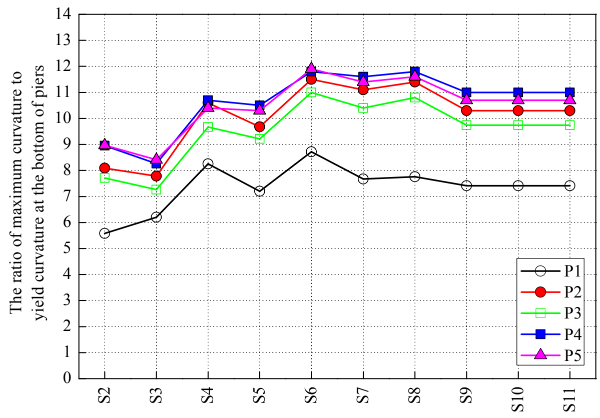


Fig. 10 Ratio of maximum curvature to yield curvature

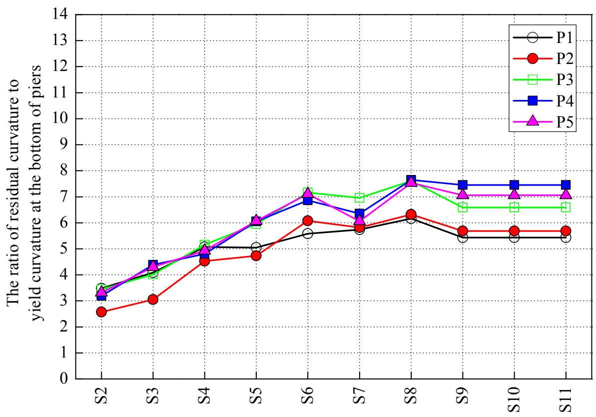


Fig. 9 Ratio of residual curvature to yield curvature

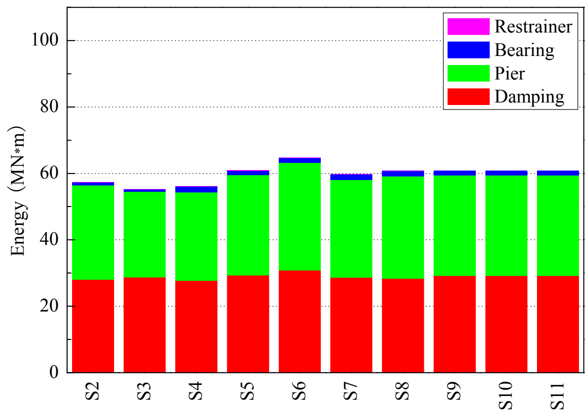


Fig. 11 Energy distribution at end of earthquake

behavior of simple supported span and continuous span by connecting them as a whole. Then cable restrainer can reduce the maximum positive pounding force between roller bearing and stopper in continuous span by averaging the positive pounding force on top of different piers. At last the less positive pounding force can lead to less residual pier inclination to a certain degree. As slack value of cable restrainer decreases, the effect becomes more obviously.

Finally, residual pier inclination and residual curvature have almost same change trend as shown in **Fig. 8** and **Fig. 9**. Residual curvature is determined by curvature time-history, especially, those curvatures which are larger than yield curvature. As slack of cable restrainer decreases, maximum curvature decreases as shown in **Fig. 10**. So, maximum curvature also has a great influence on residual pier inclination.

4.4 Ratio of energy dissipation

Distribution of total input seismic energy at end of earthquake is shown in **Fig. 11**. Calculation results indicate that cable restrainer and roller bearing dissipate a very small portion of the input energy. Since the area enclosed by the force–deformation relationship curve of restrainer and roller bearing is very small. The most part of energy is dissipated by damping and piers. Energy dissipated at piers is a clear indicator of pier damage. In addition, the less energy dissipated by piers appears in case of viaduct with smaller slack values as shown in **Fig. 11**. At the same time, the less residual pier inclination is also presented in the same case in **Fig. 8**. So energy distributed by piers has important influence on plastic deformation of piers.

5. CONCLUSIONS

Slack values of cable restrainers have effect on the viaducts damage. The presented results provide sufficient evidence for the following conclusions.

1) Calculated results demonstrate that curved viaducts without cable restrainers are more vulnerable to deck unseating damage. In addition, the possibility of deck unseating is reduced by decreasing the slack value of cable restrainers.
2) Curved viaducts with cable restrainers of smaller slack values are found less possible to tangential joint residual opening damage. But slack values of cable restrainers do not obviously affect residual joint radial displacement.
3) As slack value of cable restrainer decreases, residual pier inclination decreases. So viaduct with slack value, 2 cm and 3 cm, has no pier inclination damage. In addition, residual pier inclination, maximum curvature and residual curvature at piers bases have similar change trend in all cases.
4) From the energy perspective, viaducts with cable restrainers of smaller slack values sustain less dissipated energy. Energy dissipated at piers represents pier damage to a certain degree. So viaducts with cable restrainers of smaller slack values, present less pier damage.

REFERENCES

1) Japan Road Association (JRA): Specifications for Highway Bridges – Part V Seismic Design, Maruzen, Tokyo, 2002.
2) Scawthorn, C. and Yanev, P.I. “Preliminary report 17 January 1995, Hyogo-ken Nambu, Japanese earthquake”, Eng. Struct., 17 (3), 146-157, 1995.
3) Mendez Galindo, C., Ruiz Julian, F. D., Hayashikawa, T.: Seismic performance of isolated curved steel viaducts under level II earthquakes, Journal of Structural Engineering, JSCE, Vol. 55A, pp. 699–708, 2009.
4) Ruiz Julian, F.D., Hayashikawa, T. and Obata, T.: Seismic performance of isolated curved steel viaducts equipped with deck unseating prevention cable restrainers, Journal of Constructional Steel Research, 63 (2): 237-253, 2007.