



HOKKAIDO UNIVERSITY

Title	Seismic performance of curved highway bridges with steel bearings under serious earthquakes
Author(s)	田, 欽
Degree Grantor	北海道大学
Degree Name	博士(工学)
Dissertation Number	甲第11894号
Issue Date	2015-03-25
DOI	https://doi.org/10.14943/doctoral.k11894
Doc URL	https://hdl.handle.net/2115/58962
Type	doctoral thesis
File Information	Qin_Tian.pdf



SEISMIC PERFORMANCE OF CURVED HIGHWAY BRIDGES WITH STEEL BEARINGS UNDER SERIOUS EARTHQUAKES

QIN TIAN

Division of Engineering and Policy for Sustainable Environment
Graduate School of Engineering
Hokkaido University
Sapporo, Japan

February 2015

SEISMIC PERFORMANCE OF CURVED HIGHWAY BRIDGES WITH STEEL BEARINGS UNDER SERIOUS EARTHQUAKES

QIN TIAN

A DISSERTATION SUBMITTED IN PARTIAL FULFILLMENT
OF THE REQUIREMENTS FOR THE DEGREE
OF DOCTOR OF PHILOSOPHY

Division of Engineering and Policy for Sustainable Environment
Graduate School of Engineering
Hokkaido University
Sapporo, Japan

February 2015

ABSTRACT

As urban areas have become more congested and highway interchanges have become more complex, horizontally curved composite steel I girder bridges have become an essential component in highway infrastructure. These types of bridges are key to providing smooth transitions from one highway to the next, allowing for a seamless flow of traffic. Unfortunately, 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 serious earthquakes. So how to improve the seismic behavior of curved viaduct becomes important during strong earthquakes. Stopper installed at both sides of roller bearing in tangential direction, cable restrainer equipped between adjacent superstructures, viscous damper installed at expansion joint and shock absorber device set between roller bearing and stopper are all evaluated respectively in this thesis.

Traditional steel bearings are very common, but they are easy to be broken in earthquakes. Recently roller bearings equipped with stopper are installed on top of piers. Because stopper can be easily installed and the price is not expensive, so it has wide application prospect in future. Therefore, the effect of stopper together with cable restrainer on mitigating curved viaduct damage during serious earthquake is carried out. The relationship between different stopper values and the different types of damages is obtained. The advice for the seismic design of viaduct based on the appropriate stopper values has been put forward.

Recently, cable restrainer installed at curved viaduct can significantly reduce the possibility of deck unseating damage. In this thesis, the effect of stiffness and slack value of cable restrainer on curved viaduct damage is presented. According to the evaluation results, the seismic design of curved viaduct based on the cable restrainer with appropriate stiffness and slack value can be put forward. In addition, a comparison between restrained and unrestrained curved viaduct is also presented. Finally, the use of cable restrainer reducing curved viaduct damage is proved in this thesis.

In addition, application of viscous dampers to viaducts has gained significant attention in recent years. Due to its large resistance force and energy dissipation capacity, viscous dampers have wide application prospect. The most advantage of viscous dampers is that it shows no resistance forces under slow relative movements of segments results from thermal changes, creep and shrinkage effects. It is only activated when earthquake happens. Therefore, this thesis analyzes the overall performance of highway viaducts with different kinds of viscous dampers and different stopper values. Different kinds of viscous dampers have different stiffness and different resistance forces. So the relationships between different kinds of viscous dampers, different stopper values, and curved viaduct damage have been found respectively. A comparison between curved viaducts equipped with viscous dampers and without viscous dampers is also carried out. The calculation results demonstrated that stopper value plays a more important role in piers damage than viscous damper.

In recent years, the effects of shock absorber devices installed between adjacent viaduct segments on reducing pounding forces, have been studied by many researchers. But the effectiveness of shock absorber devices installed between roller bearing and stopper on reducing pounding damage, is lack. In this thesis, calculation results clearly show that the curved viaduct with shock absorber devices between roller bearing and stopper present less damage. In addition, the effect of stiffness and thickness of shock absorber devices on mitigating curved viaduct damage is evaluated respectively. Besides, strain hardening, strain softening and elastic shock absorber devices are also analyzed in this thesis. Conclusively, shock absorber devices with large stiffness and large thickness present less viaduct damage. Curved viaduct with strain softening shock absorber devices exhibits less viaduct damage.

ACKNOWLEDGEMENTS

First and foremost, I would like to extend my sincere gratitude to my supervisor, Professor Toshiro Hayashikawa, for his helpful suggestions in terms of the content and style of this thesis. Without his patient assistance and friendly encouragement, it would not be possible for me to complete this thesis on time without reducing its scholarly quality. I benefited much from his critical thinking, copious knowledge and scholarly expertise in three years and a half. His wide knowledge and his constant support have been of great value for me in future. I am so pleasure and honor to do research with him.

Additionally, I would like to thank my committee members Associate Professor Takashi Matsumoto, Professor Yukinori Kobayashi and Professor Mitsumasa Midorikawa, not only for their time and extreme patience, but for their intellectual contributions to the development of this work.

I want to express my thanks to Associate Prof. Takashi Matsumoto, whose advice, motivation and encouragement always created the best environment in our laboratory. I also would like to express my gratitude to Assistant Prof. Xingwen He, who gives me many useful advices and comments for my study. I extend many thanks to my colleagues and friends from the Laboratory of Bridge and Structural Engineering, especially Shoyichi Iijima, Basir Zisan, Dr. Sun and Dr. Ahmed, who give me many helpful comments and suggestions to improve the quality of this thesis. In addition, I would like to express my gratitude to Toshihiko Hosokawa and Takeshi Shimoda from bottom of my heart, who give me their kind hearted helps of the living in Japan. Without their helps in living, I shall not pay enough attention to the study during the doctor courses. And I also want to give my thanks to my Chinese friends in our laboratory, including Dr. Zhiping Gan, Di Bai, Lin Yue and Junpeng Liu for making my experience in the laboratory pleasure. Sincere thanks are extended to Javier Gil Belda and Md. Robiul Awall.

I would like to sincerely express my gratitude to China Scholarship Council (CSC) for the financial supports providing me this precious opportunity and therefore making all of this possible. I extend many thanks to Communist Party of China and Chinese government. I also express my sincere gratitude to Hokkaido University in Japan, who accepted me to study here. A very special thanks to Consulate General of China in Sapporo, who always concerned about my learning and living in Sapporo.

I owe my deepest gratitude to Prof. Wenxin Ren in China, who encourages me to study hard in Japan. His motivation and encouragement were an essential factor for the successful completion of this thesis.

I am also deeply indebted to my Japanese language class teachers, Ogawara Yoshiro, Kobayashi Yuko, Miki Hirata. Without their kind hearted helps, I shall not understand Japanese language and conveniently live in Japan. They let me know Japanese cultures, Japanese life style and real-life manners.

Finally, and most importantly, I want to thank my mother Daichun Wu, brother Hong Tian and sister-in-law Junzhi Xiang. They always supported me a lot of care, love and understanding during all my studies. My mother always helps me out of difficulties without a word of complaint. My appreciation also goes to my nephew Lin Tian, to my aunt Zhicui Tian and to my cousin Zhifeng Yang.

Qin Tian
Sapporo, Japan
February 2015

CONTENTS

TITLE PAGE

ABSTRACT

ACKNOWLEDGEMENTS

CONTENTS

LIST OF PRINCIPLE SYMBOLS

CHAPTER 1: GENERAL INTRODUCTION

1.1	Statement of the Problem	1
1.2	Brief Review of Previous Research	3
1.3	Object and Scope of Study	5
1.4	Organization of the Thesis	5
	References	7

CHAPTER 2: NONLINEAR ANALYTICAL APPROACH

2.1	Introduction	9
2.2	Natural Vibration Analysis Formulation	10
2.3	Three-dimensional Fiber Beam Element Formulation	13
2.3.1	Strain displacement relations	14
2.3.2	Equilibrium condition	15
2.3.3	Finite element formulation	15
2.3.4	Solution technique of non-linear equilibrium equation	18
2.4	Non-linear Dynamic Equilibrium Equation of Motion	20
2.4.1	Dynamic equilibrium equation	20
2.4.2	Consistent mass matrix	20
2.4.3	Damping mechanism	21
2.4.4	Integration of dynamic equilibrium equation	22
2.5	Energetic Analysis	25
2.6	Conclusions	26
	References	27

CHAPTER 3: DESCRIPTION OF THE HIGHWAY VIADUCTS

3.1	Introduction	29
3.2	Analytical Model of the Viaduct	29
3.2.1	Deck superstructure	30
3.2.2	Piers	31
3.2.3	Bearing supports	32
3.2.4	Expansion joint	34
3.3	Input Earthquake Ground Motions	35
3.4	Classification of Seismic Damage	38
3.4.1	Deck unseating damage	38

3.4.2	Pounding damage	39
3.4.3	Tangential and radial joint residual opening damage	39
3.4.4	Pier damage	40
3.5	Conclusions	40
3.6	Appendices	41
	References	47

CHAPTER 4: SEISMIC RESPONSE OF HIGHWAY VIADUCTS EQUIPPED WITH ROLLER BEARINGS OF DIFFERENT STOPPER VALUES

4.1	Introduction	49
4.2	Unseating Prevention System	49
4.2.1	Unseating prevention mechanism	49
4.2.2	Analytical model of cable restrainer	51
4.3	Analytical Model of Roller Bearings with Different Stopper Values	51
4.4	Numerical Results and Discussion	52
4.4.1	Deck unseating damage	53
4.4.2	Pounding damage	54
4.4.3	Tangential and radial joint residual opening damage	55
4.4.4	Pier damage	56
4.4.5	Relationship of bending moment and curvature	57
4.5	Conclusions	58
	References	58

CHAPTER 5: EFFECTIVENESS OF CABLE RESTRAINERS ON SEISMIC RESPONSE

5.1	Introduction	61
5.2	Cable Restrainers with Different Stiffness and Different Slack Values	61
5.3	Effect of Stiffness	62
5.3.1	Deck unseating damage	62
5.3.2	Tangential and radial joint residual opening damage	62
5.3.3	Pier damage	63
5.3.4	Relationship of bending moment and curvature	64
5.4	Effect of Slack Values	65
5.4.1	Deck unseating damage	65
5.4.2	Tangential and radial joint residual opening damage	66
5.4.3	Pier damage	66
5.4.4	Relationship of bending moment and curvature	67
5.5	Conclusions	68
	References	69

CHAPTER 6: EFFECTIVENESS OF VISCOUS DAMPER ON SEISMIC RESPONSE

6.1	Introduction	71
6.2	Analytical Model of Viscous Damper	71
6.3	Numerical Results and Discussion	73

6.3.1	Deck unseating damage	73
6.3.2	Tangential and radial joint residual opening damage	73
6.3.3	Relative displacement between superstructures	75
6.3.4	Pier damage	75
6.3.5	Relationship of bending moment and curvature	80
6.4	Conclusions	80
	References	81

CHAPTER 7: EFFECTIVENESS OF SHOCK ABSORBER DEVICES ON SEISMIC RESPONSE

7.1	Introduction	83
7.2	Analytical Model of Shock Absorber Devices	84
7.3	Analytical Model of Expansion Joint	86
7.4	Effect of Stiffness	87
7.4.1	Deck unseating damage	87
7.4.2	Bearing force	87
7.4.3	Pier damage	89
7.4.4	Displacement of superstructures	92
7.4.5	Relationship of bending moment and curvature	93
7.5	Effect of Thickness	94
7.5.1	Deck unseating damage	94
7.5.2	Bearing force	95
7.5.3	Pier damage	97
7.5.4	Displacement of superstructures	99
7.5.5	Relative displacement between two decks	100
7.5.6	Relationship of bending moment and curvature	101
7.6	Effect of SAD's Type	102
7.6.1	Deck unseating damage	102
7.6.2	Bearing force	103
7.6.3	Pier damage	104
7.6.4	Displacement of superstructures	105
7.6.5	Relationship of bending moment and curvature	106
7.7	Conclusions	108
	References	110

CHAPTER 8: SUMMARY AND CONCLUSIONS 111

LIST OF PRINCIPLE SYMBOLS

a	cross section area;
a_m, a_k	stiffness and mass proportional damping constants;
a_{max}	maximum acceleration of the input excitation;
A	cross-sectional area;
$\mathbf{A}$	transformation matrix;
A_s, A_c, A_v	steel girder, concrete slab and composite section areas;
b	equivalent cross section dimension;
c	equivalent cross section dimension;
c_z	modification factor for zone;
c_B	standard value of effective displacement coefficient;
$[C]$	the system damping matrix;
$\{D\}, \{d\}, \mathbf{d}$	global, local displacement vectors;
d	equivalent cross section dimension;
$\mathbf{d}(t)$	displacement vector of modal coordinates;
$\mathbf{d}^{i+1}, \mathbf{d}^{(i+1)}, \mathbf{d}, \mathbf{d}_t$	incremental displacement at iteration $i+1$, incremental displacement and displacement at incremental step t vectors;
$\dot{\mathbf{d}}^{i+1}, \dot{\mathbf{d}}, \dot{\mathbf{d}}_t$	incremental velocity at iteration $i+1$, incremental velocity and velocity at incremental step t vectors;
$\ddot{\mathbf{d}}^{i+1}, \ddot{\mathbf{d}}, \ddot{\mathbf{d}}_t$	incremental acceleration at iteration $i+1$, incremental acceleration and acceleration at incremental step t vectors;
$\{\ddot{\mathbf{d}}\}, \{\ddot{\mathbf{d}}_g\}$	acceleration vector and the ground acceleration history;
E	material modulus of elasticity;
E_t	tangent stiffness parameters;
E_I, E_K, E_D, E_S	input, kinetic, damping and strain energies;
e	equivalent cross section dimension;
f_w	hollow rectangular cross section dimension;
$\{f\}, \mathbf{f}, \mathbf{f}_t$	element nodal force vectors;
f, f_t	external applied forces;
F_1, F_2	isolation bearing force parameters (plastic and hardening);
$F, \{F\}_{t+1} - \{F\}_t$	unbalanced force vectors;
F_y	yield force;
$F^{(i+1)}$	incremental force at iteration $i+1$;
g	acceleration of gravity;
G	shear modulus;
$H^{(i)}, H$	incremental damping forces;
h_B	equivalent damping ratio;
$I^{(i)}, I$	incremental inertia forces;
I_y, I_z	cross-section moments of inertia about y- and z-axes;
J	cross section torsional constant;
$[k], \mathbf{k}$	element stiffness matrices;
k_h, k_{h0}	design horizontal seismic coefficient and its standard value;
K_1, K_2, K_3	isolation bearing stiffness parameters (elastic, plastic and hardening);
K_B	effective stiffness;
K_r, K_s	restrainer stiffness in tangential and radial directions;
K_i	impact stiffness;
$[K], \mathbf{K}, [\mathbf{K}]_{t+1}$	the system stiffness matrices and tangent stiffness matrix at time $t+1$;
$\mathbf{K}_t$	tangent stiffness matrix;
l	beam element length, effective pier length;
L	viaduct longitudinal mode shape;

$[m], \mathbf{m}$	element mass matrices;
$\bar{m}$	beam element mass per unit length;
M_{e0}	yield moment;
m_{ij}	components of matrix $\mathbf{m}$;
M_{xi}, M_{yi}, M_{zi}	element nodal moments in local coordinate system, node i ;
M_n^*	the effective modal mass;
M_j	the j th modal mass;
$[M], \mathbf{M}$	the system mass matrices;
M_w	earthquake moment magnitude;
$[N]$	$\mathbf{H}(x,y,z)\mathbf{A}^{-1}$, shape function;
n	ratio of Young's modulus;
P	equivalent lateral force;
$\mathbf{q}(t)$	the n -dimensional vector of modal coordinates, $\mathbf{q}(t) = \{q_1(t) \ q_2(t) \ \dots \ q_n(t)\}^T$;
Q_d	yield strength;
$\{r\}, \mathbf{r}$	displacement transformation vectors;
$\mathbf{r}, \mathbf{r}_t$	force vectors;
$R^{(i)}, R$	incremental elastic forces;
r	radius of gyration;
s	foundation excitation spatial distribution vector;
s_s, s_c, s	steel girder, concrete slab and composite section gravity center distances;
t_1, t_2, t_{11}, t_{22}	hollow rectangular cross section dimensions;
Δt	time increment from time step t to time step $t+1$;
t	equivalent cross section dimension;
T, T_0	natural periods of the fundamental mode;
t_E	earthquake total duration;
T_{sv}	Saint-Venant torsion;
$\bar{u}, \bar{v}, \bar{w}, \bar{u}_t, \bar{v}_t, \bar{w}_t$	displacements of an arbitrary point in a beam element;
$u_i, v_i, w_i, \theta_{xi}, \theta_{yi}, \theta_{zi}$	element nodal displacements in local coordinate system, node i ;
$\mathbf{u}(x,y,z)$	internal displacement at any general point along the beam element;
u_{Be}	effective design displacement;
U	internal work;
V	volume;
$\Delta w, w$	energy dissipated during stress strain cycle and equiv. linear elastic energy;
ww	hollow rectangular cross section dimensions;
W	external virtual work, superstructure weight;
$XYZ, xyz, x'y'z'$	the global system, local system, auxiliary local system coordinates;
X_i, Y_i, Z_i	element nodal forces in local coordinate system, node i ;
$\alpha_M, \alpha_C, \alpha_K$	integration constants;
$\boldsymbol{\alpha}$	generalized coordinates $\{\alpha_1, \alpha_2, \alpha_3, \alpha_4, \alpha_5, \alpha_6, \alpha_7, \alpha_8, \alpha_9, \alpha_{10}, \alpha_{11}, \alpha_{12}\}^T$;
α	strain hardening ratio;
β	the angle of rolling and Newmark's β method constant;
γ	the generalized strain and Newmark's integration parameter;
η_i, η_j, η	damping ratios;
θ_x	the cross-sectional rotation about x axis;
θ	viaduct orientation angle, skew angle;
μ	friction coefficient, shear displacement ductility ratio;
σ_{t+1}, σ	the axial Cauchy stress, second Piola-Kirchoff axial stress;
ν	Poisson ratio;
$\psi_i(x)$	shape functions;
$\omega_i, \omega_j, \omega$	natural circular frequencies;
Γ_n	modal participation factor;

$[\Phi], \Phi =$ mode shape matrices;
 $\Phi_n =$ n th mode eigenvector; and
 $\varepsilon_t =$ the strain in the deformed configuration t .

Superscripts

$\det. | | =$ determinant of matrix;
 $\delta =$ variational operator;
 $\mathbf{0} =$ zero vector;
 $('), ('') = d / dx$ and d^2 / dx^2 , respectively;
 $(\dot{ }), (\ddot{ }) = d / dt$ and d^2 / dt^2 , respectively;
 $\{ \}^T =$ transposition vector; and
 $[]^{-1} =$ inverse matrix.

Subscripts

$n =$ number modes;
 $N =$ degree of freedom; and
 $t =$ incremental step.

GENERAL INTRODUCTION

1.1 Statement of the Problem

As urban areas have become more congested and highway interchanges have become more complex, horizontally curved composite steel I girder bridges have become an essential component in highway infrastructure. These types of bridges are key to providing smooth transitions from one highway to the next, allowing for a seamless flow of traffic. Unfortunately, curved viaducts are susceptible to be damaged during serious earthquakes. Due to the collapse of a bridge as show in **Fig 1.1**, society pays for a tremendous cost. Even non-collapsed, **Fig 1.2** shows that the temporary lost of post-earthquake serviceability of important bridges may cause very costly disruption to vehicle traffic on major transportation arteries and is simply unacceptable [1]. In addition, the direct economic loss, attributed to replacement or repair cost of the damaged bridges, forms only a minor portion of the total losses. The major cost, which is often difficult to estimate accurately, is associated with the disruption of vehicle movement immediately following the earthquakes and during the long re-construction phase [2]. Curved viaducts are more subject to seismic damage than straight viaducts during serious earthquakes, due to rotation of the superstructure or displacement towards the outside of the curve. Asymmetrical and complex curved girder bridge seismic behavior has been observed in association with past earthquakes, and subsequent damage observations substantiate their vulnerability. A representative example includes a curved multispan steel girder bridge located 49 km from the epicenter of the 1992 Petrolia earthquake that suffered severe damage [3]. This damage occurred at the cross-frames and shear connections between the steel girder and concrete deck, resulting in large lateral deformations at the bearings. Based on the previous study, coupling between the bending, torsional, and lateral motions in curved bridges caused bearing and cross-frame damage [4].

However, the widely recognized susceptibility to earthquake damage of curved highway viaducts is substantially amplified with the irregularity originated by the rupture of continuity of the deck superstructure at expansion joints. Long bridges are constructed as multiple spans separated by joints in the superstructure to accommodate thermal expansions and contractions without inducing large forces to the bridge components. Recent strong earthquakes have repeatedly demonstrated that during an earthquake, adjacent spans often vibrate out-of-phase, causing two main types of displacement problems. The first type is pounding damage between superstructures. Pounding could amplify the relative displacement results in the requirement of longer seat width to support the deck [5, 6]. Moreover, pounding transfers large seismic lateral forces from one deck to another that leads to a significant change in the seismic response of the entire viaduct system [7-10]. Besides, pounding may significantly increase the reaction forces at the piers bases [11, 12]. The second type is the falling down of deck, since pounding increases the opening gap between adjacent superstructures [13-16]. The falling down of deck is the most common seismic damage and it has been observed to be considerably magnified by complex bridge configurations like curved deck geometries as show in **Fig 1.3**. In order to overcome mentioned viaducts damage, steel stopper have been equipped at both sides of roller bearings. Stopper could prevent lager displacement of roller bearing, so the damage of roller bearing could be mitigated. As stopper value is less than the gap of expansion joint, the pounding phenomenon shall not occur. Therefore, there are no pounding damages between superstructures during serious earthquakes. Besides stopper can be easily installed and the price is not expensive, so it has wide application prospect in future.



Fig. 1.1 The collapse of a bridge in Kobe earthquake



Fig. 1.2 Disturb normal traffic after earthquake



Fig. 1.3 The falling down of deck in earthquake



Fig. 1.4 Bridge pier damage in earthquake

Unfortunately, pounding force between roller bearing and stopper is so large, which could bring damages at not only roller bearing but also piers bases as show in **Fig 1.4**. Hence, rubber shock absorber device between roller bearing and stopper could be used to reduce the negative pounding effects by smooth changes of impact stiffness. Besides shock absorber devices could be easily installed and the price is cheap. Based on previous study [17, 18], the effects of shock absorber devices installed between adjacent viaduct segments on reducing pounding forces, have been studied by many researchers. The calculation results demonstrated that shock absorber devices could obviously reduce the negative effect of pounding between superstructures. But the effectiveness of shock absorber devices installed between roller bearing and stopper on mitigating negative pounding effect is lack.

Another method for overcoming the falling down of deck is the installation of cable restrainers that provide connection between adjacent spans. This type of unseating prevention system is established in addition to the seating length of the supporting substructure to provide restraint against unexpected seismic forces by limiting relative movements at the expansion joint and thus, minimizing the possibility of support loss of superstructures. Moreover, cable restrainers provide a fail-safe mechanism by supporting a fallen girder unseated in the event of a severe earthquake [19]. Although the cost of cable restrainer installation is relatively low, their performance has been generally satisfactory during recent

earthquakes. Cable restrainers appear to have been effective in preventing unseating in both the 1989 Loma Prieta and 1994 Northridge Earthquakes.

During serious earthquakes, if the input energy is dissipated by the device equipped on the viaduct, the viaduct damage could be significantly reduced. The viscous damper can consume some energy induced by earthquake. Fortunately, the viscous damper has been developed since the late 1980s [20-23]. It provides an adaptable damping force by adjusting the size of the orifice through which a viscous fluid flows when a piston moves in a hydraulic cylinder. The most advantage of viscous dampers is that it shows no resistance forces under slow relative movements of segments results from thermal changes, creep and shrinkage effects. It only becomes activity when earthquake occurs. Therefore, application of viscous dampers to viaducts has gained significant attention in recent years [24]. Due to its large resistance force and energy dissipation capacity, viscous dampers have wide application prospect.

1.2 Brief Review of Previous Research

The effect of seismic activities on horizontally curved bridge is studied by many researchers. In 1976, experimental studies on the seismic response of high curved over crossings were performed by Williams and Godden. [25]. Their experiments are related to seismic resistance of large multi-span curved over crossings similar of the type, which suffered heavy damage during the 1971 San Fernando Earthquake. Buckle [26], in New Zealand, conducted in-situ response evaluation of two bridges subjected to earthquake loads, one straight and the other curved and continuous over many spans. Time histories of acceleration and displacement were collected and analyzed using spectral analysis. Abdel-Salam and Heins. [27] carried out a study on the seismic response of curved bridges. They examined moments and shear occurring at abutments, bents, and column bases of their model by analyzing the response history of the bridge. Chen [28] presented the effects of various modeling and analysis methods on seismic responses of curved bridges. Important system parameters such as bridge length, soil-structure interactions and bridge skews were examined. DesRoches and Fenves [29], in 1997, evaluated the earthquake response of a curved highway bridge. They developed a three-dimensional calibrated bridge model to investigate and compare the significant differences between uniform free-field and non-uniform input motion. More recently, a study by Hayashikawa et al. [30] evaluated in detail the influence of various bearing conditions on curved bridges. This investigation showed that the effects of steel bearing support conditions dramatically affect the seismic behavior of curved bridges. In 2011, Ates and Constantinou perform [31] a parametrical study associated with the effect of the earthquake ground motions on the seismic response of isolated and non-isolated curved bridges including soil-structure interaction. The maximum and minimum response values of the isolated and non-isolated bridges are compared with each other for different cases. In 2012, Seo and Linzell [32] focused on detailed description of the fragility curve generation and application process during earthquakes, with a focus on computational work involved to create the curves. Seo and Linzell [33], in 2013, studied the effective of the radius of curvature and number of spans on bearing deformation and column curvature ductility. The calculation results indicated that 17-22% variation in bearing deformation and column curvature ductility caused by variation of the radius of curvature and number of spans. In 2014, Wilson, Mahamoud and Chen [34] carried out a study on the combined effects of skew and curvature on the seismic performance of reinforced concrete bridges. The results show a considerable impact on the seismic performance due to the effects of skew and curvature, with stacking effects observed in the combined geometries. Insights on the complexities of curvature, skew, loading direction and support condition are also made. Abdelnaby et al. [35], in 2014, studied on the bridges piers behavior during earthquakes. In his study, a hybrid

simulation test is conducted on a curved four-span bridge. A detailed numerical finite element, fiber-based model of the whole bridge system is established. The analytical predictions of this model are then used for comparison with the hybrid simulation test results.

During serious earthquakes, the application of cable restrainers on viaducts to prevent unseating of deck superstructures get researchers attention. Previous research on seismic cable restrainer design has included laboratory testing of cable restrainer devices [36] and evaluation and development of design procedures [37]. Post-earthquake evaluations from the 1989 Loma Prieta and the 1994 Northridge Earthquakes have shown that many cable restrainers were observed to have worked effectively during the earthquakes [38], preventing simply-supported spans from falling from their supports. However, the collapse of bridges such as the Gavin Canyon undercrossing and the Route 14/5 separation during the 1994 Northridge Earthquake proved that inadequate cable restrainer design can have catastrophic results [39]. Large seismic forces are likely to cause either the cables to break or the bridge diaphragm walls at the two ends of the cables to suffer a punch-through action during a severe earthquake. Kawashima and Shoji [40] investigated the effect of restrainers to mitigate pounding damage at expansion joints. Watanabe and Kawashima [41] evaluated the effectiveness of cable restrainers for mitigating rotation of skewed bridges during earthquakes. Kim et al. [42] studied the possibility of incorporating energy dissipating restrainers for seismic protection of bridge structures. Ruiz Julian, F. D. and Hayashikawa, T. [43, 44] evaluated the nonlinear seismic response of curved highway viaducts with different cable restrainers, such evaluation was based on a parametric study on design of cable restrainers for curved highway viaducts under Level II earthquakes. Shehata [17] used cable restrainers and natural rubber shock absorber to mitigate the adverse effects. He concluded that cable restrainers together with shock absorber devices could be effective in reducing the relative opening displacements and impact forces due to pounding at expansion joint. However, there is no literature available regarding to the roller bearing together with cable restrainers to mitigate curved viaducts damage during earthquakes.

In addition, due to its large resistance force and energy dissipation capacity, viscous dampers has gained significant attention in recent years. Ruangrassamee and Kawashima [24] investigate the effectiveness of variable dampers in improving the seismic response including pounding effect. It is found that the friction-type damping force scheme gives the largest reduction of deck displacement, relative displacement between two decks, and the flexural hysteretic curvature at the plastic hinge of a pier. In 1994, Kawashima and Unjoh [23] presents a variable damper proposed to control the response of bridges against earthquakes. The feasibility of the variable damper is shown through an analytical simulation of a bridge. A prototype variable damper is developed and a series of shake-table tests are performed. However, the effectiveness of roller bearing together with viscous damper on reducing curved viaducts damage has not been done.

Pounding between superstructures bring damage not only at contact face, but also at piers bases due to transfer forces from pounding. In order to reduce the negative pounding effects, some researchers set shock absorber devices between adjacent viaduct segments. In 2000, Kawashima [45] study on the effect of shock absorber device on deck's acceleration, deck's displacement and impact forces between superstructures. Zhu, Abe and Fujino [46], in 2004, set shock absorber device between roller adjacent superstructures to reduce viaducts damage due to negative effect of pounding. The pounding force and acceleration of superstructures could be significantly reduced. In 2009, Shehata [17] set rubber shock absorber devices between adjacent superstructures combined with cable restrainers to mitigate the pounding forces. Besides, the relative displacement between superstructures could also be reduced. But the effectiveness of shock absorber devices installed between roller bearing and stopper on reducing pounding damage, have not been done.

1.3 Object and Scope of Study

The main objective of this thesis is to find appropriate method or technology to improve the seismic behavior of viaduct with steel bearings during earthquakes.

In order to accomplish the main objective, three progressive stages are carried out. The first stage is developing a three-dimensional model of the viaduct, which plenty represents the essential mechanical and structural properties of real bridge structures. The second stage is finding some parameters to evaluate the viaduct damage, which represents the falling down of deck, tangential and radial joint residual opening damage, pounding damage and pier damage. The third stage consists in many technologies to improve seismic behavior of viaduct during earthquakes.

Stopper equipped at both sides of roller bearing can prevent large displacement of roller bearing and pounding damage at expansion joint. The effect of stopper value on the damage of viaduct with cable restrainer is carried out. Based on the relationship of stopper value and viaducts damages, appropriate stopper value could be put forward for the seismic design.

Cable restrainer can be used to reduce the possibility of the falling down of deck by limiting large relative opening displacement between adjacent superstructures. However, stiffness and slack value are two important parameters of cable restrainer, which play an important role in the behavior of cable restrainers. The effects of the stiffness and slack values of cable restrainers together with roller bearings on viaduct damage are carried out. Therefore, the seismic design of curved viaduct installed of the cable restrainer with appropriate stiffness and slack value can be put forward. In order to evaluate the seismic response accurately, the study combines non-linear dynamic analysis with a three-dimensional bridge model.

Moreover, application of viscous dampers to viaducts could dissipate some input energy during serious earthquakes. So, viscous damper has gained significant attention in recent years. Generally, researchers use lead rubber bearings combined with viscous dampers to reduce viaduct damage. However, the study on roller bearings combined with viscous dampers to reduce damage is lack. In this thesis, the overall performance of highway viaducts with different kinds of viscous dampers together with roller bearing is evaluated during serious earthquakes. The relationship between different kinds of viscous dampers, different stopper values, and curved viaduct damage could be found respectively. According to the relationship, the advice for the seismic design of curved viaduct based on the appropriate stopper values could be put forward.

Finally, since pounding force between roller bearing and stopper is so large that could bring damages at not only roller bearing but also piers bases. Hence, the application of shock absorber device between roller bearing and stopper has get researchers' attentions. Shock absorber device can significantly reduce the negative effect of pounding by smooth changes of impact stiffness. Based on the previous study, the effects of shock absorber devices installed between adjacent viaduct segments on reducing pounding forces are proved by many researchers. However, the study on the effectiveness of shock absorber devices installed between roller bearing and stopper on reducing pounding damage is lack. Moreover, strain hardening, strain softening and elastic shock absorber devices are also taken into account in recent years. Therefore, the study on shock absorber devices installed between roller bearing and stopper together with viscous damper is carried out. The effect of stiffness, thickness and three types of shock absorber devices on curved viaduct damage is presented respectively.

1.4 Organization of the Thesis

The thesis contents are organized in eight chapters, which are further divided into several sections and subsections. The chapters have individual introduction, which give brief

orientations of the subject under investigation. For a detailed discussion of various aspects of the subject, many references have been included in each chapter. The specific objectives of each chapter are as follows:

Chapter 1 provides the general introduction and motivating objectives of this research.

Chapter 2 presents the theoretical approach for a sophisticated three-dimensional analytical program capable of simulating structural behavior in severely inelastic response and accurately predicting seismic response of bridges. The theoretical foundation of the non-linear finite element formulation includes the best features of several previous studies. A three-dimensional element, capable of modeling geometric and material non-linearities, is developed based on the fiber element model. An incremental-iterative method based on the Newmark direct integration method and the Newton-Raphson method is employed for obtaining the solutions of the non-linear dynamic equilibrium equations. Finally, the energetic analysis formulation of the problem is also presented.

Chapter 3 presents the analytical model of the curved viaduct in this study. Three dimensional (3D) analytical model is adopted in this analysis. Cross-sectional properties of the nonlinear elements are prescribed using fiber elements which are considered as the most promising models for nonlinear analysis of structural members. For roller bearings, both Coulomb frictional force and the stopper effect are accounted for. The pounding phenomenon between superstructures is modeled using impact spring elements. In addition, the classification method of viaduct damage is also presented in this chapter.

Chapter 4 evaluates the effect of stopper value on the damage of viaduct equipped with cable restrainer. Stopper values are divided into three cases. They are respectively less than slack value of cable restrainer (4 cm), larger than slack value of cable restrainer (4 cm) and less than the gap of expansion joint (10 cm), larger than the gap of expansion joint (10 cm). According to the relationship between stopper value and viaduct damages, the advice for the seismic design of viaduct based on the appropriate stopper values can be put forward.

Chapter 5 presents the effects of stiffness and slack value of cable restrainer on nonlinear seismic response of curved highway viaducts with roller bearings. The effects of stiffness and slack value of cable restrainer on deck unseating damage, tangential and radial joint residual opening damage, pier damage, and relationship of bending moment and curvature at pier base have been evaluated respectively. In addition, a comparison between restrained and unrestrained curved viaduct is also presented in this chapter.

Chapter 6 respectively presents the overall performance of highway viaducts with different kinds of viscous dampers together with different stopper values. Application of viscous dampers and stopper on the curved viaduct plays important role in viaduct damages. The first stage is study the effect of different types of viscous dampers on the damage of curved viaduct with a constant stopper value. The second stage consists in the effect of different stopper values on the damage of curved viaduct with a constant viscous damper. Additionally, a comparison between curved viaduct equipped with viscous dampers and without viscous dampers is presented. According to the calculation results, the advice based on the appropriate stopper value together with viscous damper could be proposed.

In **Chapter 7**, the effect of shock absorber devices between roller bearing and stopper on mitigating the damage of curved viaduct with viscous damper by smooth changes of impact stiffness is carried out. The stiffness and thickness of shock absorber are very important parameters. Therefore, the effect of stiffness and thickness of shock absorber devices on mitigating viaduct damage is evaluated. In addition, the influence of three types of shock absorber devices including strain hardening, elastic and strain softening on curved viaduct damage is also carried out during serious earthquakes.

Chapter 8 presents the significant finds and conclusions in this research. The advice for the seismic design of curved viaduct based on the finds and conclusions can be put forward.

References

- [1] Hashimoto, S., Abe, M., and Fujino, Y.: Damage analysis of Hanshin Expressway viaducts during 1995 Kobe earthquake. III: Three-span continuous girder bridges, *Journal of Bridge Engineering*, ASCE, Vol. 10, No. 1, pp. 61-68, 2005.
- [2] Rose, A., and Lim, D.: Business interruption losses from natural hazards: conceptual and methodological issues in the case of the Northridge earthquake, *Environmental Hazards*, Vol. 4, pp. 1-14, 2002.
- [3] Itani, A. M., Bruneau, M., Carden, L., and Buckle, I. G.: Seismic behavior of steel girder bridge superstructures, *Journal of Bridge Engineering*, ASCE, Vol. 9, No. 3, pp. 243-249, 2004.
- [4] Ruiz Julian, F. D., Hayashikawa, T., Obata, T.: Seismic performance of isolated curved steel viaducts equipped with deck unseating prevention cable restrainers, *Journal of Constructional Steel Research*, Vol. 63, No. 2, pp. 237-253, 2007.
- [5] Hayashikawa, T., Hirooka, T., Ikeda, K., Abdel Raheem, S. E.: Dynamic behaviour of viaducts with PC cables in consideration of pounding of girders and energy absorption, *JSCE Proceedings of Hokkaido*, Vol. 59, Japan Soc Civil Eng, pp. 50-53, 2003.
- [6] Ruangrassamee, A., Kawashima, K.: Relative displacement response spectra with pounding effect, *Earthquake Engineering & Structural Dynamics*, Vol. 30, No. 10, pp. 1511-1538, 2001.
- [7] Jankowski, R., Wilde, K., Fujino, Y.: Reduction of pounding effects in elevated bridges during earthquakes, *Earthquake Engineering & Structural Dynamics*, Vol. 29, No. 2, pp. 195-212, 2000.
- [8] Ruiz Julian, F. D., Hayashikawa, T., Obata, T.: Seismic performance of isolated curved steel viaducts equipped with deck unseating prevention cable restrainers, *Journal of Constructional Steel Research*, Vol. 63, No. 2, pp. 237-253, 2007.
- [9] Zanardo, G., Hao, H., Modena, C.: Pounding effects in multi-span simply supported bridges induced by spatially varying earthquake ground motions, *Proceedings of the 7th International Seminar on Seismic Isolation, Passive Energy Dissipation and Active Control of Seismic Vibrations of Structures*, Assisi, Italy, 2001.
- [10] DesRoches, R., Muthukumar, S.: Effect of pounding and restrainers on the seismic response of multi-frame bridges, *ASCE Journal of Structural Engineering*, Vol. 128, No. 7, pp. 1-10, 2002.
- [11] Taflanidis, A.A.: Optimal probabilistic design of seismic dampers for the protection of isolated bridges against near-fault seismic excitations, *Engineering Structures*, Vol. 33, No. 12, pp. 3496-3508, 2011.
- [12] Zanardo, G., Hao, H., Modena, C.: Seismic response of multi-span simply supported bridges to a spatially varying earthquake ground motion, *Earthquake Engineering & Structural Dynamics*, Vol. 31, No. 6, pp. 1325-1345, 2002.
- [13] Watanabe, E., Sugiura, K., Nagata, K., Kitane, Y.: Performances and damages to steel structures during the 1995 Hyogoken-Nanbu earthquake, *Engineering Structures*, Vol. 20, No. 4-6, pp. 282-90, 1998.
- [14] Scawthorn, C., Yanev, P. I.: Preliminary report 17 January 1995, Hyogo-ken Nambu, Japanese earthquake, *Engineering Structures*, Vol. 17, No. 3, pp. 146-57, 1995.
- [15] Kawashima, K., Unjoh, S.: Impact of Hanshin/Awaji earthquake on seismic design and seismic strengthening of highway bridges, *Structural Engineering/Earthquake Engineering*, JSCE, Vol. 13, No. 2, pp. 211s-240s, 1996.
- [16] Otsuka, H., Unjoh, S., Terayama, T., Hoshikuma, J., Kosa, K.: Damage to highway bridges by the 1995 Hyogoken Nanbu earthquake and the retrofit of highway bridges in Japan, In: *Proceedings of the 3rd U.S.-Japan Workshop on Seismic Retrofit of Bridges*, Osaka, Japan, pp. 10-11, 1996.
- [17] Abdel Raheem, S. E.: Pounding mitigation and unseating prevention at expansion joints of isolated multi-span bridges, *Engineering Structures*, Vol. 31, No. 10, pp. 2345-2356, 2009.
- [18] Zhu, P., Abe, M., Fujino, Y.: Evaluation of pounding countermeasures and serviceability of elevated bridges during seismic excitation using 3D modeling, *Earthquake Engineering & Structural Dynamics*, Vol. 33, No. 5, pp. 591-609, 2004.
- [19] Ruiz Julian, D., and Hayashikawa, T.: Effects of restrainers on 3D nonlinear seismic response of simply supported skewed highway viaducts, *Journal of Constructional Steel*, JSSC, Vol. 12, pp. 55-62, 2004.
- [20] Kawashima, K., Unjoh, J.: Development of hybrid control technology, *Civil Engineering Journal*, Vol.32, No.6, pp. 2-5, 1989.
- [21] Kawashima, K., Unjoh, S., Nagashima, H., Shimizu, H.: Current research efforts in Japan for passive and active control of highway bridges against earthquakes, *Proceedings of the 23rd Joint Meeting, US-Japan Panel on Wind and Seismic Effect*, Tsukuba, Japan, pp. 187-209, 1991.
- [22] Kawashima, K., Unjoh, S., Shimizu, K.: Experiments on dynamic characteristics of variable damper, *Proceedings of Japan National Symposium on Structural Response Control*, pp.121-128, 1992.
- [23] Kawashima, K., Unjoh, S.: Seismic response control of bridges by variable dampers, *Journal of Structural Engineering*, ASCE, Vol.120, No.9, pp. 2583-2601, 1994.

- [24] Ruangrassamee, A., Kawashima, K.: Control of nonlinear bridge response with pounding effect by variable dampers, *Engineering Structures*, Vol.25, No.5, pp. 593-606, 2003.
- [25] Williams, D., and Godden, W. G.: Experimental model studies on the seismic response of high curved overcrossings, *Report No. 76/18 UCB/EERC*, Earthquake Engineering Research Center, University of California at Berkeley, 1976.
- [26] Buckle, I.G.: Field tests of a curved box girder bridge using simulated earthquake loads, *Proceedings of the 8th European Conference on Earthquake Engineering*, Lisbon, Portugal, Vol. 4, pp. 1-8, 1986.
- [27] Abdel-Salam, M. N. and Heins, C. P.: Seismic response of curved steel box girder bridges, *Journal of Structural Engineering*, Vol. 114, pp. 2790-2800, 1988.
- [28] Chen, Y.: Effects of modeling and analysis methods on seismic responses of curved bridges, *Engineering Mechanics: Proceedings of the 10th Conference*, Colorado, USA, Vol. 2, pp. 1078-1081, 1995.
- [29] DesRoches, R., and Fenves, G. L.: Evaluation of recorded earthquake response of a curved highway bridge, *Earthquake Spectra*, Vol. 3, No. 3, pp. 363-386, 1997.
- [30] Hayashikawa, T., Otake, A., and Nakajima, A.: Nonlinear behavior of curved viaducts subjected to three-dimensional earthquake ground motions, Paper No. G1-20, *The 10th Earthquake Engineering Symposium Proceedings*, Architectural Institute of Japan, Tokyo, Japan, Vol. 2, pp. 2391-2396, 1998.
- [31] Ates, S., Constantinou, M. C.: Example of application of response history analysis for seismically isolated curved bridges on drilled shaft with springs representing soil, *Engineering Structures*, Vol. 31, No. 3, pp. 334-350, 2011.
- [32] Seo, J., Linzell, D. G., Horizontally curved steel bridge seismic vulnerability assessment, *Engineering Structures*, Vol. 34, pp. 21-32, 2012.
- [33] Seo, J., Linzell, D. G.: Nonlinear Seismic Response and Parametric Examination of Horizontally Curved Steel Bridges Using 3D Computational Models, *Journal of Bridge Engineering*, ASCE, Vol. 18, No. 3, pp. 220-231, 2013.
- [34] Wilson, T., Mahamoud, H., Chen, S.: Seismic performance of skewed and curved reinforced concrete bridges in mountainous states, *Engineering Structures*, Vol. 70, No. 1, pp. 158-167, 2014.
- [35] Abdelnaby, A., Frankie, T., Elnashai, A. S., Spencer, B. F., Kuchma, D. A. et al.: Numerical and hybrid analysis of a curved bridge and methods of numerical model calibration, *Engineering Structures*, Vol. 70, No. 1, pp. 234-245, 2014.
- [36] DesRoches, R., Pfeifer, T., Leon, R. T., and Lam, T.: Full-scale tests of seismic cable restrainer retrofits for simply supported bridges, *Journal of Bridge Engineering*, ASCE, Vol. 8, No. 4, pp. 191-198, 2003.
- [37] Saiidi, M., Randall, M., Maragakis, E., and Isakovic, T.: Seismic restrainer design methods for simply supported bridges, *Journal of Bridge Engineering*, ASCE, Vol. 6, No. 5, pp. 307-315, 2001.
- [38] Yashinsky, M.: Chapter 30: Earthquake damage to structures. In: Chen, W. F., editor. *Structural Engineering Handbook*, Boca Raton, CRC Press, 1999.
- [39] Duan, L., and Chen, W. F.: Chapter 18: Bridges. In: Chen, W. F., and Scawthorn, C., editors. *Earthquake Engineering Handbook*, Boca Raton, CRC Press, 1999.
- [40] Kawashima, K., and Shoji, G.: Effect of restrainers to mitigate pounding between adjacent decks subjected to a strong ground motion, *12th World Conference on Earthquake Engineering*, Paper No. 1435, 2000.
- [41] Watanabe, G., and Kawashima, K.: Effectiveness of cable-restrainer for mitigating rotation of a skewed bridge subjected to strong ground shaking, *13th World Conference on Earthquake Engineering*, Paper No. 789, Vancouver, Canada, August 1-6 2004.
- [42] Kim, J. M., Feng, M. Q., and Shinozuka, M.: Energy dissipating restrainers for highway bridges, *Soil Dynamics and Earthquake Engineering*, Vol. 19, No. X, pp. 65-69, 2000.
- [43] Hayashikawa, T., and Ruiz Julian, D.: Parametric study on design of cable restrainers for curved highway viaducts under Level II earthquakes, *Proceedings of Hokkaido Chapter of the Japan Society of Civil Engineers*, JSCE, Vol. 61, CD ROM, I-12, 2005.
- [44] Ruiz Julian, F. D., and Hayashikawa, T.: Study on nonlinear seismic response of curved highway viaducts with different cable restrainers, *Journal of Structural Engineering JSCE*, Vol. 51A, pp. 701-712, 2005.
- [45] Kawashima, K., Shoji, G.: Effect of restrainers to mitigate pounding between adjacent decks subjected to a strong ground motion, *12th World Conference on Earthquake Engineering-12WCEE*, Paper no. 1435, 2000.
- [46] Zhu, P., Abe, M., Fujino, Y.: Evaluation of pounding countermeasures and serviceability of elevated bridges during seismic excitation using 3D modeling, *Earthquake Engineering & Structural Dynamics*, Vol. 33, No. 5, pp. 591-609, 2004.

NONLINEAR ANALYTICAL APPROACH

2.1 Introduction

Recently, many bridges are damaged in serious earthquakes. So how to evaluate bridge dynamic behavior becomes very important. Common seismic design philosophies for bridges allow some degree of damage without collapse. To control and evaluate damage, a post-elastic non-linear analysis is required [1, 2]. Inelastic dynamic analysis provides a realistic measure of structural response because the inelastic model accounts for the redistribution of internal actions due to the non-linear force displacement behavior of the components. A non-linear inelastic dynamic analysis is not difficult in concept, but it requires careful structural modeling and intensive computing effort. During the process to ensure the safeness, inelastic dynamic analysis is commonly used for the safety evaluation of bridges to determine their inelastic response when subjected to strong earthquake ground motions. Actually, the three-dimensional frame modeling is considered to simulate the seismic behavior of bridge structures. In the analytical model, the $M-\phi$ relation for non-linearities at the member level is used to define the real behavior of material non-linear. However, shortcomings have been pointed out in the analysis of frame bridges in which a change in an axial force occurs, as use the $M-\phi$ model. On the other hand, the fiber model has an advantage in that the effects of a change in the axial force bi-directional bending are automatically considered. Therefore, a remarkable technique for bridge seismic analysis based on a fiber model can be successfully established. In spite of this, it seems that few results of the dynamic analysis using this technique are available because its application to the actual practice, where the dynamic analysis using a constant energy law is not so complicated, is limited. Additionally, linear analysis tends to underestimate structural responses and overestimate their strengths. Therefore, the appropriate dynamic analysis, including geometrical non-linearities, becomes necessary in this analysis. Actually, the behavior of highway viaducts under serious earthquake excitation should consider the material non-linearity together with the geometric non-linearity [3, 4].

In recent years, because there is great improvement in computer, which can save the amount of computing time, the finite element method is recognized as an accurate and versatile powerful numerical tool for bridge analysis. Dynamic analysis of bridges is complex and consequently, time required to obtain the numerical solution is considerable. In addition, a large number of input and output data also needs to consider the effect of time consume. However, the recent advances in computer and modeling techniques have significantly reduced the amount of computing time, and analysis of complex structural phenomena in bridges can be successfully carried out [5 - 9]. This chapter presents an efficient procedure for the non-linear elastoplastic dynamic analysis under strong seismic excitation, considering both geometric non-linearity and material non-linearity. In the analysis, the non-linear space beam elements can be used to simulate structural components. The element has two nodes with six degrees of freedom per node, and can transmit an axial force, two shear forces, two bending moments and a torque. In the analysis, the total Lagrangian formulation is adopted considering all coupling between bending, twisting and stretching deformations for beam element by the geometrical non-linear beam theory. In addition, the kinematic hardening model, uniaxial yield criterion flow relations are used to describe the behavior of steel material non-linearity. For inelastic behavior, the spread of plasticity can be achieved through portions of the member cross section in addition to the ability to model zones of plasticity along the

member length utilizing the member segments. Because the fiber model allows modeling of strain hardening, axial-flexural interaction, residual stresses and gradual spread of yielding within the cross section, it is a more realistic model than the more common plastic hinge beam-column model. The fiber model used in this study is physically motivated from actual uniaxial test data of structural steel bars. Explicit expressions for stress in terms of strain also give the instantaneous stiffness, making the model ideal for iterative non-linear analysis. For compression and tension, the stress-strain relation is the same. Residual stresses shift the curve along the strain axis until the residual stress level is located at zero strain. Therefore, adjacent fibers can have different levels of residual stress. The stress pattern is defined in such a way that equilibrium is maintained, but the maximum residual stress can be chosen. Based on the tangent stiffness operator, the effects of both the geometrical variations and material non-linearity are accounted for. In order to solve the non-linear dynamic equilibrium equations, an incremental-iterative method is employed in this analysis, which is based on the Newmark direct integration method and the Newton-Raphson iteration scheme.

2.2 Natural Vibration Analysis Formulation

Natural vibration characteristics of viaducts play an important role in the prediction of their structural performance during strong earthquakes. So the calculation of natural vibration characteristics of viaducts is necessary in the analysis. It is fundamental to calculate the natural frequencies and mode shapes of a bridge structure for the solution of its dynamic responses due to seismic loads. The mathematical idealization for three-dimensional free vibrations is discussed in this section taken into account that the highway viaduct considered in the analysis is composed of thin-walled beam members. Based on the finite element approach, free vibration of three-dimensional framework structures is analyzed by using the consistent mass matrix.

In order to calculate natural vibration of the viaduct, two different basic types of coordinate systems of structures are considered: distributed coordinate systems and discrete systems [10]. The first type is the distributed-parameter system, which is applied to structures whose physical properties are continuously distributed in three-dimensional space. In this case, the basic relation between forces and displacements for a beam segment subjected to axial, flexural and torsional vibration is formulated by general solutions of differential equations of motion. The above general solutions lead to a dynamic stiffness matrix, which includes natural circular frequencies of vibrating structures. This approach results in exact solutions of vibration and is called the continuous mass method or the eigen stiffness matrix method. The second type is the lumped-parameter system, which defines forces and displacements at a set of discrete nodal points in terms of components having specified directions. The lumping procedure is most effective in treating systems in which a large proportion of the total mass actually is concentrated at a few discrete points. The analytical procedure for this type can be greatly simplified as an eigenvalue problem because the inertia forces are developed only at these points. In this case, it is necessary to define the displacements and accelerations only at these discrete locations. In the space framework structures, the lumped mass matrix is derived as a diagonal matrix by applying half the mass and its associated mass-moment of each beam segment to the appropriate nodal point. Moreover, the influence coefficients are evaluated by a procedure similar to that used to determine the static stiffness coefficients. The resulting matrix is called the consistent mass matrix, and it contains many off-diagonal terms due to the effect of mass coupling. The choice of the discrete or distributed coordinate systems depends upon the geometry of the bridge structure, the location and nature of constraints, the distribution of the applied forces and the necessary information concerning deflections.

$$\{f\} = [k]\{d\} \quad (2.1)$$
$$\{\mathbf{f}\} = \{X_i \ Y_i \ Z_i \ M_{xi} \ M_{yi} \ M_{zi} \ X_j \ Y_j \ Z_j \ M_{xj} \ M_{yj} \ M_{zj}\}^T \quad (2.2)$$

For a three-dimensional frame member the stability function-based stiffness matrix has the following form:

11

where I_y and I_z are the cross section moments of inertia with respect to the principal axes labeled as y and z , respectively, and l , A and J are the length, cross-sectional area and torsional constant of the beam element, and E is the Young's modulus. In the same manner, the consistent mass matrix type is selected for uniform beam segment of a three dimensional frame, and their components are expressed by combining the consistent mass matrices for axial, torsional and flexural mass effects neglecting the rotary inertia effects as follows:

$$[m] = \frac{\bar{m}l}{420} \begin{bmatrix} 140 & 0 & 0 & 0 & 0 & 0 & 70 & 0 & 0 & 0 & 0 & 0 \\ & 156 & 0 & 0 & 0 & 22l & 0 & 54 & 0 & 0 & 0 & -13l \\ & & 156 & 0 & -22l & 0 & 0 & 0 & 54 & 0 & 13l & 0 \\ & & & 140J/A & 0 & 0 & 0 & 0 & 0 & 70J/A & 0 & 0 \\ & & & & 4l^2 & 0 & 0 & 0 & -13l & 0 & -3l^2 & 0 \\ & & & & & 4l^2 & 0 & 13l & 0 & 0 & 0 & -3l^2 \\ & & & & & & 140 & 0 & 0 & 0 & 0 & 0 \\ & & & & & & & 156 & 0 & 0 & 0 & -22l \\ & & & & & & & & 156 & 0 & 22l & 0 \\ & & & & & & & & & 140J/A & 0 & 0 \\ & & & & & & & & & & 4l^2 & 0 \\ & & & & & & & & & & & 4l^2 \end{bmatrix} \quad (2.5)$$

The formulation of the problem implies that the motion of the bridge structure subjected to earthquake loading is assumed to be described by a set of N simultaneous second order linear differential equations in the time domain given by,

$$[M]\{\ddot{d}\} + [C]\{\dot{d}\} + [K]\{d\} = -[M]\{r\}\{\ddot{d}_g\} \quad (2.6)$$

where $[M]$, $[C]$ and $[K]$ are respectively the system mass, damping and stiffness matrices, obtained by assembling the mass, damping and stiffness element matrices presented in Eqs. 2.4 and 2.5. While $\{\ddot{d}\}$, $\{\dot{d}\}$ and $\{d\}$ are the acceleration, velocity and displacement vectors, respectively, $\{r\}$ is a displacement transformation vector that expresses the displacement of each structure degree of freedom due to static application of a unit support displacement, $\{\ddot{d}_g\}$ is the earthquake ground acceleration history applied at the supports. In order to determine the frequencies of the structure, eigenvalue analysis is performed. The free vibrations of an undamped system is described by,

$$[M]\{\ddot{d}\} + [K]\{d\} = \{0\} \quad (2.7)$$

By assuming general solutions in form of harmonic oscillation ($\ddot{d} = -\omega^2 d$), where ω is the natural circular frequency. Then, the state of free vibration of a structure is represented by the eigenvalue problem as,

$$|K - \omega^2 M| = 0 \quad (2.8)$$

The computation of eigenvalues in this study is carried out through the Householder's method. For an eigenvalue problem the solution can be expanded in terms of the system eigenfunctions. Indeed, a linear transformation with eigenfunctions as a transformation matrix permits the reduction of system equation to a set of independent equations. The mode shape matrix is n -dimensional vector space, which consists of n linear independent and mutually

orthogonal eigenvectors. The displacement vector can be expressed in terms of any set of n independent vectors.

$$\mathbf{d}(t) = \sum_{r=1}^n \phi_r q_r(t) = \Phi \mathbf{q}(t) \quad (2.9)$$

where Φ is $n \times n$ mode shape matrix, $\mathbf{q}(t) = \{q_1(t) \ q_2(t) \ \dots \ q_n(t)\}^T$ is the n -dimensional vector of modal coordinates. For the foundation excitation the time variation and spatial distribution is defined by $\{\ddot{d}_g\}$ and $s = -\mathbf{m} \mathbf{r}$, respectively. The excitation distribution can be expressed as:

$$s = -\sum_{r=1}^n m_r r = -\sum_{r=1}^n \Gamma_r m \phi_r \quad (2.10)$$

because of the orthogonality relation, premultiplying each term in this equation by ϕ_n^T , gives

$$\Gamma_n = \frac{\phi_n^T \mathbf{m} \mathbf{r}}{\phi_n^T \mathbf{m} \phi_n}, \quad s_n = \Gamma_n \mathbf{m} \phi_n \quad (2.11)$$

the s_n denotes the contribution of n th mode to the excitation vector $\mathbf{s}$, and the Γ_n is defined as modal participation factor, implying that it is a measure of the degree to which the n th mode participates in the response. The effective modal mass M_n^* that represent the part of the total mass responding to the earthquake in each mode, can calculated as:

$$M_n^* = \frac{(\sum_{j=1}^n m_j \phi_{jn})^2}{\sum_{j=1}^n m_j \phi_{jn}^2} \quad (2.12)$$

2.3 Three-dimensional Fiber Beam Element Formulation

In order to analyze the elasto-plastic dynamic response of frame structures subjected to strong seismic excitations, a non-linear dynamic finite element technique has been developed. An elastoplastic tangent stiffness matrix of three-dimensional beam element is derived by using the total Lagrangian formulation. This procedure considers both sources of geometric and material non-linearities. An incremental-iterative technique based on the Newmark direct integration method and the Newton-Raphson iteration scheme is employed for obtaining the solutions of the non-linear dynamic equilibrium equations. The finite displacement theory, which reflects the deformed configuration of a body, is composed of a strain-displacement relation, an equilibrium condition and stress-strain relation as well. Since the theory must discern a deformed state from the initial configuration, time- or displacement-dependent variables, necessarily expresses any point of a body. In solid mechanics, an initial configuration of a body is usually given, so it is chosen as a reference state with an affiliated coordinate system. This is called the Lagrangian approach, in which the coordinates of a body at the reference state are chosen as independent variables and solutions such as strains, stresses or displacements are expressed as functions of those variables. If the displacements under consideration are excessively large, the change of the geometrical shape cannot be disregarded in the derivation of the governing equations.

2.3.1 Strain displacement relations

In general, the strain displacement is geometrical relation irrespective of external forces and material used, and there are two methods of describing the deformations of a continuous body. The Lagrangian and the Eulerian, the Lagrangian method employs the coordinates of each particle in the initial position as the independent variables, where as in the Eulerian method the independent variables are the coordinate of each material particle at the time of interest or at the present position. In the Lagrangian method, the initial configuration remains constant. This simplifies the computation. Therefore, the Lagrangian description is selected for the finite element formulation. Recently, a formulation of fiber model has been established. A three-dimensional beam element of six degrees of freedom at each node, in which all coupling among bending; twisting and stretching deformations for beam element is incorporated according to the geometrical non-linear beam theory, has been considered. The deflection of a general point $(x, y$ and $z)$ inside the beam can be expressed as,

$$\bar{u}(x, y, z) = u(x) - y \frac{dv(x)}{dx} - z \frac{dw(x)}{dx} \quad (2.13a)$$

$$\bar{v}(x, y, z) = v(x) \quad (2.13b)$$

$$\bar{w}(x, y, z) = w(x) \quad (2.13c)$$

where $\bar{u}$, $\bar{v}$ and $\bar{w}$ are the displacements of a point in a beam corresponding to x -, y - and z -axes, respectively, u , v and w are the displacements of a point in the centroid axis of a beam corresponding to x -, y - and z - axes, respectively. The strain in the deformed configuration ε_t can be expressed in terms of the displacement at the equilibrium state at any specified point in the beam cross-section $(x, y$ and $z)$ that define the state of strain at any arbitrary point. The strain displacement equation (Green-Lagrange representation of axial strain ε_t) is given in the following:

$$\varepsilon_t = \frac{d\bar{u}_t}{dx} + \frac{1}{2} \left(\frac{d\bar{u}_t}{dx} \right)^2 + \frac{1}{2} \left(\frac{d\bar{v}_t}{dx} \right)^2 + \frac{1}{2} \left(\frac{d\bar{w}_t}{dx} \right)^2 \quad (2.14)$$

in which it is assumed that $\frac{d\bar{u}_t}{dx} \ll 1$. Thus, $\left(\frac{d\bar{u}_t}{dx} \right)^2$ is ignored compared with its linear term.

So the normal strain ε_t is calculated using the displacement to give:

$$\varepsilon_t = \frac{d\bar{u}_t}{dx} + \frac{1}{2} \left(\frac{d\bar{v}_t}{dx} \right)^2 + \frac{1}{2} \left(\frac{d\bar{w}_t}{dx} \right)^2 \quad (2.15)$$

where $\bar{u}_t$, $\bar{v}_t$ and $\bar{w}_t$ are the element displacement vector in x -, y - and z -axis directions, respectively. The strain in $t+1$ configuration is unknown, which can be written incrementally from configuration t as follows:

$$\varepsilon_{t+1} = \frac{d\bar{u}_{t+1}}{dx} + \frac{1}{2} \left(\frac{d\bar{v}_{t+1}}{dx} \right)^2 + \frac{1}{2} \left(\frac{d\bar{w}_{t+1}}{dx} \right)^2 \quad (2.16a)$$

and its variation is,

$$\delta \varepsilon_{t+1} = \frac{d\bar{u}}{dx} + \frac{d\bar{v}_t}{dx} \cdot \frac{d\delta\bar{v}}{dx} + \frac{d\bar{w}_t}{dx} \cdot \frac{d\delta\bar{w}}{dx} + \frac{1}{2} \delta \left(\frac{d\bar{v}}{dx} \right)^2 + \frac{1}{2} \delta \left(\frac{d\bar{w}}{dx} \right)^2 \quad (2.16b)$$

2.3.2 Equilibrium condition

For the principal of energy, that the external work is equivalent to the internal work, the equilibrium equation in the deformed configuration $t+1$ can be expressed terms of the principle of virtual displacements

$$\delta U - \delta W = 0 \quad (2.17)$$

$$\int_V \sigma_{t+1} \delta \varepsilon_{t+1} dV + \int_l T_{sv} \frac{d\theta_x}{dx} dx + \frac{1}{2} \int_l GJ \left(\frac{d\theta_x}{dx} \right)^2 dx - \int_s f_{t+1}^T \delta d_{t+1} dS = 0 \quad (2.18)$$

where U = internal work, W = external virtual work, σ_{t+1} = axial Cauchy stress, V = volume, δ = virtual quantity, f = external applied force and θ_x is the cross-sectional rotation about x axis. Since $t+1$ configuration is unknown, which can be written incrementally from configuration t consistent with a non-linear Lagrangian scheme as:

$$\int_V (\sigma_t + \sigma) \delta \varepsilon_{t+1} dV + \int_l T_{sv} \frac{d\theta_x}{dx} dx + \frac{1}{2} \int_l GJ \left(\frac{d\theta_x}{dx} \right)^2 dx - \int_s (f_t^T + f^T) \delta (d_t + d) dS = 0 \quad (2.19)$$

where σ = second Piola-Kirchhoff axial stress

$$\begin{aligned} \int_V (\sigma_t + \sigma) (d\delta u / dx + dv_t / dx \cdot d\delta v / dx + dw_t / dx \cdot d\delta w / dx + \delta (dv / dx)^2 / 2 + \delta (dw / dx)^2 / 2) dV \\ + \int_l T_{sv} \frac{d\theta_x}{dx} dx + \frac{1}{2} \int_l GJ \left(\frac{d\theta_x}{dx} \right)^2 dx - \int_s (f_t^T + f^T) \delta (d_t + d) dS = 0 \end{aligned} \quad (2.20)$$

$$\begin{aligned} \int_V \{ \sigma_t (d\delta u / dx + dv_t / dx \cdot d\delta v / dx + dw_t / dx \cdot d\delta w / dx) + \sigma_t \delta (dv / dx)^2 / 2 + \sigma_t \delta (dw / dx)^2 / 2 \} + \int_l T_{sv} \frac{d\theta_x}{dx} dx \\ + \frac{1}{2} \int_l GJ \left(\frac{d\theta_x}{dx} \right)^2 dx + \sigma (d\delta u / dx + dv_t / dx \cdot d\delta v / dx + dw_t / dx \cdot d\delta w / dx) dV - \int_s (f_t^T + f^T) \delta (d_t + d) dS = 0 \end{aligned} \quad (2.21)$$

$$\varepsilon^* = du / dx + dv_t / dx \cdot dv / dx + dw_t / dx \cdot dw / dx \quad (2.22)$$

$$\int_V \{ \sigma \varepsilon^* + \sigma_t \delta (dv / dx) / 2 + \sigma_t \delta (dw / dx) / 2 \} dV + \frac{1}{2} \int_l GJ \left(\frac{d\theta_x}{dx} \right)^2 dx - \int_s f^T \delta d dS = \int_s f_t^T \delta d dS - \int_V \sigma_t \varepsilon^* dV - \int_l T_{sv} \frac{d\theta_x}{dx} dx \quad (2.23)$$

2.3.3 Finite element formulation

The displacement transformation matrix of beam element relates the element internal displacement to the nodal point displacement

$$\mathbf{u}(x, y, z) = \begin{Bmatrix} \bar{u}(x, y, z) \\ \bar{v}(x, y, z) \\ \bar{w}(x, y, z) \\ \bar{\theta}_x(x, y, z) \end{Bmatrix} = \begin{Bmatrix} u(x) - y dv(x) / dx - z dw(x) / dx \\ v(x) \\ w(x) \\ \theta_x(x) \end{Bmatrix} = \mathbf{H}(x, y, z) \boldsymbol{\alpha} = \mathbf{H}(x, y, z) \mathbf{A}^{-1} \mathbf{d} \quad (2.24)$$

where $\mathbf{u}(x, y, z)$ is the displacement at any general point along the beam element, and four nodal point displacements for expressing $\bar{u}(x, y, z)$, $\bar{v}(x, y, z)$, $\bar{w}(x, y, z)$ and $\theta_x(x, y, z)$ are considered. The shape functions are assumed for the local element displacements u , v , w and θ_x in the local coordinate variable x :

The nodal displacement vector is composed of six components, two displacements and a rotational deformation for each nodal point, and they can be expressed with the transformation matrix $\mathbf{A}$, which can be a function of the generalized displacements if geometrical non-linear effects are taken into account. The local-global transformation is expressed

$$\begin{aligned} u &= \alpha_1 + \alpha_2 x \\ v &= \alpha_3 + \alpha_4 x + \alpha_5 x^2 + \alpha_6 x^3 \\ w &= \alpha_7 + \alpha_8 x + \alpha_9 x^2 + \alpha_{10} x^3 \\ \theta_x &= \alpha_{11} + \alpha_{12} x \end{aligned} \quad (2.25)$$

The unknown coefficients $\alpha_1, \alpha_2, \dots, \alpha_{12}$, which are also called the generalized coordinates, will be expressed in terms of the unknown element nodal displacements $\mathbf{d}$.

To satisfy continuity requirements, the interpolation functions must be assumed continuous in displacement and slope on the element boundaries. Thus appropriate nodal quantities are displacements and rotations at each end of the element. Therefore, the $\mathbf{H}$ matrix is

$$\mathbf{H}(x, y, z) = \begin{Bmatrix} 1 & x & 0 & -y & -2xy & -3x^2y & 0 & -z & -2xz & -3x^2z & 0 & 0 \\ 0 & 0 & 1 & x & x^2 & x^3 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 1 & x & x^2 & x^3 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & x \end{Bmatrix} \quad (2.26)$$

$$\boldsymbol{\alpha} = \{\alpha_1 \ \alpha_2 \ \alpha_3 \ \alpha_4 \ \alpha_5 \ \alpha_6 \ \alpha_7 \ \alpha_8 \ \alpha_9 \ \alpha_{10} \ \alpha_{11} \ \alpha_{12}\}^T \quad (2.27)$$

$$\{d\} = [\mathbf{A}]\{\alpha\} \quad (2.28)$$

in which

$$\mathbf{A} = \begin{Bmatrix} 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 1 & l & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & l & l^2 & l^3 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 1 & l & l^2 & l^3 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & l \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 2l & 3l^2 & 0 & 0 \\ 0 & 0 & 0 & 1 & 2l & 3l^2 & 0 & 0 & 0 & 0 & 0 & 0 \end{Bmatrix} \quad (2.29)$$

$$\mathbf{u}(x, y, z) = \mathbf{H}(x, y, z)\mathbf{A}^{-1}\mathbf{d} \quad (2.30)$$

Continuing with expression of Eq. 2.22, strains are rewritten in terms of the derivatives of the shape functions through the matrix $\mathbf{B}$

$$\varepsilon^* = du/dx + dv_t/dx \cdot dv/dx + dw_t/dx \cdot dw/dx = \mathbf{B}_1 \alpha - y \mathbf{B}_2 \alpha - z \mathbf{B}_3 \alpha = (\mathbf{B}_1 - y \mathbf{B}_2 - z \mathbf{B}_3) \alpha \quad (2.31a)$$

$$(dv/dx)^2/2 = (\alpha^T \mathbf{B}_4^T \mathbf{B}_4 \alpha)/2$$

$$(dw/dx)^2/2 = (\alpha^T \mathbf{B}_5^T \mathbf{B}_5 \alpha)/2$$

$$(d\theta_x/dx)^2/2 = (\alpha^T \mathbf{B}_6^T \mathbf{B}_6 \alpha)/2 \quad (2.31b)$$

where

$$\begin{aligned} \mathbf{B}_1 &= \{0 \quad 1 \quad 0 \quad dv_t/dx \quad 2xdv_t/dx \quad 3x^2 dv_t/dx \quad 0 \quad dw_t/dx \quad 2xdw_t/dx \quad 3x^2 dw_t/dx \quad 0 \quad 0\} \\ \mathbf{B}_2 &= \{0 \quad 0 \quad 0 \quad 0 \quad 2 \quad 6x \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0\} \\ \mathbf{B}_3 &= \{0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 2 \quad 6x \quad 0 \quad 0\} \\ \mathbf{B}_4 &= \{0 \quad 0 \quad 0 \quad 1 \quad 2x \quad 3x^2 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0\} \\ \mathbf{B}_5 &= \{0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 1 \quad 2x \quad 3x^2 \quad 0 \quad 0\} \\ \mathbf{B}_6 &= \{0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 1\} \end{aligned} \quad (2.32)$$

Due to the considered non-linearities, the expression of the incremental virtual work is fully non-linear, and it is obtained by substituting in Eq. 2.23. Variation of the internal energy is given as,

$$\begin{aligned} \int_V \{\sigma \varepsilon^* + \sigma_t \delta(dv/dx)/2 + \sigma_t \delta(dw/dx)/2\} dV + \frac{1}{2} \int_I GJ \left(\frac{d\theta_x}{dx}\right)^2 dx \\ = \int_V \{E_T \varepsilon^* \delta \varepsilon^* + \sigma_t \delta(dv/dx)/2 + \sigma_t \delta(dw/dx)/2\} dV + \frac{1}{2} \int_I GJ \left(\frac{d\theta_x}{dx}\right)^2 dx \\ = \int_V E_T \delta \alpha^T (\mathbf{B}_1^T - y \mathbf{B}_2^T - z \mathbf{B}_3^T) (\mathbf{B}_1 - y \mathbf{B}_2 - z \mathbf{B}_3) \alpha dV + \int_V \sigma_t \delta \alpha^T (\mathbf{B}_4^T \mathbf{B}_4 + \mathbf{B}_5^T \mathbf{B}_5) \alpha dV + \int_I GJ \delta \alpha^T (\mathbf{B}_6^T \mathbf{B}_6) \alpha dx \\ = \delta \alpha^T \int_V E_T \{\mathbf{B}_1^T \mathbf{B}_1 - y(\mathbf{B}_2^T \mathbf{B}_1 + \mathbf{B}_1^T \mathbf{B}_2) + y^2 \mathbf{B}_2^T \mathbf{B}_2 - z(\mathbf{B}_3^T \mathbf{B}_1 + \mathbf{B}_1^T \mathbf{B}_3) + z^2 \mathbf{B}_3^T \mathbf{B}_3 + zy(\mathbf{B}_2^T \mathbf{B}_3 + \mathbf{B}_3^T \mathbf{B}_2)\} dV \alpha \\ \quad + \delta \alpha^T \int_V \sigma_t (\mathbf{B}_4^T \mathbf{B}_4 + \mathbf{B}_5^T \mathbf{B}_5) dV \alpha + \int_I GJ \delta \alpha^T (\mathbf{B}_6^T \mathbf{B}_6) \alpha dx \\ = \delta \alpha^T \int \{E_1 \mathbf{B}_1^T \mathbf{B}_1 - E_2 (\mathbf{B}_2^T \mathbf{B}_1 + \mathbf{B}_1^T \mathbf{B}_2) + E_3 \mathbf{B}_2^T \mathbf{B}_2 - E_4 (\mathbf{B}_3^T \mathbf{B}_1 + \mathbf{B}_1^T \mathbf{B}_3) + E_5 \mathbf{B}_3^T \mathbf{B}_3 + E_6 (\mathbf{B}_2^T \mathbf{B}_3 + \mathbf{B}_3^T \mathbf{B}_2)\} \alpha dx \\ \quad + \delta \alpha^T \int_I F_x (\mathbf{B}_4^T \mathbf{B}_4 + \mathbf{B}_5^T \mathbf{B}_5) dx \alpha + \delta \alpha^T \int_I GJ (\mathbf{B}_6^T \mathbf{B}_6) \alpha dx \end{aligned} \quad (2.33)$$

whereas the work done by the external forces results in

$$\begin{aligned} \int_V \sigma_t \varepsilon^* dV + \int_I T_{sv} \frac{d\theta_x}{dx} dx = \int_V \sigma_t (\mathbf{B}_1 - y \mathbf{B}_2 - z \mathbf{B}_3) \delta \alpha dV + \int_I T_{sv} \mathbf{B}_6 \delta \alpha dx \\ = \delta \alpha^T \int_V \sigma_t (\mathbf{B}_1^T - y \mathbf{B}_2^T - z \mathbf{B}_3^T) dV + \delta \alpha^T \int_I T_{sv} \mathbf{B}_6^T dx = \delta \alpha^T \int (F_x \mathbf{B}_1^T - M_z \mathbf{B}_2^T - M_y \mathbf{B}_3^T + T_{sv} \mathbf{B}_6^T) dx \end{aligned} \quad (2.34)$$

in which,

$$\begin{aligned} E_1 &= E_T \int dydz, \quad E_2 = E_T \int y dydz, \quad E_3 = E_T \int y^2 dydz, \quad E_4 = E_T \int z dydz, \quad E_5 = E_T \int z^2 y dydz, \quad E_6 = E_T \int zy dydz \\ F_x &= \int \sigma_t dydz, \quad M_z = \int \sigma_t y dydz, \quad M_y = \int \sigma_t z dydz \end{aligned} \quad (2.35a)$$

That can be determined through numerical integration over the cross section fiber segments as follow

$$\int_A \sigma_t yz dydz = \int_A \sigma_t yz dA = \sum_{k=1}^n \sigma_{tk} y_k z_k A_k \quad (2.35b)$$

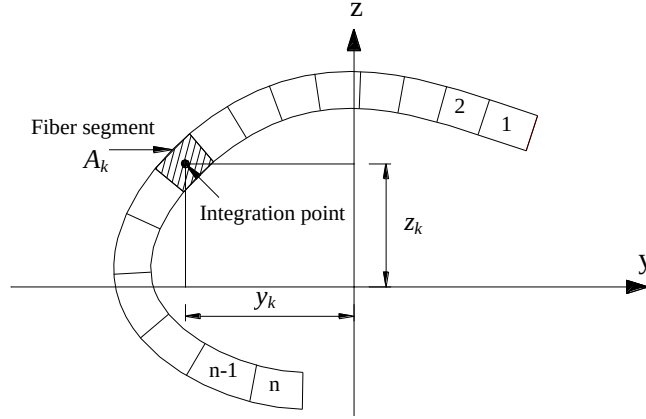


Fig. 2.2 Fiber model of arbitrary cross section

Substitution Eqs. 2.28, 2.33 and 2.34 into Eq. 2.23, the equation of motion can be expressed as

$$\delta \mathbf{d}^T (\mathbf{k}_t \mathbf{d} - \mathbf{f}_t) = \delta \mathbf{d}^T (\mathbf{f} - \mathbf{r}_t) \quad (2.36)$$

from which the tangent stiffness matrix can be obtained as follows,

$$\begin{aligned} \mathbf{k}_t = (\mathbf{A}^{-1})^T \int \{ & E_1 \mathbf{B}_1^T \mathbf{B}_1 - E_2 (\mathbf{B}_2^T \mathbf{B}_1 + \mathbf{B}_1^T \mathbf{B}_2) + E_3 \mathbf{B}_2^T \mathbf{B}_2 - E_4 (\mathbf{B}_3^T \mathbf{B}_1 + \mathbf{B}_1^T \mathbf{B}_3) + E_5 \mathbf{B}_3^T \mathbf{B}_3 \\ & + E_6 (\mathbf{B}_2^T \mathbf{B}_3 + \mathbf{B}_3^T \mathbf{B}_2) + F_x (\mathbf{B}_4^T \mathbf{B}_4 + \mathbf{B}_5^T \mathbf{B}_5) + GJ (\mathbf{B}_6^T \mathbf{B}_6) \} dx \mathbf{A}^{-1} \end{aligned} \quad (2.37)$$

In order to capture the spread of plastic zone in individual element, the beam element is divided along its length and cross section directions, and based on the stress states of integration points in the section, the stiffness quantities of the section are calculated along the cross section as shown in **Fig. 2.2**, then the stiffness quantities of the element are obtained by integrating along the length of the beam, in which the plastic development of the material is automatically considered. The stiffness matrix calculations of the elements are completed by numerical integration procedure. After obtaining the stiffness matrix of the elements with respect to the local coordinate system, transforming them from the element local coordinate system to global coordinate system, then assembling the stiffness matrix of the elements using standard procedures and getting the globe stiffness matrix. During the incremental process, the effects of both the geometrical variations and material non-linearity are accounted for through the tangent stiffness operator.

2.3.4 Solution technique of non-linear equilibrium equation

In using the Newton-Raphson method to the equilibrium iteration, the stiffness matrix $\mathbf{K}$ and equivalent residual force vector have to be updated for each iteration step. The equivalent stiffness matrix can thus be constructed. The non-linear problem is solved by taking a series of linear steps. In structural terms this process is explained by writing equilibrium equations in the incremental form

$$\mathbf{K}_t \mathbf{d} = \mathbf{f}_{t+1} - \mathbf{r}_t \quad (2.38)$$

The non-linear equilibrium equation is solved using the Newton-Raphson method, as shown in **Fig. 2.3**. The equilibrium point is approached by successive approximations to reduce the unbalanced load in iteration. The stiffness or slope of the force-displacement curve is updated at each iteration. The stiffness matrix $\mathbf{k}_n$ is a function of displacements $\mathbf{d}$ because the problem is non-linear. The current tangent stiffness, $\mathbf{k}_n$, is used to compute the next step, $\mathbf{d}$. Then both displacement and stiffness are updated for the next iteration, at the equilibrium state t , supposing that the displacement $\mathbf{d}_t$ that correspond to load $\mathbf{r}_t$ is known. In this way, a load versus displacement curve is approximated by a series of straight line segments.

Considering the load versus displacement function,

$$\mathbf{r}_{t+1} = \mathbf{f}_{t+1} \quad (2.39)$$

and supposing that the displacement $\mathbf{d}_n$ that correspond to the load $\mathbf{r}_n$ is known. The load is now increased to $\mathbf{r}_{t+1}$ and the corresponding displacement $\mathbf{d}_{t+1}$ is sought. A Taylor series expansion of $\mathbf{r}_{t+1}$ about $\mathbf{d}_t$ truncated after the first derivative is

$$\mathbf{r}_{t+1} = \mathbf{r}_t + \frac{\delta \mathbf{r}_t}{\delta \mathbf{d}_t} \mathbf{d} \quad (2.40)$$

where the tangent stiffness could be expressed as

$$\mathbf{K}_t = \frac{\delta \mathbf{r}_t}{\delta \mathbf{d}_t} \quad (2.41)$$

A new displacement $\mathbf{d}$ is seek such that

$$\mathbf{K}_t \mathbf{d} = \mathbf{r}_{t+1} - \mathbf{r}_t \quad (2.42)$$

where $\mathbf{r}_{t+1} - \mathbf{r}_t$ can be interpreted as the difference between the applied load and the resisting force of the structure. Recurrence relations of the form characterize the Newton-Raphson method could be written as follows:

$$\begin{aligned} \mathbf{K}_t^{(i-1)} \mathbf{d}^{(i)} &= \Gamma^{(i-1)} \\ \mathbf{K}_t^{(i)} &= \frac{\delta \mathbf{r}_{t+1}^{(i)}}{\delta \mathbf{d}_{t+1}^{(i)}}; \quad \Gamma^{(i)} = \mathbf{f}_{t+1} - \mathbf{r}_{t+1}^{(i)}; \quad \mathbf{r}_{t+1}^{(0)} = \mathbf{r}_t; \quad \mathbf{f}_{t+1}^{(0)} = \mathbf{f}_t; \quad \mathbf{d}_{t+1}^{(i)} = \mathbf{d}_{t+1}^{(i-1)} + \mathbf{d}^{(i)} \end{aligned} \quad (2.43)$$

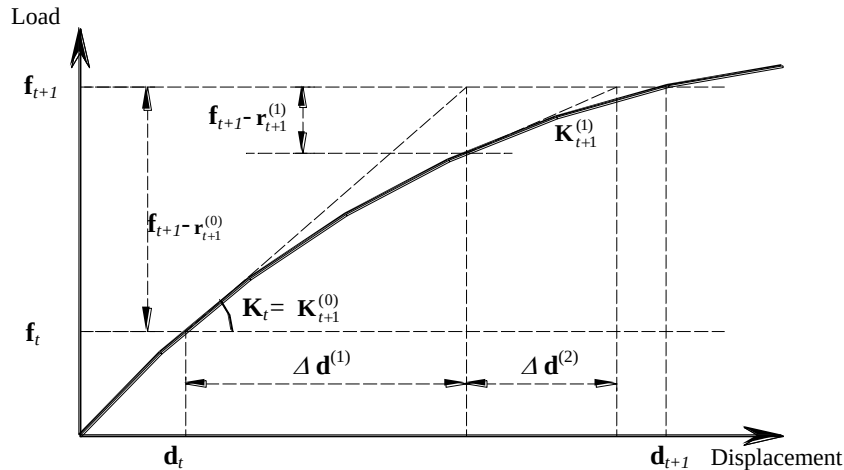


Fig. 2.3 Newton-Raphson integration method

2.4 Non-linear Dynamic Equilibrium Equation of Motion

2.4.1 Dynamic equilibrium equation

The governing non-linear dynamic equation of the viaduct response can be derived by the principle of energy that the external work is absorbed by the work of internal, inertial and damping for any small admissible motion that satisfies compatibility and boundary condition. By assembling the element dynamic equilibrium equation for the time $t+1$ over all the elements, the incremental FEM dynamic equilibrium equation can be obtained as:

$$[M]\{\ddot{d}\}_{t+1} + [C]\{\dot{d}\}_{t+1} + [K]_{t+1}\{\Delta d\}_{t+1} = \{F\}_{t+1} - \{F\}_t \quad (2.44)$$

where $[M]$, $[C]$ and $[K]_{t+1}$ are the system mass, damping and tangent stiffness matrices at time $t+1$. The tangent stiffness matrix takes into consideration the material non-linearities through bilinear stress strain relation for the beam column element, and the geometrical non-linearities for the case of in-plane, out-plane bending deformations and linear torsional deformations. $\{\ddot{d}\}$, $\{\dot{d}\}$ and $\{\Delta d\}$ are the accelerations, velocities and incremental displacements vectors at time $t+1$, respectively, $\{F\}_{t+1} - \{F\}_t$ is the unbalanced force vector. It can be noticed that the dynamic equilibrium equation of motion takes into consideration the different sources of non-linearities both geometrical and material non-linearities, which affect the calculation of the tangent stiffness and internal forces.

2.4.2 Consistent mass matrix

A mass matrix is evaluated with the same shape functions used for the derivation of the stiffness coefficient is called a consistent mass matrix, cubic Hermitian polynomials can be used as the shape functions for evaluating the mass coefficients for the flexural inertial forces, while a linear polynomial may be used for modeling the axial and torsional inertia forces. The mass coefficient m_{ij} as the force at nodal coordinate i due to a unit acceleration at nodal coordinate j while all other coordinate are maintained at zero acceleration. To calculate the coefficient for the consistent mass matrix, it is necessary first to determine the displacement function by solving the governing equation of response motion to a unit displacement at one of the model coordinates. The shape function of axial or torsional deformations is defined to be linear functions as follows:

$$\psi_1(x) = 1 - \xi, \quad \psi_2(x) = \xi \quad \xi = x/l \quad (2.45a)$$

and that of flexural deformations is defined to be cubic Hermitian functions as follows:

$$\begin{aligned} \psi_3(x) &= 1 - 3\xi^2 + 2\xi^3, & \psi_4(x) &= \xi l (1 - \xi)^2 \\ \psi_5(x) &= 3\xi^2 - 2\xi^3, & \psi_6(x) &= \xi l (\xi - 1) \end{aligned} \quad (2.45b)$$

The consistent mass coefficient m_{ij} related to axial and torsional inertial forces of prismatic beam segment, which has distributed mass $\bar{m}$ per unit of length l , could be calculated from

$$m_{ij} = \int_0^l \bar{m} \psi_i(x) \psi_j(x) dx \quad (i, j = 1, 2) \quad (2.46)$$

while the consistent mass coefficient m_{ij} related to flexural deformations includes translational and rotary inertia forces of beam segment, which has mass moment inertia ($\bar{I}_c = \bar{m}I_c / A$, $c = y$ or z axis) per unit of length, and could be calculated from

$$m_{ij} = \int_0^l \bar{m} \psi_i(x) \psi_j(x) dx + \int_0^l \bar{I}_c \psi'_i(x) \psi'_j(x) dx \quad (i, j = 3 - 6) \quad (2.47)$$

2.4.3 Damping mechanism

A correct representation of structural damping cannot be formulated from a practical standpoint. Accordingly, simplified assumptions have to be used. The physical causes of damping in dynamic analysis are any processes, which dissipate energy or reduce the structural response through internal friction. Plastic yield in the non-linear materials automatically absorbs energy when the structure follows a loading and unloading cycle. This is an actual hysteresis effect that produces an accurate form of damping. In this analysis, damping is treated in two categories. First, a phenomenological damping approach is used by considering physical elastic-plastic hysteresis loss in energy dissipation devices. This damping includes the internal hysteresis that occurs in materials such as rubber, friction in bearings, and other non-linear effects such as plastic strain in metals. In the second category, an equivalent viscous damping is introduced, as constant matrix terms, in the system in the form of damping matrix $[C]$. In this study, different spectral damping schemes are used. A simplified damping assumption is used only to clarify vertical ground motion and damping scheme effects on the bridge dynamic response, where the attenuation of tower structure is adopted the viscous damping of mass proportional type with damping coefficient to the first fundamental natural vibration. Rayleigh's damping scheme is used to form damping matrix $[C]$ as a linear combination of mass and stiffness matrices, which effectively captures the tower structures damping and is also computationally efficient.

$$[C] = a_m[M] + a_k[K] \quad (2.48)$$

where

$$a_m = \frac{2\omega_i\omega_j(\eta_i\omega_j - \eta_j\omega_i)}{\omega_j^2 - \omega_i^2}, \quad a_k = \frac{2(\eta_i\omega_j - \eta_j\omega_i)}{\omega_j^2 - \omega_i^2} \quad (2.49)$$

The stiffness and mass proportional damping constants (a_m , a_k) are given in terms of two different natural circular frequencies (ω_i , ω_j) and the corresponding critical damping ratio (η_i , η_j). Damping ratios for other modes can be found as

$$\eta = \frac{1}{2}(a_k\omega + a_m/\omega) \quad (2.50)$$

The use of more than two proportional damping constants is expensive. In order to prepare for situation where is possible to have more than two damping ratios incorporated in the analysis, a generalized damping matrix could be introduced.

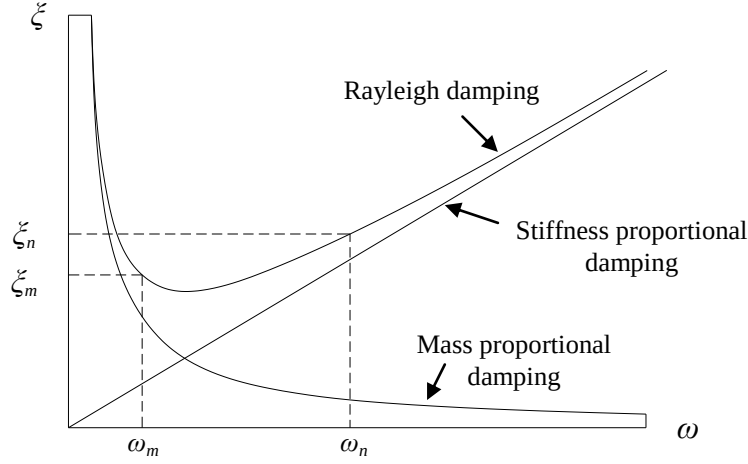


Fig. 2.4 Relationship between damping ratio and frequency for Rayleigh damping type

$$[\Phi]^T [C] [\Phi] = \begin{bmatrix} 2\omega_1\eta_1 & 0 & \cdots & 0 \\ 0 & 2\omega_2\eta_2 & & 0 \\ & & \ddots & \vdots \\ 0 & 0 & \cdots & 2\omega_n\eta_n \end{bmatrix} \quad (2.51)$$

where $[\Phi]$ is the modal matrix. The damping matrix can then be determined

$$[C] = [M] \left(\sum_{j=1}^{j=n} \frac{2\eta_j\omega_j}{\bar{M}_j} \{\phi_j\}\{\phi_j\}^T \right) [M] \quad (2.52)$$

where n is the number of modes considered for damping and $\bar{M}_j$ is the j th modal mass. One disadvantage of this approach is that a full damping matrix $[C]$ is obtained regardless of whether the stiffness and mass matrices are banded or not. An approximation has to be introduced in this regard by discarding elements of damping matrix off the stiffness matrix bandwidth. The relationships between damping ratio and frequency are shown graphically in **Fig. 2.4**.

2.4.4 Integration of dynamic equilibrium equation

Considering the direct integration method, the following three equations represent an implicit scheme:

$$(1-\alpha_M)[M]\{\ddot{d}\}_{t+1} + \alpha_M[M]\{\ddot{d}\}_{t+1} + (1-\alpha_C)[C]\{\dot{d}\}_{t+1} + \alpha_C[C]\{\dot{d}\}_t + (1-\alpha_K)[K]_{t+1}\{\Delta d\}_{t+1} = \{F\}_{t+1} - \{F\}_t \quad (2.53)$$

$$\{\dot{d}\}_{t+1} = \{\dot{d}\}_t + (1-\gamma)\Delta t \{\ddot{d}\}_t + \gamma\Delta t \{\ddot{d}\}_{t+1} \quad (2.54a)$$

$$\{d\}_{t+1} = \{d\}_t + \Delta t \{\dot{d}\}_t + (0.5-\beta)\Delta t^2 \{\ddot{d}\}_t + \beta\Delta t^2 \{\ddot{d}\}_{t+1} \quad (2.54b)$$

where α_M , α_C , α_K , β and γ are five constants and Δt is the time increment from time step t to time step $t+1$; the above two equations give

$$\{\ddot{d}\}_{t+1} = (1/\beta\Delta t^2)(\{\Delta d\}_{t+1} - \Delta t\{\dot{d}\}_t - (0.5-\beta)\Delta t^2\{\ddot{d}\}_t) \quad (2.55)$$

$$\{\dot{d}\}_{t+1} = (\gamma/\beta \Delta t)(\{\Delta d\}_{t+1} - (1-\beta/\gamma)\Delta t\{\dot{d}\}_t - (0.5 - \beta/\gamma)\Delta t^2 \{\ddot{d}\}_t) \quad (2.56)$$

The substitution of Eqs. 2.55 and 2.56 into Eq. 2.53 leads to the following incremental equivalent stiffness equation for determining the incremental displacement vector:

$$[K]\{\Delta d\}_{t+1} = \{F\} \quad (2.57)$$

in which effective stiffness matrix and the equivalent incremental load vector can be written as:

$$[K] = (1-\alpha_M) (1/\beta \Delta t^2) [M] + (1-\alpha_C) (\gamma/\beta \Delta t) [C] + (1-\alpha_K) [K]_{t+1} \quad (2.58)$$

$$\begin{aligned} \{F\} = \{F\}_{t+1} - \{F\}_t + (1/\beta \Delta t) ((1-\alpha_M) [M] + \Delta t (\gamma\beta - \alpha_C \gamma) [C]) \{\dot{d}\}_t \\ + (1/2\beta) ((1-2\beta - \alpha_M) [M] + \Delta t (\gamma/2\beta) (1-\alpha_C) [C]) \{\ddot{d}\}_t \end{aligned} \quad (2.59)$$

if $\alpha_C = \alpha_K$, then it represents the generalized- α trapezoidal scheme. If $\alpha_M = 0$ and $\alpha_K \neq 0$, then it results in the Hilber-Hughes-Taylor scheme. If $\alpha_K = 0$ and $\alpha_M \neq 0$, then it leads to the Wood-Bossak-Zienkiewicz scheme. If $\alpha_K = 0$ and $\alpha_M = 0$, then it reduce to the Newmark's β scheme. For the Newmark's β scheme, if $\beta = 1/4$ and $\gamma = 1/2$ then it results in the constant acceleration scheme, while $\beta = 1/6$ and $\gamma = 1/2$ leads in the linear acceleration scheme. An alternative efficient time integration scheme is the generalized- θ mid-point scheme.

In this study, the incremental equation of motion, Eq. 2.44 that is derived from consideration of static equilibrium at time t and $t+1$ can be written as follows:

$$I + H + R = F \quad (2.60)$$

where I , H , R and F are the incremental inertia, damping, elastic and unbalanced forces, respectively. To solve the equation of motion, the Newmark's step-by-step integration method is used for the integration of equation of motion, since it has been experienced that the Newmark's β method is the most suitable for non-linear analysis; it has the lowest period elongation and has no amplitude decay or amplifications. In addition, the stability concern is not a problem with the variable ratio of time increment to natural period. Using the Newmark's integration method, the equation 2.60 terms can be expressed as

$$\begin{aligned} I = \mathbf{M} \ddot{\mathbf{d}} = \mathbf{M} \left[\frac{1}{\beta \Delta t^2} \mathbf{d} - \frac{1}{\beta \Delta t} \dot{\mathbf{d}}_t - \frac{1}{\beta} \ddot{\mathbf{d}}_t \right], \quad R = \mathbf{K}_t \mathbf{d} \\ H = \mathbf{C} \dot{\mathbf{d}} = \mathbf{C} \left[\frac{\gamma}{\beta \Delta t} \mathbf{d} - \frac{\gamma}{\beta} \dot{\mathbf{d}}_t - \left(\frac{\gamma}{2\beta} - 1 \right) \Delta t \ddot{\mathbf{d}}_t \right] \end{aligned} \quad (2.61)$$

where γ and β are Newmark's integration parameters that can be chosen according to the accuracy and stability concerns. The algorithm is unconditionally stable if $\beta \geq (\gamma + 0.5)^2/4$. In this study the Newmark's β of constant acceleration scheme is considered for which γ and β are equal to 0.5 and 0.25 respectively.

The equation of motion is solved for the incremental displacement $\{\Delta d\}_{t+1}$ using the Newton Raphson iteration method where the stiffness matrix is updated at each increment to consider the geometrical and material nonlinearities and to speed the convergence rate. As the

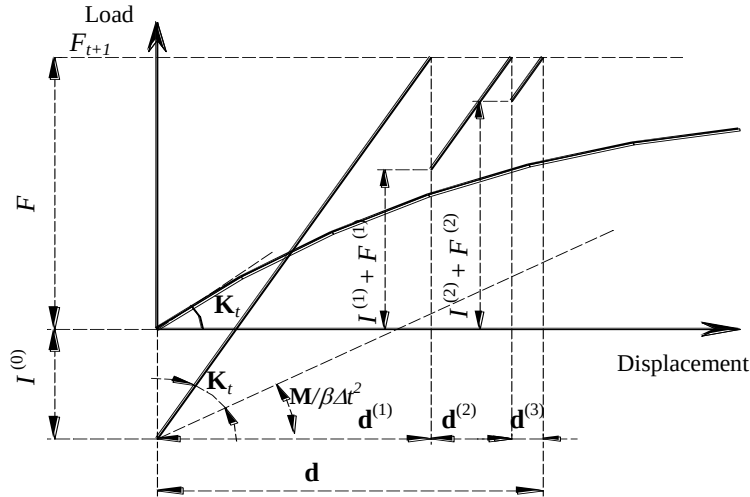


Fig. 2.5 Newmark's β method and Newton-Raphson iteration technique

incremental displacement is determined by the solution of Eq. 2.57, the response displacement at the end of the time step and the incremental displacement, velocity and acceleration components increment measured from the previous convergence configuration at time t can be expressed as

$$\begin{aligned}\mathbf{d} &= \mathbf{d}_{t+1} - \mathbf{d}_t = \Delta t \dot{\mathbf{d}}_t + (0.5 - \beta) \Delta t^2 \ddot{\mathbf{d}}_t + \beta \Delta t^2 \ddot{\mathbf{d}}_{t+1} \\ \dot{\mathbf{d}} &= \dot{\mathbf{d}}_{t+1} - \dot{\mathbf{d}}_t = \frac{\gamma}{\beta \Delta t} \mathbf{d} - \frac{\gamma}{\beta} \dot{\mathbf{d}}_t - \left(\frac{\gamma}{2\beta} - 1\right) \Delta t \ddot{\mathbf{d}}_t \\ \ddot{\mathbf{d}} &= \ddot{\mathbf{d}}_{t+1} - \ddot{\mathbf{d}}_t = \frac{1}{\beta \Delta t^2} \mathbf{d} - \frac{1}{\beta \Delta t} \dot{\mathbf{d}}_t - \frac{1}{\beta} \ddot{\mathbf{d}}_t\end{aligned}\tag{2.62}$$

The equation of equilibrium, using Eq. 3.61, can be expressed as

$$(\mathbf{K}_t + \frac{\gamma}{\beta \Delta t} \mathbf{C} + \frac{\gamma}{\beta \Delta t^2} \mathbf{M}) \mathbf{d} = F - H^{(0)} - I^{(0)} \quad (2.63)$$

in which $H^{(0)}$ and $I^{(0)}$ are directly obtained by substitution for the incremental displacement value $\mathbf{d} = \mathbf{0}$ in Eq. 2.61. A schema of Newmark's β method and Newton Raphson iteration technique is illustrated in **Fig 2.5**. Recurrence relations of the form characterize the Newmark's β method and Newton-Raphson iteration technique in the form of a pseudo static equation for the displacements at the end of the time step could be written as follows

$$(\mathbf{K}_{t+1}^{(i)} + \frac{\gamma}{\beta \Delta t} \mathbf{C} + \frac{\gamma}{\beta \Delta t^2} \mathbf{M}) \mathbf{d}^{(i+1)} = F - R^{(i)} - H^{(i)} - I^{(i)} = \Gamma^{(i)} \quad (2.64)$$

$$\begin{aligned}\mathbf{d}^{i+1} &= \mathbf{d}_{t+1}^{i+1} - \mathbf{d}_{t+1}^i \\ \dot{\mathbf{d}}^{i+1} &= \dot{\mathbf{d}}_{t+1}^{i+1} - \dot{\mathbf{d}}_{t+1}^i \\ \ddot{\mathbf{d}}^{i+1} &= \ddot{\mathbf{d}}_{t+1}^{i+1} - \ddot{\mathbf{d}}_{t+1}^i\end{aligned}\tag{2.65}$$

The pseudo stiffness matrix and the total displacement for each time step are defined as

$$\hat{\mathbf{K}}_{t+1}^{(i+1)} = \mathbf{K}_{t+1}^{(i)} + \frac{\gamma}{\beta \Delta t} \mathbf{C} + \frac{\gamma}{\beta \Delta t^2} \mathbf{M}, \mathbf{d} = \sum_1^i d^{(i)} \quad (2.66)$$

The iterative process is terminated when $\mathbf{F}^{(i+1)}$ and $\mathbf{d}^{(i+1)}$ fall with prescribed tolerance. In this study, the convergence is fixed for

$$\frac{\mathbf{d}^{(i)}}{\mathbf{d}} \leq 0.001 \quad (2.67)$$

2.5 Energetic Analysis

Generally speaking, the process for an earthquake to damage a structure can be viewed as an energy transferring process. Energy is produced by the collision of underground plates, and the movement of the ground transfers part of the energy to structures to induce structural vibration. Although the motion of the ground at a specific site is independent of the structure built at this site, the response of the structure and the amount of energy inputted to the structure depend highly on the structural characteristics such as the periods of vibration and damping. This input energy is then transferred to different energy forms such as strain energy, kinetic energy, and damping energy. Therefore, studying the energy characteristics and energy transfer process in structures during earthquakes will provide a good basis for protecting structures.

Housner [11] first propose an energy method to quantify seismic structural response. His research results indicate that a ground motion effectively feeds energy into a given structure, some of this energy being dissipated through damping and the remainder stored in the structure in the forms of kinetic energy or motion of the mass, and strain energy or deformation of the structural members. Housner formulated the fundamental concept that at any instant the total input energy must be equal to the sum of the kinetic energy, strain energy, energy dissipated through normal damping, and energy dissipated through permanent deformation. This provided the initial impetus to the later developments of energy methods in earthquake engineering. However, this proposal has remained an isolated episode until the last decade, when the availability of a great quantity of records from severe earthquakes had determined a decisive impulse to researches on energy criteria. During the last two decades many researchers have studied the energy concepts applied to the non-linear seismic analysis of structures. Uang and Bertero [12] provided mathematically consistent definitions of energy methods, Zahrah and Hall [13] studied seismic energy absorption in SDOF systems, and Akiyama [14] used an energy method based on the equation of Housner for the design of steel buildings. In the last few years, the seismic design methodology based on energy criteria has been introduced in the Japanese Seismic Code as an effective tool in the earthquake-resistance design of new structures and in the seismic assessment of the existing structures [4].

In this analysis, based on the equilibrium of energy, the non-linear seismic response of the bridge is also calculated, observing that energy methods produce goods indicators of the non-linear inelastic seismic structural performance.

The equation of motion of a multi-degree-of-freedom system subjected to ground motion can be written as follows

$$[M]\{\ddot{d}\} + [C]\{\dot{d}\} + [K]\{d\} = -[M]\{\ddot{d}_g\} \quad (2.68)$$

where $[M]$, $[C]$ and $[K]$ are the mass, damping, and stiffness matrices, respectively, $\{\ddot{d}\}$, $\{\dot{d}\}$ and $\{d\}$ are acceleration, velocity, and the relative displacement vectors, respectively, and $\{\ddot{d}_g\}$ is the earthquake ground acceleration vector. By multiplying both sides of Eq. 2.68 by $\{\dot{d}^T\}$ and by integrating the product over the entire duration of the earthquake, i.e. from $t = 0$ to t_0 , where t_0 refers to the instant when the earthquake fades away, the energy balance equation becomes

$$\int_0^t \{\dot{d}^T\} [M] \{\ddot{d}\} dt + \int_0^t \{\dot{d}^T\} [C] \{\dot{d}\} dt + \int_0^t \{\dot{d}^T\} [K] \{d\} dt = - \int_0^t \{\dot{d}^T\} [M] \{\ddot{d}_g\} dt \quad (2.69)$$

where

$$E_I = E_K + E_D + E_S \quad (2.70)$$

E_I is the absolute input energy where the inertia force applied to the structure is expressed in terms of the total acceleration relative to a fixed reference axis. E_K represents the absolute kinetic energy that includes the effects of the rigid body translation of the structure. E_D is the consumed energy due to damping mechanism of the system, and E_S corresponds to the absorbed energy, which is composed of recoverable elastic strain energy and irrecoverable hysteretic energy.

Then, at the end of the motion, Eq. 2.70 becomes

$$E_I = E_D + E_S \quad (2.71)$$

i.e. the total energy imparted to the structure by the earthquake must be dissipated by damping and inelastic deformations. Evaluating Eq. 2.69 over the entire duration of an earthquake motion, the cumulative energy balance is obtained.

2.6 Conclusions

In this study, the viaduct is expected to experience complex behavior during an earthquake. Static analysis could not describe this structural system. Therefore, based on the dynamic analysis, the calculation results can verify the safety of the bridge under seismic loading. A non-linear finite element formulation for the non-linear dynamic analysis of bridge structures is developed to obtain realistic prediction of the structural response. Hence, the design procedures and seismic performance of the viaduct can be effectively improved. This formulation includes the best features of several previous studies, where a fiber flexural element is developed for tower characterization; the element accounted for both geometric and material non-linearities, which are vital to understanding the complex structural mechanical behavior. The fiber flexural element modeling is adopted to capture the spread of plastic zone in individual element, the beam element is divided along its length and cross section directions. Based on the stress states of integration points in the section, the stiffness quantities of the section are calculated along the cross section, then the stiffness quantities of the element are obtained by integrating along the length of the beam, in which the plastic development of the material is automatically considered. Besides, the stiffness matrix calculations of the elements are completed by numerical integration procedure. After the stiffness matrix of the elements with respect to the local coordinate system is obtained, the

stiffness matrix of the elements is transformed from the element local coordinate system to global coordinate system. Finally the globe stiffness matrix is obtained by assembling the stiffness matrix of the elements using standard procedures.

Based on the actual uniaxial test data of structural steel bars, the fiber element model used in this study is physically motivated. Explicit expressions for stress in terms of strain also give the instantaneous stiffness, making the model ideal for iterative non-linear analysis. For compression and tension, the stress-strain relation is the same. Residual stresses shift the curve along the strain axis until the residual stress level is located at zero strain. Thus, adjacent fibers can have different levels of residual stress. The stress pattern is defined in such a way that equilibrium is maintained, but the maximum residual stress can be chosen. The differential equations of motion are solved in incremental form by employing the Newmark method. In convergent process, the Newton-Raphson method is used for the solution of the nonlinear problem, and the iteration continues until the tolerance reaches an acceptable specified value. During the incremental process, based on the tangent stiffness operator, the effects of both the geometrical variations and material non-linearity are accounted for.

In addition, the nonlinear seismic response of the bridge is calculated on the basis of the equilibrium of energy. Energetic analysis is formulated under the principle that at any instant the sum of the kinetic energy, strain energy, energy dissipated through normal damping, and energy dissipated through permanent deformation, must be equal to the total input energy.

References

- [1] Committee of Earthquake Engineering: *The 1995 Hyogoken-Nanbu Earthquake, Investigation into Damage to Civil Engineering Structures*, Japan Society of Civil Engineers, June, 1996.
- [2] Moehle, J. P.: Northridge earthquake of January 17, 1994: reconnaissance report, volume 1 - highway bridges and traffic management, *Earthquake Spectra*, Vol.11, pp. 287-372, 1995.
- [3] Ali, H. M. and Abdel-Ghaffar, A. M.: Modeling the nonlinear seismic behavior of cable-stayed bridges with passive control bearings, *Computers and Structures*, Vol. 54, No. 3, pp. 461-492, 1995.
- [4] Japan Road Association, *Specification for Highway Bridges - Part V Seismic Design*, Chapter 1-8, 1996.
- [5] Bathe, K. J., Ramm, E. and Wilson E. L.: Finite element formulations for large deformation dynamics, *International Journal for Numerical Methods in Engineering*, Vol. 9, pp. 353-386, 1975.
- [6] Bathe, K. J. and Bolourchi S.: Large displacement analysis of three-dimensional beam structures, *International Journal for Numerical Methods in Engineering*, Vol. 14, pp. 961-986, 1979.
- [7] Yang, T. Y. and Saigal S.: A simple element for static and dynamic response of beams with material and geometric nonlinearities, *International Journal for Numerical Methods in Engineering*, Vol. 20, pp. 851-867, 1984.
- [8] Shi, G. and Atluri, S. N.: Elasto-plastic large deformation analysis of space frames, *International Journal for Numerical Methods in Engineering*, Vol. 26, pp. 589-615, 1988.
- [9] Bathe, K. J.: *Finite element procedures in engineering analysis*, Englewood Cliffs, New Jersey, Prentice-Hill, 1982.
- [10] Clough, W. R. and Penzien, J.: *Dynamic of structures*, 2nd ed., New York, McGraw-Hill, 1993.
- [11] Housner G. W.: Behavior of structures during earthquakes, *Journal of Engineering Mechanics ASCE*, Vol. 25, pp. 85-109, 1959.
- [12] Uang, C. M. and Bertero V. V.: Evaluation of seismic energy in structures, *Earthquake Engineering and Structural Dynamics*, Vol. 19, pp. 77-90, 1990.
- [13] Zahrah, T. and Hall J.: Earthquake energy absorption in SDOF structures, *Journal of Structural Engineering ASCE*, Vol. 110, No. 8, pp. 1757-1772, 1984.
- [14] Akiyama H.: Earthquake resistant limit state design for buildings, *University of Tokyo Press*, Tokyo, 1985.

DESCRIPTION OF THE HIGHWAY VIADUCTS

3.1 Introduction

Horizontally curved bridges are commonly used for highway interchanges and have become a significant component of modern transportation systems. Increasingly complex interchanges and the desire to conform to existing geometric restrictions have made these types of structures more attractive [1-5]. Horizontally curved bridges make up of a significant portion of the bridge population in Japan, and they exist in all regions of the country.

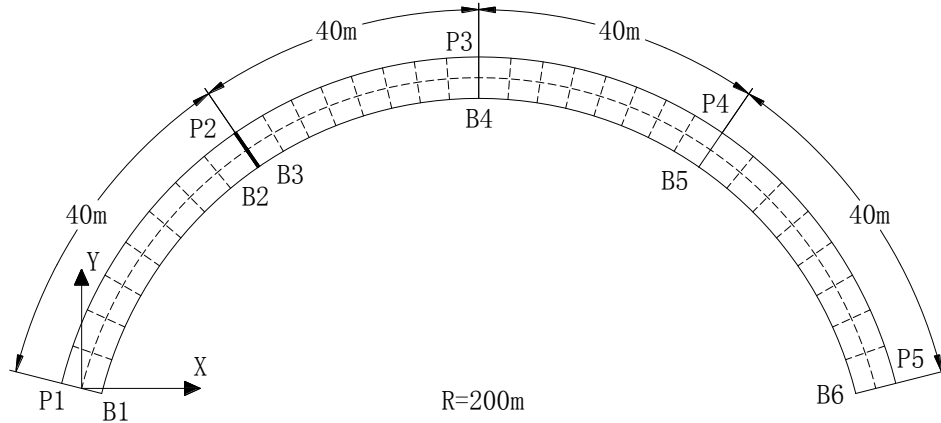
Based on the previous study [6-7], curved steel bridges under seismic loadings had larger nonlinear bearing and substructure response than straight bridges because of their curvature and geometric parameters, such as girder and cross-frame spacing. A representative example includes a curved multispan steel girder bridge located 49 km from the epicenter of the 1992 Petrolia earthquake that suffered severe damage [8]. This damage occurred at the cross-frames and shear connections between the steel girder and concrete deck, resulting in large lateral deformations at the bearings. During an earthquake, adjacent spans often vibrate out-of-phase, causing two types of displacement problems. One is the pounding between superstructures. Pounding phenomenon could amplify the relative displacement between adjacent superstructures [9, 10]. In addition, pounding could transfers large seismic lateral forces from one deck to the other, which obviously increase the reaction forces at the piers bases [11, 12]. The other problem is that pounding could increase the opening gap between adjacent superstructures, which may result the falling down of deck [13-16]. Therefore, four types of bridge damage are illustrated in this chapter.

In Japan, many curved viaducts supporting on steel bearings were easy to be damaged during serious earthquakes. Recently, some researchers and engineers use lead rubber bearing to support viaduct instead of steel bearing. However, both the material price and installation price of lead rubber bearing is expensive. So, in order to reduce the viaduct damage, stoppers are installed at both sides of roller bearings. Stopper could prevent large displacement of roller bearing. Besides, as stopper value is less than the gap between superstructures, the pounding phenomenon between superstructures could be prevented. Therefore, in this chapter, stopper is equipped at both sides of roller bearings in tangential direction.

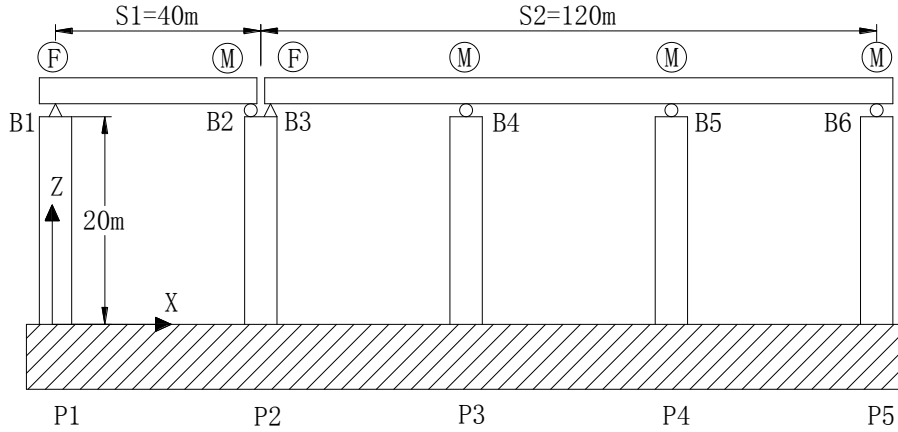
In order to exactly study the behavior of viaduct, the present study combines the use of a non-linear dynamic analysis methodology with a precise non-linear modeling of the entire bridge structural system. The non-linear viaduct seismic response has been accurately analyzed to provide a refined estimation of seismic demands on critical components with particular emphasis focused on the expansion joint behavior and its interaction with the remaining bridge structure.

3.2 Analytical Model of the Viaduct

In order to obtain actual and realistic prediction of the viaduct structural responses, three dimensional (3D) analytical model is adopted in this analysis. Three dimensional analytical model allows for the evaluation of the combined effect of longitudinal, transverse and vertical earthquake excitations on the response of actual viaduct. This fact provides a valuable environment for the non-linear behavior due to material and geometrical nonlinearities of the relatively large deflection of the structure on the stresses and forces. Characterizations of the non-linear structural elements of piers and bearings are also considered.



(a) Plan view of curved highway viaduct model



(b) Elevation view of curved highway viaduct model

Fig. 3.1 Analytical model of viaduct

The analytical highway viaduct is composed by a single simply-supported span connected to a three-span continuous span. The overall viaduct length of 160 m is divided in equal spans of 40 m as shown in **Fig. 3.1**. The width of superstructure is 11.9 m. The viaduct alignment is horizontally curved in a circular arc with a radius of curvature of 200 m. The curvature is measured from the origin of the circular arc to the centre-line of the deck superstructure. Tangential configuration for both piers and bearing supports is adopted, respect to the global coordinate system for the viaduct as shown in **Fig. 3.1**, in which the X- and Y- axes lie in the horizontal plane while the Z-axis is vertical. The fundamental natural period for simple supported span and continuous span is 0.770 s and 1.393 s respectively. This fundamental natural period corresponds to the modal shape in the longitudinal direction of the viaduct.

3.2.1 Deck superstructure

The viaduct superstructure is composed of a concrete deck slab and three I-section steel girders (G1, G2 and G3). The distance between adjacent is 2.1 m girders in transverse direction. 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

Table 3.1 Cross sectional properties of deck

Girder	A (m ²)	I_z (m ⁴)	I_y (m ⁴)
G1	0.2350	0.1005	0.0994
G2	0.4000	0.1609	0.2182
G3	0.2350	0.1005	0.0994

Table 3.2 Cross sectional properties of piers

Pier	A (m ²)	I_x (m ⁴)	I_y (m ⁴)	M_{e0} (MN m)
P1	0.4500	0.3798	0.3798	77.63
P2	0.4700	0.4329	0.4329	84.76
P3	0.4700	0.4329	0.4329	84.76
P4	0.4700	0.4329	0.4329	84.76
P5	0.4500	0.3798	0.3798	77.63

girders is assumed for the linear elastic elements of the superstructure model, which is treated as a three dimensional grillage beam system as shown in **Fig. 3.2**. Determination of these quantities is discussed in the Appendix A. The cross sectional properties of the deck are summarized in **Table 3.1**. End cross-frames provide an important load-path for the seismically induced lateral loads. Seismic forces at the deck are transmitted from the cross-frames to arrive at the top of the bearings, to reach the substructure and finally the ground. As a result the stiffness of all these components will affect the dynamic behavior of the bridge. The in-plane stiffness of the deck is very high in comparison with the end-diaphragms and it is assumed to be rigid. This means that all points of the deck will displace equally [17]. At the pier locations, the viaduct deck is modeled in the transverse direction as a rigid bar of length equal to the deck width. The interaction between deck and pier motions could be successfully simulated by the transverse rigid bar [18].

3.2.2 Piers

According to the seismic design code in Japan, the viaduct piers have been successfully designed. Box-section steel piers are viable solutions to support highway bridges in space-restricted urban areas. Furthermore, due to their high strength/mass ratios and significant ductility, their structural performance during strong earthquakes is greatly enhanced [19]. The height of all hollow box section steel piers is 20 m. The cross sectional properties of the viaduct piers are summarized in **Table 3.2**. It can be seen from **Table 3.2** that central piers (P2, P3 and P4) have larger cross section sizes than outer piers (P1 and P5). Since the length of corresponding span is longer for central piers, and consequently, larger forces due to gravity loading are transferred to central piers. In addition, steel and concrete densities are 7850 kg/m³ and 2500 kg/m³ respectively. Size and thickness of the pier plate elements are also selected considering that width/thickness ratios of the members must not exceed specified limits for box-section design, usually achieved by welding transverse to the thin-walled plates [20]. The seismic design of piers, performed using the seismic coefficient method given in the Specifications for Highway Bridges [21], is explained in detail in Appendix B. Note that same structural properties of piers have been considered in transverse direction due to the square box section.

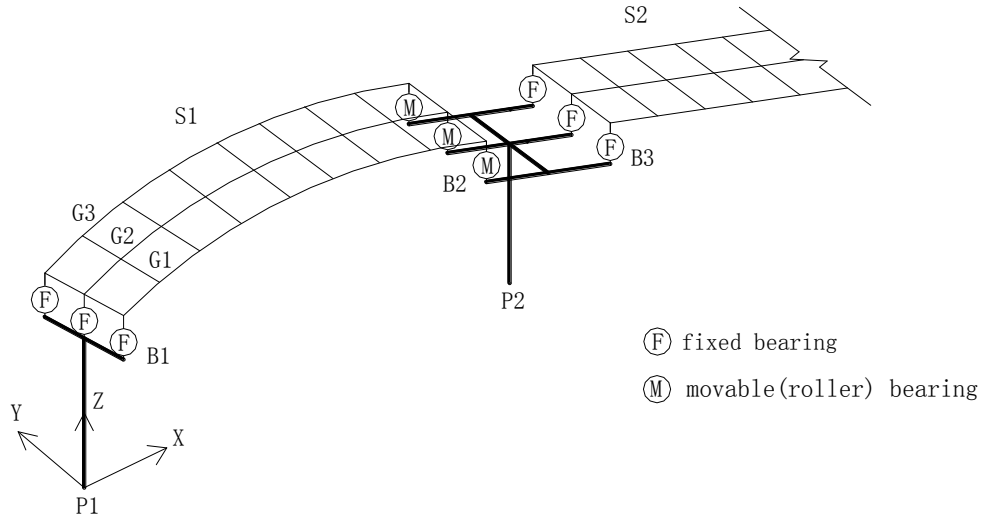
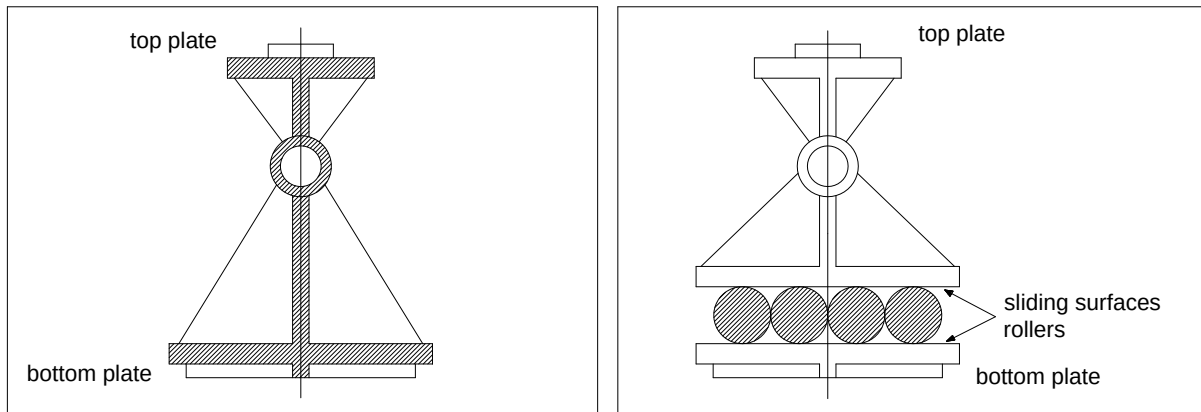


Fig. 3.2 Curved viaduct finite element model

Fiber element could be used to simulate characterization of structural pier elements [22]. The fiber element model is actually considered an accurate and practical technique for computing the response of structural members. In the fiber element model developed for characterization of structural bridge elements in this study, the element is subdivided into a discrete number of longitudinal and transversal fibers. The geometric characteristics of each fiber are its location in the local reference system and the fiber area. The accuracy of the element response and, consequently, that of the entire structure are affected by the mesh selection that is the number of sections used in the discretization of a given member, and by the number of fibers in a section. A large number of fibers certainly could obtain better results, but calculation time increases. Therefore it is very important to choose appropriate number of fibers in this analysis. The element is divided in 5 longitudinal parts, which, as well are subdivided in 12 transverse divisions. The element stress resultants are determined by integration of the fiber zone stresses over the cross section of the element. The steel stress-strain behavior of the beam element is described by a bilinear non-linear model with uniaxial yield criterion and kinematic hardening flow rule. The strain hardening in plastic area is equal to 0.01. The yield stress is 235.4 MPa, and the modulus of elasticity is 200 GPa.

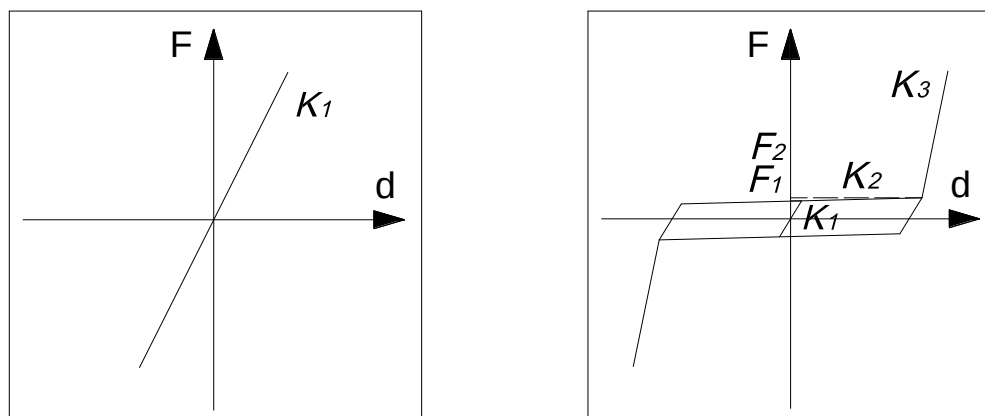
3.2.3 Bearing supports

Generally speaking, steel bearings have been extensively installed in bridges to transfer vertical and horizontal forces from the superstructure to the substructure. An appropriate combination of fixed and roller steel bearing is considered optimal to support the superstructure weight providing accommodation for thermal expansion, contraction and rotation to the deck superstructure. However, recent serious earthquakes, including the HN Earthquake, have clearly demonstrated that failure of steel bearings resulted in collapse of highway viaducts [23]. Therefore, the bearing support is one of the essential structural members of a bridge. A typical steel fixed bearing, illustrated in Fig. 3.3 (a), consists of a parted metal casing with each part welded to the top and bottom steel plates. The top steel plate of the bearing is connected to the deck girder by connection bolts and the bottom steel plate is connected to the pier by anchor bolts. Steel roller bearings, shown in Fig. 3.3 (b) have similar details of fixed bearings, except the casings, which have a special configuration for



(a) Fixed bearing

(b) Roller bearing

Fig. 3.3 Schematic representation of steel bearings

(a) Fixed bearing

(b) Roller bearing

Fig. 3.4 Analytical model of steel bearing supports**Table 3.3** Structural properties of steel bearing supports

Bearing type	Component	K_1 (MN/m)	K_2 (MN/m)	K_3 (MN/m)	F_1 (MN)	F_2 (MN)
Fixed	Longitudinal	980.0	-	-	-	-
	Transverse	980.0	-	-	-	-
Roller	Longitudinal	49.0	0.0098	980.0	0.0735	variable
	Transverse	980.0	-	-	-	-

allowing free movement to the supported deck in longitudinal direction. Roller supports are free to translate along the surface upon which the roller rests, allowing the bridge structure to move under seismic and non-seismic loadings.

Fixed bearing supports (**Fig. 3.4** (a)) are installed on top of left pier (P1) in simple supported span, roller bearing supports (**Fig. 3.4** (b)) are rested on the top of right pier (P2). For continuous span, fixed bearing supports are installed on top of left pier (P2). However, roller bearings are rested on top of other piers (P3, P4 and P5). Fixed bearing supports movements are restrained in both longitudinal and transverse radial direction. Roller bearing supports have freedom movement in longitudinal direction, but have been restrained in transverse radial direction. **Table 3.3** shows structural properties of steel bearing supports. It

Table 3.4 Properties of the selected earthquake ground motion

Earthquake record	Comp.	PGA (gal)	PGV (kine)	PGD (cm)	RMSA (gal)	RMSV (kine)	I_A (cm/sec)	T_1 (sec)
TAK	L	641.7	127.1	35.8	140.4	35.0	90997.6	1.20
	T	666.2	120.7	32.7	128.0	30.3	75718.2	1.17
	V	289.7	16.0	4.5	51.4	8.9	12172.6	0.13
KOB	L	817.8	81.3	17.7	131.4	15.6	79741.2	0.68
	T	617.1	74.3	19.9	105.7	12.8	51654.5	0.69
	V	332.2	38.3	10.3	61.7	7.5	17559.8	0.32

should be noted at this point that steel bearings of this study have a large reserve of strength against gravity loads, thus the bearings are vertically stiff enough to prevent vertical deformations. Bearings are also adequately anchored against potential uplift, while attachment details are consistent with the characteristics of the bridge. Transverse modeling is exclusively considered in case of the 3D analytical model with rotations allowed to occur in the horizontal plane. A simple element of high stiffness is used for the fixed bearings. On the other hand, Coulomb friction force is taken into account in numerical analysis for roller bearings, which are represented by using a trilinear element (**Fig. 3.4 (b)**). The Coulomb frictional force of roller supports (F_1) is obtained by multiplying the vertical reaction due to the dead load acting on the support by the coefficient of static friction, as shown in Eq. 3.1,

$$F_1 = R * \mu = 150 * 0.05 = 7.5 \text{ tf (0.0735 MN)} \quad (3.1)$$

where R is the vertical reaction force due to the superstructure weight, and $\mu = 0.05$ is the static friction coefficient of roller bearing support which remains constant and independent of pressure and velocity. It indicates that steel stoppers are provided to roller bearings in order to limit the relative movement of the upper and lower portion of those bearings, thus reducing the risk that rollers will be dislodged from the bearing assembly. Additionally, steel stoppers are installed at both sides of roller bearing supports in longitudinal direction. In the analytical model, F_2 delimitates the clearance to stoppers. Pounding stiffness between roller bearing supports and steel stopper is K_3 which has a high stiffness slope.

3.2.4 Expansion joint

There is a gap between simply-supported span and continuous span, which allows for contraction and expansions of the road deck from creep, shrinkage, temperature fluctuations and traffic without generating constraint forces in the structure. During the 1995 HN Earthquake, pounding damage at the expansion joints leaded to fracture of bearing supports, local damage and potential contribution of falling down of bridge decks. The large pounding forces between superstructures result in a significant additional loading transmitted to the piers bases of the bridge. In addition, the large pounding forces between superstructures increases the possibility of brittle fracture of structural members such as bearing supports. Based on the previous study [24, 25], the pounding forces between superstructures is less, as the gap between superstructures is either small or large enough to avoid collisions. However, enlarging the expansion joints between girders is an expensive and undesirable solution, because vehicles crossing large expansion joints are subjected to noise, vibration and traffic problems. Recent strong earthquakes have repeatedly demonstrated that during an earthquake, adjacent spans often vibrate out-of-phase, causing excessively large separation at the expansion joint. In this study, the gap between superstructures is 10 cm. As stopper value is less than the gap at the expansion joint, both the falling down of deck and the pounding

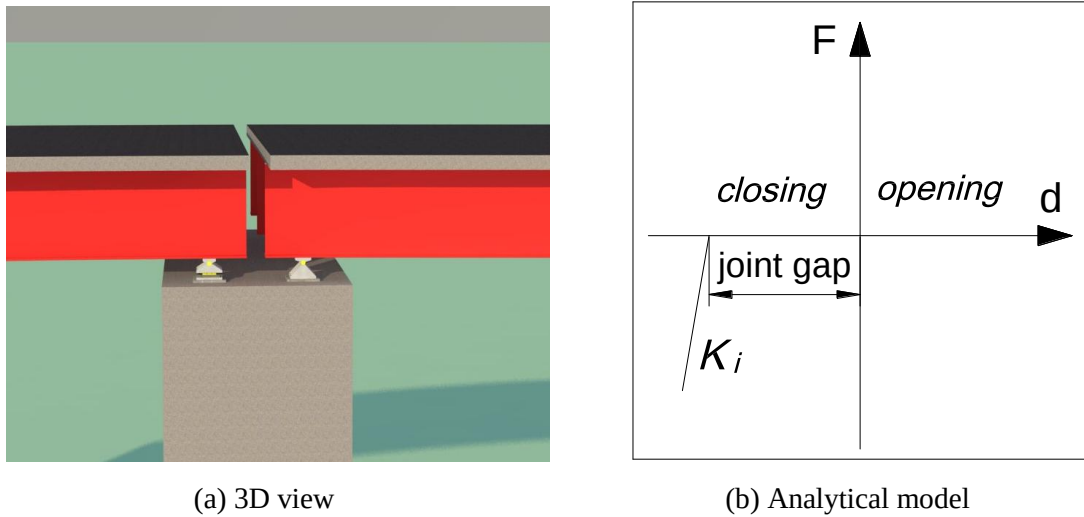


Fig. 3.5 3D view and analytical model of expansion joint

phenomenon between superstructures shall not occur. It is stopper that resists the large relative displacement between superstructures. However, as stopper value is larger than the gap at the expansion joint, the falling down of deck could occur. Besides, the pounding phenomenon between superstructures may also happen. The pounding phenomenon, defined as taking place at the three girder ends, is modeled using impact spring elements for which the compression-only bilinear gap element is provided with a spring of stiffness $K_i = 980.0 \text{ MN/m}$ as shown in **Fig. 3.5**. In order to simplify, the effects of the expansion joint in the transverse direction are neglected.

3.3 Input Earthquake Ground Motions

The 17th of January 1995 at 5.46 a.m. a violent earthquake, 7.2 on the Richter scale, struck Kobe, a highly developed and congested modern city in Japan. The depth of its epicenter was 14 km below ground level. Since the earthquake motion progressed toward the faults directly below Kobe, the HN Earthquake caused severe damage and destruction to a large number of structures throughout the Hanshin-Awaji areas [26]. Many bridges were damaged in this earthquake. In particular, damage to highway bridges was unprecedented, both in scale and quantity [27]. The Japan Society of Civil Engineers (JSCE) issued the “Proposal on Earthquake Resistance for Civil Engineering Structures” [28], in which two types of earthquake ground motions were defined for earthquake resistant design of structures. One type, referred to as Level I (L-I), represents ground motions of moderate intensity, which are likely to be experienced by the structures once or twice during their lifetime. The other is Level II (L-II) ground motion of extreme intensity rarely experienced during their lifetime. L-II ground motions represent the near-fault earthquakes, with accelerograms modified from direct-inland-stricken type earthquake records characterized by a short duration, high intensity, pulse-like shaking like the devastating 1995 HN Earthquake.

Therefore, the Japanese Seismic Specifications were revised to performance-based design in 2000 [20]. According to the performance-based philosophy, a structure should satisfy requirements to assess the seismic performance under different specified levels of seismic excitation. Because L-II motions can drive the bridge structure to its ultimate limit design state, leading to the highest damage potential. Consequently, L-II motions are basically used to check the ultimate capacity of structures beyond their elastic limits. L-II motions generated by active inland faults are, in principle, determined based on identification of threatening

active faults and estimation of their fault mechanism. Consideration on L-II motions is needed even in an area where significant active faults are not mapped until today [29]. The near-fault region of an earthquake is considered to be the area within several kilometers of the extension to the ground surface of the fault rupture plane. Near-fault earthquake ground motions contain large, rapid pulses, which result in large ground displacements, generally exceeding 0.5 m, which take place at rapid ground velocities, usually on the order of 1 m/sec, duration in the range of 1-3 sec, and significant long-period content. In order to evaluate the seismic performance of the viaduct, the non-linear model is subjected to two different sets of earthquake records. The two input ground motions are accelerograms obtained during the HN Earthquake ($M_w = 6.9$) at JR Takatori Station (TAK) and the Kobe Observatory of Japan Meteorological Agency (KOB). The large magnitude records used in this study, classified as near-fault motions, are characterized by the presence of high peak accelerations and strong velocity pulses with a long period component as well as large ground displacements [30].

These serious earthquakes have been selected due to the destructive potential of long duration pulses on structures that can lead to large deformations, probably exciting the bridge into its nonlinear range as well as inducing significant opening and pounding phenomenon at the expansion joint. Acceleration, velocity and displacement-time histories, including the two earthquake components, are shown in **Figs. 3.6-3.8** respectively. The input ground motion records include three direction earthquake waves which are longitudinal (L), transverse (T) and vertical (V) respectively. The longitudinal earthquake component shakes the highway viaduct parallel to the X-axis of the global coordinate system, while the transverse and vertical components are acting in the Y- and Z-axes, respectively.

The maximum amplitude of recorded acceleration, termed as peak ground acceleration (*PGA*), is a basic measure of earthquake potential and is a commonly used index to describe the severity of earthquake ground motions. However, examination of several recorded seismic events has shown that a large *PGA* is not always followed by severe structural damage, while ground motions with a very low *PGA* could produce an unexpectedly high level of destruction. This fact can be explained since large recorded *PGA* often corresponds to short duration impulses of high frequency, which are out of the range of the natural frequencies of most structures. For this reason, the *PGA* cannot be considered as a totally reliable factor. Instead, the peak ground velocity (*PGV*) or integral parameters seems to be a more representative measure of earthquake intensity as they are directly correlated with energy demands.

The input earthquakes of this study have been characterized in terms of peak and integral parameters. Among various alternative possibilities ground motion indices are selected to represent the measure of the earthquake risk. The peak parameters, peak ground acceleration (*PGA*) and velocity (*PGV*), are obtained as the maximum value in the time-history records. The integral parameters, the root mean square acceleration (*RMSA*), velocity (*RMSV*) and the Arias intensity (I_A), are calculated using Eqs. 3.2 and 3.3.

$$RMSX = \left[\frac{1}{t_E} \int_0^{t_E} x^2(t) dt \right]^{1/2} \quad (3.2)$$

where $x(t)$ is either the ground acceleration, velocity or displacement, and t_E is the total duration of the earthquake. The Arias intensity, I_A , is proportional to the integral over time of the acceleration squared, and is expressed in units of velocity, defined as follows,

$$I_A = \frac{\pi}{2g} \cdot t_E \cdot RMSA^2 = \frac{\pi}{2g} \int_0^{t_E} \ddot{x}_g(t)^2 dt \quad (3.3)$$

where g is the gravity acceleration, and $\ddot{x}_g(t)$ is the ground acceleration.

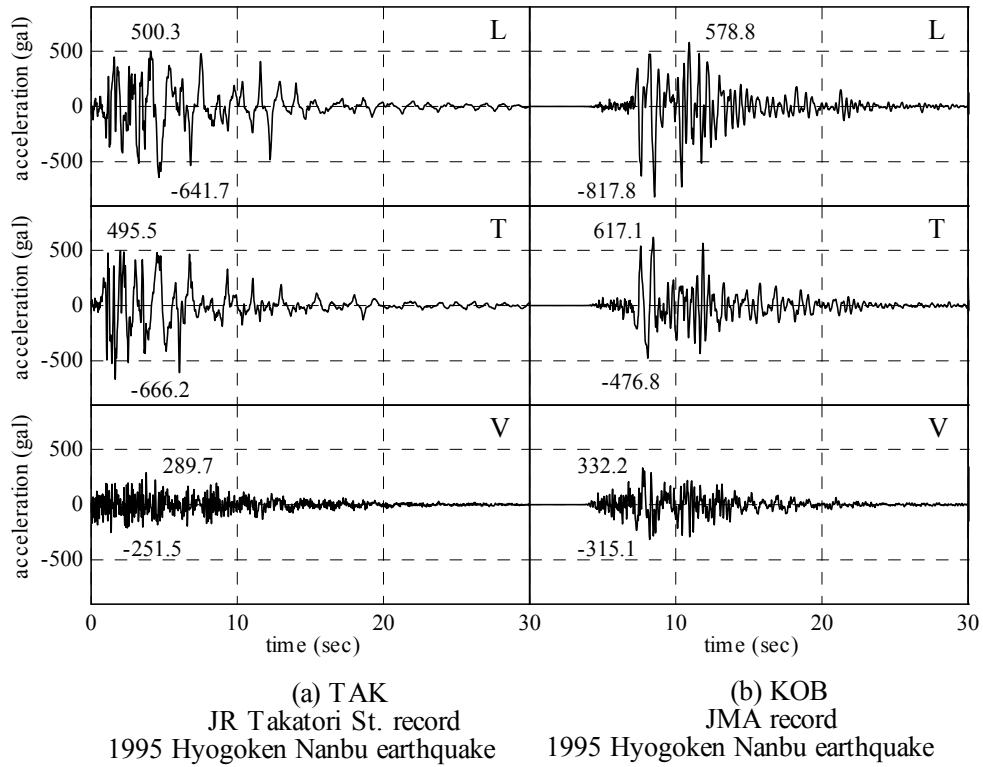


Fig. 3.6 Input earthquake accelerations

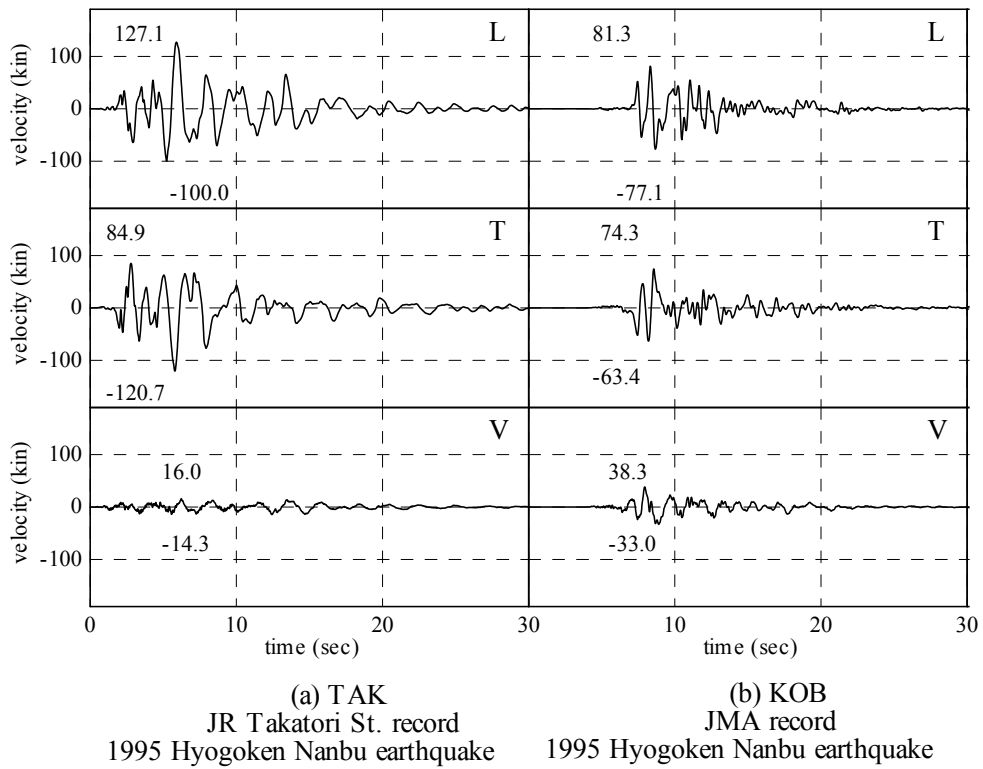


Fig. 3.7 Input earthquake velocities

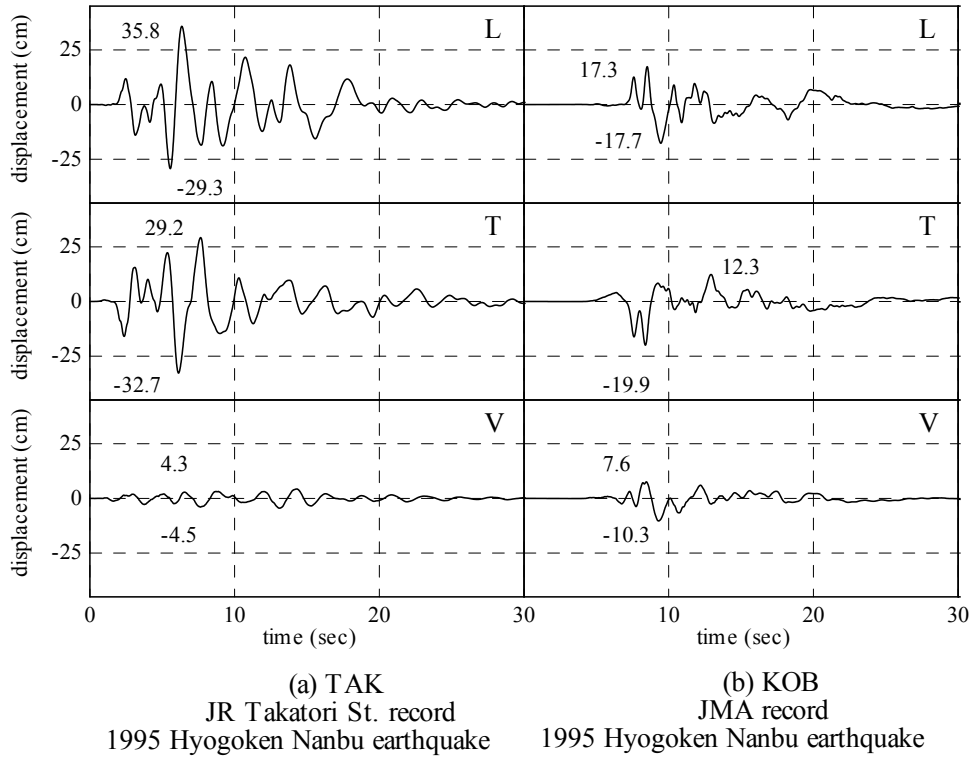


Fig. 3.8 Input earthquake displacements

All the seismic parameters previously defined have been calculated for the two input ground motions and summarized in **Table 3.4**. The predominant period of the earthquakes, determined by examining the spectra of the free-vibration records, is also included in the table. The *PGV* and the integral parameters (*RMSA*, *RMSV*, and *I_A*) are found to provide plenty identification with the near-fault motion characteristics, which are dominated by pulses of intensity larger in a direction perpendicular (L-component) than that parallel to the fault plane (T-component). In addition, the acceleration response spectrum for a damping ratio ($\xi=0.02$) is shown in **Fig. 3.9**.

3.4 Classification of Seismic Damage

Bridges in recent strong seismic events provides one of the best means of assessment of seismic resistance in new construction as well as for retrofitting of existing bridge structures. For this reason, most expected types of earthquake damages to bridges with similar characteristics to those considered in this study have been evaluated.

3.4.1 Deck unseating damage

One of the most catastrophic seismic damages to bridge superstructures is the failure due to deck unseating [31, 32] as shown in **Fig. 3.10** (a). During an earthquake, adjacent spans can vibrate out-of-phase, resulting in relative displacements at expansion joints. In simple supported span, the induced relative displacements to steel roller bearings can exceed the seat width at the pier top, causing the dislodgment of the rollers from the bearing assembly and the subsequent collapse due to the falling down of deck. The maximum roller bearing (B2) displacement in negative tangential direction has been established as the damage index to evaluate the potential possibility of deck unseating. A limit of 0.35 m has been considered to

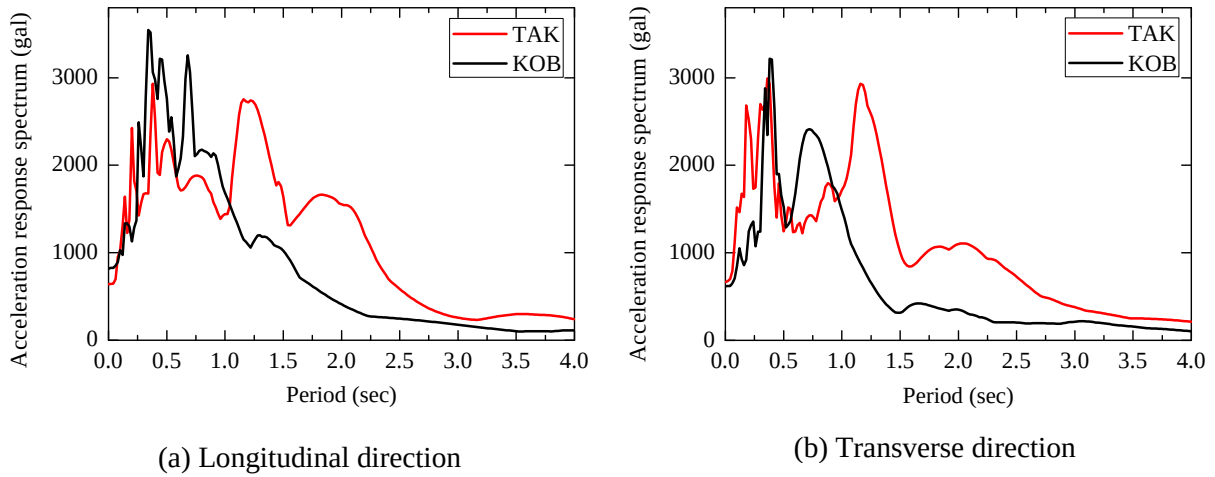


Fig. 3.9 Acceleration response spectra ($\xi = 0.02$)

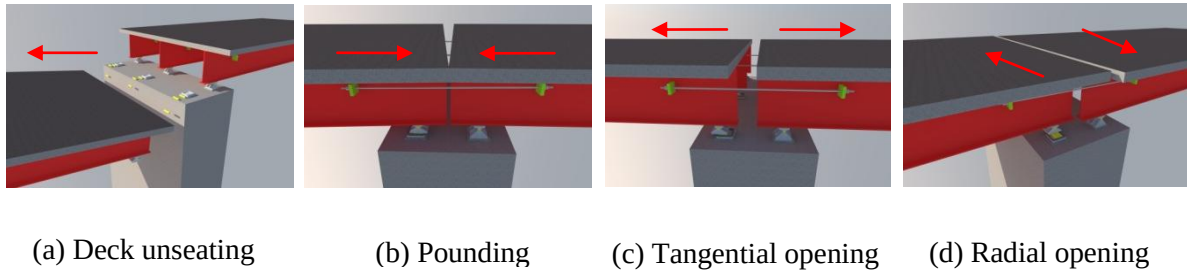


Fig. 3.10 Schematical representation of seismic damages at top of pier 2

indicate the high unseating probability for existing bridges with narrow steel pier caps that provide short seat widths.

3.4.2 Pounding damage

Pounding phenomenon between superstructures is shown in **Fig. 3.10** (b). Large pounding force between superstructures can lead to large local damage at colliding area. Besides high impact forces are transmitted to bearing supports, make bearing supports particularly vulnerable to failure, which could result in the collapse of the bridge. Ratios of maximum impact force to the deck weight greater than 1.0 have been observed to provide a good estimation of significant transmitted forces to bearing supports.

3.4.3 Tangential and radial joint residual opening damage

This damage is shown in **Fig. 3.10** (c) and **Fig. 3.10** (d). Permanent tangential offsets of expansion joints observed at the end of recent earthquakes have substantially interfered with the post earthquake serviceability of highway viaducts. In several cases, traffic was impeded and the disruption of the bridge usability in the aftermath of the earthquakes resulted in a critical problem for rescue activities. The residual joint tangential displacement (RJTD) has been calculated in order to perform the post-earthquake serviceability evaluation on the viaduct. 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 [33]. 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, thus sensibly restricting the operability of the bridge in such critical moments.

3.4.4 Pier damage

During the 1995 Kobe earthquake, a large number of piers supporting highway viaducts were severely damaged due to large local inelastic cyclic strains [34]. As a consequence, piers sustained significant residual deformations, which were dependent on the maximum ductility reached during the seismic loading [35]. Bridges supported on piers with large residual inclination may lose their serviceability, becoming largely unsafe and probably irreparable. Due to this fact, immediately after this earthquake, the 1996 Japanese specifications imposed the limitation of 1% maximum inclination of the bridge pier height in post-earthquake pier demolition for important bridges [20]. Therefore, the residual pier inclination (RPI) has been considered as an important damage index in this investigation, being computed as the final pier position in the orbit of the two horizontal directions at the end of the earthquake, in order to consider the bi-directional loading condition in the horizontal plane.

3.5 Conclusions

Horizontally curved bridges are common in Japan. But many curved bridges were damaged in the Hyogoken-Nanbu earthquake of 1995. Therefore, in this chapter, the model of curved viaduct is carried out and the classification method of viaduct damage is also presented.

(1) Three dimensional (3D) analytical model is adopted in this analysis, which allows for the evaluation of the combined effect of longitudinal, transverse and vertical earthquake excitations on the response of actual viaduct. The non-linear behavior due to material and geometrical nonlinearities of the relatively large deflection of the structure on the stresses and forces is accounted for. In addition, characterizations of the non-linear structural elements of piers and bearings are also considered.

(2) The continuous bridge superstructure consists of a concrete deck slab that rests on three I-shape steel girders, modeled with a box cross-section element which represents the composite effect of the continuous steel reinforced concrete deck and the steel girders, identified as a homogeneous material. Pier is simulated by fiber element. The fiber element model is actually considered an accurate and practical technique for computing the response of structural members. The steel stress-strain behavior of the pier element is described by a bilinear non-linear model with uniaxial yield criterion and kinematic hardening flow rule. Fixed bearings are simulated by a simple element of high stiffness. Coulomb friction force is taken into account in numerical analysis for roller bearings, which are represented by using a trilinear element. Pounding stiffness between roller bearing and steel stopper is a high stiffness slope.

(3) The pounding phenomenon between superstructures is simulated by a spring of high stiffness. In order to simplify, the effects of the expansion joint in the transverse direction are neglected. The non-linear model of viaduct is subjected to two different sets of earthquake records. The input ground motions are accelerograms obtained during the HN Earthquake at JR Takatori Station and the Kobe Observatory of Japan Meteorological Agency.

(4) There are many types of viaduct damage during serious earthquakes. In this chapter, four types of viaduct damage are presented including deck unseating damage, tangential and radial joint residual opening damage, pounding damage and pier damage. The definition and evaluation index of these damages are carried out.

3.6 Appendices

(1) Appendix A: Calculation of deck superstructure composite section

For composite plate girder bridges, the steel girders and the concrete slab are joined by shear connectors. In this way, the concrete slab becomes integral with the girder and usually takes most of the compression component of the bending moment while the steel plate girder takes the tension. For this reason, composite girders are much more effective than the simply tiered girder.

A typical portion of a composite I-girder bridge consisting of a concrete deck and built-up plate girder I-section with cross frames is represented in **Fig. 3.11**. The deck of the bridge, shown in **Fig. 3.12**, is made from reinforced concrete segments with a mass of $7.5 \times 10^3 \text{ kg/m}$. The span length and the width of one superstructure segment are 40.0 and 11.9 m, respectively. The steel reinforced concrete slab is supported on three steel girders at the interval of 4.2 m, one of which is plotted in **Fig. 3.13**.

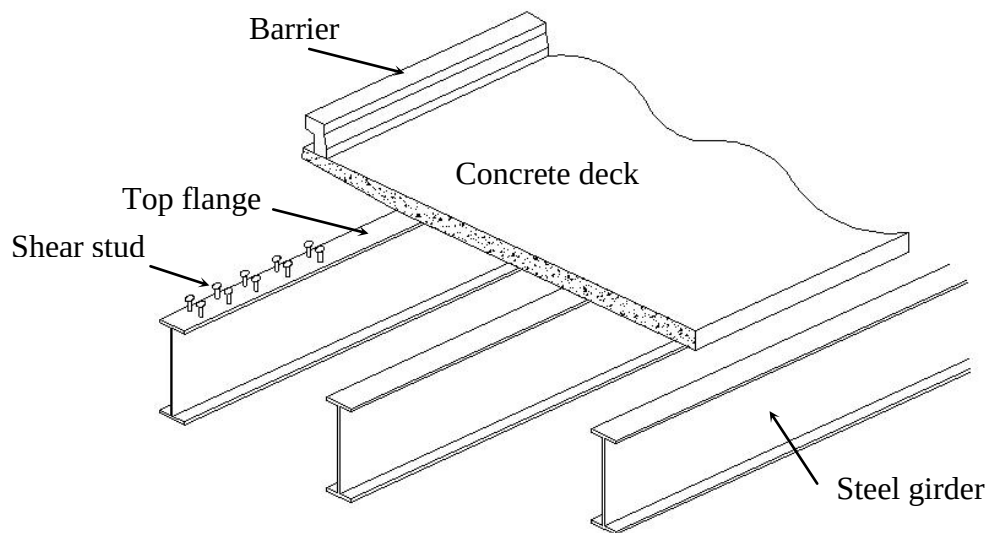


Fig. 3.11 Components of composite I-girder bridge

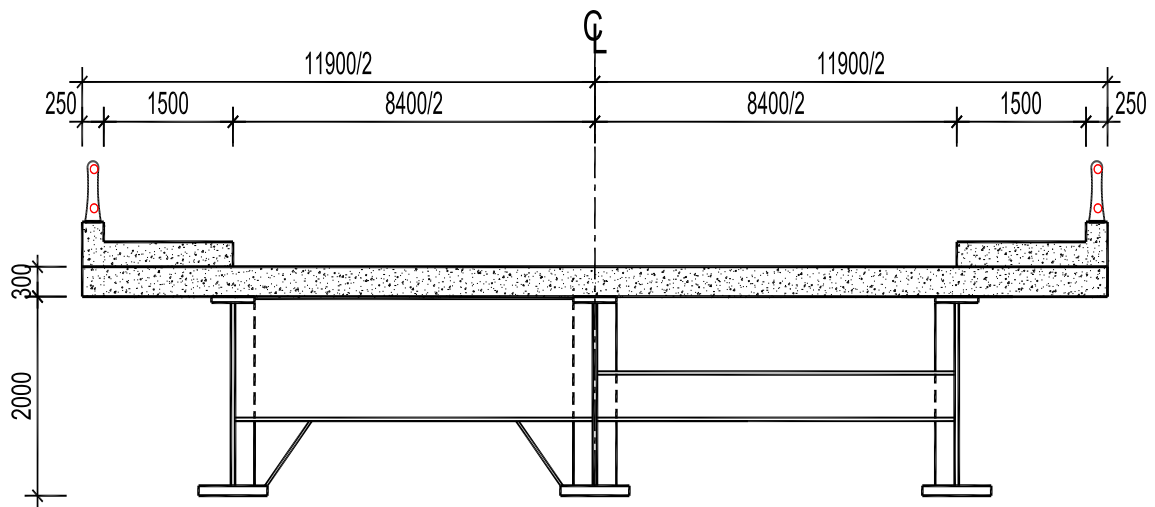


Fig. 3.12 Section of deck superstructure

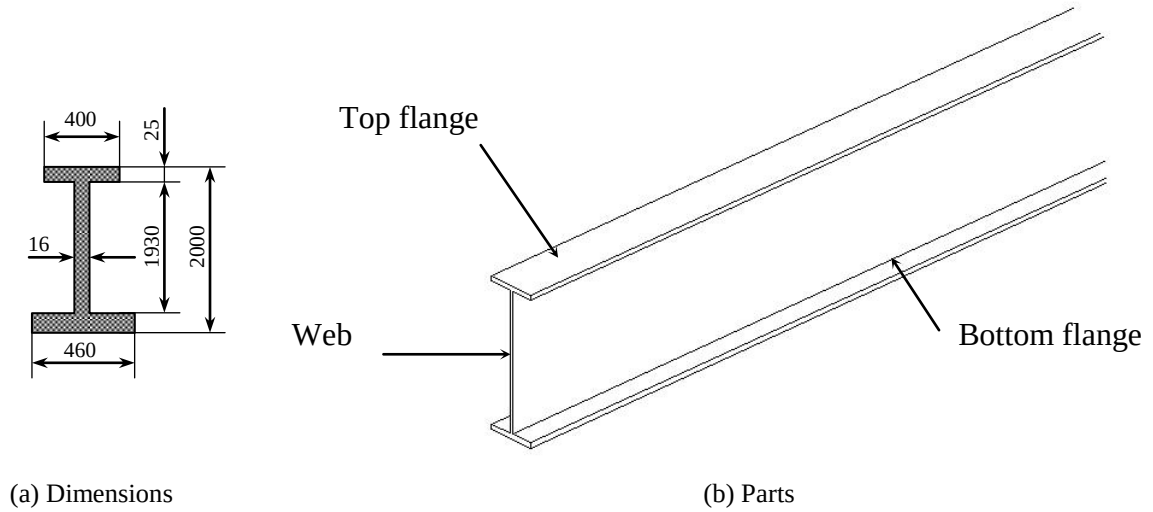


Fig. 3.13 Dimensions (mm) and parts of steel girder

In order to calculate the concrete slab-steel girder composite section properties of the superstructure, the cross-sectional area and moment of inertia of the concrete slab and the steel girder are separately calculated, as summarized in **Table 3.5**.

In computation of the properties of the composite section, the concrete slab, ignoring the haunch area, is transformed into an equivalent steel area. For the purpose, the concrete area is divided by the modular ratio $n = 7$ (ratio of Young's modulus between steel and concrete). Therefore, the composite area, A_v , can be calculated to represent it as a homogeneous material using Eq. 3.4.

$$A_v = A_s + \frac{1}{n} A_c \quad (3.4)$$

Then, $A_v = 0.52954 \text{ m}^2$. The first step of determining the moment of inertia of the composite section is to find the neutral axis, in **Fig. 3.14**, through Eqs. 3.5 and 3.6.

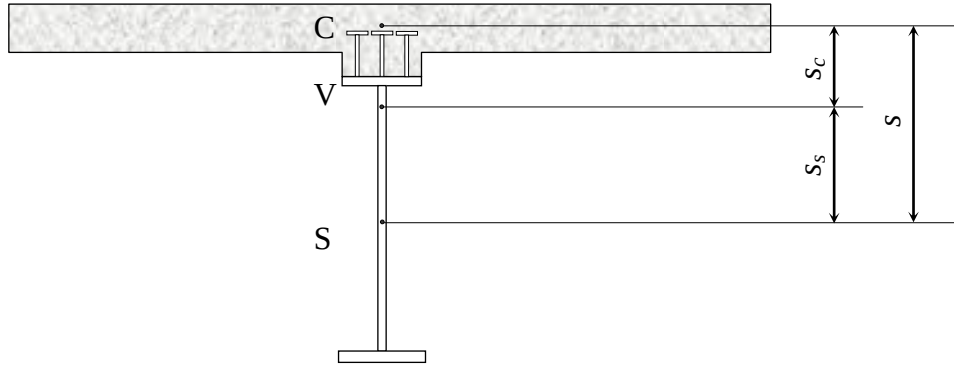
$$s_c = \frac{A_s}{A_c} s \quad (3.5)$$

$$s_s = \frac{A_c}{n \cdot A_v} s \quad (3.6)$$

where C , S and V represent the center of gravity of the reinforced concrete slab, the steel girder and the composite model, respectively. s , s_c and s_s are the distances between these parameters, being s equal to 1.3825m. The moment of inertia of the composite cross-section, represented as a homogeneous material is calculated by summing the moments in various components about the neutral axis as follows,

Table 3.5 Structural properties of slab and girder

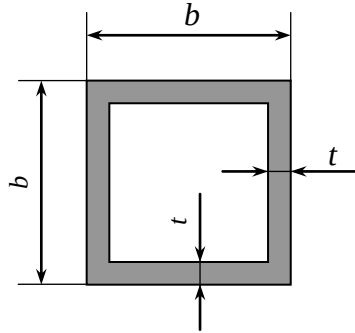
Component	Area (m^2)	Moment of inertia (m^4)
Concrete slab	$A_c = 3.27250$	$I_c = 0.020097$
Steel girder	$A_s = 0.06204$	$I_s = 0.018960$

**Fig. 3.14** Composite cross section

$$I_v = I_s + A_s \cdot s_s^2 + \frac{1}{n} (I_c + A_c \cdot s_c^2) \quad (3.7)$$

being obtained the results: $s_c = 0.2621$ m, $s_s = 1.1204$ m, $I_v = 0.1963$ m⁴.

And finally, the parameters b and t of the equivalent model, the box cross section model represented in **Fig. 3.15**, are calculated by Eqs. 3.8 and 3.9 to obtain approximately same values of area and moment of inertia,

**Fig. 3.15** Equivalent box cross-section model of superstructure

$$A_v = b^2 - (b - 2t)^2 \quad (3.8)$$

$$I_v = \frac{b^4 - (b - 2t)^4}{12} \quad (3.9)$$

being obtained values of $b = 1612$ mm, and $t = 82$ mm,

Superstructure is modeled with a box cross-section element, which represents the composite effect of the continuous steel reinforced concrete deck and the steel girders, identified as a homogeneous material. This analytical model plenty represents their mechanical and structural characteristics, and is considered adequate for its implementation in the finite element analysis.

(2) Appendix B: Calculation of pier cross section

Cross-section geometry dimensions of the bridge piers are calculated based on the stability criteria under static loading. Although steel box sections possess high flexural capacity, their cross-sections, composed of thin-walled plates, are susceptible to local buckling if the members are not adequately proportioned or detailed. The parameters governing the structural behavior of members composed of thin-walled plates are the width/thickness ratios of the plates and the pier slenderness parameter [36]. Size and thickness of the pier plate elements are selected considering that width/thickness ratios of the members must not exceed specified limits for box-section design, usually achieved by welding transverse to the thin-walled plates.

The seismic design of the bridge piers has been based on the seismic coefficient method given in the Specifications for Highway Bridges in Japan (Part II) [37]. Firstly, the design horizontal seismic coefficient is calculated.

(a) Design Horizontal Seismic Coefficient

The response of a bridge to earthquake ground motions varies according to the strength of the earthquake motion, its periodicity and duration, the ground type, structural type of the bridge, and particularly the structure of its foundation. The standards for the design horizontal seismic coefficient are established accounting for these elements and are also in accordance with the stipulations concerning seismic design in the previous Specification for Seismic Design (February 1990) [37]. The standard values for the design horizontal seismic coefficient have been established by integrating past practices and experience, and are based on the results, accounting for the acceleration response spectrum curves by ground type found through analysis of the strong motion records of 394 components observed in Japan, the characteristics of bridge damage caused in recent earthquakes, and other facts obtained through observations and experimental studies.

The design horizontal seismic coefficient is calculated by the following equation

$$k_h = c_z k_{h0} \quad (3.10)$$

where k_h is the design horizontal seismic coefficient used with the seismic coefficient method, k_{h0} is the standard value of the design horizontal seismic coefficient used with the seismic coefficient method, based on **Table 3.6**, and c_z is a modification factor for zone (=1.0). The design horizontal seismic coefficient is taken, $k_h = 0.20$.

Table 3.6 Standard values of the design horizontal seismic coefficient

Ground Classification	Value of k_{h0} for the Natural Period T (s)		
	$T < 0.1$	$0.1 \leq T \leq 1.1$	$1.1 < T$
Class I	$k_{h0} = 0.431 \cdot T^{1/3}$ But, $k_{h0} \leq 0.16$	$k_{h0} = 0.2$	$k_{h0} = 0.213T^{2/3}$
	$T < 0.2$	$0.2 \leq T \leq 1.3$	$1.3 < T$
Class II	$k_{h0} = 0.427 \cdot T^{1/3}$ But, $k_{h0} \leq 0.20$	$k_{h0} = 0.25$	$k_{h0} = 0.298T^{2/3}$
	$T < 0.34$	$0.34 \leq T \leq 1.5$	$1.5 < T$
Class III	$k_{h0} = 0.430 \cdot T^{1/3}$ But, $k_{h0} \leq 0.24$	$k_{h0} = 0.3$	$k_{h0} = 0.393T^{2/3}$

The equivalent lateral force P , acting at the top of the pier and representing the effect of earthquake loads is calculated. This lateral force depends on the weight of the superstructure W , and it is expressed on Eq. 3.11.

$$P = k_h W = 0.20 \times 8840 = 1768 \text{ kN} \quad (3.11)$$

The bending moment, maximum at the base of the bridge pier, depends on the pier height H and it is calculated using Eq. 3.12.

$$M = P H = 1768 \times 20 = 35360 \text{ kNm} \quad (3.12)$$

The compressive stress, σ_c , and the bending compressive stress are calculated

$$\sigma_b = \frac{M}{I} y = \frac{35360}{0.3798} \times 1.15 = 107066.9 \text{ kN/m}^2 = 107.1 \text{ N/mm}^2 \quad (3.13)$$

$$\sigma_c = \frac{R}{A} = \frac{4125}{0.4500} = 12277.8 \text{ kN/m}^2 = 9.2 \text{ N/mm}^2 \quad (3.14)$$

where M is the bending moment at the pier base previously calculated, I is the cross-section moment of inertia, y is the distance to the external cross-sectional fiber, R is the axial compressive load, and A is the cross-sectional area.

(b) Allowable Stress Evaluation

The allowable stress is evaluated based on the criteria,

$$\sigma_a \geq \sigma \quad (3.15)$$

The allowable axial compressive stress without local buckling is calculated using Eq. 3.16,

$$\sigma_{cag} = 125 - 0.68 \cdot \left(\frac{l}{r} - 19 \right) = 125 - 0.68 \times \left(\frac{42}{0.9135} - 19 \right) = 106.0 \text{ N/mm}^2 \quad (3.16)$$

where r is the radius of gyration and l is the effective length of the pier.

The local buckling allowable stress, σ_{cal} , is

$$\sigma_{cal} = 125 \text{ N/mm}^2 \quad (3.17)$$

the upper limit of the allowable axial compressive stress without local buckling is

$$\sigma_{cao} = 125 \text{ N/mm}^2 \quad (3.18)$$

and the allowable axial compressive stress is calculated by using Eq. 3.19

$$\sigma_{ca} = \frac{\sigma_{cag} \cdot \sigma_{cal}}{\sigma_{cao}} = \frac{106 \cdot 125}{125} = 106 \text{ N/mm}^2 \quad (3.19)$$

$$\sigma_{bag} = 140 \text{ N/mm}^2 \quad (3.20)$$

The allowable Euler buckling stress is calculated with Eq. 3.21

$$\sigma_{ea} = 12000000 \cdot \left(\frac{r}{l}\right)^2 = 12000000 \times \left(\frac{0.9135}{42}\right)^2 = 556 \text{ N/mm}^2 \quad (3.21)$$

where r is the radius of gyration and l is the effective length of the pier.

And finally verifications stipulated in the Specifications for the geometrical characteristics of pier cross-section is carried out by using Eqs. 3.22 and 3.23

$$\frac{\sigma_c}{\sigma_{ca}} + \frac{\sigma_b}{\sigma_{bag} \left(1 - \frac{\sigma_c}{\sigma_{ea}}\right)} \leq 1 \quad (3.22)$$

$$\sigma_c + \frac{\sigma_b}{1 - \frac{\sigma_c}{\sigma_{ea}}} \leq \sigma_{cal} \quad (3.23)$$

and numerically,

$$\frac{9.2}{106} + \frac{107.1}{125 \times \left(1 - \frac{9.2}{556}\right)} = 0.958 \leq 1 \quad (3.24)$$

$$9.2 + \frac{107.1}{1 - \frac{9.2}{556}} = 118.1 \leq \sigma_{cal} = 125 \text{ N/mm}^2 \quad (3.25)$$

The yield moment of the pier, M_{e0} , is calculated through Eq. 3.26, where σ_y is the yield stress and a , b , and c are geometrical properties represented in **Fig. 3.16**.

$$M_{e0} = \frac{4\sigma_y}{3c} (c^3 d - (c - 2e)^3 (d - 2e)) = 77.6 \text{ MNm} \quad (3.26)$$

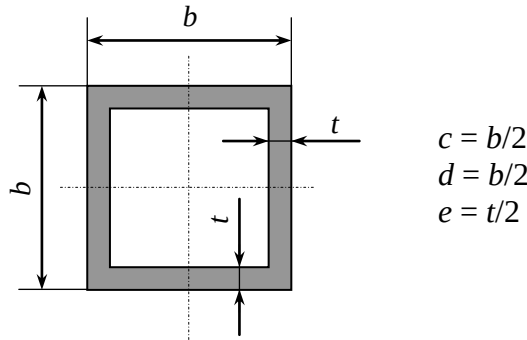


Fig. 3.16 Equivalent box cross-section model of bridge piers

References

- [1] Seo, J., Linzell, D. G.: Nonlinear Seismic Response and Parametric Examination of Horizontally Curved Steel Bridges Using 3D Computational Models, *Journal of Bridge Engineering*, ASCE, Vol. 18, No. 3, pp. 220-231, 2013.
- [2] Jerome, F. H., Dan, K., Luis, P.: Measured behavior of a curved composite I-girder bridge, *Journal of Constructional Steel Research*, Vol. 66, No. 3, pp. 351-368, 2010.
- [3] Seo, J., Linzell, D. G., Horizontally curved steel bridge seismic vulnerability assessment, *Engineering Structures*, Vol. 34, pp. 21-32, 2012.
- [4] Zureick, A., Linzell, D., Leon, R.T., Burrell, J.: Curved steel I-girder bridges: experimental and analytical studies, *Engineering Structures*, Vol. 22, No. 2, pp. 180-190, 2000.
- [5] Robiul Awall, Md., Hayashikawa, T., Matsumoto, T., He, X. W.: Effects of bottom bracings on torsional dynamic characteristics of horizontally curved twin I-girder bridges with different curvatures, *Earthquake Engineering and Engineering Vibration*, Vol. 11, No. 2, pp. 149-162, 2012.
- [6] Mendez Galindo, C.: Seismic Response Of Curved Highway Viaducts With Isolation Bearings and Cable Restrainers, *Doctoral Dissertation*, Graduate School of Engineering, Hokkaido University, Japan, February 2010.
- [7] Mendez Galindo, C., Hayashikawa, T., Ruiz Julian, D.: Curvature effect on seismic response of curved highway viaducts equipped with unseating cable restrainers, *Journal of Structural Engineering*, JSCE, Vol. 55A, pp. 699–708, 2009.
- [8] Itani, A.M., Bruneau, M., Carden, L., Buckle, I. G.: Seismic behavior of steel girder bridge superstructures, *Journal of Bridge Engineering*, ASCE, Vol. 9, No. 3, pp. 243–249, 2004.
- [9] Hayashikawa, T., Hirooka, T., Ikeda, K., Abdel Raheem, S. E.: Dynamic behaviour of viaducts with PC cables in consideration of pounding of girders and energy absorption, *JSCE Proceedings of Hokkaido*, Vol. 59, Japan Soc Civil Eng, pp. 50-53, 2003.
- [10] Ruangrassamee, A., Kawashima, K.: Relative displacement response spectra with pounding effect, *Earthquake Engineering and Structural Dynamics*, Vol. 30, No. 10, pp. 1511-1538, 2001.
- [11] Taflanidis, A.A.: Optimal probabilistic design of seismic dampers for the protection of isolated bridges against near-fault seismic excitations, *Engineering Structures*, Vol. 33, No. 12, pp. 3496-508, 2011.
- [12] Zanardo, G., Hao, H., Modena, C.: Seismic response of multi-span simply supported bridges to a spatially varying earthquake ground motion, *Earthquake Engineering and Structural Dynamics*, Vol. 31, No. 6, pp. 1325-1345, 2002.
- [13] Watanabe, E., Sugiura, K., Nagata, K., Kitane, Y.: Performances and damages to steel structures during the 1995 Hyogoken-Nanbu earthquake, *Engineering Structures*, Vol. 20, No. 4-6, pp. 282-90, 1998.
- [14] Scawthorn, C., Yanev, P. I.: Preliminary report 17 January 1995, Hyogo-ken Nambu, Japanese earthquake, *Engineering Structures*, Vol. 17, No. 3, pp. 146-57, 1995.
- [15] Kawashima, K., Unjoh, S.: Impact of Hanshin/Awaji earthquake on seismic design and seismic strengthening of highway bridges, *Structural Engineering/Earthquake Engineering*, JSCE, Vol. 13, No. 2, pp. 211s–240s, 1996.
- [16] Otsuka, H., Unjoh, S., Terayama, T., Hoshikuma, J., Kosa, K.: Damage to highway bridges by the 1995 Hyogoken Nanbu earthquake and the retrofit of highway bridges in Japan, In: *Proceedings of the 3rd U.S.-Japan Workshop on Seismic Retrofit of Bridges*, Osaka, Japan, pp. 10-11, 1996.
- [17] Maleki, S.: Effect of deck and support stiffness on seismic response of slab-girder bridges, *Engineering Structures*, Vol. 24, No. 2, pp. 219-226, 2002.
- [18] Dicleli, M.: Simplified seismic analysis of a class of regular steel bridges, *Engineering Structures*, Vol. 24, No. 11, pp. 1409-1422, 2002.
- [19] Hsu, H. L. and Juang, J. L.: Performance of thin-walled box columns strengthened with internal braces, *Thin-Walled Structures*, Vol. 37, No. 3, pp. 241-258, 2000.
- [20] Japan Road Association (JRA), *Specification for Highway Bridges - Part V Seismic Design*, Tokyo (Japan), Maruzen, 2002.
- [21] Japan Road Association (JRA), *Specification for Highway Bridges - Part II Steel Bridges*, Chapter 1-13, Tokyo (Japan), Maruzen, 1996.
- [22] Ishizawa, T. and Iura, M.: Analysis of tubular steel bridge piers, *Earthquake Engineering and Structural Dynamics*, Vol. 34, No. 8, pp. 985-1004, 2005.
- [23] Tanimura, S., Sato, T., Umeda, T., Mimura, K. and Yoshikawa, O.: A note of dynamic fracture of the bridge bearing due to the great Hanshin-Awaji earthquake, *International Journal of Impact Engineering*, Vol. 27, No. 2, pp. 153-160, 2002.
- [24] Jankowski, R., Wilde, K., Fujino, Y.: Reduction of pounding effects in elevated bridges during earthquakes, *Earthquake Engineering and Structural Dynamics*, Vol. 29, No. 2, pp. 195-212, 2000.
- [25] Jankowski, R., Wilde, K., Fujino, Y.: Pounding of superstructure segments in isolated elevated bridge

- during earthquakes, *Earthquake Engineering and Structural Dynamics*, Vol. 27, No. 5, pp. 487-502, 1998.
- [26] Kelly, J. M.: A seismic base isolation: review and bibliography, *Soil Dynamics and Earthquake Engineering*, Vol. 5, No. 4, pp. 202-216, 1986.
 - [27] Bruneau, M.: Performance of steel bridges during the 1995 Hyogoken-Nanbu (Kobe, Japan) earthquake-a North American perspective, *Engineering Structures*, Vol. 20, No. 12, pp. 1063-1078, 1998.
 - [28] Japan Society of Civil Engineers: Proposal on earthquake resistance for civil engineering structures (Special Task Committee of Earthquake Resistance of Civil Engineering Structures), The 1995 Hyogoken-Nambu earthquake investigation into damage of civil engineering. Earthquake Engineering Committee, Japan Society of Civil Engineers, 1996.
 - [29] Toda, S., Hataya, R., Abe, S. and Miyakoshi, K.: The 1995 Kobe earthquake and problems of evaluation of active faults in Japan, *Engineering Geology*, Vol. 43, No. 2-3, pp. 151-167, 1996.
 - [30] Somerville, P. G.: Magnitude scaling of the near fault rupture directivity pulse, *Physics of the Earth and Planetary Interiors*, Vol. 137, No. 1-4, pp. 201-212, 2003.
 - [31] Dicleli, M., Bruneau, M.: Seismic performance of multispan simply supported slab-on-girder steel highway bridges, *Engineering Structures*, Vol. 17, No. 1, pp. 4-14, 1995.
 - [32] Rashidi, S., Ala Saadeghvaziri, M.: Seismic modeling of multi-span simply-supported bridges using Adina, *Computers and Structures*, Vol. 64, No. 5-6, pp. 1025-1039, 1997.
 - [33] Zhu, P., Abe, M., Fujino, Y.: Evaluation of pounding countermeasures and serviceability of elevated bridges during seismic excitation using 3D modeling, *Earthquake Engineering and Structural Dynamics*, Vol. 33, No. 5, pp. 591-609, 2004.
 - [34] Kitada, T., Yamaguchi, T., Matsumura, M., Okada, J.: New technologies of steel bridges in Japan, *Journal of Constructional Steel Research*, Vol. 58, No. 1, pp. 21-70, 2002.
 - [35] Fujino, Y., Hashimoto, S., Abe, M.: Damage analysis of Hanshin expressway viaducts during 1995 Kobe earthquake. I: Residual inclination of reinforced concrete piers, *Journal of Bridge Engineering*, ASCE, Vol. 10, No. 1, pp. 45-53, 2005.
 - [36] Basu, B. and Gupta, V. K.: A damage-based definition of effective peak acceleration, *Earthquake Engineering and Structural Dynamics*, Vol. 27, No. 5, pp. 503-512, 1998.
 - [37] Japan Road Association (JRA), *Specification for Highway Bridges - Part V Seismic Design*, Tokyo (Japan), Maruzen, 1990.

SEISMIC RESPONSE OF HIGHWAY VIADUCTS EQUIPPED WITH ROLLER BEARINGS OF DIFFERENT STOPPER VALUES

4.1 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. 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].

The poor seismic performance of steel bearing supports has been highlighted due to the disastrous consequences for the overall bridge seismic performance. Failure of steel bearings resulted in collapse of highway viaducts during the 1995 Kobe earthquake [2, 3]. Moreover, traffic vehicle flow was impeded in some cases by superstructure falling on to the surface of the substructure. This severe damage to highway viaducts that resulted from inadequate performance of bearing supports emphasizes the need to carefully evaluate the role of bearings as important bridge structural elements.

Traditional steel bearings are very common, but they are easy to be broken in earthquakes. Recently roller bearings equipped with stopper are installed on top of piers. Since stopper can be easily installed and the price is not expensive, so it has wide application prospect in future. Therefore, the purpose of the present study is to analyze the response of curved steel viaduct installed of one type of cable restrainer, based on roller bearing supports equipped with stoppers. The relationship between different stopper values and the different types of damages is obtained. According to the relationship, the advice for the seismic design of viaduct based on the appropriate stopper values could be put forward.

4.2 Unseating Prevention System

4.2.1 Unseating prevention mechanism

Spans dropped off from too narrow support seats are the major cause of bridge collapse in the event of strong earthquake ground motion. Even non-collapsing, residual expansion joint separations may cause very costly disruption to traffic on major transportation arteries and is simply unacceptable [4]. An incapacitated bridge is a life-safety issue due directly to the damaged bridge and the possible loss of a lifeline. A lost bridge can be expensive to repair at a time when a region's resources are most strained and a compromised commercial route could result in losses to the regional economy [5].

Continuity of bridge superstructures is extremely important and is the easiest and cheapest insurance to obtain. If structures are not continuous, they may be tied together at deck joints to prevent them from pulling apart and collapsing during an earthquake [6]. The purpose is not only to prevent collapse, but also to provide serviceability after a major earthquake. Since the HN Earthquake, unseating prevention systems have been gradually incorporated to highway bridges in Japan, linking spans at the expansion joints, to prevent bridge spans from unseating during earthquakes [7]. Several retrofit programs are still in the process of installing restrainer devices as a measure of upgrading the seismic performance of the bridges. These retrofit measures, which focus on the superstructure, are generally less expensive and easier to perform compared to retrofits for the piers and/or foundations [8].

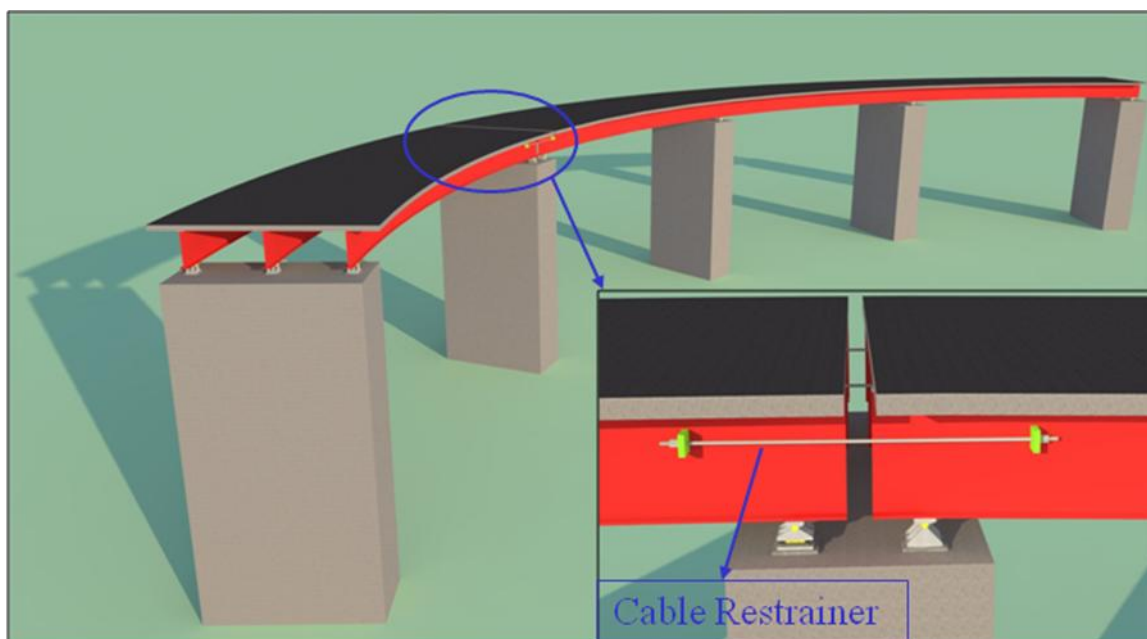


Fig. 4.1 3D views of curved viaduct equipped with cable restrainers

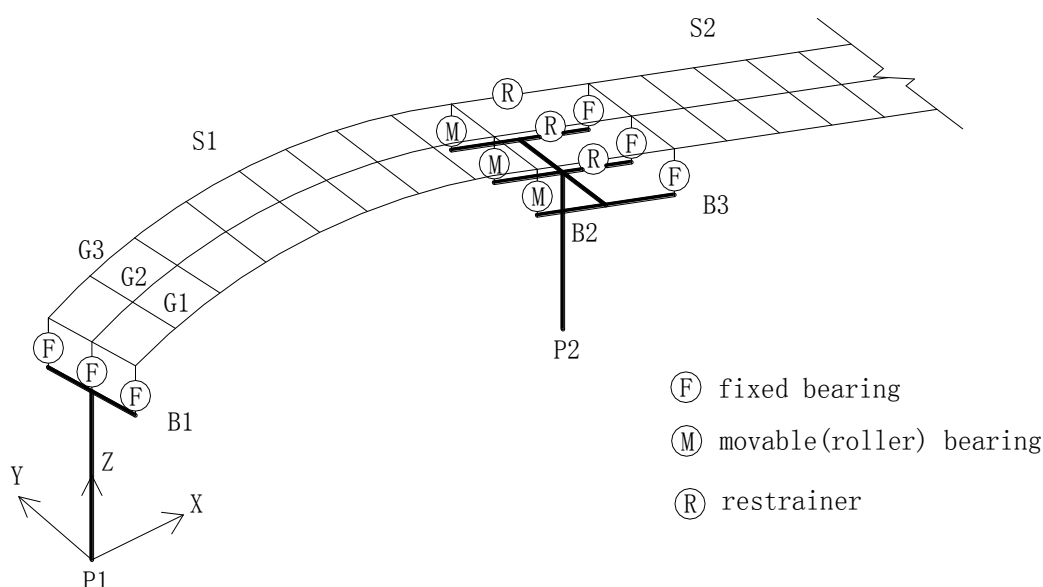
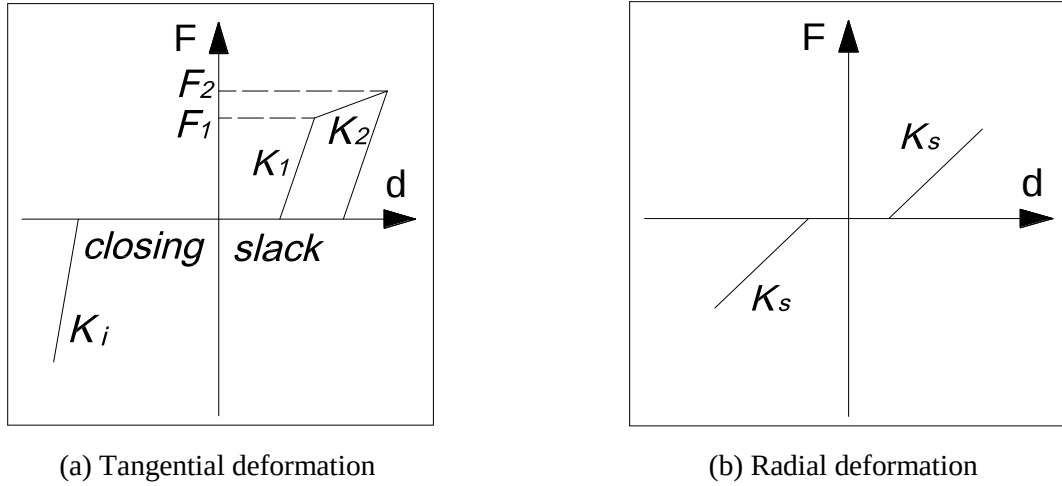


Fig. 4.2 Finite element model detail of curved viaduct equipped with cable restrainers

The Japanese Specifications for Highway Bridges establishes the unseating prevention system as a fail-safe structure to prevent the superstructure from collapsing in the event of unexpected failure of the structural systems. The unseating prevention system is intended to supplement the role of the seat length of a girder at the support, coming into operation before an unexpected relative displacement between the superstructure and the substructure and reaches the seat length of a girder at the support. From the experience of past seismic disasters, the Specifications give priority and particular careful examination with regard to the unseating prevention system for complex bridges with substantially different substructure types (i.e. bearing supports), bridges in which sizes of adjoining superstructures are substantially different, and bridges with small curvature radius. The unseating prevention system is effective to reduce the possibility of the unseating of girders.

**Fig. 4.3** Analytical model of cable restrainers**Table 4.1** Structural properties of cable restrainer

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

4.2.2 Analytical model of cable restrainer

Cable restrainers are relatively simple structures. Previous research on cable restrainer performance and design has included laboratory testing of cable restrainers [9] and evaluation and development of design procedures [10-15]. Post-earthquake evaluations from the 1989 Loma Prieta and the 1994 Northridge Earthquakes have shown that many cable restrainers were observed to have worked effectively during the earthquakes [16], preventing simply-supported spans from falling from their supports.

In this analysis, cable restrainers units are anchored to the three girder ends connecting both adjacent superstructures across the expansion joint as shown in **Fig. 4.1**. The detail of the finite element model is presented in **Fig. 4.2**. Seismic restrainers, illustrated in **Fig. 4.3a**, have been tangentially modeled as tension-only spring elements provided with a slack of 4 cm, a value fitted to accommodate the expected deck thermal movements limiting the activation of the system specifically for earthquake loading. 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. 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. 4.3b**. In addition, the bridge seismic performance has been evaluated on one type of cable restrainer (R4), structural properties of cable restrainer (R4) is shown in **Table 4.1**.

4.3 Analytical Model of Roller Bearings with Different Stopper Values

The bridge seismic performance has been evaluated on models equipped with steel bearings (SB), as described in Section 3.2. Steel stoppers are installed at both sides of roller

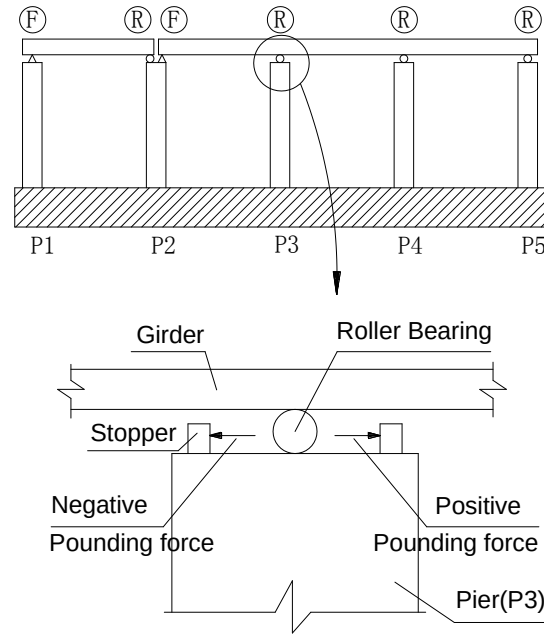


Fig. 4.4 Schematical diagram of pounding force between roller bearing and stopper

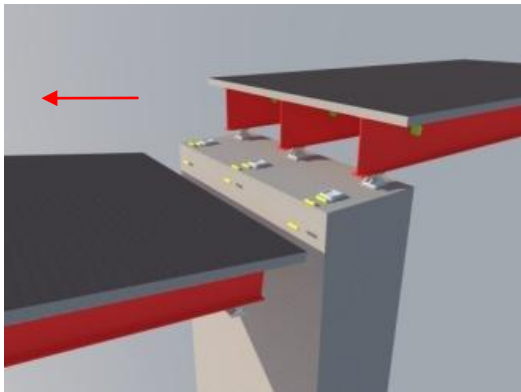


Fig. 4.5 Schematical diagram of deck unseating

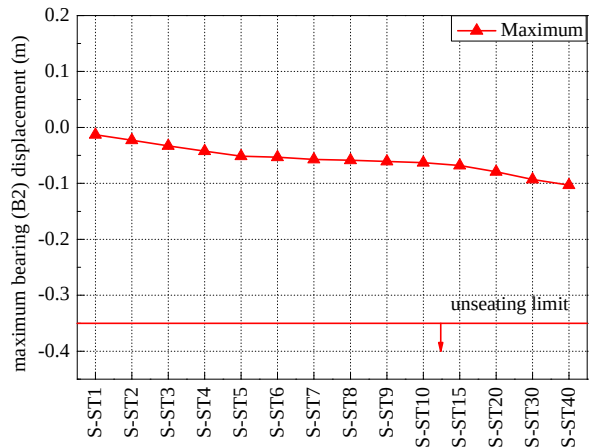


Fig. 4.6 Possibility of deck unseating damage

bearing supports in longitudinal direction. In this analysis, the stopper values are variable. Besides the stopper values are divided into three parts. In the first part, stopper values are from 1 cm to 4 cm which are less than cable restrainer slack value of 4 cm. In the second part, stopper values are from 5 cm to 10 cm with the increase of 1 cm which is larger than cable restrainer slack value and less than expansion joint gap of 10 cm. In the third part, stopper values are larger than expansion joint gap including 15 cm, 20 cm, 30 cm and 40 cm. Based on the different stopper values, the calculation includes fourteen cases in order to observe the effect of stopper value.

4.4 Numerical Results and Discussion

The overall three-dimensional seismic response of the viaduct is investigated in detail through non-linear dynamic response analysis. In order to explain the following analysis

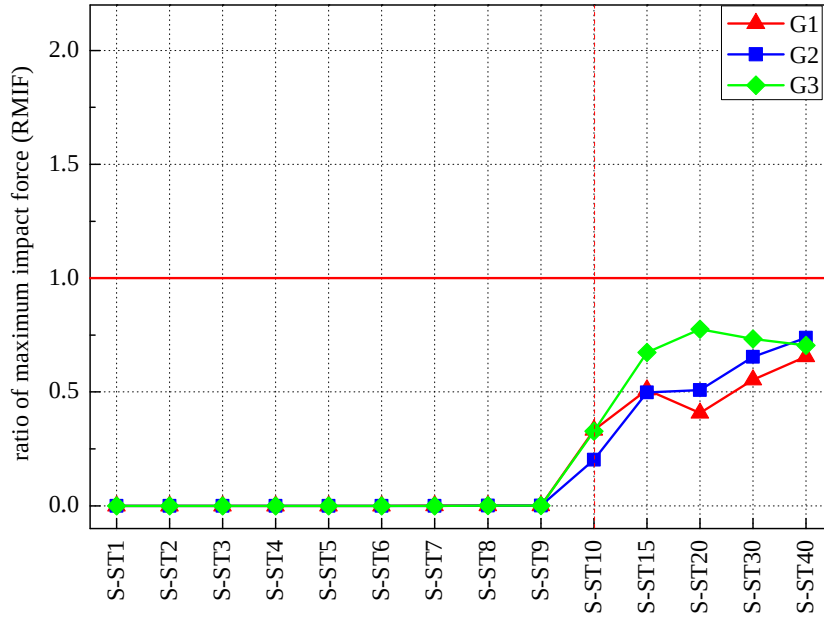


Fig. 4.7 Pounding damage

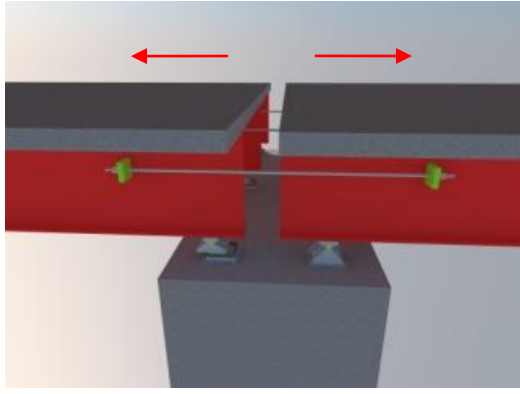
results clearly, pounding forces in positive and negative directions are shown in **Fig. 4.4**. Pounding forces in positive direction could lead to positive displacements on top of piers. However, pounding forces in negative direction could cause negative displacements on top of piers. Kobe earthquake measured by the Takatori (TAK) stations is adopted as an input ground motion.

4.4.1 Deck unseating damage

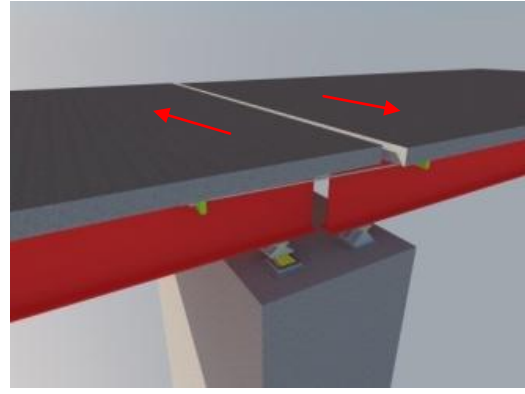
One of the most catastrophic seismic damages to bridge superstructures is the failure due to deck unseating [17, 18] as shown in **Fig. 4.5**. During an earthquake, adjacent spans can vibrate out-of-phase, resulting in large relative displacements at expansion joints. In simple supported span and continuous span, the induced relative displacements to steel roller bearings can exceed the seat width at the pier top, causing the dislodgment of the rollers from the bearing assembly and the subsequent collapse due to deck superstructure unseating. The maximum roller bearing (B2) displacement in negative tangential direction has been established as the damage index to evaluate the potential possibility of deck unseating. A limit of 0.35 m has been considered to indicate the high unseating probability for existing bridges with narrow steel pier caps that provide short seat widths.

It can be seen from **Fig. 4.6** that collapse due to deck unseating does not occur for all cases. In the cases that stopper values are 1 cm, 2 cm and 3 cm, the maximum absolute steel bearing displacement in negative direction is same with stopper values respectively. Since the cable restrainer slack value is 4 cm which is larger than stopper values in these cases. So cable restrainer could not be activated when earthquake happens. In above cases, it is that the stopper value determines the maximum absolute steel bearing (B2) movements in negative direction. As stopper value increases, maximum absolute steel bearing (B2) movements in negative direction gradually increase.

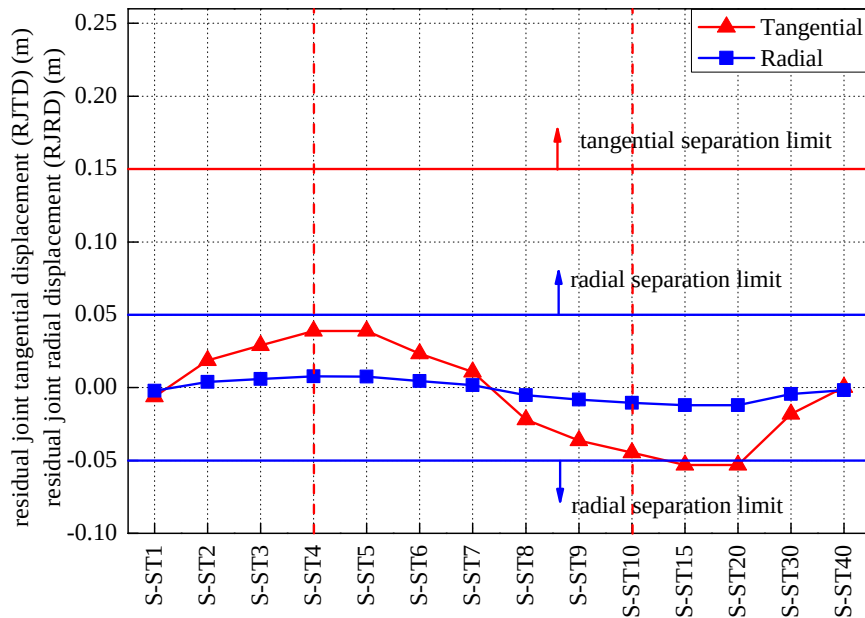
In the cases that stopper values are from 4 cm to 9 cm, cable restrainer and stopper work together to resist the external force. So the maximum absolute bearing displacements in negative direction are determined by all of them. Maximum absolute bearing displacements in negative direction increase with the increase of stopper value.



(a) Tangential opening



(a) Radial opening

Fig. 4.8 Schematical diagram of tangential and radial opening damage**Fig. 4.9** Tangential and radial joint residual opening

In other cases that stopper values are larger than 10 cm, the roller bearing (B2) does not impact stopper, cable restrainers and roller bearings on top of other piers resisting the external force. So the maximum absolute displacements in negative direction are determined by the cable restrainer's performance and the effect of pounding forces between roller bearing and stopper on top of other piers as earthquake happens. As stopper value increases, maximum absolute steel bearing (B2) movements in negative direction also increase.

4.4.2 Pounding damage

As stopper value is larger than expansion joint gap of 10 cm, pounding between superstructures occurs. Due to this pounding phenomenon a remarkable point to note is that, in addition to the expected local damage at colliding girders, high impact forces are transmitted to bearing supports located in the proximity of the expansion joint. The large spikes analytically observed in both, tangential and radial, components of reaction forces make the steel bearing supports particularly vulnerable to failure, which could result in the

collapse of the bridge. Ratios of Maximum Impact Force (RMIF) to the deck weight greater than 1.0 have been observed to provide a good estimation of significant transmitted forces to bearing supports.

According to the presented results in **Fig. 4.7**, ratio of maximum impact forces are smaller than 1 for all cases. As stopper values are smaller than 10 cm, pounding forces between girders are zero. Since expansion joint gap is 10 cm which is larger than stopper value in above cases, it is stopper that resists the impact force.

As stopper value is larger than 10 cm, there is impact force between girders. There is a sharp increased trend in the cases that stopper value is 10 cm. When stopper value is larger than 15 cm, the curve is becoming smooth and steady. Considering bridge curve alignment, the deck irregularly translates because it does not concentrate movement exclusively in the tangential direction. Impacts are observed to be non-uniformly distributed across the expansion joint, thus particular girders, especially G3, resist most of the impact force.

4.4.3 Tangential and radial joint residual opening damage

This damage is shown in **Fig. 4.8a** and **Fig. 4.8b**. Permanent tangential offsets of expansion joints observed at the end of recent earthquakes have substantially interfered with the post earthquake serviceability of highway viaducts. In several cases, traffic was impeded and the disruption of the bridge usability in the aftermath of the earthquakes resulted in a critical problem for rescue activities. The residual joint tangential displacement (RJTD) has been calculated in order to perform the post-earthquake serviceability evaluation on the viaduct. 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 [19]. Besides, a positive value of RJTD corresponds to an opening (outward) of the expansion joint while a negative value of RJTD corresponds to a closing (inward). 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, thus sensibly restricting the operability of the bridge in such critical moments.

It can be seen from **Fig. 4.9** that RJTD and RJRD are all under the limit value for all cases, so there are no damages of RJTD and RJRD in earthquakes.

As stopper value is from 1 cm to 4 cm, RJTD presents increasing trend in **Fig. 4.9**. Since the slack value of cable restrainer is 4 cm which is larger than above stopper value. In addition, expansion joint gap of 10 cm is also larger than stopper value. So cable restrainer does not work and pounding phenomenon between superstructures also does not happen. Finally, due to the influence of the pounding forces between roller bearing and stopper, RJTD presents increasing trend.

As stopper value is from 5 cm to 10 cm, RJTD presents decreasing trend. Pounding phenomenon between superstructures does not happen because of large expansion joint gap of 10 cm which is larger than stopper value. However, cable restrainer begins to be activated because of its small slack value (4 cm) which is smaller than stopper value. Based on the cable restrainer's tension effect, residual joint tangential displacement becomes small.

As stopper value is larger than 10 cm, RJTD presents increasing trend. Roller bearing (B2) does not impact on stopper, but pounding phenomenon between superstructures happens because expansion joint gap of 10 cm which is smaller than stopper value. So, cable restrainer and pounding phenomenon between superstructures play an important role in RJTD. Although pounding times between superstructures are almost same with cable restrainer tension times, pounding force is larger than cable restrainer tension. Finally, RJTD shows increasing trend.

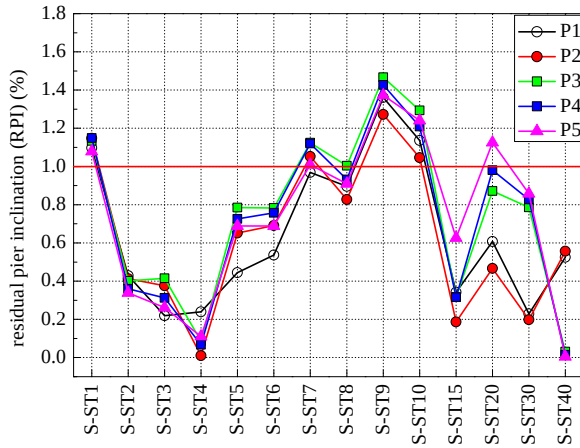


Fig. 4.10 Residual pier inclination

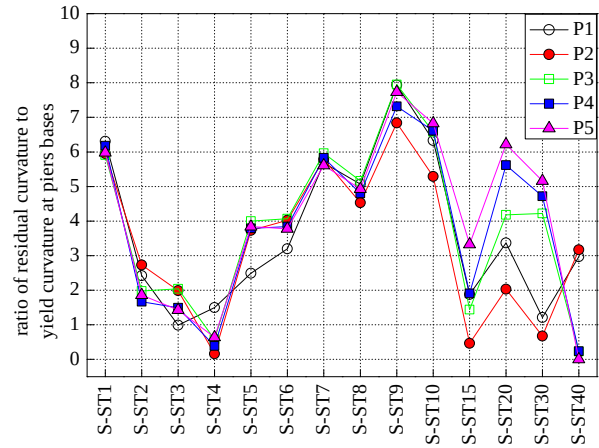


Fig. 4.11 Ratio of residual curvature to yield curvature

In addition, since radical direction of all bearings is restraint, residual joint radial displacement (RJRD) is near zero as shown in **Fig. 4.9**.

4.4.4 Pier damage

A large number of piers were damaged in 1995 Kobe earthquake. As a consequence, piers sustained significant residual deformations that were dependent on the maximum ductility reached during the seismic loading. Viaducts supported on piers with large residual inclination may lose their serviceability, even possibly results in collapse. 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]. Therefore, residual pier inclination (RPI) has been considered as an important damage index in this analysis.

Fig. 4.10 shows that pier inclination damage happens in some cases. The installation of lateral stoppers at both sides of roller bearing supports in longitudinal direction is found relatively effective since the maximum bending moment acting on the pier with fixed bearings tends to be reduced. However, adverse effects are appreciated for piers with roller supports, being the possibility of large residual displacement extended to all top of piers as shown in **Fig. 4.10**. Because large impact forces acting on stoppers of roller bearings are transferred to the piers with a sharp increment of bending moment at those piers bases. Consequently, all piers present residual pier inclination. In addition, residual pier inclination and residual curvature present almost same change trend as shown in **Fig. 4.10** and **Fig. 4.11**. Residual curvature is determined by curvature time-history, especially, those curvatures which are equal to yield curvature. Maximum curvature plays an important role in residual pier inclination to some extent.

As stopper value is from 1 cm to 4 cm, residual pier inclination decreases with stopper value increasing as shown in **Fig. 4.10**. Slack value of cable restrainer and closing value between superstructures are all larger than stopper value. So it is that stopper resists external forces.

As stopper value is from 5 cm to 9 cm, residual pier inclination increases in **Fig. 4.10**. It is stopper and cable restrainer that work together to resist external forces. At the beginning, top of piers presents positive direction trend for TAK input. As simple supported span move toward negative direction, cable restrainer resists the force instead of stopper. Cable restrainer transfer the negative pounding force to continuous span, do not directly apply on the pier 2. Besides as stopper value increases, maximum positive curvature at pier base increases. So, it

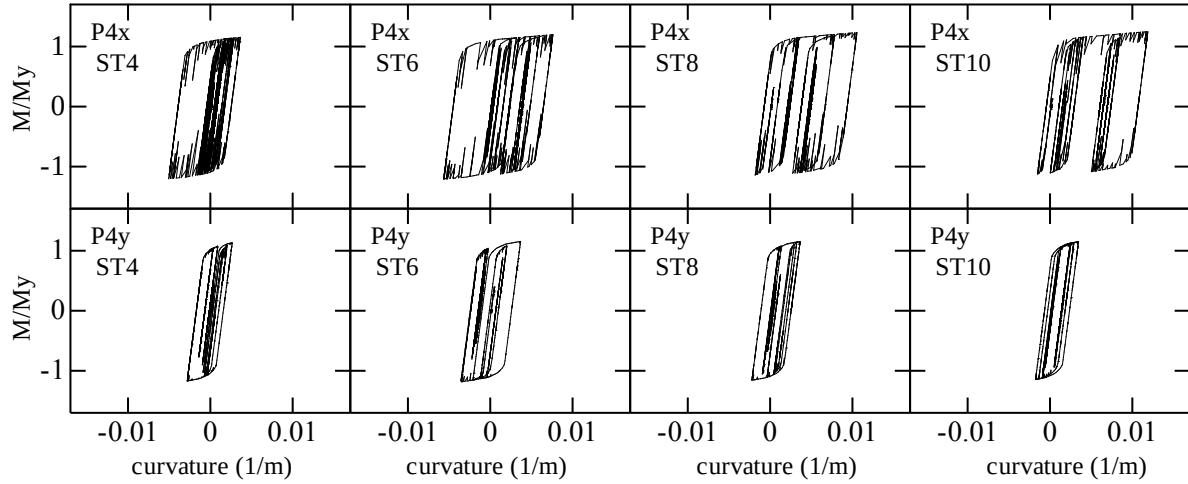


Fig. 4.12 Bending moment ratio-curvature relationship at pier (P4) base

results in increasing of residual curvature at piers bases as shown in **Fig. 4.11**. Finally, residual displacement on the top of piers present similar change trend with residual curvature at piers bases as shown in **Fig. 4.10** and **Fig. 4.11**.

As stopper value is larger than 10 cm, residual pier inclination presents fluctuating curve as shown in **Fig. 4.10**. Since stopper value is larger than 10 cm, there is no pounding force between roller bearing (B2) and stopper. In other words, there is not directive pounding force between simple supported span and pier 2. So, simple supported span and continuous span shall present respective different behavior because of different masses. In addition, the variation degree of different behavior shall increases because of directive pounding force between superstructures. The residual curvature at piers bases respectively presents different values for simple supported span and continuous span as shown in **Fig. 4.11**. Similarly, the piers of simple supported span and continuous span present obviously different residual pier inclination in **Fig. 4.10**.

Fortunately, when stopper values are from 2 cm to 6 cm, residual pier inclination is under 1% presenting no pier inclination damage. Although as stopper value is larger than expansion joint gap, residual pier inclination may present less value, it results in local damage at contact area because of pounding phenomenon between superstructures. Conclusively, the region, stopper value of nearby 4 cm which equal to slack value of cable restrainer, shows no damage.

4.4.5 Relationship of bending moment and curvature

During an earthquake, the section of the pier that suffers higher demands is the bottom one, where the bending moments reach to the highest value. The maximum curvatures transmitted to the base of the pier can be considerate as an appropriate measure of seismic structural damage. In this analysis, central pier (P4) of continuous span, supporting more weight, shall afford more inertial force during earthquakes, is likely subjected to large pounding forces between roller bearing and stopper during earthquakes. In addition, stopper value is larger than 4 cm that allows for contraction and expansions of the road deck from creep, shrinkage and temperature fluctuations without generating constraint forces in the structure. Besides stopper value is less than 10 cm which could eliminate local damage due to pounding between superstructures. Therefore, the relationship between curvature and the ratio of bending moment (M) to yield bending moment (M_y) for P4 in X and Y direction in case of

4 cm, 6 cm, 8 cm and 10 cm are shown **Fig. 4.12**. The relationship of bending moment and curvature at other piers bases present similar results.

It can be seen from **Fig. 4.12** that the absolute value of M/M_y overpasses 1 in both X and Y directions, which indicates plastic deformation at pier base in these two directions. Maximum bending moment and maximum curvature in X direction is larger than those in Y direction respectively. Since all the bearings are restrained in Y direction that leads to relatively less bearing forces. So, maximum bending moment and maximum curvature at piers bases in X direction play more important effect on the damages of piers bases. Moreover, as stopper value increases, maximum bending moment and maximum curvature at piers bases in X direction increase.

4.5 Conclusions

The effectiveness of roller bearing stopper as preventing roller bearing excessive movement is evaluated in this study. In order to perform a complete investigation of its role on the seismic performance of the viaduct, the dynamic behavior of the structure has been analyzed when the roller bearings are equipped with different stopper values. The presented results provide sufficient evidence for the following conclusions:

(1) The calculated results clearly demonstrate that roller bearing with stopper value could provide an effective means for overcoming the potential problems associated with deck unseating. In addition, as stopper value decreases, the possibility of deck unseating damage decreases.

(2) It is additionally highlighted that as stopper value is larger than 10 cm, there is obvious impact force between girders, but the ratio of maximum impact force is below 1 which is not problem for the bridge's safety. Therefore, stopper is appropriate device for reducing the ratio of maximum impact force between girders.

(3) It is found that that viaduct, supported on roller bearings with stoppers, does not present tangential and radial joint residual opening damages at expansion joint during earthquakes. The tension of cable restrainer could reduce residual joint tangential displacement for opening direction.

(4) According to evaluation of pier inclination damage, the results could be divided into three parts. In first part, as stopper value is less than cable restrainer's slack value of 4 cm, residual pier inclination decreases with stopper value increasing. In second part, as stopper value is larger than cable restrainer's slack value but less than expansion joint gap of 10 cm, residual pier inclination increases with stopper value increasing. In third part, as stopper value is larger than expansion joint gap of 10 cm, residual pier inclination presents fluctuating curve. In addition, residual pier inclination and residual curvature at piers bases present almost same change trend in case of different stopper values. In other words, residual curvature at piers bases plays an important role in residual pier inclination.

(5) As stopper value is larger than cable restrainer's value and less than expansion joint gap, plastic deformations at piers bases appear in both X and Y directions during earthquakes. Maximum bending moment and maximum curvature at pier base in X direction is larger than those in Y direction respectively. So, maximum bending moment and maximum curvature at piers bases in X direction play more important effect on the damages of piers bases. As stopper value increases, maximum bending moment and maximum curvature at piers bases in X direction increase.

References

- [1] Japan Road Association (JRA), *Specification for Highway Bridges - Part V Seismic Design*, Tokyo (Japan), Maruzen, 2002.

- [2] Watanabe, E., Sugiura, K., Nagata, K. and Kitane, Y.: Performances and damages to steel structures during the 1995 Hyogoken-Nanbu earthquake, *Engineering Structures*, Vol. 20, No. 4-6, pp. 282-290, 1998.
- [3] Scawthorn, C., and Yanev, P.I.: Preliminary report 17 January 1995, Hyogo-ken Nambu, Japanese earthquake, *Engineering Structures*, Vol.17, No.3, pp. 146-157, 1995.
- [4] Zhang, R.: Chapter 41: Seismic isolation and supplemental energy dissipation. In: Chen, W. F., and Duan, editors. *Bridge Engineering Handbook*, Boca Raton, CRC Press, 1999.
- [5] Rose, A., and Lim, D.: Business interruption losses from natural hazards: conceptual and methodological issues in the case of the Northridge earthquake, *Environmental Hazards*, Vol. 4, pp. 1-14, 2002.
- [6] Takeno, S., Ohno, H., and Izuno, K.: Velocity-based design of seismic unseating prevention cable and shock absorber for bridges, *Structural Engineering and Earthquake Engineering*, Vol. 21, No. 2, pp. 175-188, 2004.
- [7] Hayashi, H., Marui, T., Taniguchi, N., and Kayano, S.: Restoration of Hanshin Expressway after Kobe/Awaji earthquake – challenge of 623 days before opening, *Cement and Concrete Composites*, Vol. 22, pp. 29-38, 2000.
- [8] DesRoches, R., Choi, E., Leon, R. T., and Pfeifer, T. A.: Seismic response of multiple span steel bridges in central and southeastern United States. II: Retrofitted, *Journal of Bridge Engineering ASCE*, Vol. 9, No. 5, pp. 473-479, 2004.
- [9] DesRoches, R., Pfeifer, T., Leon, R. T., and Lam, T.: Full-scale tests of seismic cable restrainer retrofits for simply supported bridges, *Journal of Bridge Engineering ASCE*, Vol. 8, No. 4, pp. 191-198, 2003.
- [10] Saiidi, M., Maragakis, E.A., and Feng, S.: An evaluation of the current Caltrans seismic restrainer design method, *Rep. No. CCEER-92-8*, Civil Engineering Department, University of Nevada, Reno, 1992.
- [11] Saiidi, M., Maragakis, E., and Feng, S.: Parameters in bridge restrainer design for seismic retrofit, *Journal of Structural Engineering ASCE*, Vol. 122, No. 1, pp. 61-68, 1996.
- [12] Fenves, G., and DesRoches, R.: Response of the Northwest Connector in the Landers and Big Bear earthquakes, *Rep. No. UCB/EERC-94/12*, Earthquake Engineering Research Center, University of California, Berkeley, 1994.
- [13] Yang, Y. S., Priestley, M.J. N., and Ricles, J. M.: Longitudinal seismic response of bridge frames connected by restrainers, *Rep. No. SSRP-94/09*, Structural System Research, University of California, San Diego, San Diego, 1994.
- [14] Trochalakis, P., Eberhard, M.O., and Stanton, J. F.: Evaluation and design of seismic restrainers for in-span hinges, *Rep. No. WA-RD 387.1*, Washington State Transportation Center, Seattle, 1995.
- [15] Priestley, M. J. N., Seible, F., and Calvi, M.: Seismic design and retrofit of bridges, *Wiley*, New York, 1996.
- [16] Yashinsky, M.: Chapter 30: Earthquake damage to structures. In: Chen, W. F., editor. *Structural Engineering Handbook*, Boca Raton, CRC Press, 1999.
- [17] Dicleli, M., Bruneau, M.: Seismic performance of multispan simply supported slab-on-girder steel highway bridges, *Engineering Structures*, Vol.17, No.1, pp. 4-14, 1995.
- [18] Rashidi, S., Ala Saadeghvaziri, M.: Seismic modeling of multi-span simply-supported bridges using Adina, *Computers & Structures*, Vol. 64, No. 5-6, pp. 1025-1039, 1997.
- [19] Zhu, P., Abe, M., Fujino, Y.: Evaluation of pounding countermeasures and serviceability of elevated bridges during seismic excitation using 3D modeling, *Earthquake Engineering & Structural Dynamics*, Vol. 33, No. 5, pp. 591-609, 2004.

EFFECTIVENESS OF CABLE RESTRAINERS ON SEISMIC RESPONSE

5.1 Introduction

One of the most catastrophic seismic damages to bridge superstructures is the failure due to deck unseating. The application of cable restrainers on the viaduct could significantly reduce the possibility of deck unseating damage. Even when the structures have been provided with enough seating lengths in order to support the structure, the occurrence of unexpected seismic forces requires the installation of cable restrainers. This technology limits relative displacements at the expansion joint, minimizing by this the possibility of collapse of superstructures. One of the most remarkable advantages of using cable restrainers is that those cables provide a fail-safe mechanism by supporting a fallen girder unseated in the event of a severe earthquake [1]. Besides, the installation of cable restrainers is very cheap. In addition, their performance has been generally satisfactory during recent earthquakes [2].

The study combines the use of non-linear dynamic response analysis with a precise three-dimensional non-linear modeling of the entire bridge structural system to evaluate the global dynamic behavior of the bridge system with particular emphasis focused on the interaction of unseating prevention devices with the remaining bridge structure. The effectiveness of cable restrainers to mitigate earthquake damage of highway viaducts is examined under the action of severe earthquake ground motions [3]. There are two important parameters of cable restrainer including stiffness and slack value. The two parameters play an important role in the behavior of cable restrainers.

Therefore, there are two main purposes of this chapter study.

One is to analyze the overall performance of highway viaducts with different kinds of cable restrainers. Different kinds of cable restrainers have different stiffness. So the effect of cable restrainer's stiffness on deck unseating damage, tangential and radial joint residual opening damage, and pier damage is analyzed. The relationship between viaduct damage and cable restrainer's stiffness will be found. According to the relationship, provide advice for the seismic design of curved viaduct based on the cable restrainer with appropriate stiffness. In addition, a comparison between restrained and unrestrained curved viaduct is also presented. The use of cable restrainer reducing curved viaduct damage is proved in this chapter.

The other 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, and pier damage is analyzed. The relationship between viaduct damage and cable restrainer's slack values could 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.

5.2 Cable Restrainers with Different Stiffness and Different Slack Values

In this chapter, two important parameters including stiffness and slack value are analyzed respectively. Viaduct equipped with cable restrainers of different stiffness shall present different behavior during earthquakes. Similarly, viaduct equipped with cable restrainers of

Table 5.1 Structural properties of cable restrainers

Restrainer	K_1 (MN/m)	K_2 (MN/m)	F_1 (MN)	F_2 (MN)
R0	0	0	0	0
R1	131.069	6.553	1.649	1.938
R2	162.442	8.122	1.938	2.280
R3	168.814	8.441	2.242	2.622
R4	204.058	10.203	2.584	3.040
R5	214.343	10.717	2.964	3.477
R6	156.863	7.843	4.178	4.761

different slack values also shows variable behavior in earthquakes. In addition, cable restrainer mechanism and displacement-force relationship are presented in Section 4.2.

First, the application of cable restrainers with different stiffness on the viaduct is presented. The viaduct seismic performance has been evaluated on 6 different kinds of cable restrainers (R0 indicating no cable restrainers), structural properties of cable restrainers are shown in **Table 5.1**. Besides, stopper value is 8 cm, slack value of cable restrainer is 4 cm.

Moreover, viaducts equipped with cable restrainers of different slack values are also studied in this chapter. The slack value adopted in this analysis is from 2 cm to 11 cm with the increase of 1 cm. Therefore, there are 10 calculation cases included in this chapter. Besides, stopper value is 8 cm, cable restrainer (R4) is adopted in this analysis.

5.3 Effect of Stiffness

5.3.1 Deck unseating damage

Firstly, the effect of stiffness on deck unseating damage is analyzed. In simple supported span, the induced relative displacements to steel roller bearings can exceed the seat width at the pier top, causing the dislodgment of the rollers from the bearing assembly and the subsequent collapse due to deck superstructure unseating. A limit of 0.35 m has been considered to indicate the high unseating probability for existing bridges with narrow steel pier caps that provide short seat widths.

The values are all smaller than limit value for all cases, as shown in **Fig. 5.1**. Maximum absolute roller bearing (B2) displacement for negative direction can be reduced in case of cable restrainers. As stiffness of cable restrainer increases, maximum absolute roller bearing (B2) displacement for negative direction decreases. Since the extension of cable restrainers could reduce maximum absolute negative displacement of roller bearing (B2). In addition, large stiffness lead to large extension of cable restrainers which results in less maximum absolute roller bearing (B2) displacement for negative direction.

5.3.2 Tangential and radial joint residual opening damage

As mention in the previous chapter (Section 4.4.3), permanent tangential offset of bridge expansion joints observed at the end of recent earthquakes have substantially interfered the post-earthquake serviceability of highway. The residual joint tangential displacement (RJTD) has been calculated in order to perform the post-earthquake serviceability evaluation on the viaduct. 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, thus sensibly restricting the operability of the bridge in such critical moments.

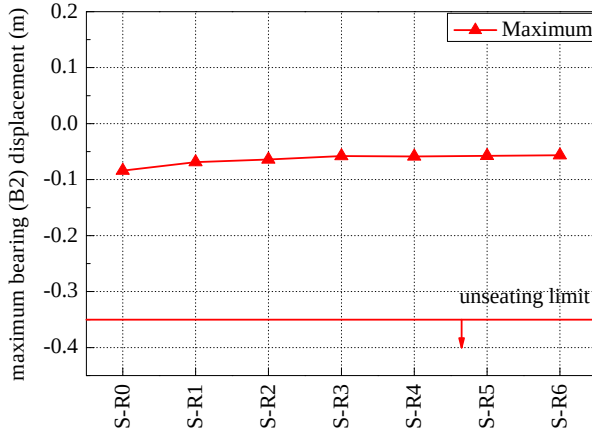


Fig. 5.1 Possibility of deck unseating damage

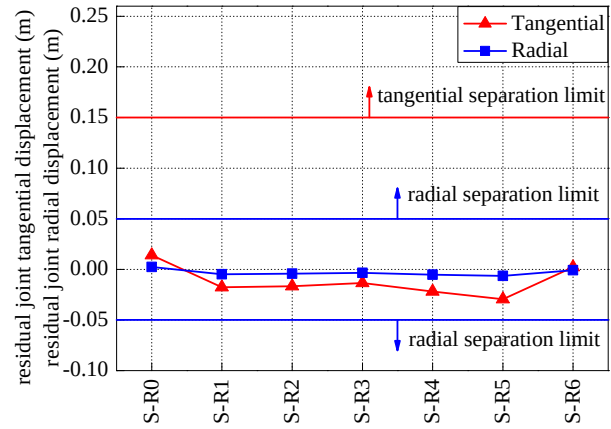


Fig. 5.2 Tangential and radial joint residual opening

Both residual joint tangential displacement (RJTD) and residual joint radial displacement (RJRD) are all under the limit line for all cases in **Fig. 5.2**, so there is no damage for RJTD and RJRD in earthquake. Curved bridges installed of cable restrainers present less residual joint tangential displacement. Moreover, since radical direction of all bearings is restraint, residual joint radial displacement is near zero.

5.3.3 Pier damage

As mention in the previous chapter (Section 4.4.4), viaducts supported on piers with large residual inclination may lose their serviceability, even possibly results in collapse. Residual pier inclination (RPI) has been considered as an important damage index in this analysis.

Detailed evaluation of calculated results presented in **Fig. 5.3** shows that pier inclination damage occurs in some cases. The installation of lateral stoppers to roller bearings is found relatively effective since the maximum bending moment acting on the pier with fixed bearings tends to be reduced. However, adverse effects are appreciated for piers with roller supports, being the possibility of large residual displacement extended to all top of piers in **Fig. 5.3**. Since large impact forces acting on stoppers of roller bearings are transferred to the piers with a sharp increment of bending moment at those pier bottoms. Consequently, all piers present residual pier inclination.

In addition, cable restrainers obviously play an important role in residual pier inclination as shown in **Fig. 5.3**. Viaducts with cable restrainers present less residual pier inclination than those without cable restrainers. Moreover, as stiffness or yield force of cable restrainer increases, residual pier inclination decreases. Finally, viaduct with cable restrainer R5 and R6 presents no pier inclination damage. At the beginning, top of piers presents 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. At last it results in less positive displacement on top of piers (P3, P4 and P5). As stiffness or yield force of cable restrainer increases, the effect of cable restrainer reducing positive displacement on top of piers (P3, P4 and P5) could become more obviously.

Viaducts with cable restrainers present more centralized residual pier inclination for all piers than those without cable restrainers as shown in **Fig. 5.3**. In other words, cable restrainers can reduce the degree of difference for the behavior of simple supported span and continuous span by connecting them as a whole. So, cable restrainer can reduce the maximum

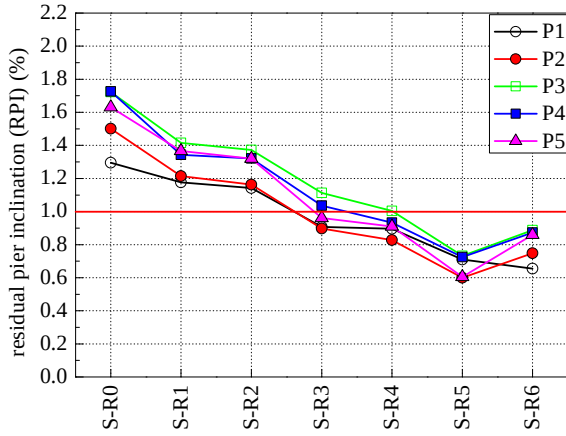


Fig. 5.3 Residual pier inclination

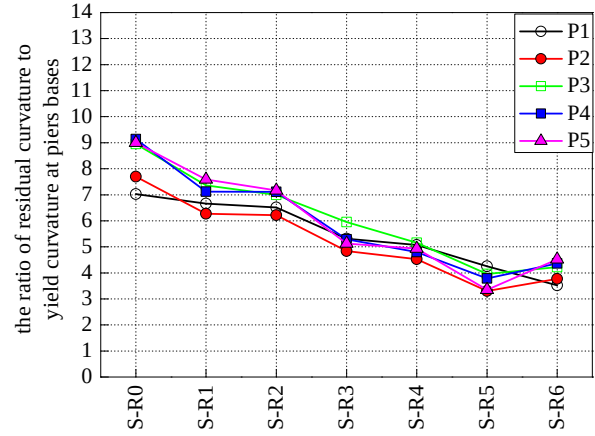


Fig. 5.4 Ratio of residual curvature to yield curvature

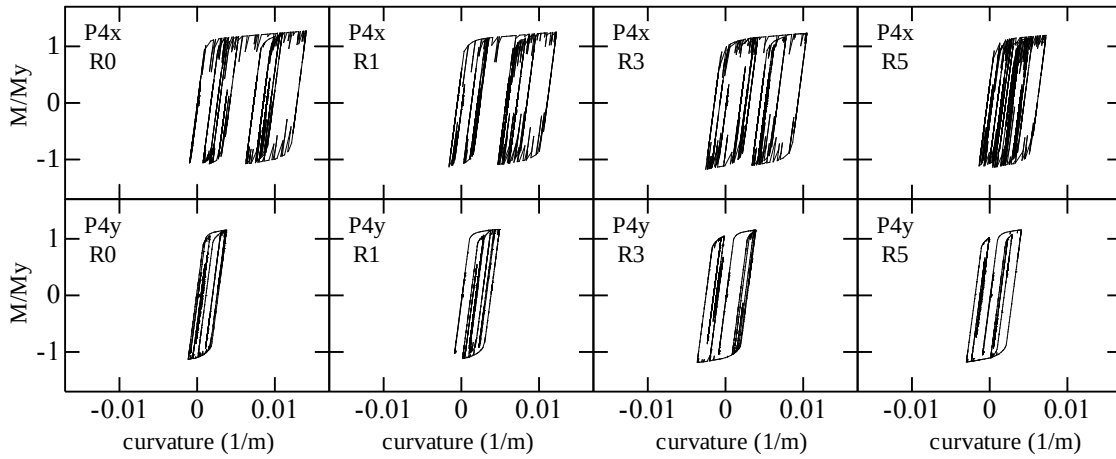


Fig. 5.5 Bending moment ratio-curvature relationship at pier (P4) base

positive pounding force between roller bearing and stopper in continuous span by averaging the positive pounding force on top of different piers. Therefore the less positive pounding force can lead to less residual pier inclination to a certain degree.

Finally, residual pier inclination and residual curvature present almost same change trend as shown in Fig. 5.3 and Fig. 5.4. Large bending moment at piers bases could lead to large curvature. In other words, curvature at piers bases indicates bending moment value to a certain degree. Residual curvature is determined by curvature time-history, especially, those curvatures which are equal to yield curvature. Residual curvature could determine residual pier inclination to some extent.

5.3.4 Relationship of bending moment and curvature

In this analysis, central pier (P4) of continuous span, supporting more weight, shall afford more inertial force during earthquakes, is likely subjected to large pounding forces between roller bearing and stopper during earthquakes. Therefore, the relationship between curvature and the ratio of bending moment (M) to yield bending moment (M_y) for P4 in X and Y direction in case of R0, R1, R3, R5 are shown Fig. 5.5. The relationship of bending moment and curvature at other piers bases present similar results.

It is well known that large bending moment at pier base lead to large curvature. According to the results for the bending moments in the X direction as shown in **Fig. 5.5**, the absolute value of M/M_y overpasses 1 in all cases. It indicates that there is plastic deformation in the X direction. In addition, maximum bending moments in the X direction are larger than the ones in the Y direction. Because roller bearings have been restrained in the Y direction. So, maximum bending moment in the X direction plays more important role in piers bases damage than Y direction. Besides **Fig. 5.5** also obviously show that the residual curvature at pier bases is determined by the bending moment which is larger than yield bending moment, especially maximum bending moment.

In addition, maximum bending moments at pier bases in the X direction, in the case of cable restrainers, is smaller than the case without cable restrainers as shown in **Fig. 5.5**. In addition, as cable restrainer stiffness increases, maximum bending moments in the X direction decreases. Besides, viaduct also exhibits slight pier damage in the Y direction.

5.4 Effect of Slack Values

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

5.4.1 Deck unseating damage

Based on the previous chapter (Section 4.4.1), the maximum roller bearing (B2) displacement in negative tangential direction has been established as the damage index to evaluate the potential possibility of deck unseating. A limit of 0.35 m has been considered to indicate the high unseating probability for existing bridges with narrow steel pier caps that provide short seat widths.

It can be seen from **Fig. 5.6** that collapse due to deck unseating does not happen for all cases. Maximum absolute bearing (B2) displacements for negative direction increase with the slack value of cable restrainers increasing.

As slack values are from 2 cm to 5 cm which are much smaller than stopper value, 8 cm, the maximum absolute 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 of cable restrainers determine the maximum absolute roller bearing displacement in negative direction.

As slack values are from 6 cm to 8 cm which are a little smaller than stopper value, the maximum absolute negative displacement of roller bearing (B2) 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. So, 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. So the slack value of cable restrainer does not affect on the maximum absolute bearing (B2) displacements for negative direction.

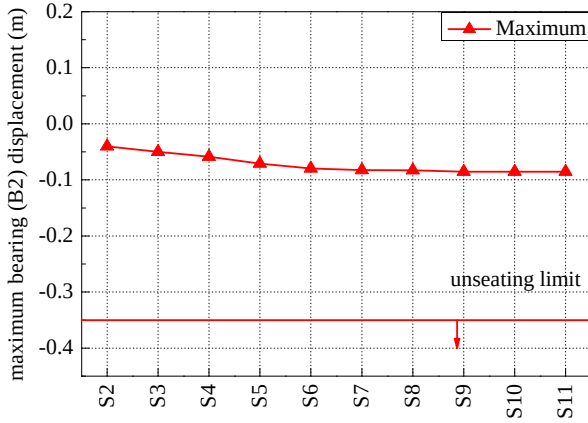


Fig. 5.6 Possibility of deck unseating damage

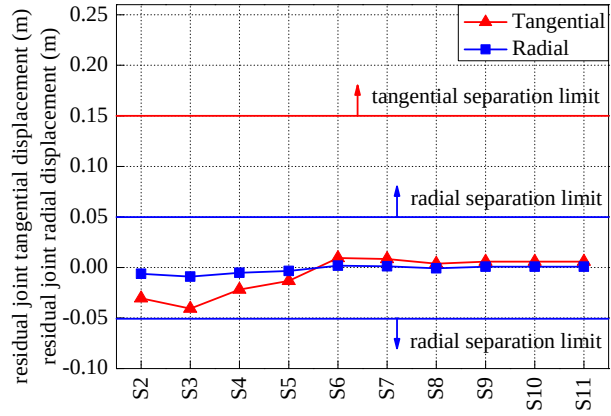


Fig. 5.7 Tangential and radial joint residual opening

5.4.2 Tangential and radial joint residual opening damage

Based on the previous chapter (Section 4.4.3), the limit value of residual joint tangential displacement (RJTD) is 0.15m. However, the limit value of residual joint radial displacement (RJRD) is 0.05m.

It can be seen from **Fig. 5.7** that RJTD and RJRD are all under the limit line for all cases. So, viaduct does not present any damages of 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 exhibit the closing trend. Since slack values are much smaller than stopper value between roller bearing and stopper, cable restrainers work earlier and more intensively. Moreover, since radial direction of all bearings is restraint, residual joint radial displacement (RJRD) is near 0.

5.4.3 Pier damage

Based on the previous chapter (Section 4.4.4), 1% maximum inclination of the bridge pier height is used as an evaluation of pier damage.

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, presents 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. So 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 larger 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 exhibit more centralized residual pier inclination for all piers than those without cable restrainers as shown in **Fig. 5.8**. In other

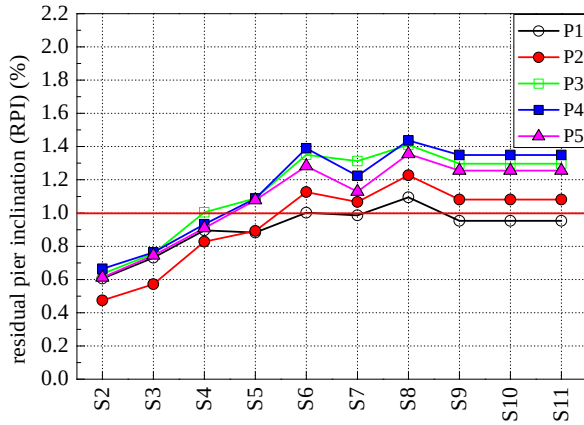


Fig. 5.8 Residual pier inclination

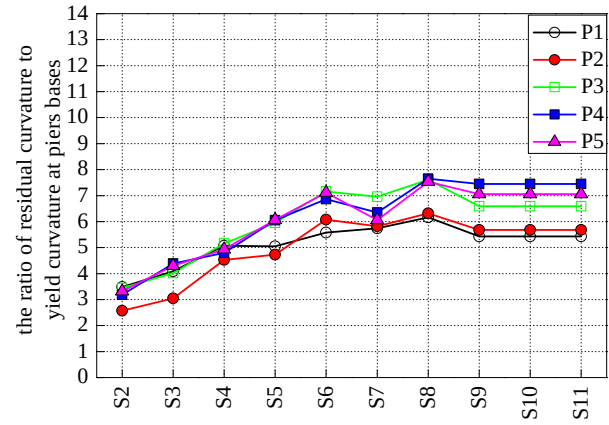


Fig. 5.9 Ratio of residual curvature to yield curvature

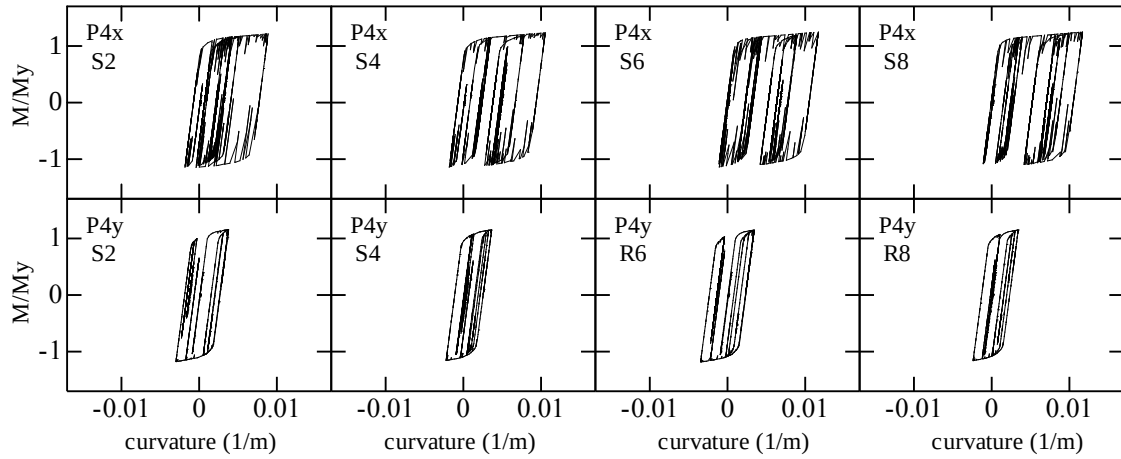


Fig. 5.10 Bending moment ratio-curvature relationship at pier (P4) base

words, cable restrainers can reduce the degree of difference for the behavior of simple supported span and continuous span by connecting them as a whole. So 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 present almost same change trend as shown in **Fig. 5.8** and **Fig. 5.9**. In other word, residual curvature at piers bases plays an important role in residual pier inclination.

5.4.4 Relationship of bending moment and curvature

In this analysis, central pier (P4) of continuous span, supporting more weight, shall afford more inertial force during earthquakes, is likely subjected to large pounding forces between roller bearing and stopper during earthquakes. Therefore, the relationship between curvature and the ratio of bending moment (M) to yield bending moment (M_y) for P4 in X and Y direction in case of S2, S4, S6, S8 are shown **Fig. 5.10**. The relationship of bending moment and curvature at other piers bases present similar results.

It can be seen from **Fig. 5.10** that the absolute value of M/M_y overpasses 1 for all cases in the X direction. It indicates that there is plastic deformation at pier base in the X direction. As slack value of cable restrainer increases, maximum bending moments in the X direction increase. Moreover, due to roller bearings restrained in the Y direction, maximum bending moments in the X direction are larger than the ones in the Y direction. Therefore, maximum bending moment in the X direction plays more important role in piers bases damage than Y direction. However viaduct also exhibits slight pier damage in the Y direction. Besides residual curvature at pier base is determined by the bending moment which is larger than yield bending moment, especially maximum bending moment.

5.5 Conclusions

The effects of stiffness and slack value of cable restrainer on nonlinear seismic response of curved highway viaducts have been analyzed respectively. The effects of stiffness and slack value of cable restrainer on deck unseating damage, tangential and radial joint residual opening damage, pier damage, and relationship of bending moment and curvature at pier base have been evaluated respectively. The investigation results provide sufficient evidence for the following conclusions:

(1) In case of different stiffness of cable restrainer, curved viaducts without cable restrainers are more vulnerable to deck unseating damage. In addition, the possibility of deck unseating is reduced by increasing the stiffness of cable restrainers. Besides, curved viaducts without cable restrainers are found more possible to tangential joint residual opening damage. However, residual joint radial displacements are very small in all cases. Cable restrainers do not obviously affect residual joint radial displacement.

(2) In case of different stiffness of cable restrainer, according to evaluation of pier inclination damage, viaducts with cable restrainers present less residual pier inclination than those without cable restrainers. Moreover, as stiffness of cable restrainer increases, residual pier inclination decreases. So cable restrainers with large stiffness (R5 and R6) can be used to significantly mitigate the pier damage. In addition, residual pier inclination and residual curvature at piers bases present similar change trend in all cases. In other words, residual curvature at pier base plays an important role in residual pier inclination.

(3) In case of different stiffness of cable restrainer, as stiffness of cable restrainer increases, maximum bending moments at piers bases in the X direction decreases. In addition, maximum bending moments at piers bases in the X direction is larger than the ones in the Y direction. So cable restrainers with large stiffness are appropriate to be used to reduce pier damage.

(4) In case of different slack values of cable restrainer, the possibility of deck unseating is reduced by decreasing the slack value of cable restrainers. In addition, 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.

(5) In case of different slack values of cable restrainer, as slack value of cable restrainer decreases, residual pier inclination decreases. So viaduct with slack value, 2 cm and 3 cm, presents no pier inclination damage. In addition, residual pier inclination and residual curvature at piers bases exhibit similar change trend in all cases.

(6) In case of different slack values of cable restrainer, as slack value of cable restrainer increases, maximum bending moments at piers bases in the X direction increases. Moreover, maximum bending moments at piers bases in the X direction is larger than the ones in the Y direction. Therefore cable restrainers with less slack value are appropriate to be used to reduce pier damage.

References

- [1] Hayashikawa, T., and Ruiz Julian, D.: Parametric study on design of cable restrainers for curved highway viaducts under Level II earthquakes, *Proceedings of Hokkaido Chapter of the Japan Society of Civil Engineers*, JSCE, Vol. 61, CD ROM, I-12, 2005.
- [2] Roberts, J., and Maroney, B.: Chapter 40: Seismic retrofit practice. In: Chen, W. F., and Duan, editors. *Bridge Engineering Handbook*, Boca Raton, CRC Press, 1999.
- [3] Japan Road Association (JRA), *Specification for Highway Bridges - Part V Seismic Design*, Tokyo (Japan), Maruzen, 2002.

EFFECTIVENESS OF VISCOUS DAMPER ON SEISMIC RESPONSE

6.1 Introduction

During the 1995 Kobe earthquake, the failure of steel bearings resulted in unseating of decks [1, 2]. Damage of pounding between adjacent viaduct segments, has also been observed in earthquakes [3]. To mitigate above damages, roller bearings equipped with stopper are installed on top of piers. Although seismic isolation device such as lead rubber bearing has been used to reduce curved viaduct damage recently, the price is more expensive. Due to its cheap price and simple installation, stopper has been used in this analysis.

The viscous damper has been developed since the late 1980s [4-7]. It provides an adaptable damping force by adjusting the size of the orifice through which a viscous fluid flows when a piston moves in a hydraulic cylinder. The most advantage of viscous dampers is that it shows no resistance forces under slow relative movements of segments results from thermal changes, creep and shrinkage effects. It only begins to work when earthquake happens. Therefore, application of viscous dampers to viaducts has gained significant attention in recent years [8]. Due to its large resistance force and energy dissipation capacity, viscous dampers have wide application prospect. Generally, researchers use lead rubber bearings combined with viscous dampers to reduce viaduct damage. However, roller bearings combined with viscous dampers to reduce damage do not be done.

Therefore, the purpose of this chapter is to analyze the overall performance of highway viaducts with different kinds of viscous dampers and different stopper values. Different kinds of viscous dampers have different stiffness and different resistance forces. So the relationship between different kinds of viscous dampers, different stopper values, and curved viaduct damage could be found respectively. According to the relationship, the advice for the seismic design of curved viaduct based on the appropriate stopper values could be put forward. Additionally, a comparison between curved viaduct equipped with viscous dampers and without viscous dampers is presented. Application of viscous dampers and stopper reducing some kinds of viaduct damages is also proved in this chapter.

6.2 Analytical Model of Viscous Damper

Schematic of viscous damper is shown in **Fig. 6.1**. In this analysis, viscous dampers units are anchored to the three girder ends connecting both adjacent superstructures across the expansion joint as shown in **Fig. 6.2**. The relationship between resistance force and relative velocity is nonlinear as shown in **Fig. 6.3a**. C is attenuation coefficient which is shown in **Table 6.1**. Viscous damper presents no resistance force under slow relative movements of segments results from thermal changes, creep and shrinkage effects. It only quickly begins to work when earthquake happens. In other words, viscous dampers do not have influence on viaduct normal service in static state condition that is very well for viaduct. Three stopper values (4 cm, 6 cm and 8 cm) can be used in this analysis. Stopper values have obviously effect on the relative velocity between adjacent superstructures. According to the calculation results, the maximum relative velocity between superstructures is 0.6 m/s, 0.7 m/s and 0.8 m/s respectively. The resistance forces of viscous dampers are derived from the maximum relative velocity as shown in **Table 6.1**. Another relationship between force and displacement for viscous damper is shown in **Fig. 6.3b**. The force-displacement loop is nearly rectangular which presents relative large energy dissipation capacity. In addition, the bridge seismic

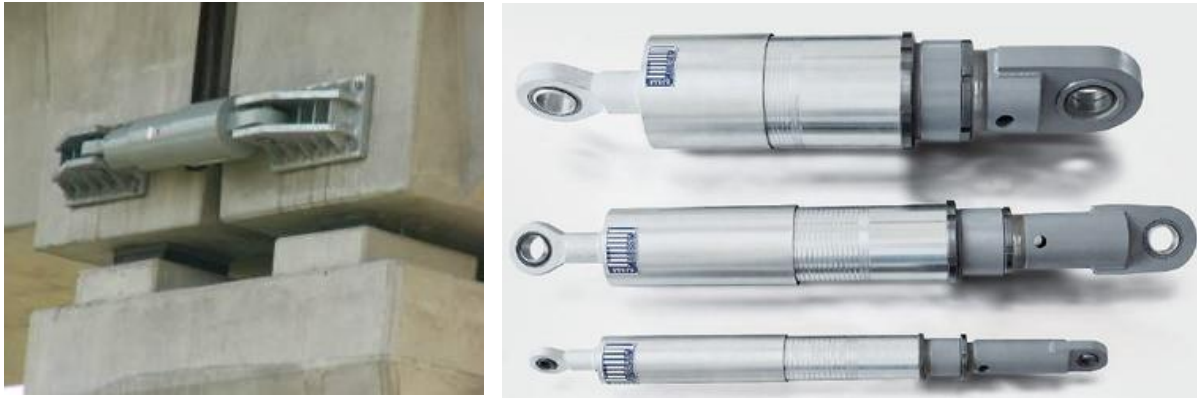


Fig. 6.1 Schematic diagram of viscous damper

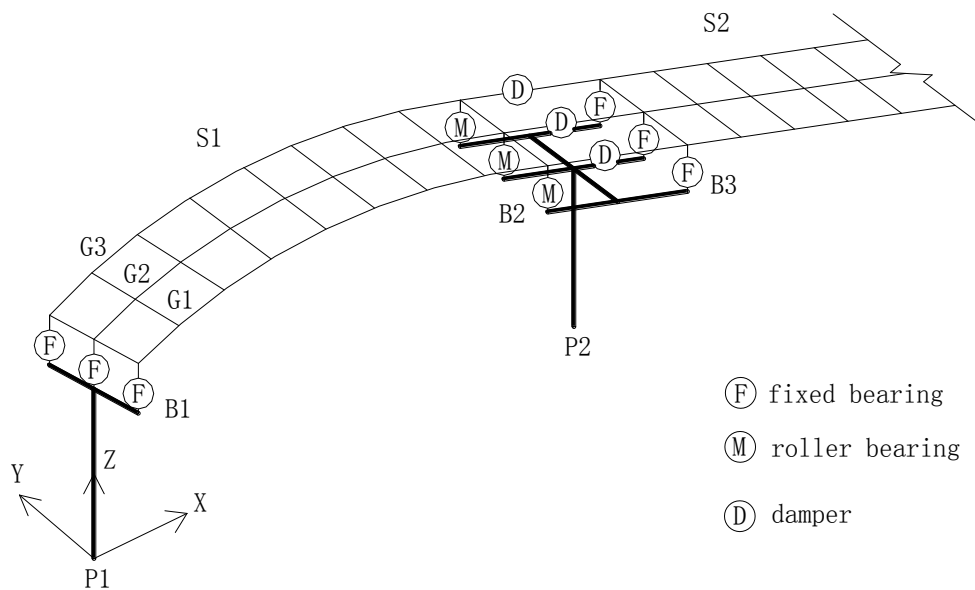


Fig. 6.2 Finite element model detail of curved viaduct equipped with viscous damper

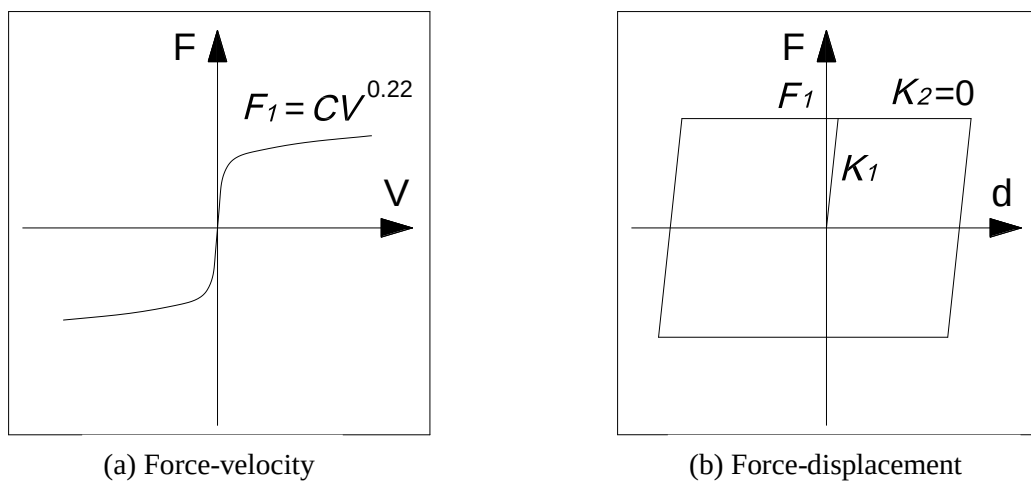


Fig. 6.3 Analytical model of viscous damper

performance has been evaluated on 5 different kinds of viscous dampers (D0 indicating no viscous dampers), structural properties of viscous dampers are shown in **Table 6.1**.

Table 6.1 Structural properties of viscous dampers

Damper type	Attenuation coefficient C	Power exponent α	Stiffness K_1 (MN/m)	Stopper=4 cm Resistance force F_1 (MN)	Stopper=6 cm Resistance force F_1 (MN)	Stopper=8cm Resistance force F_1 (MN)
D0	-	-	-	-	-	-
D100	117	0.22	90	0.105	0.108	0.111
D300	350	0.22	130	0.313	0.324	0.333
D500	583	0.22	200	0.521	0.539	0.555
D1000	1165	0.22	400	1.041	1.077	1.109
D2000	2330	0.22	800	2.082	2.154	2.218

6.3 Numerical Results and Discussion

The overall three-dimensional seismic response of the viaduct is investigated in detail through non-linear dynamic response analysis. For an easy identification of the different calculation cases, a specific nomenclature is adopted in this research. “D” refers to viscous damper. So “D100” presents that the viscous damper type is D100. And “D0” indicates the viaduct without viscous dampers. Kobe earthquake measured by the Takatori (TAK) stations is adopted as an input ground motion.

6.3.1 Deck unseating damage

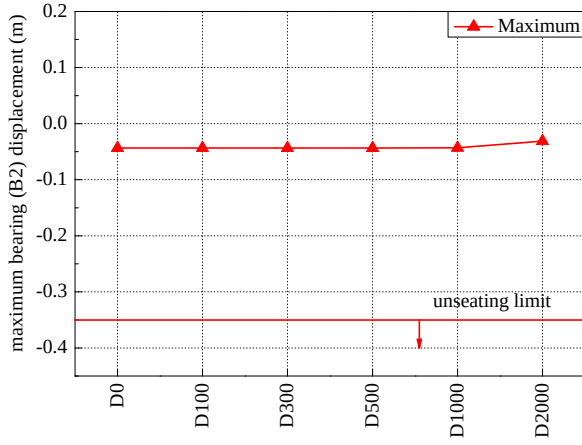
Based on the previous chapter (Section 4.4.1), the maximum roller bearing (B2) displacement in negative tangential direction has been established as the damage index to evaluate the potential possibility of deck unseating. A limit of 0.35 m has been considered to indicate the high unseating probability for existing bridges with narrow steel pier caps that provide short seat widths.

Absolute values are all smaller than limit absolute value for all cases as shown in **Fig. 6.4**. Maximum absolute bearing (B2) displacement for negative direction presents nearly the same value with stopper value except the case of D2000. Since viscous damper’s stiffness and resistance force is not large enough to resist the structures movements. It is stopper that prevents superstructures movements. So as stopper value increases, maximum absolute bearing (B2) displacement for negative direction increases. Additionally, maximum absolute bearing (B2) displacement for negative direction presents smallest value as D2000 is installed in the viaduct in case of the same stopper value respectively. Since the resistance force and stiffness of D2000 is largest one among all the selected viscous dampers. So the roller bearing (B2) move relatively hard, comparing with other viscous dampers, that leads to smaller maximum displacement for negative direction.

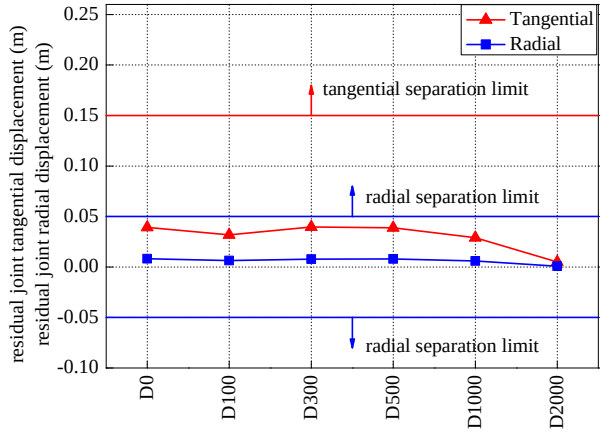
6.3.2 Tangential and radial joint residual opening damage

Based on the previous chapter (Section 4.4.3), the limit value of residual joint tangential displacement (RJTD) is 0.15m. However, the limit value of residual joint radial displacement (RJRD) is 0.05m.

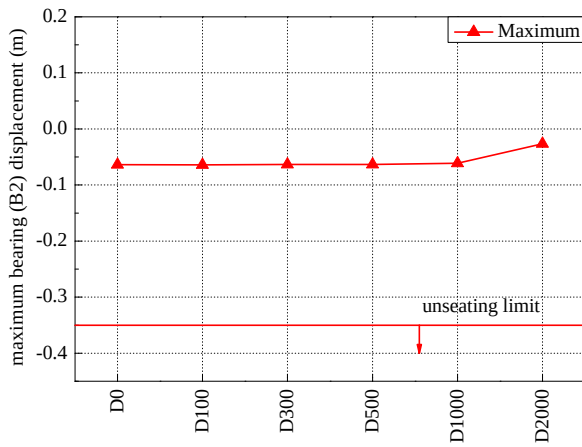
Both residual joint tangential displacement (RJTD) and residual joint radial displacement (RJRD) are all smaller than the limit value in all cases as shown in **Fig. 6.5**. However,



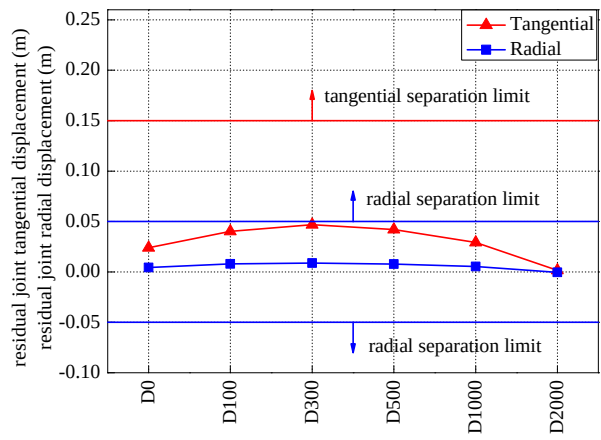
(a) Stopper=4 cm



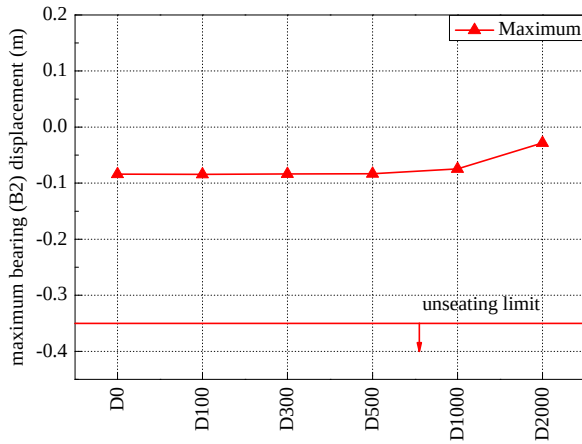
(a) Stopper=4 cm



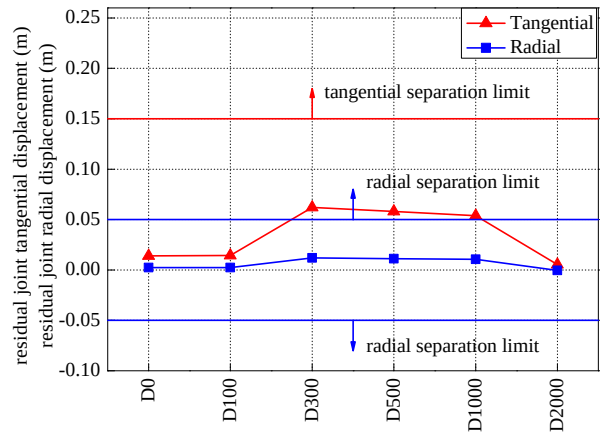
(b) Stopper=6 cm



(b) Stopper=6 cm



(c) Stopper=8 cm



(c) Stopper=8 cm

Fig. 6.4 Possibility of deck unseating damage

Fig. 6.5 Tangential and radial joint residual opening damage

residual joint tangential displacement presents smallest value in case of D2000. Since the resistance force and the stiffness of D2000 is largest one which leads to superstructures move relatively hard. According to **Fig. 6.5**, residual joint tangential displacement is smaller than stopper value respectively. Moreover, because radial direction of all bearings is restraint, residual joint radial displacement is near zero.

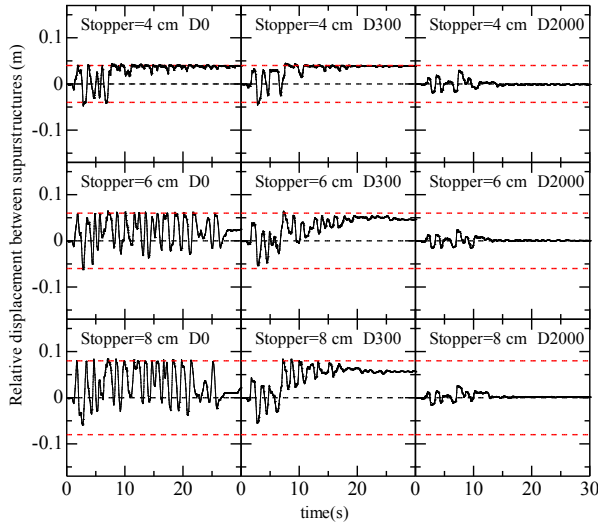


Fig. 6.6 Relative displacement between superstructures

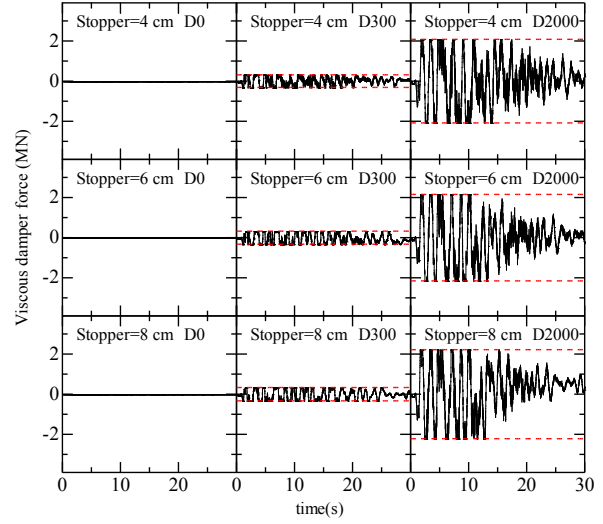


Fig. 6.7 Viscous damper force time history

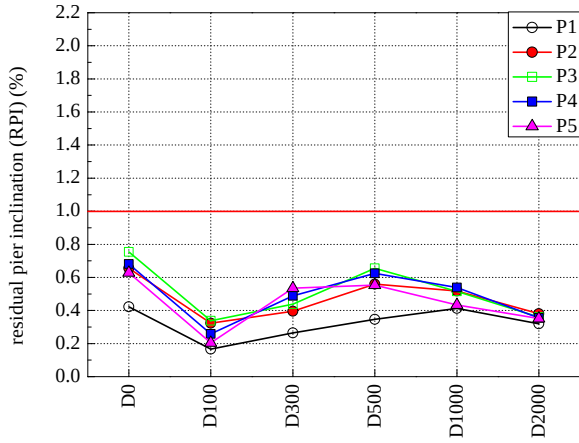
6.3.3 Relative displacement between superstructures

Relative displacement time history between superstructures, without viscous damper and with viscous damper, is presented in **Fig. 6.6**. The results with viscous dampers (D300 and D2000) are only presented here since the results with other viscous dampers exhibit the similar response. The dashed red line presents stopper value. A positive relative displacement of the expansion joint corresponds to opening of the joint gap (outward) while a negative relative displacement corresponds to a closing (inward). Comparing with maximum closing relative displacement when no damper is provided, it is significantly reduced in the case of having viscous dampers. In addition, maximum closing relative displacement obviously decreases as viscous damper's resistance force and stiffness increases. However, maximum positive relative displacements present the same value with stopper value respectively. Since positive relative displacement (outward) is larger than stopper value, pounding between roller bearing (B2) and stopper occurs except the case of D2000. It is stopper that prevents maximum positive relative displacement becoming larger. But since the resistance force and stiffness of D2000 is very large, it could obviously reduce both positive and negative relative displacement. **Fig. 6.7** shows viscous damper force time history. D2000 presents largest resistance force that can reduce relative displacement between superstructures. Since there is no resistance damper force in case of D0, negative relative displacement exhibits largest one.

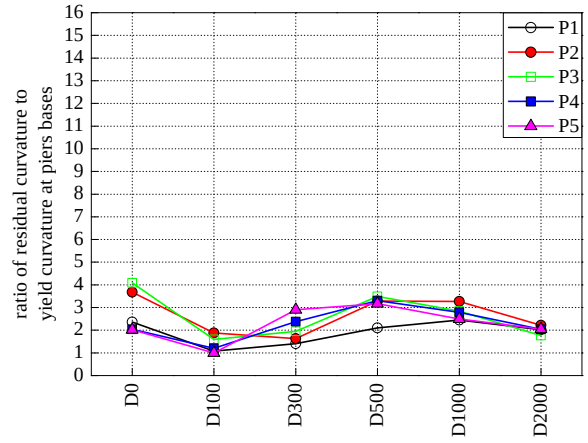
6.3.4 Pier damage

As mention in the previous chapter (Section 4.4.4), viaducts supported on piers with large residual inclination may lose their serviceability, even possibly results in collapse. Residual pier inclination (RPI) has been considered as an important damage index in this analysis. Besides 1% maximum inclination of the bridge pier height is used as an evaluation of pier damage.

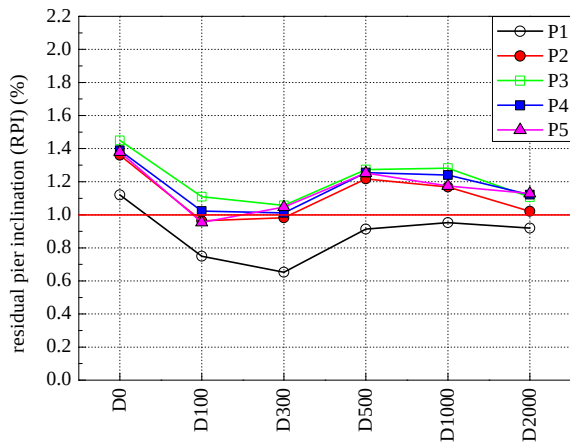
Residual pier inclination and residual curvature present almost same change trend as shown in **Fig. 6.8** and **Fig. 6.9**. Similarly, maximum pier inclination and maximum curvature at piers bases also exhibit same change trend as shown in **Fig. 6.10** and **Fig. 6.11**. Besides maximum inclination and maximum curvature present similar change trend to residual pier inclination and residual curvature as shown in **Fig. 6.8**, **Fig. 6.9** and **Fig. 6.10** and **Fig. 6.11**.



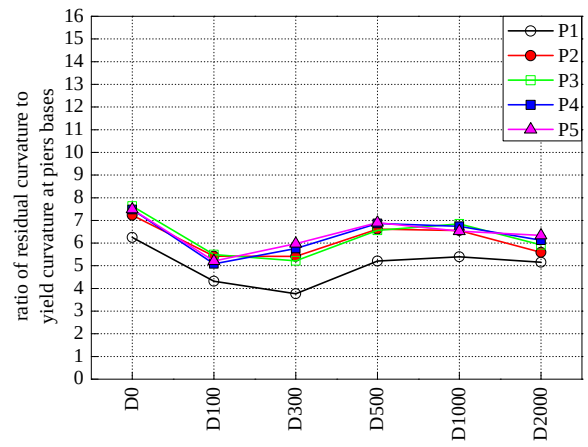
(a) Stopper=4 cm



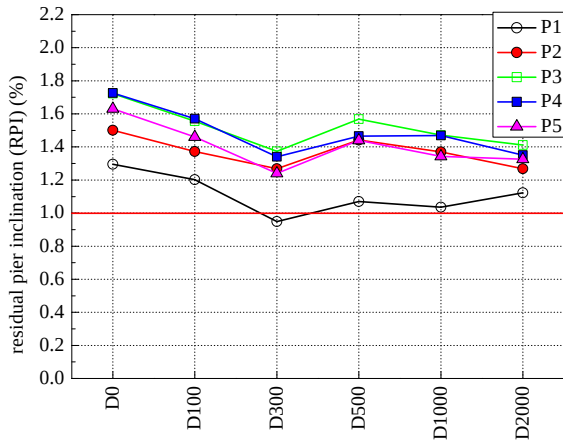
(a) Stopper=4 cm



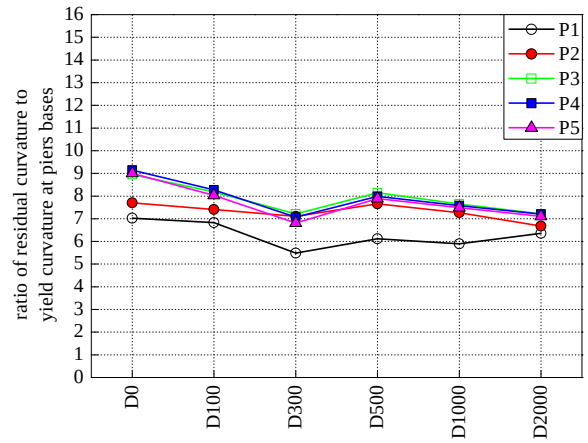
(b) Stopper=6 cm



(b) Stopper=6 cm



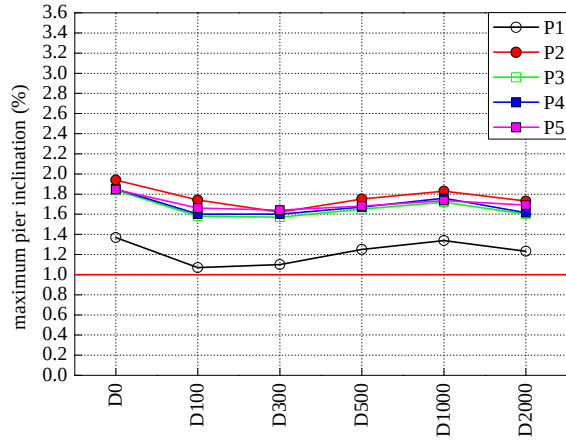
(c) Stopper=8 cm



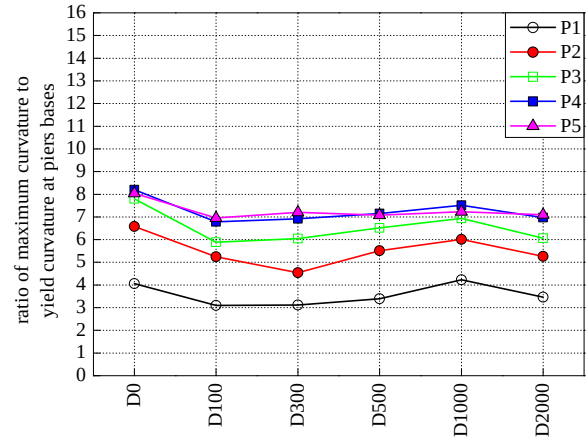
(c) Stopper=8 cm

Fig. 6.8 Residual pier inclination**Fig. 6.9** Ratio of residual curvature to yield curvature

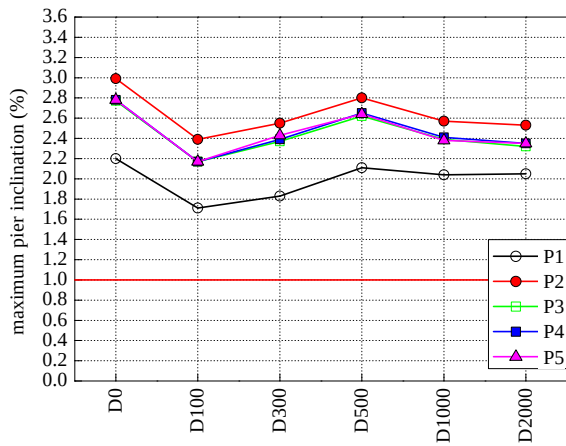
Since residual curvature is determined by curvature time-history, especially, those curvatures which are larger than yield curvature. Therefore maximum curvature at piers bases plays an important role in residual curvature and residual pier inclination. In other words, large maximum curvature at piers bases could lead to large residual curvature at piers bases and residual pier inclination.



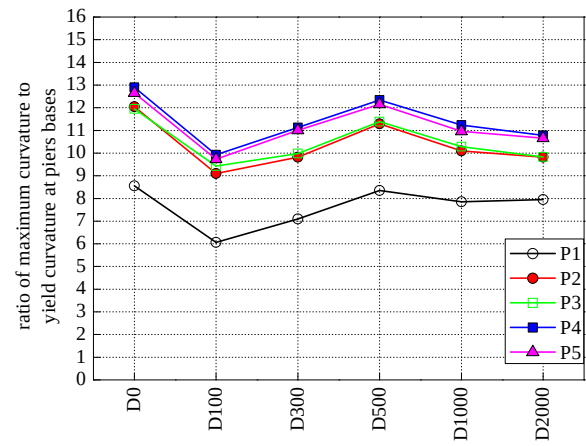
(a) Stopper=4 cm



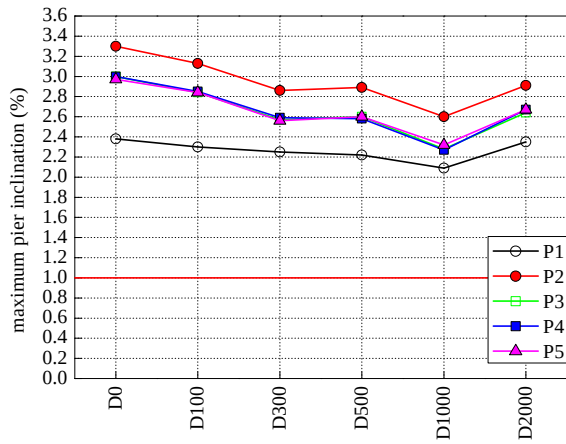
(a) Stopper=4 cm



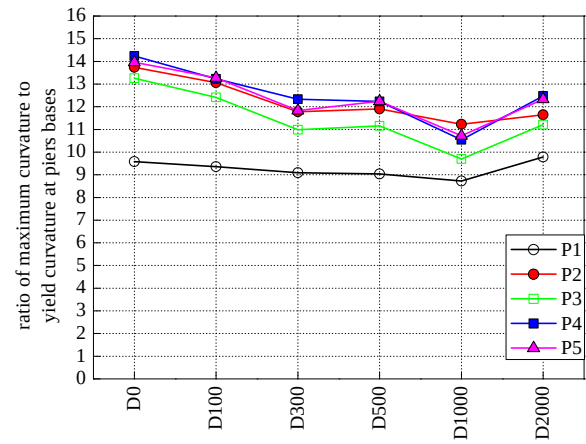
(b) Stopper=6 cm



(b) Stopper=6 cm



(c) Stopper=8 cm

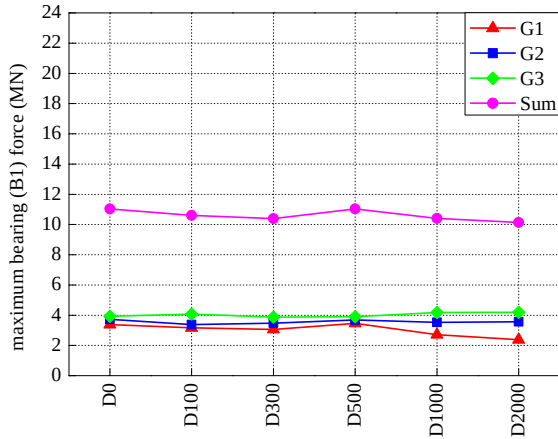


(c) Stopper=8 cm

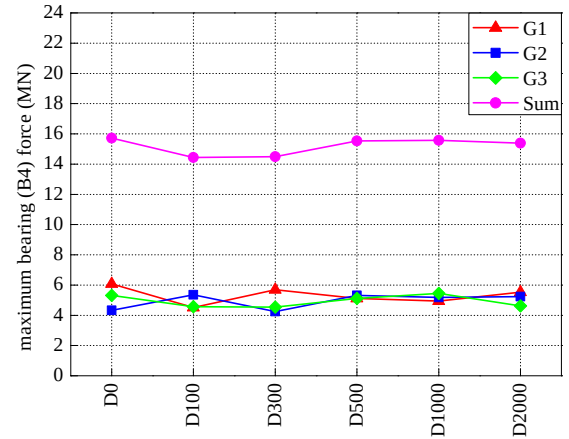
Fig. 6.10 Maximum pier inclination

Fig. 6.11 Ratio of maximum curvature to yield curvature

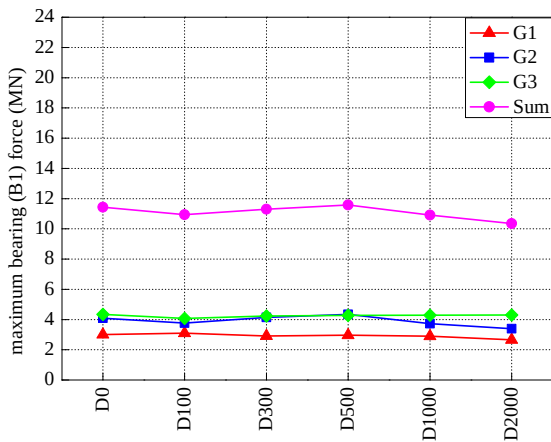
In addition, as stopper value is 4 cm, there is no pier inclination damage in Fig. 6.8. As stopper value increases, residual pier inclination gradually increases. However, viaduct piers present damage when stopper value is 6 cm and 8 cm. Since the pounding force between roller bearing and stopper increases with stopper value increasing as shown in Fig. 6.12 and Fig. 6.13. The bearing (B1 and B4) forces are only presented here, representing fixed bearing



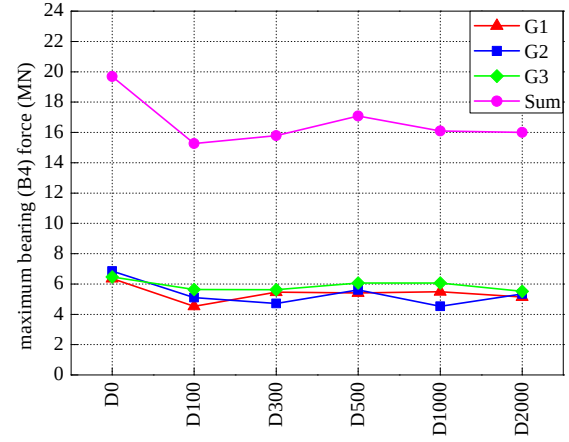
(a) Stopper=4 cm



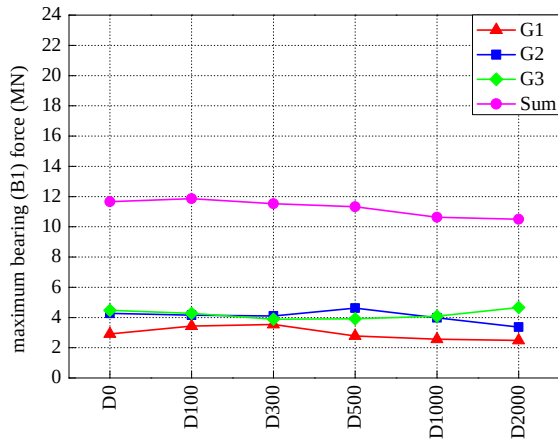
(a) Stopper=4 cm



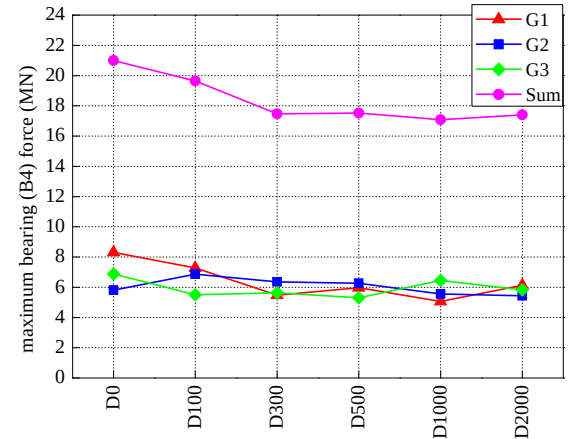
(b) Stopper=6 cm



(b) Stopper=6 cm



(c) Stopper=8 cm



(c) Stopper=8 cm

Fig. 6.12 Maximum bearing (B1) forces for tangential direction

Fig. 6.13 Maximum bearing (B4) forces for tangential direction

and roller bearing force respectively, because the other bearings forces exhibit the similar change trend. Comparing with other piers, P1 exhibits smallest residual pier inclination. Since pounding force between roller bearing and stopper is much larger than fixed bearing force. Fixed bearings and roller bearings are used on top of P1, P3 respectively.

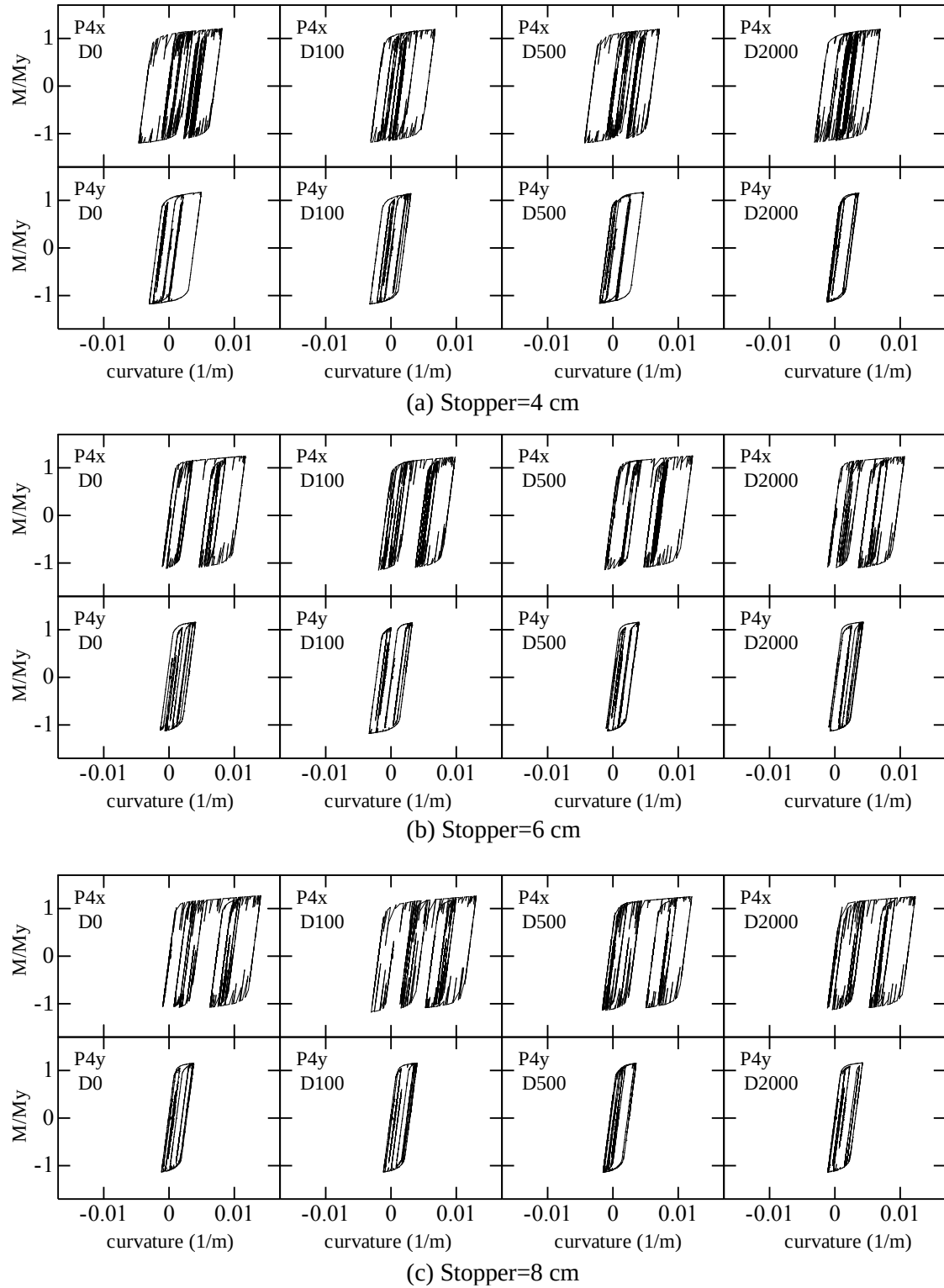


Fig. 6.14 Bending moment ratio-curvature relationship at pier (P4) base

Finally, residual pier inclination does not present obvious difference value when different viscous dampers are provided in **Fig. 6.8**. Viscous damper's resistance force and stiffness does not have obvious influence on residual pier inclination. However, residual pier inclination without viscous dampers presents larger value than the ones with viscous dampers in the case of same stopper value. Since viscous dampers could dissipate some energy to a certain degree.

6.3.5 Relationship of bending moment and curvature

In this analysis, central pier (P4) of continuous span, supporting more weight, shall afford more inertial force during earthquakes, is likely subjected to large pounding forces between roller bearing and stopper during earthquakes. Therefore, the relationship between curvature and the ratio of bending moment (M) to yield bending moment (M_y) for P4 in X and Y direction in case of D0, D100, D500, D2000 are shown **Fig. 6.14**. The relationship of bending moment and curvature at other piers bases present similar results.

Fig. 6.14 shows that the absolute value of M/M_y overpasses 1 in both X and Y directions, which indicates plastic deformation at pier base in these two directions. Maximum bending moment and maximum curvature in X direction is larger than those in Y direction respectively. Since all the bearings are restrained in Y direction that leads to relatively less bearing forces. So, maximum bending moment and maximum curvature at piers bases in X direction play a more important effect on the damages of piers bases. As stopper value increases, maximum bending moment and maximum curvature at piers bases in X direction obviously increases. However, maximum bending moment and maximum curvature at piers bases does not exhibit significantly difference in case of different viscous dampers applied on the viaduct. In other words, the type of viscous dampers does not have effect on maximum bending moment and maximum curvature at piers bases. Besides, viaduct equipped with viscous damper presents less maximum bending moment and less maximum curvature at piers bases than the ones without viscous dampers. Since viscous damper could dissipate some energy to a certain degree. Conclusively, stopper value plays a more important role in maximum bending moment and maximum curvature at piers bases than viscous dampers.

6.4 Conclusions

The overall performance of highway viaducts with different kinds of viscous dampers and different stopper values is evaluated during serious earthquakes in this chapter. Application of viscous dampers and stopper reducing some kinds of viaduct damages is also proved in this analysis. Stopper value plays a more important role in piers damage than viscous damper. The investigation results provide sufficient evidence for the following conclusions.

(1) Calculated results demonstrate that stopper could effectively reduce deck unseating possibility. However, deck unseating possibility could also be significantly reduced in case of D2000 because of its large resistance force and stiffness. Besides, curved viaduct with D2000 presents smallest residual joint tangential displacement. However, residual joint radial displacements are very small in all cases. The type of viscous damper and stopper value do not obviously affect residual joint radial displacement.

(2) Maximum closing relative displacement could be greatly reduced in case of viscous dampers. In addition, maximum closing relative displacement obviously decreases as damper's resistance force and stiffness increases. However, stopper could greatly reduce maximum opening relative displacement.

(3) Viaducts with smaller stopper value present less residual pier inclination. As stopper value increases, residual pier inclination increases. Comparing with other piers, P1, fixed bearings on top of it, exhibits smallest residual pier inclination. Since pounding force between roller bearing and stopper is much larger than fixed bearing force. Viaducts with viscous dampers present less residual pier inclination than those without viscous dampers. But damper's resistance force and stiffness does not obviously affect on the residual pier inclination. Finally, residual pier inclination, maximum pier inclination, residual curvature and maximum curvature at piers bases present similar trend for all cases.

(4) Plastic deformations at piers bases appear in both X and Y directions during earthquakes. Maximum bending moment and maximum curvature at piers bases in X direction is larger than those in Y direction. So, maximum bending moment and maximum curvature at piers bases in X direction play a more important role in the damages of piers bases. In case of same viscous damper, as stopper value increases, maximum bending moment and maximum curvature at piers bases in X direction obviously increases. Viaduct equipped with viscous damper presents less maximum bending moment and less maximum curvature at piers bases than the ones without viscous dampers. However, the type of viscous dampers does not have effect on maximum bending moment and maximum curvature at piers bases. Stopper value plays a more important role in maximum bending moment and maximum curvature at piers bases than viscous dampers.

References

- [1] Watanabe, E., Sugiura, K., Nagata, K. and Kitane, Y.: Performances and damages to steel structures during the 1995 Hyogoken-Nanbu earthquake, *Engineering Structures*, Vol. 20, No. 4-6, pp. 282-290, 1998.
- [2] Scawthorn, C., and Yanev, P.I.: Preliminary report 17 January 1995, Hyogo-ken Nambu, Japanese earthquake, *Engineering Structures*, Vol.17, No.3, pp. 146-157, 1995.
- [3] Abdel Raheem, S.E.: Pounding mitigation and unseating prevention at expansion joints of isolated multi-span bridges, *Engineering Structures*, Vol.31, No.10, pp. 2345-2356, 2009.
- [4] Kawashima, K., Unjoh, J.: Development of hybrid control technology, *Civil Engineering Journal*, Vol.32, No.6, pp. 2-5, 1989.
- [5] Kawashima, K., Unjoh, S., Nagashima, H., Shimizu, H.: Current research efforts in Japan for passive and active control of highway bridges against earthquakes, *Proceedings of the 23rd Joint Meeting*, US-Japan Panel on Wind and Seismic Effect, Tsukuba, Japan, pp. 187-209, 1991.
- [6] Kawashima, K., Unjoh, S., Shimizu, K.: Experiments on dynamic characteristics of variable damper, *Proceedings of Japan National Symposium on Structural Response Control*, pp.121-128, 1992.
- [7] Kawashima, K., Unjoh, S.: Seismic response control of bridges by variable dampers, *ASCE Journal of Structural Engineering*, Vol.120, No.9, pp. 2583-2601, 1994.
- [8] Ruangrassamee, A., Kawashima, K.: Control of nonlinear bridge response with pounding effect by variable dampers, *Engineering Structures*, Vol.25, No.5, pp. 593-606, 2003.

EFFECTIVENESS OF SHOCK ABSORBER DEVICES ON SEISMIC RESPONSE

7.1 Introduction

During the last decades, curved viaducts with steel bearings are widely used in modern transportation networks as a viable option at complicated interchanges or river crossings where geometric restrictions and constants of limited site space make extremely complicated the adoption of standard straight viaducts. The reports of damage to curved viaducts during the 1995 Kobe earthquake indicate that failures of steel bearings could lead to local damage and falling down of viaduct decks [1-4]. Pounding between adjacent viaduct segments also could amplify the relative displacement results in the requirement of longer seat width to support the deck [5, 6]. Moreover, pounding transfers large seismic lateral forces from one deck to another that leads to a significant change in the seismic response of the entire viaduct system [7-10]. Besides, pounding may significantly increase the reaction forces at the piers bases [11, 12].

Recently, seismic isolation devices such as lead rubber bearings have been used to replace the vulnerable steel bearings to reduce seismic force [13-15], it could lead to large displacements and therefore increase the possibility of pounding between adjacent viaduct segments. In order to reduce the negative pounding effects, some researchers set shock absorber devices between adjacent viaduct segments [16, 17]. Since the cost of lead rubber bearing replacing steel bearing is relatively expensive and the installation of lead rubber bearing is relatively complicated. So in order to mitigate above mentioned damages, conventional roller steel bearings equipped with stopper are installed on top of piers. Due to its cheap price and simple installation, stopper shall have very good application prospect. However, pounding force between roller bearing and stopper is so large that could bring damages at not only roller bearing but also piers bases. Therefore, rubber shock absorber device between roller bearing and stopper could be used to reduce the negative pounding effects by smooth changes of impact stiffness. Besides shock absorber devices could be easily installed and the price is not expensive. Recently, the effects of shock absorber devices installed between adjacent viaduct segments on reducing pounding forces, have been studied by many researchers. But the effectiveness of shock absorber devices installed between roller bearing and stopper on reducing pounding damage, have not been done. The stiffness and thickness of shock absorber devices play an important effect on the viaduct damages during earthquakes. Moreover, strain hardening, strain softening and elastic shock absorber devices are also taken into account in recent years.

In addition, application of viscous dampers to viaducts has gained significant attention in recent years [18, 19]. Due to its large resistance force and energy dissipation capacity, viscous dampers have wide application prospect. The most advantage of viscous dampers is that it shows no resistance forces under slow relative movements of segments results from thermal changes, creep and shrinkage effects.

Therefore, there are three main purposes of this chapter study. The first purpose is to analyze the overall performance of highway viaducts with different stiffness and resistant forces of rubber shock absorber devices, considering one kind of viscous damper. Then the relationship between different rubber shock absorber device's stiffness and curved viaduct damage could be found. According to the relationship, the advice for the seismic design of curved viaduct based on the appropriate stiffness of shock absorber device could be put

forward. In addition, a comparison between curved viaducts equipped with shock absorber devices and without shock absorber devices is presented.

The second purpose is to analyze the overall performance of highway viaducts with different thickness of rubber shock absorber devices, considering one kind of viscous damper. The relationship between different thickness of rubber shock absorber device and curved viaduct damage could be found. According to the relationship, the advice for the seismic design of curved viaduct based on the appropriate thickness of shock absorber device could be put forward. In addition, a comparison between curved viaducts equipped with shock absorber devices and without shock absorber devices is presented.

The third purpose is to analyze the overall performance of highway viaducts installed of three types of shock absorber devices including strain hardening, elastic and strain softening shock absorber devices. The relationship between different types of shock absorber devices and curved viaduct damage could be obtained respectively. Finally, according to the relationship, the advice for the seismic design of curved viaduct based on the appropriate type of shock absorber device could be put forward.

7.2 Analytical Model of Shock Absorber Devices

Viaduct superstructures are supported on steel bearings. For simple supported span, fixed bearing supports (**Fig. 7.1a**) are rested on top of left pier (P1), roller bearing supports without shock absorber device (**Fig. 7.1b**) are rested on top of right pier (P2). For continuous span, fixed bearing supports are installed on top of left pier (P2). However, roller bearings without shock absorber device (**Fig. 7.1b**) are rested on top of other piers (P3, P4 and P5). Fixed bearing supports movements are restrained in both longitudinal and transverse radial directions. Roller bearing supports have freedom movement in longitudinal direction, but have been restrained in transverse radial direction. In this analysis, coulomb friction force is taken into account for roller bearings, which are modeled by using the bilinear rectangle displacement-load relationship as shown in **Fig. 7.1b**. The coefficient of friction is 0.05. In addition, steel stoppers are installed at both sides of roller bearing supports in longitudinal direction. Pounding between roller bearing supports and steel stopper was modeled using an impact spring element. The spring stiffness is K_3 , which is a high stiffness slope as shown in **Fig. 7.1b**. In this analysis, the gap between roller bearing and steel stopper is 8 cm, which is smaller than the gap of expansion joint. As the gap between roller bearing and steel stopper is smaller than the gap of expansion joint, pounding damage between superstructures could be eliminated during earthquakes.

The schematic diagrams of roller bearings without/with shock absorber device (SAD) are shown in **Fig. 7.2a** and **Fig. 7.2b** respectively. In case of roller bearing with shock absorber devices (SADs), shock absorber devices are installed at both sides of all the roller bearings in longitudinal direction as shown in **Fig. 7.2b**. Therefore, the total number of shock absorber devices is 24 in the viaduct. Based on the previous study, it is obvious that the effectiveness of energy dissipation of SADs is much less significant in decreasing the compression force in devices [20]. The energy dissipation in SADs is also very small in viaduct system [20]. In other words, the energy dissipation capacity is not the requirement with significant importance for shock absorber device. So, force-displacement relation of strain hardening shock absorber device could be simulated in **Fig. 7.1d**. The analytical model of roller bearing support with strain hardening shock absorber devices is shown in **Fig. 7.1c**. According to previous studies, shock absorber is effective in reducing negative effects of pounding by smooth changes of impact stiffness.

In order to analyze the effect of SAD's stiffness, the section area of SAD is variable and the thickness of SAD is 4 cm. Generally, large area of SAD shall have large stiffness.

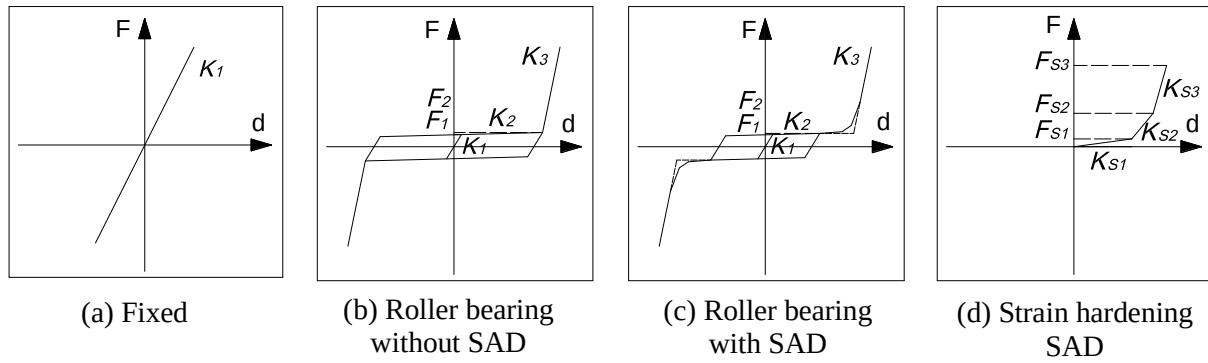


Fig. 7.1 Analytical model of steel bearing supports and SAD

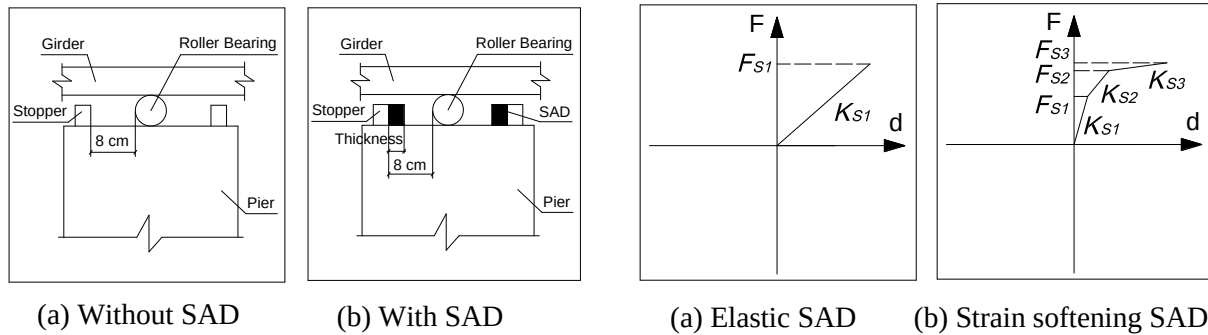


Fig. 7.2 Schematic diagram of roller bearing without/with SAD

Fig. 7.3 Analytical model of elastic and strain softening SADs

Table 7.1 Structural properties of shock absorber device

SAD type	Area (cm ²)	Thickness (cm)	K_{s1} (MN/m)	K_{s2} (MN/m)	K_{s3} (MN/m)	F_{s1} (MN)	F_{s2} (MN)	F_{s3} (MN)
A0	-	-	-	-	-	-	-	-
A1	15×8	4	5.00	60	120	0.12	0.6	1.56
A2	20×8	4	6.67	80	160	0.16	0.8	2.08
A3	25×8	4	8.33	100	200	0.20	1.0	2.60
A4	30×8	4	10.00	120	240	0.24	1.2	3.12

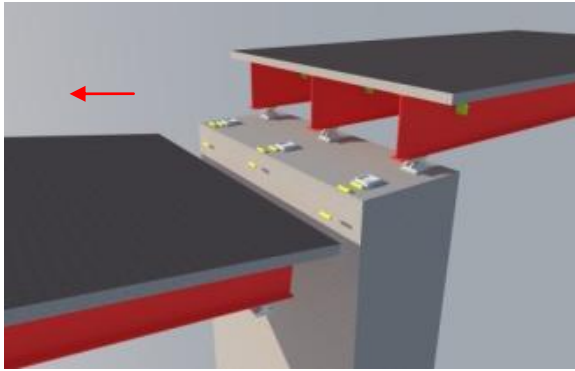
Table 7.2 Structural properties of strain hardening shock absorber device

SAD type	Thickness (cm)	K_{s1} (MN/m)	K_{s2} (MN/m)	K_{s3} (MN/m)	F_{s1} (MN)	F_{s2} (MN)	F_{s3} (MN)
T0	-	-	-	-	-	-	-
T2	2	8.33	100	200	0.10	0.50	1.30
T4	4	8.33	100	200	0.20	1.00	2.60
T6	6	8.33	100	200	0.30	1.50	3.90
T8	8	8.33	100	200	0.40	2.00	5.20

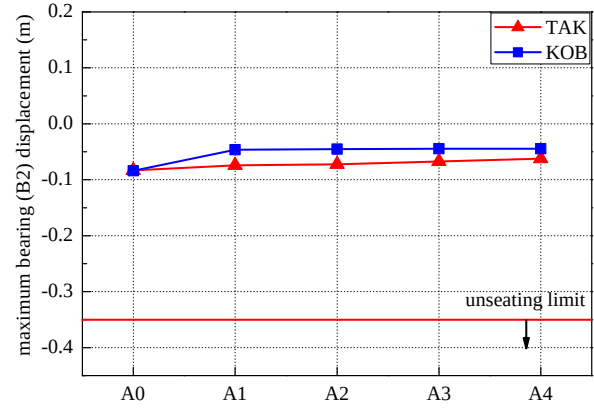
Structural properties of SAD are shown in **Table 7.1**. Moreover, the viaduct seismic performance has been evaluated, based on 4 different stiffness of SAD (A0 indicating no shock absorber device).

Table 7.3 Structural properties of strain hardening, elastic and strain softening SADs

SAD type	K_{s1} (MN/m)	K_{s2} (MN/m)	K_{s3} (MN/m)	F_{s1} (MN)	F_{s2} (MN)	F_{s3} (MN)
Strain hardening	6.67	80	160	0.16	0.80	2.08
Elastic	52	-	-	2.08	-	-
Strain softening	160	80	6.67	1.28	1.92	2.08



(a) Schematical of deck unseating damage



(b) Maximum bearing (B2) displacement

Fig. 7.4 Deck unseating damage for TAK and KOB

In order to analyze the effect of SAD's thickness, the thickness of SAD is from 0 to 8 cm with increment of 2 cm, in order to keep stiffness as constant value, the section area of SAD is variable. **Table 7.2** shows the structural properties of strain hardening shock absorber devices. Therefore, the viaduct seismic performance has been evaluated, based on 5 different thickness of strain hardening SAD (0 cm indicating no SAD).

Finally, it is possible to develop a strain softening SAD by providing many small voids or holes in a SAD [20]. For such purpose, it may be required to use natural rubber with higher modulus. It is also possible to develop a SAD with almost elastic property even under high compression. In order to find what type of SAD is appropriate for mitigating viaduct damage, in addition to strain hardening (SH) SAD, strain softening (SS) and elastic (EL) SADs are also evaluated in this analysis. Force-displacement relation of EL and SS SADs could be simulated in **Fig. 7.3**. The section area of SH, EL and SS SADs is 20 cm × 8 cm and the thickness is 4 cm. Structural properties of SH, EL and SS SADs are shown in **Table 7.3**.

7.3 Analytical Model of Expansion Joint

In this analysis, the gap of expansion joint is 10 cm which is larger than the gap between roller bearing and stopper. So there is not pounding phenomenon between superstructures during serious earthquakes. It is roller bearing and stopper that resist relatively large displacement of superstructures. Therefore, expansion joint could be protected from damage due to pounding between superstructures during severe earthquakes.

Based on previous study, viscous damper could be used to reduce pier damage to some extent. Therefore viscous dampers are anchored to the three girder ends connecting both adjacent superstructures across the expansion joint in this chapter. So, 3 viscous dampers are installed in the viaduct. The best advantage of viscous damper is that it presents no resistance force under slow relative movements of segments results from thermal changes, creep and

shrinkage effects. It is only activated when earthquake happens. Viscous dampers do not have negative effects on viaduct normal service in static state condition. The gap between roller bearing and stopper has obviously effect on the relative velocity between adjacent superstructures. According to the calculation results, the maximum relative velocity between superstructures is 0.8 m/s as the gap between roller bearing and stopper is 8 cm. The resistance force of viscous damper can be derived from the maximum relative velocity. In this analysis, viscous damper of D500 has been used in the viaduct. Structural properties of viscous damper for KVD500 are shown in **Table 6.1** of Section 6.2.

7.4 Effect of Stiffness

7.4.1 Deck unseating damage

Based on the previous chapter (Section 4.4.1), the maximum roller bearing (B2) displacement in negative tangential direction has been established as the damage index to evaluate the potential possibility of deck unseating. A limit of 0.35 m has been considered to indicate the high unseating probability for existing bridges with narrow steel pier caps that provide short seat widths.

According to **Fig. 7.4**, absolute values are all smaller than limit absolute value for all cases. Maximum absolute bearing (B2) displacement for negative direction is 8 cm in case of A0 for both TAK and KOB earthquakes. Since there is not shock absorber device between roller bearing and stopper, the gap between roller bearing and stopper is 8 cm. However, as shock absorber device equipped in the viaduct, maximum absolute bearing (B2) displacement for negative direction presents smaller than 8 cm. In other words, the stiffness and resistance force of shock absorber device could reduce maximum absolute bearing (B2) displacement for negative direction that reduces the possibility of deck unseating as shown in **Fig. 7.4**. As stiffness and resistance force of SAD increase, maximum absolute bearing (B2) displacement for negative direction decreases. Since shock absorber device with large stiffness and resistance force could afford same external force with relative smaller deformation that leads to smaller maximum absolute bearing (B2) displacement for negative direction.

7.4.2 Bearing force

Large bearing force results in not only bearing damage but also pier base damage. So how to reduce pounding force between roller bearing and stopper is very important. In this analysis, shock absorber devices are installed between roller bearing and stopper. The effect of shock absorber device's stiffness on bearing force is studied. Since continuous span (supported by B3, B4, B5 and B6) which has large inertial force comparing with simple supported span in earthquakes, the bearings forces of continuous span are relative large. Therefore, bearing (B5 and B3) forces are presented as shown in **Fig. 7.5** and **Fig. 7.6**, representing roller bearing and fixed bearing forces of continuous span respectively. Since other bearings forces exhibit the similar change trend.

According to **Fig. 7.5**, maximum roller (B5) forces with shock absorber devices are much smaller than the case without shock absorber devices. It is apparent that shock absorber devices are effective to reduce the pounding force in all cases through smooth changes of pounding stiffness. As stiffness and resistance force of shock absorber device increase, the maximum pounding force between roller bearing and stopper gradually decreases. Therefore, roller bearing supports with shock absorber devices of large stiffness shall have less bearing damage and less pier base damage. In addition, maximum roller bearing (B5) forces are much larger than fixed bearing (B3) forces for TAK and KOB respectively in **Fig. 7.5** and **Fig. 7.6**.

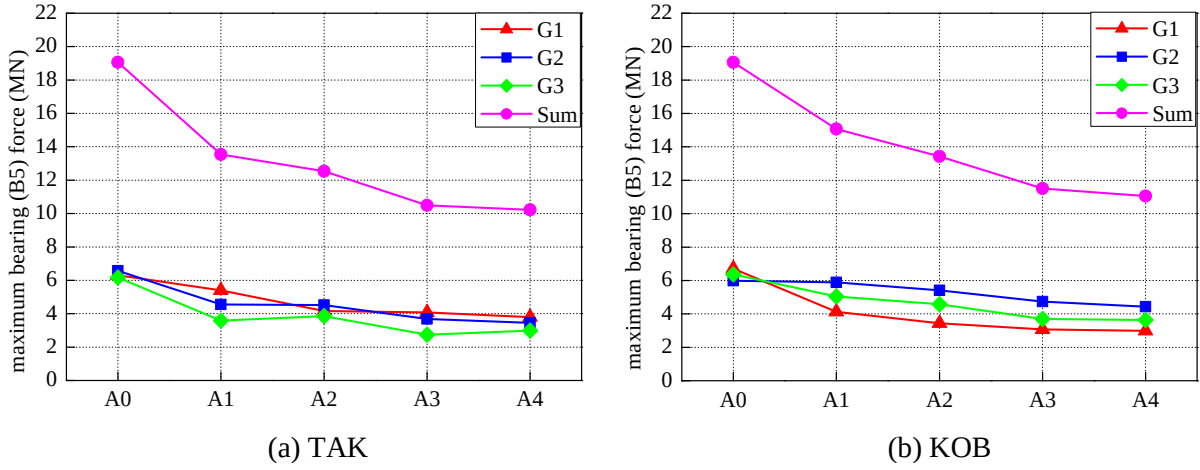


Fig. 7.5 Maximum roller bearing (B5) forces for TAK and KOB

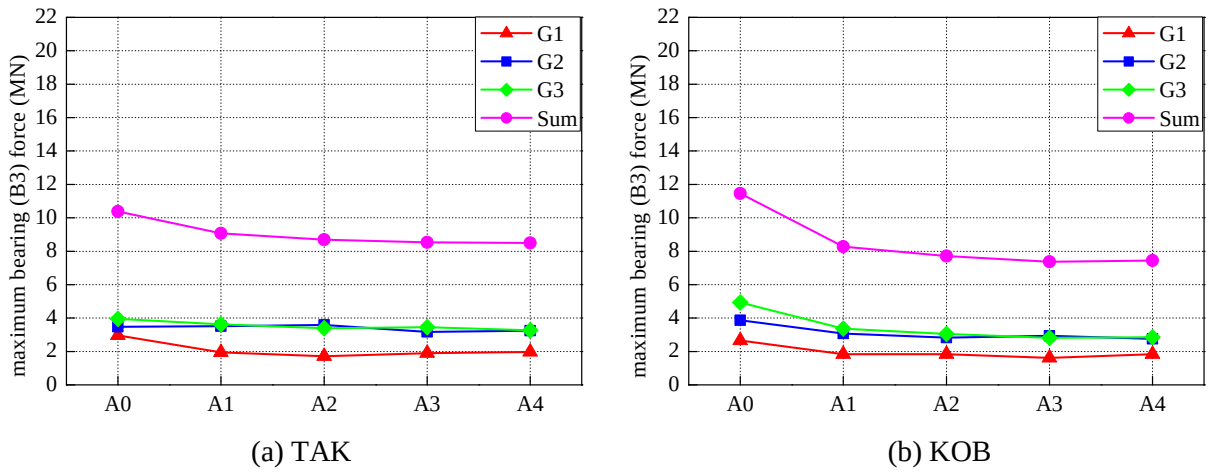


Fig. 7.6 Maximum fixed bearing (B3) forces for TAK and KOB

Pounding forces between roller bearing and stopper are much larger than fixed bearing (B3) forces in case of with/without shock absorber devices. As stiffness and resistance forces of shock absorber device increase, fixed bearing (B3) forces could also be reduced by the indirectly effect of maximum roller (B5) bearing forces. Pounding forces between roller bearing and stopper have more important effect on bearing damage and pier base damage than fixed bearing forces.

The bearing (B5) force time history between roller bearing and stopper without/with SADs of different stiffness and resistance forces is analyzed. The bearing supporting inside girder (G1) and combined with the shock absorber device types (A0, A1 and A4) are shown in **Fig. 7.7**. In order to comparing with roller bearing (B5) forces, fixed bearing (B3) force time history is also presented in **Fig. 7.7**. The application of shock absorber device leads to greatly reduction of roller bearing (B5) forces for both positive and negative directions in TAK and KOB input earthquakes. Since shock absorber devices are installed at both sides of all the roller bearings for tangential direction. In addition, due to earlier activation of shock absorber device and smooth changes of pounding stiffness, contacts time last longer but the pounding forces between roller bearing (B5) and stopper are much smaller. As stiffness and resistance force of SAD increase, pounding forces between roller bearing (B5) and stopper for both positive and negative directions decrease. In addition, fixed bearing (B3) forces can also be gradually reduced in case of roller bearings equipped with shock absorber devices. It can

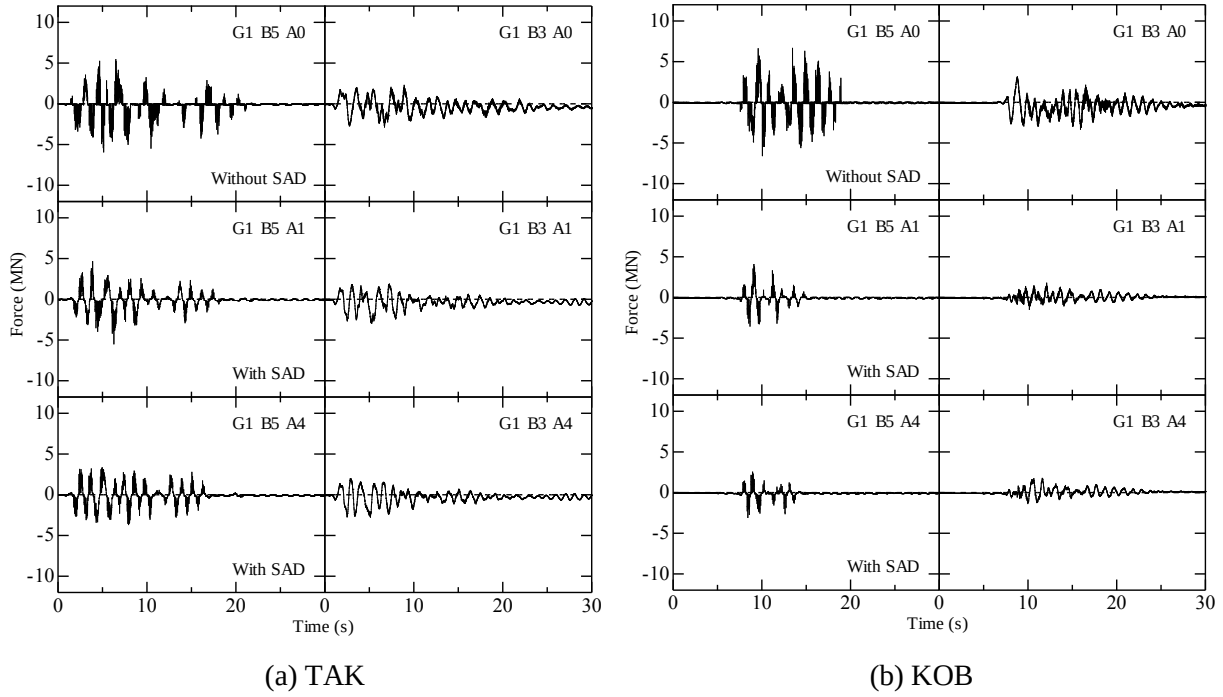


Fig. 7.7 Bearing (B5 and B3) forces time history for TAK and KOB

be also seen in **Fig. 7.7**, pounding forces between roller bearing (B5) and stopper for both positive and negative directions, are larger than fixed bearing (B3) forces respectively.

7.4.3 Pier damage

As mention in the previous chapter (Section 4.4.4), residual pier inclination (RPI) has been considered as an important damage index in this analysis. Besides 1% maximum inclination of the bridge pier height is used as an evaluation of pier damage.

In this analysis, top of piers presents residual pier inclination of positive direction for TAK and KOB input earthquakes. Residual pier inclination and maximum roller bearing forces present similar change trend as shown in **Fig. 7.5** and **Fig. 7.8**. Since large pounding forces acting on stoppers of roller bearings are transferred to the piers with a sharp increment of bending moment and curvature at piers bases that lead to large residual pier inclination. As it can be seen from **Fig. 7.8**, the installation of shock absorber devices to roller bearings is found obviously effective to reduce residual pier inclination of positive direction for TAK and KOB input earthquakes. Moreover, as stiffness and resistance forces of shock absorber device increases, residual pier inclination of positive direction decreases. At last, viaducts with shock absorber devices do not present serious pier damage for both TAK and KOB input earthquakes as shown in **Fig. 7.8**. However, residual pier inclination of viaduct without shock absorber devices is larger than 1% for TAK, which indicates serious pier damage. Since shock absorber devices between roller bearing and stopper could be used to significantly reduce both positive and negative pounding forces by smooth changes of pounding stiffness. Relative smaller positive and negative pounding forces between roller bearing and stopper could lead to smaller positive residual pier inclination. As stiffness and resistant forces of shock absorber device increase, the effect of shock absorber device on reducing residual pier inclination could become more obviously. In addition, residual pier inclination of pier 1 (P1), supporting fixed bearings, presents less value than the one of other piers in **Fig. 7.8**. Since fixed bearing forces is much smaller than pounding forces between roller bearing and stopper that leads to

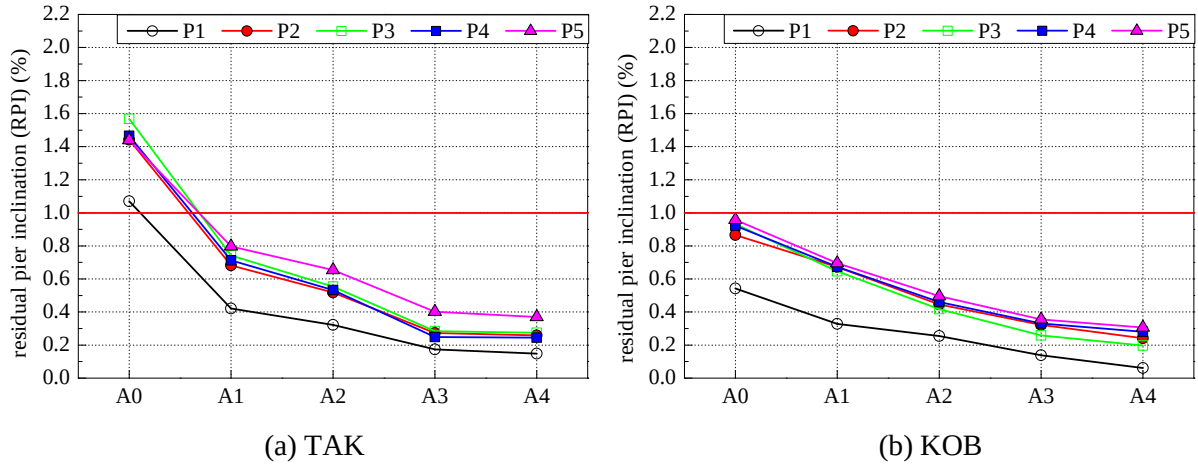


Fig. 7.8 Residual pier inclination for TAK and KOB

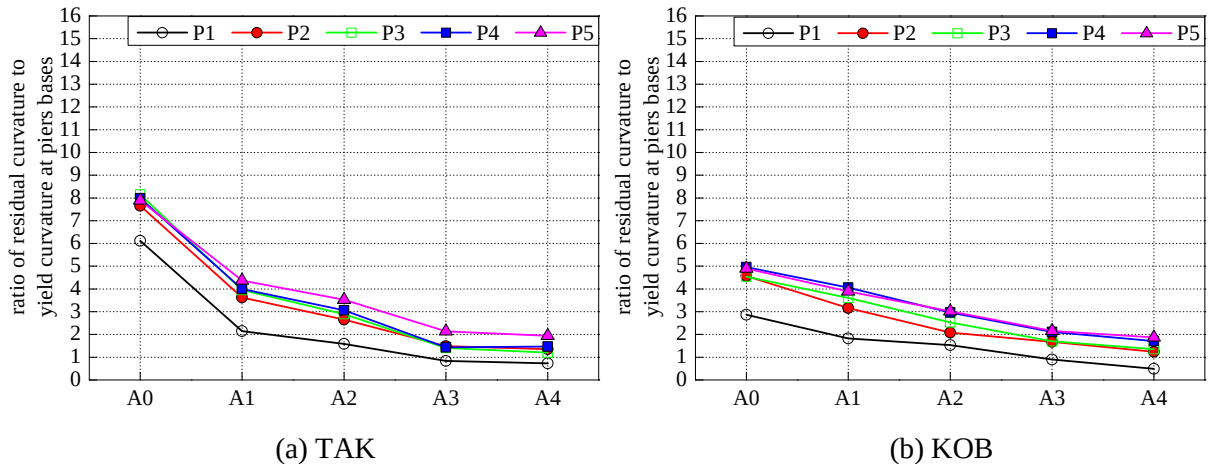


Fig. 7.9 Ratio of residual curvature to yield curvature for TAK and KOB

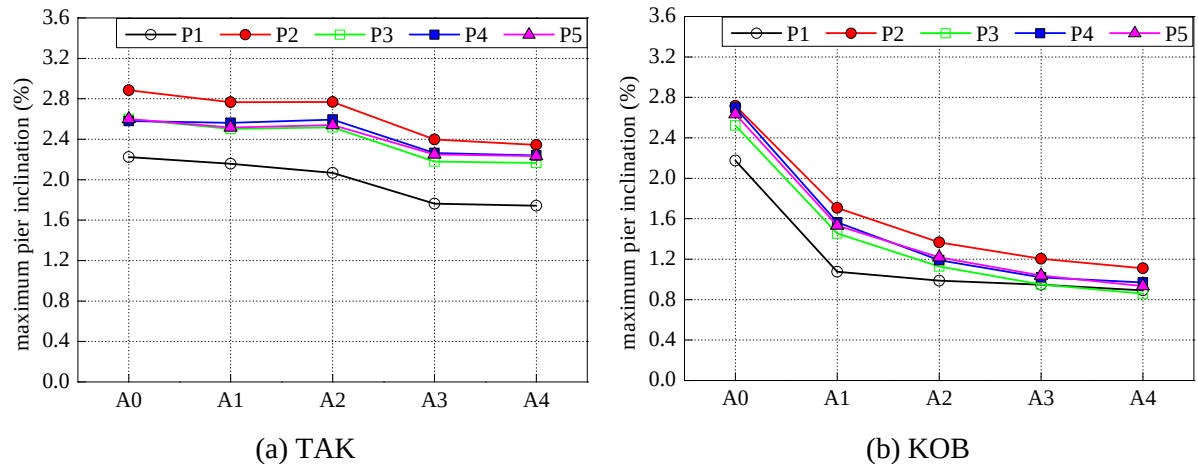


Fig. 7.10 Maximum pier inclination for TAK and KOB

less residual displacement on top of pier.

As can be seen in **Fig. 7.8** and **Fig. 7.9**, residual pier inclination and residual curvature present almost same change trend. Besides maximum pier inclination and maximum curvature at pier base also present same change trend as shown in **Fig. 7.10** and **Fig. 7.11**. In other

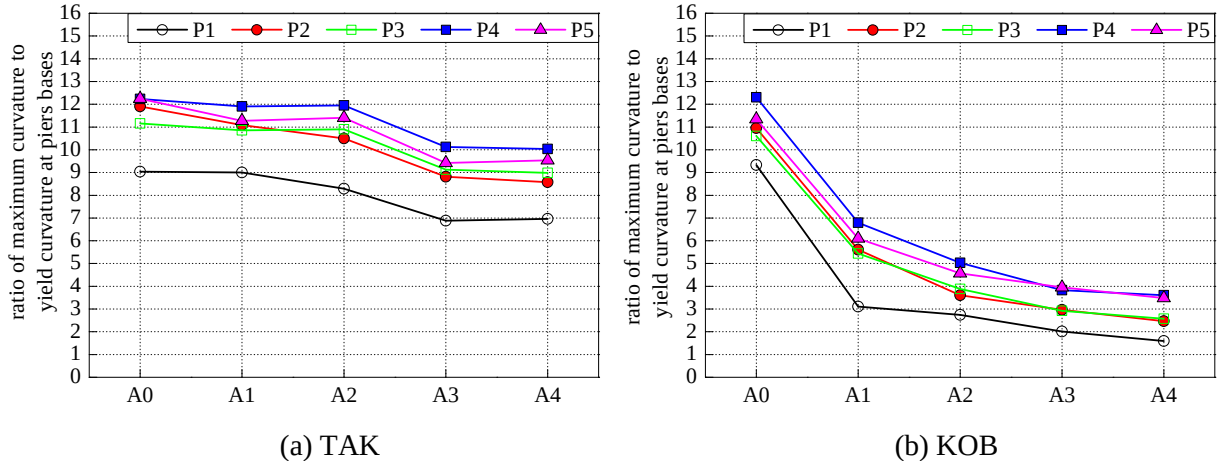


Fig. 7.11 Ratio of maximum curvature to yield curvature for TAK and KOB

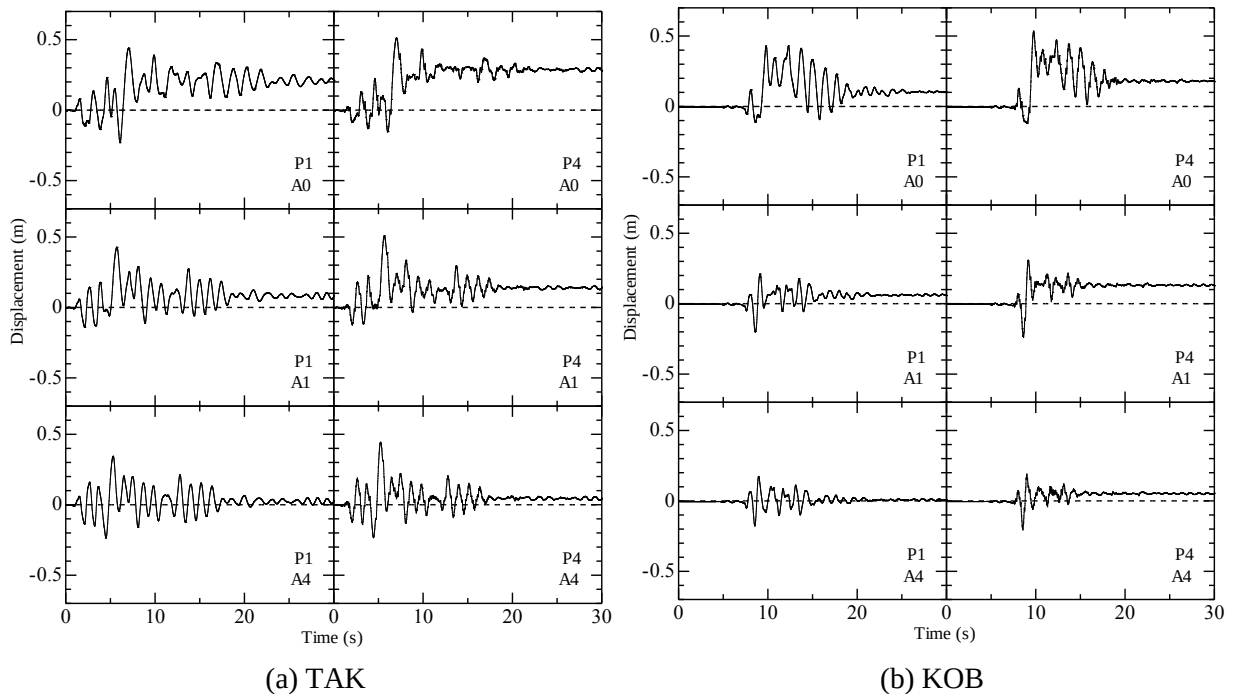


Fig. 7.12 Displacement time-history on top of piers (P1 and P4) for TAK and KOB

words, residual curvature at pier base lead to residual displacement on top of pier which results in residual pier inclination. Maximum curvature at pier base could cause maximum pier inclination to a certain degree. In addition, residual/maximum pier inclinations and residual/maximum curvature at pier base also present similar change trend as can be seen in **Fig. 7.8**, **Fig. 7.9**, **Fig. 7.10** and **Fig. 7.11**. Since residual curvature is determined by curvature time history, especially, those curvatures which are larger than yield curvature. Therefore, maximum curvature at pier base has important effect on residual curvature to a certain degree. Since pounding forces between roller bearing and stopper is much larger than fixed bearing forces, residual and maximum curvature at pier (P1) base, on top of which is fixed bearing, presents smallest value.

Finally, displacement time-history on top of piers (in case of A0, A1 and A4) for TAK and KOB input earthquakes are shown in **Fig. 7.12**. At the beginning, displacement on top of piers presents positive direction trend for TAK and KOB input earthquakes. Viaducts with

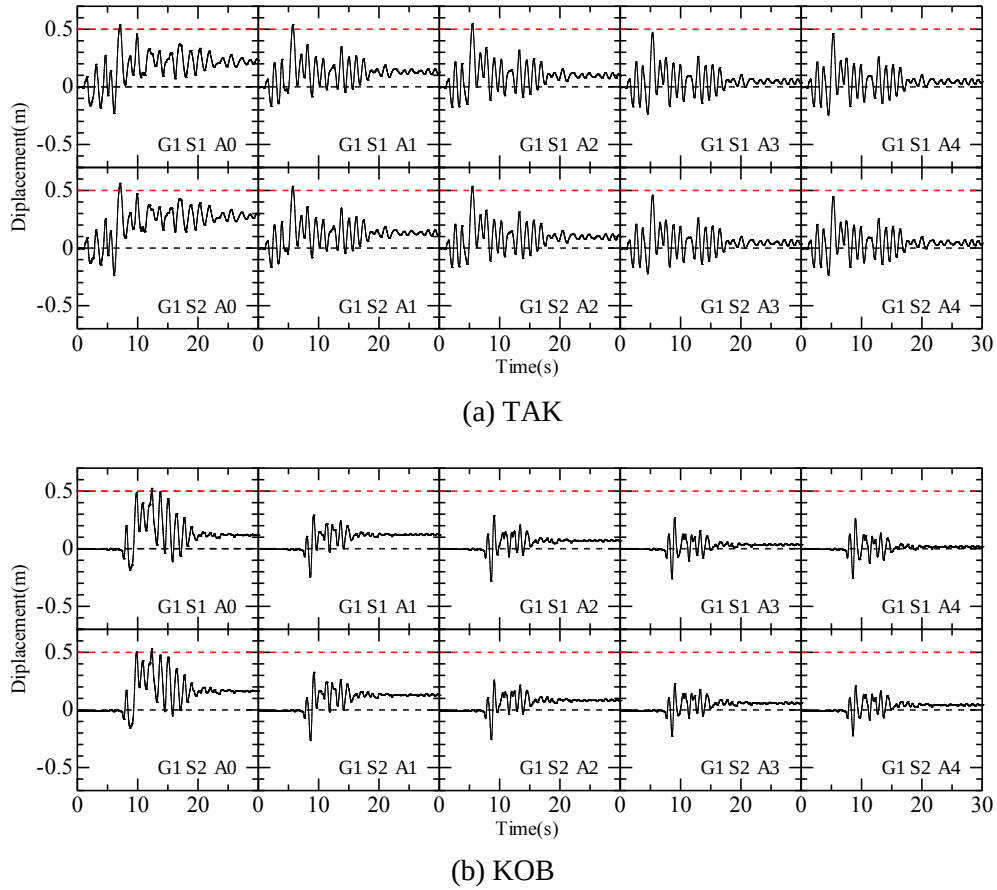
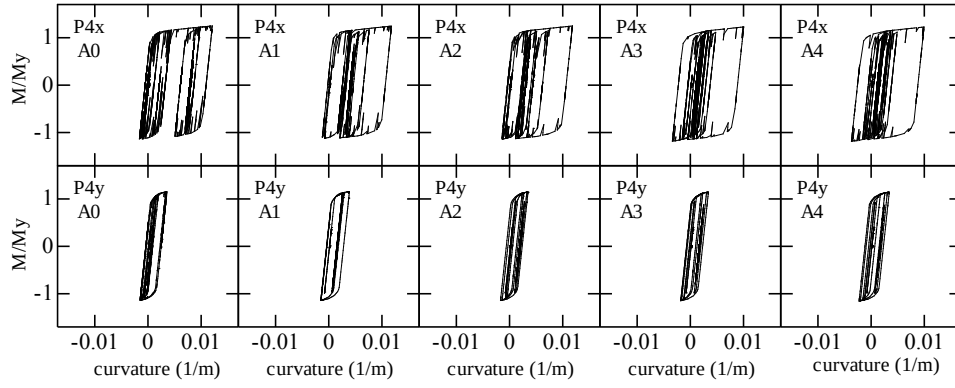


Fig. 7.13 Displacement time-history of simple supported span (S1) and continuous span (S2) for TAK and KOB

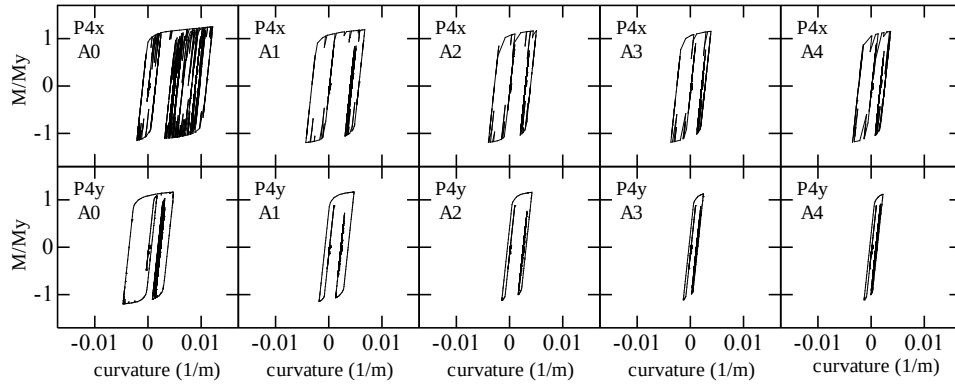
shock absorber devices present less maximum and less residual displacements for positive direction than those viaducts without shock absorber devices. As stiffness of shock absorber devices increases, the effect of shock absorber device reducing positive displacement becomes more obviously. Since shock absorber device could reduce maximum pounding force between roller bearing and stopper, which leads to less maximum bending moment and less maximum curvature at piers bases. Consequently, less curvature at piers bases results in less maximum and residual displacements on top of piers. In addition, maximum and residual displacement on top of P4 is larger than the one of P1 respectively, since pounding forces between roller bearing and stopper is significantly larger than fixed bearing (supported on top of P1) forces.

7.4.4 Displacement of superstructures

Displacements at the right end of simple supported span (S1) and the left end of continuous span for TAK and KOB input earthquakes are shown in **Fig. 7.13**. According to the previous analysis, large pounding force between roller bearing and stopper could result in large bending moment and curvature at piers bases, which leads to large maximum and residual displacements on top of piers. Therefore, maximum and residual displacements on top of piers have significant influence on displacement of superstructures. Hence, at the beginning, displacement of simple supported span and continuous span presents similarly positive direction trend for TAK and KOB input earthquakes comparing with displacement on top of piers. Shock absorber device could obviously reduce maximum and residual



(a) TAK



(b) KOB

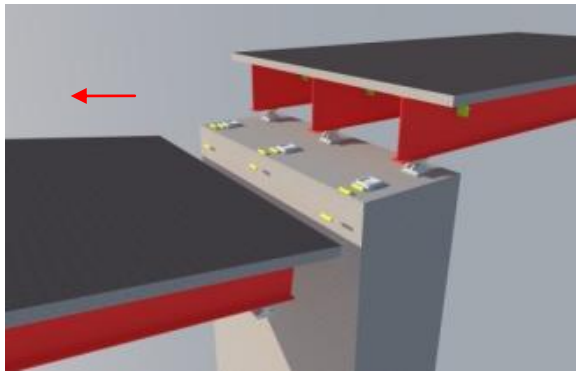
Fig. 7.14 Bending moment ratio-curvature relationship at pier (P4) base for TAK and KOB

displacements of simple supported span (S1) and continuous span (S2). As stiffness and resistance forces of shock absorber device increase, maximum and residual displacements of superstructures (S1 and S2) gradually decrease. As it also can be seen from **Fig. 7.13**, maximum and residual displacement of simple supported span (S1) and continuous span (S2) presents larger value respectively for TAK input.

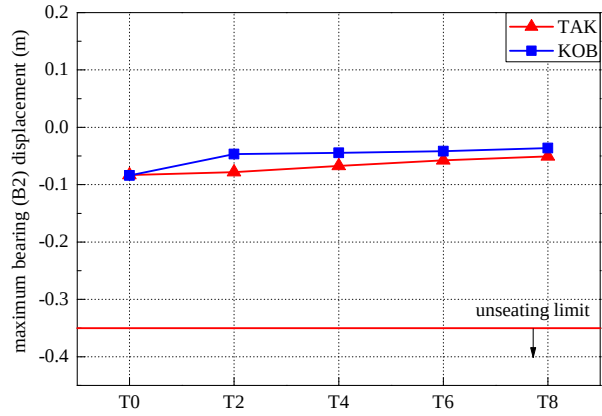
7.4.5 Relationship of bending moment and curvature

Generally, maximum and residual curvatures transmitted to piers bases can be considered to be an appropriate measure of seismic structural damages. In this analysis, central pier (P4) of continuous span, supported by roller bearing (B5), is likely subjected to large pounding forces between roller bearing and stopper during earthquakes. So, the relationship between curvature and the ratio of bending moment (M) to yield bending moment (M_y) for P4 in X and Y direction is shown **Fig. 7.14**. The relationship for other piers presents similar results.

Firstly, the absolute value of M/M_y overpasses 1 for both X and Y directions, which indicates plastic deformation at pier base in these two directions for TAK and KOB inputs as shown in **Fig. 7.14**. Moreover, maximum bending moment and curvature in X direction is larger than those in Y direction respectively. Since all the bearings are restrained in Y direction that leads to relative less bearing forces. So, maximum bending moment and curvature in X direction play more important role in the damages of pier base. Maximum and residual curvatures at pier base in case of shock absorber devices are less than the case without shock absorber devices. Finally, as stiffness of shock absorber devices increase,



(a) Schematical of deck unseating damage



(b) Maximum bearing (B2) displacement

Fig. 7.15 Deck unseating damage for TAK and KOB

maximum and residual curvatures gradually decrease for TAK and KOB input earthquakes. Since shock absorber devices could reduce pounding forces between roller bearing and stopper by smooth changes of pounding stiffness. As stiffness of shock absorber devices increase, the effect of shock absorber devices on reducing maximum and residual curvatures at pier base present more apparently. As it also can be seen from **Fig. 7.14**, maximum bending moment presents the similar change trend with maximum curvature at pier base.

Conclusively, viaduct with large stiffness of shock absorber devices between roller bearing and stopper shall present less maximum bending moment and less maximum curvature at piers bases during earthquakes.

7.5 Effect of Thickness

For an easy identification of the different calculation cases, a specific nomenclature is adopted in this research. “T” refers to thickness of shock absorber device. So “T2” presents that the thickness of shock absorber device is 2 cm. And “T0” indicates the viaduct without shock absorber device. Two earthquake ground motions were used as seismic excitations. The first ground motion was recorded at the JR Takatori Station in 1995 Kobe Earthquake. The second ground motion was recorded at the JMA Kobe observatory in 1995 Kobe Earthquake.

7.5.1 Deck unseating damage

Fig. 7.15 shows that absolute values of maximum bearing (B2) displacements in negative direction are all smaller than limit absolute value for all cases. Maximum absolute bearing (B2) displacements in negative direction are 8 cm in case of T0 for both TAK and KOB inputs. Since there is not shock absorber device installed between roller bearing and stopper, the gap between roller bearing and stopper is 8 cm. However, as shock absorber device is equipped between roller bearing and stopper, maximum absolute bearing (B2) displacement in negative direction presents smaller than 8 cm. Shock absorber device could reduce maximum bearing (B2) displacements in negative direction due to its stiffness and resistance forces. It also can be seen from **Fig. 7.15** that as thickness of shock absorber device increases, maximum absolute bearing (B2) displacement for negative direction decreases. In other words, the possibility of unseating of a deck decreases with thickness of shock absorber device increasing. Since large thickness of shock absorber leads to smaller gap value between shock absorber device and roller bearing as shown in **Fig. 7.2**, shock absorber device could be activated earlier during earthquakes that could afford external forces earlier.

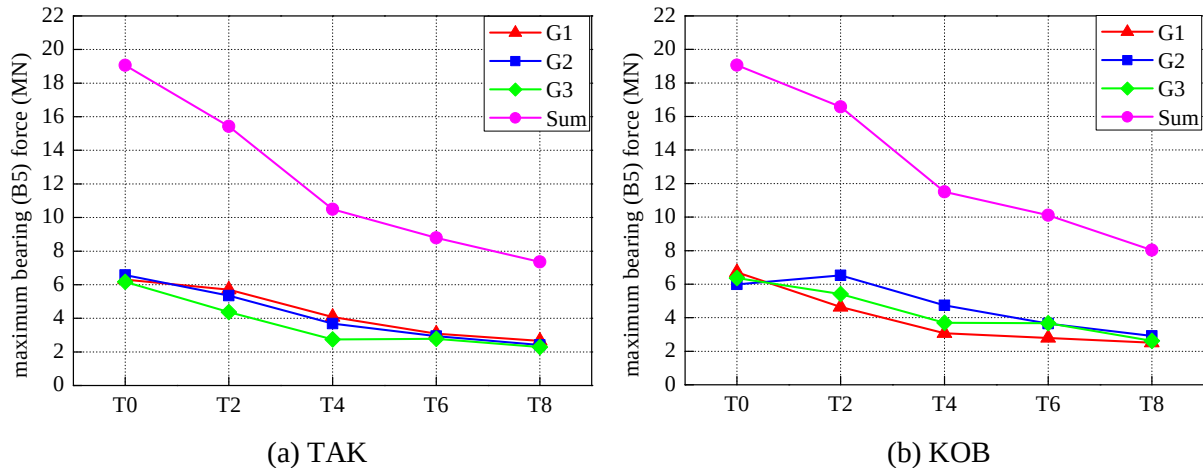


Fig. 7.16 Maximum roller bearing (B5) forces for TAK and KOB

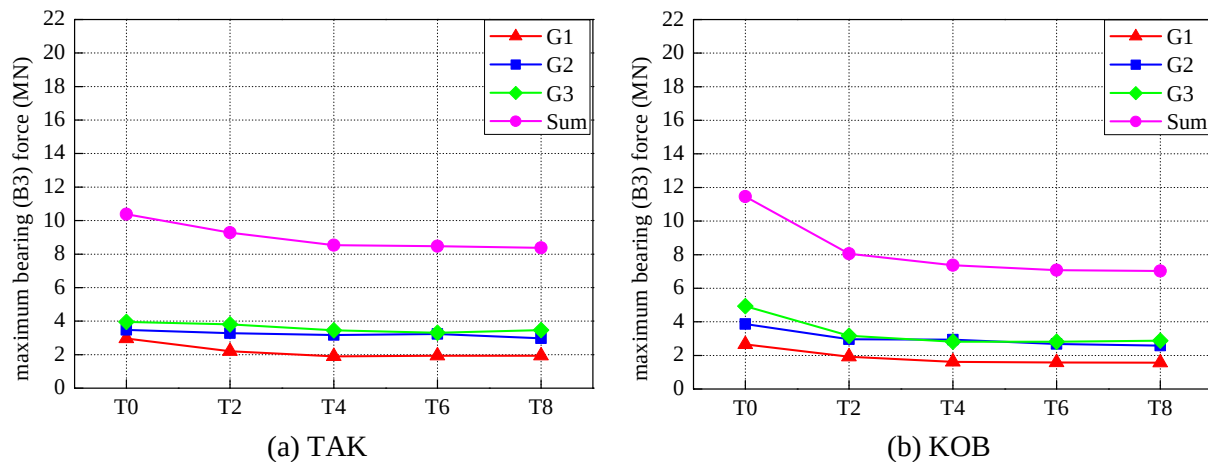
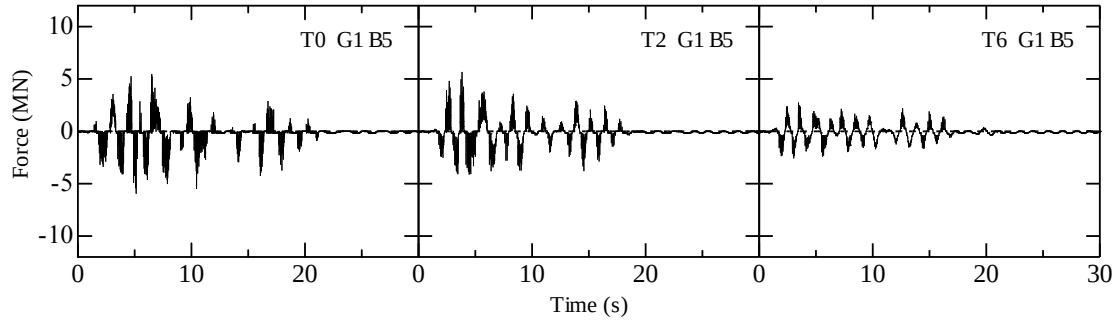


Fig. 7.17 Maximum fixed bearing (B3) forces for TAK and KOB

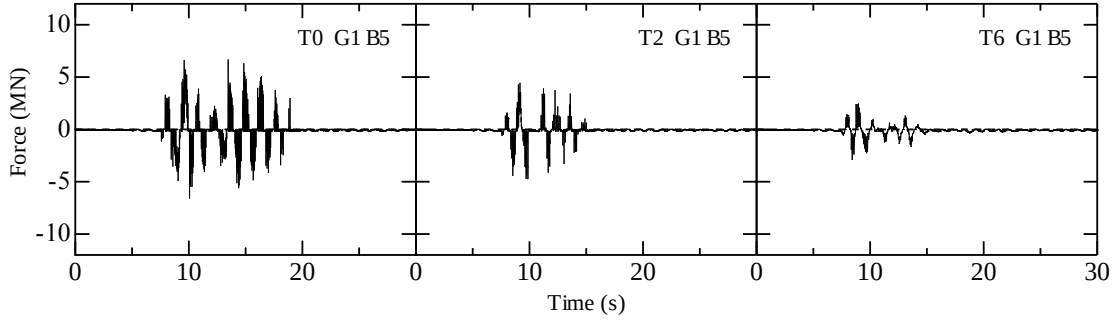
7.5.2 Bearing force

Relatively large bearing forces could bring damage at not only bearing supports but also at the piers bases. Bearing support damage is one of the reasons for falling down of superstructures. However, the damage of piers bases is always catastrophic and irreparable, which leads to the viaducts lose their serviceability. So it is very important to find an appropriate method to reduce maximum bearing forces. In order to reduce the pounding forces, shock absorber devices are installed between roller bearing and stopper in this analysis. Because continuous span (supported on B3, B4, B5 and B6) has larger inertial force than simple supported span, the bearings forces of continuous span are relatively large. Therefore, maximum bearing (B5 and B3) forces are presented in **Fig. 7.16** and **Fig. 7.17**, representing maximum roller bearing and maximum fixed bearing forces of continuous span respectively. Other maximum bearings forces exhibit the similar change trend.

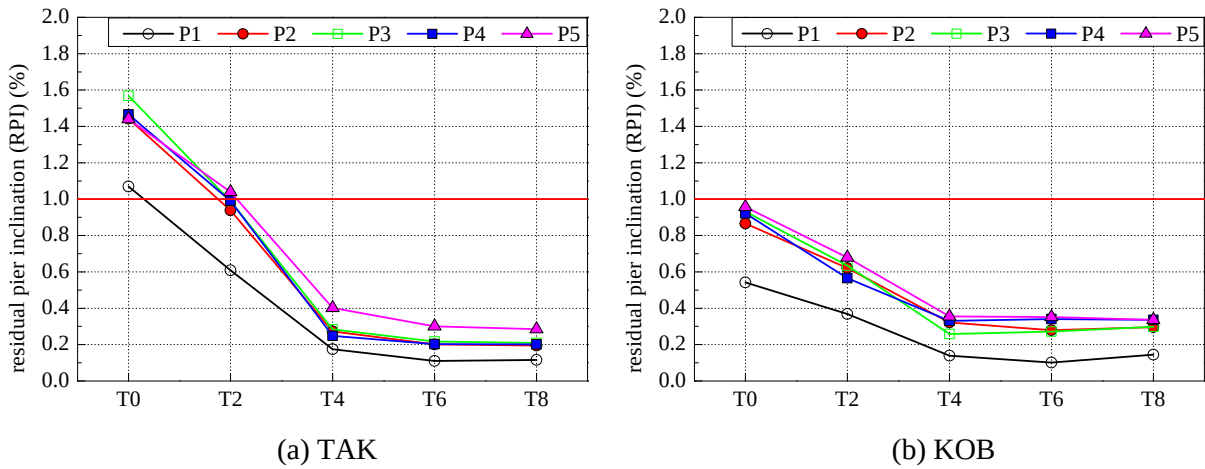
It can be seen from **Fig. 7.16** that maximum roller (B5) forces with shock absorber devices are smaller than the case without shock absorber devices. Shock absorber devices can significantly reduce the pounding forces between roller bearing and stopper through smooth changes of impact stiffness. Maximum pounding force between roller bearing and stopper decreases with the increase of thickness of shock absorber device. Since the gap value between roller bearing and shock absorber device is less as thickness of shock absorber device



(a) TAK



(b) KOB

Fig. 7.18. Roller bearing (B5) force time history for TAK and KOB inputs

(a) TAK

(b) KOB

Fig. 7.19 Residual pier inclination for TAK and KOB

is large one. In this case, during strong earthquakes, shock absorber device could be activated earlier that results in affording external forces earlier. Therefore, viaduct supported on roller bearing supports with large thickness of shock absorber device, shall subject to less damage at bearing supports and piers bases. Moreover, **Fig. 7.16** and **Fig. 7.17** show that maximum roller bearing (B5) forces are much larger than maximum fixed bearing (B3) forces for TAK and KOB inputs respectively. In other words, pounding forces between roller bearing and stopper play an important role in bearing damage and pier base damage than fixed bearing forces. Besides as thickness of shock absorber device increases, fixed bearing (B3) forces could also be gradually reduced by the indirectly effect of maximum roller (B5) bearing forces.

The pounding force time history between roller bearing (B5) and stopper without/with shock absorber device of different thickness is analyzed. The force of roller bearing

supporting inside girder (G1) and combined with different thickness (T0, T2 and T6) of shock absorber device is presented in **Fig. 7.18**. Other roller bearing force time history presents the similar change trend. **Fig. 7.18** shows that the application of shock absorber device could obviously reduce roller bearing (B5) forces in both positive and negative directions for TAK and KOB inputs. Since shock absorber devices are installed at both sides of all the roller bearings in tangential direction. Moreover, due to earlier activation of shock absorber device and smooth changes of impact stiffness, contacts time last longer but the pounding forces between roller bearing (B5) and stopper are much smaller. As thickness of shock absorber device increases, the pounding forces between roller bearing (B5) and stopper in both positive and negative directions gradually decrease. Since large thickness of shock absorber device leads to less gap between roller bearing and shock absorber device that results in earlier activation of shock absorber device.

7.5.3 Pier damage

At the beginning, top of pier presents residual pier inclination in positive direction for TAK and KOB inputs in this analysis. It can be seen from **Fig. 7.16** and **Fig. 7.19** that residual pier inclination and maximum roller bearing forces exhibit similar change trend. Since large pounding forces acting on stoppers of roller bearings are transferred to the piers with a sharp increment of bending moment and curvature at piers bases, which lead to large residual pier inclination. Viaducts with shock absorber devices present less residual pier inclination than those without shock absorber devices. In addition, as thickness of shock absorber device increases, residual pier inclination decreases. Residual pier inclination of viaduct without shock absorber devices is larger than 1% for TAK input in **Fig. 7.19a**, which indicates serious pier damage. Besides, viaduct with shock absorber device whose thickness is 2 cm also presents serious damage at piers bases for TAK input. However, there is not obvious damage at piers bases in all cases for KOB input as shown in **Fig. 7.19b**. Since shock absorber devices between roller bearing and stopper could significantly reduce both positive and negative pounding forces through smooth changes of impact stiffness. Less positive and negative pounding forces between roller bearing and stopper could result in less positive residual pier inclination. As thickness of shock absorber device increases, shock absorber device could be activated earlier. Therefore, the effect of shock absorber device on reducing residual pier inclination could become more obviously. **Fig. 7.19** also shows that residual pier inclination of pier 1 (P1), supported on fixed bearings, presents less value than the one of other piers. Since maximum fixed bearing forces are much smaller than pounding forces between roller bearing and stopper that leads to less residual displacement on top of pier.

In addition, residual pier inclination and residual curvature at pier base present almost same change trend for TAK and KOB inputs respectively in **Fig. 7.19** and **Fig. 7.20**. In other words, residual curvature at pier base could lead to residual displacement on top of pier which results in residual pier inclination. Residual pier inclination and residual curvature at pier base significantly decreases, as thickness of shock absorber device increases up to half gap between roller bearing and stopper. Therefore, shock absorber device with thickness of half gap could provide economical and effective design that could obviously reduce residual pier inclination and residual curvature at pier base. **Fig. 7.21** and **Fig. 7.22** show that maximum pier inclination and maximum curvature at pier base in positive direction also present same change trend. Maximum curvature at pier base could cause maximum pier inclination to some extent. Moreover, residual/maximum pier inclinations and residual/maximum curvatures at piers bases exhibit similar change trend in **Fig. 7.19**, **Fig. 7.20**, **Fig. 7.21** and **Fig. 7.22**. Since residual curvature is determined by curvature time history, especially, those curvatures which are equal to yield curvature. Therefore, maximum curvature plays an important role in

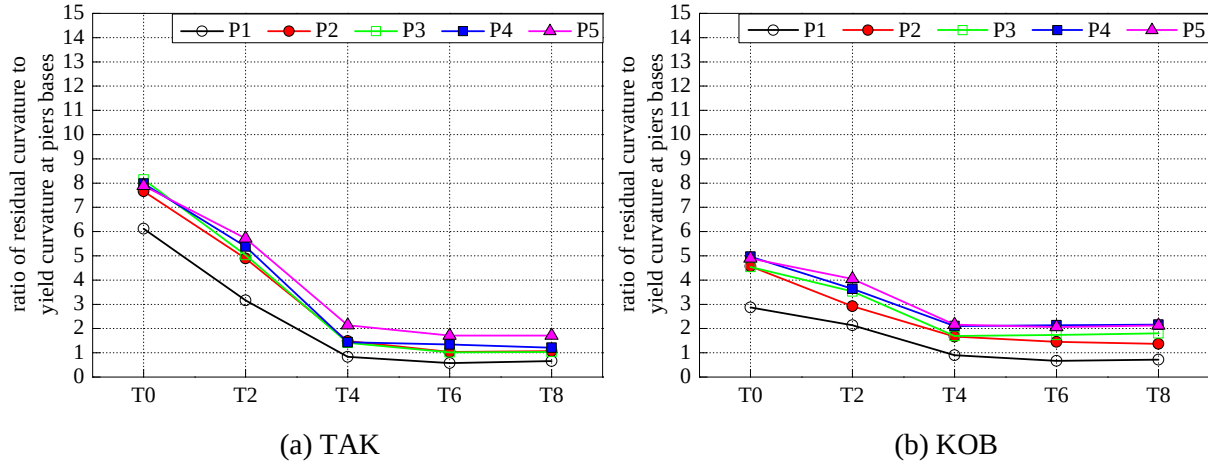


Fig. 7.20 Ratio of residual curvature to yield curvature for TAK and KOB inputs

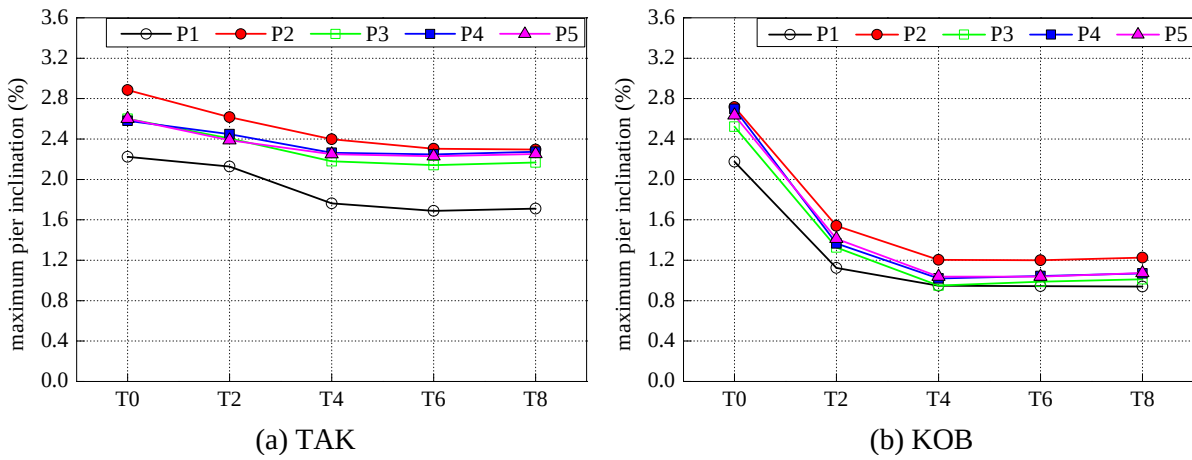


Fig. 7.21 Maximum pier inclination for TAK and KOB inputs

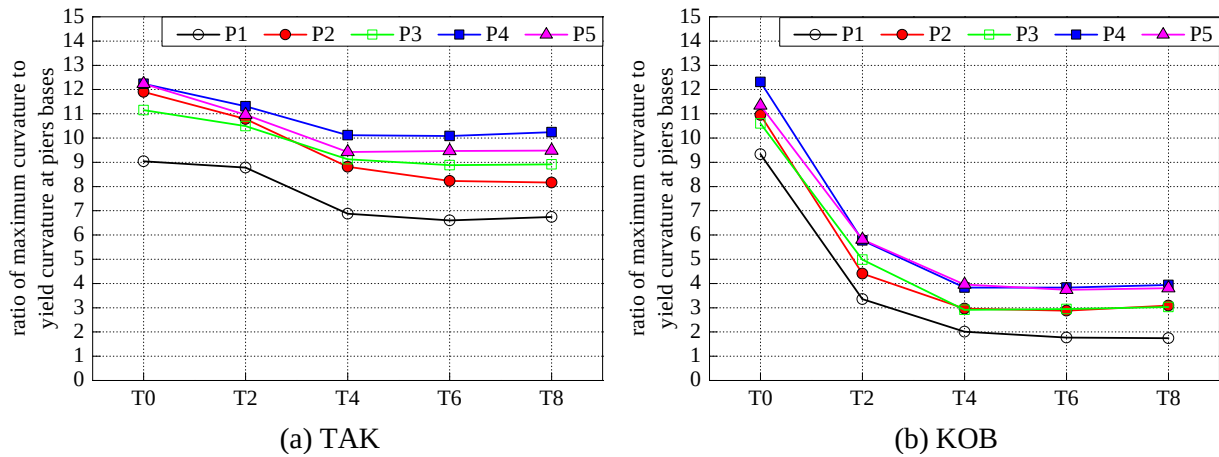


Fig. 7.22 Ratio of maximum curvature to yield curvature for TAK and KOB inputs

residual curvature at pier base. However, because fixed bearing forces are much less than pounding forces between roller bearing and stopper, residual and maximum curvatures at pier (P1) base, whose top is fixed bearing, present smallest value.

In order to study residual and maximum pier inclinations clearly, displacement time-history on top of piers (in case of T0, T2 and T6) for TAK and KOB inputs are presented in

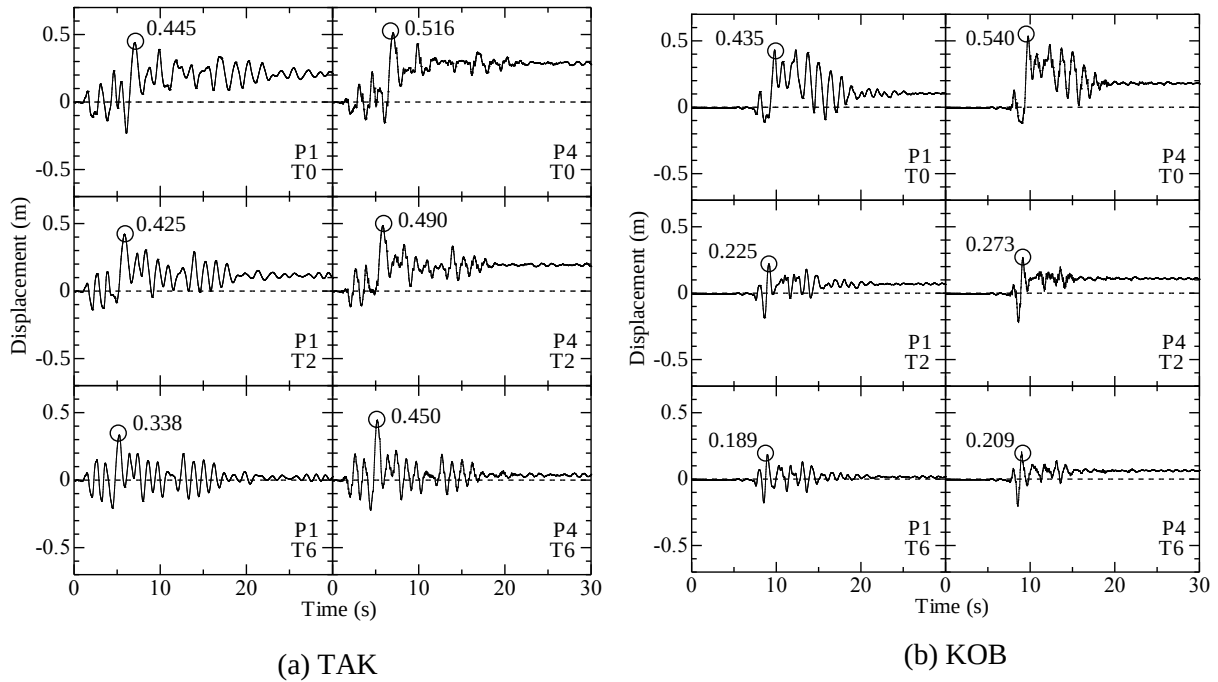


Fig. 7.23 Displacement time-history on top of piers (P1 and P4) for TAK and KOB inputs

Fig. 7.23. Other cases including T4 and T8 present similar change trend. During TAK and KOB input earthquakes, displacements on top of piers present positive direction trend. Viaducts with shock absorber devices present less maximum and residual displacements in positive direction than those viaducts without shock absorber devices. In addition, maximum and residual displacements in positive direction gradually decrease with the increase of thickness of shock absorber device. Due to both sides of roller bearings equipped with shock absorber devices in longitudinal direction, shock absorber device could reduce maximum roller bearing forces in both positive and negative directions which results in less maximum bending moment and curvature at piers bases. Consequently, less maximum curvature at piers bases in positive direction results in less maximum and residual displacements on top of piers in positive direction as shown in **Fig. 7.23**. Moreover, pounding forces between roller bearing (supported on top of P4) and stopper is obviously larger than fixed bearing (supported on top of P1) forces. So, maximum and residual displacements on top of P4 are larger than the one of P1 respectively.

7.5.4 Displacement of superstructures

It can be seen from **Fig. 7.19**, **Fig. 7.21** and **Fig. 7.24** that residual/maximum pier inclinations and residual/maximum displacements of superstructures present similar change trend. Maximum and residual displacements of superstructures are determined by maximum and residual displacements on top of piers. Viaduct with shock absorber device presents less maximum and residual displacements of superstructures. As thickness of shock absorber device increases, maximum and residual displacements of superstructures decrease as shown in **Fig. 7.24**. It is shock absorber device that reduces the pounding forces between roller bearing and stopper.

Fig. 7.25 shows displacements at the right end of simple supported span (S1) and the left end of continuous span (S2) for TAK and KOB input earthquakes. At the beginning, displacement of simple supported span (S1) and continuous span (S2) presents similarly positive direction trend for TAK and KOB input earthquakes comparing with displacement on

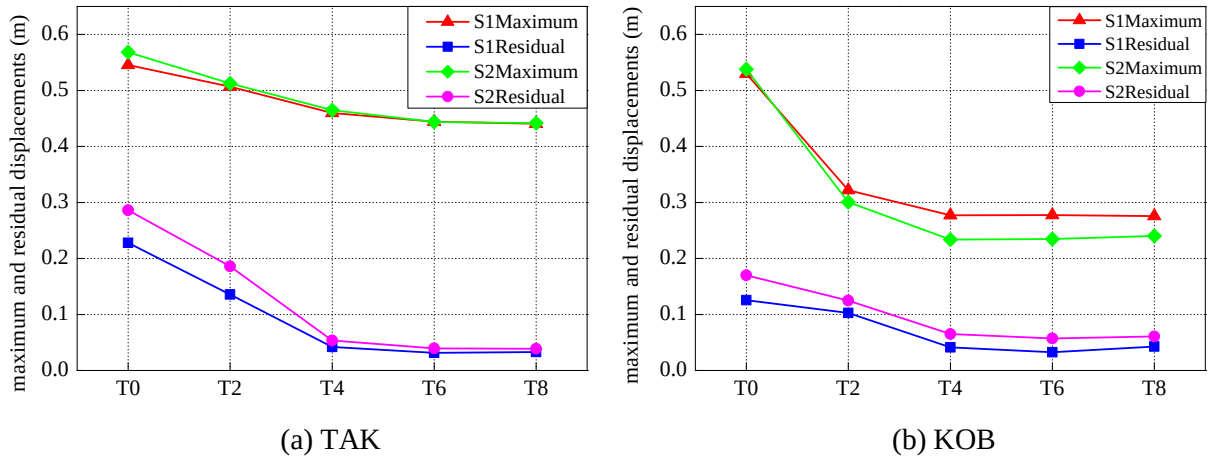


Fig. 7.24 Maximum and residual displacements of simple supported span (S1) and continuous span (S2) for TAK and KOB inputs

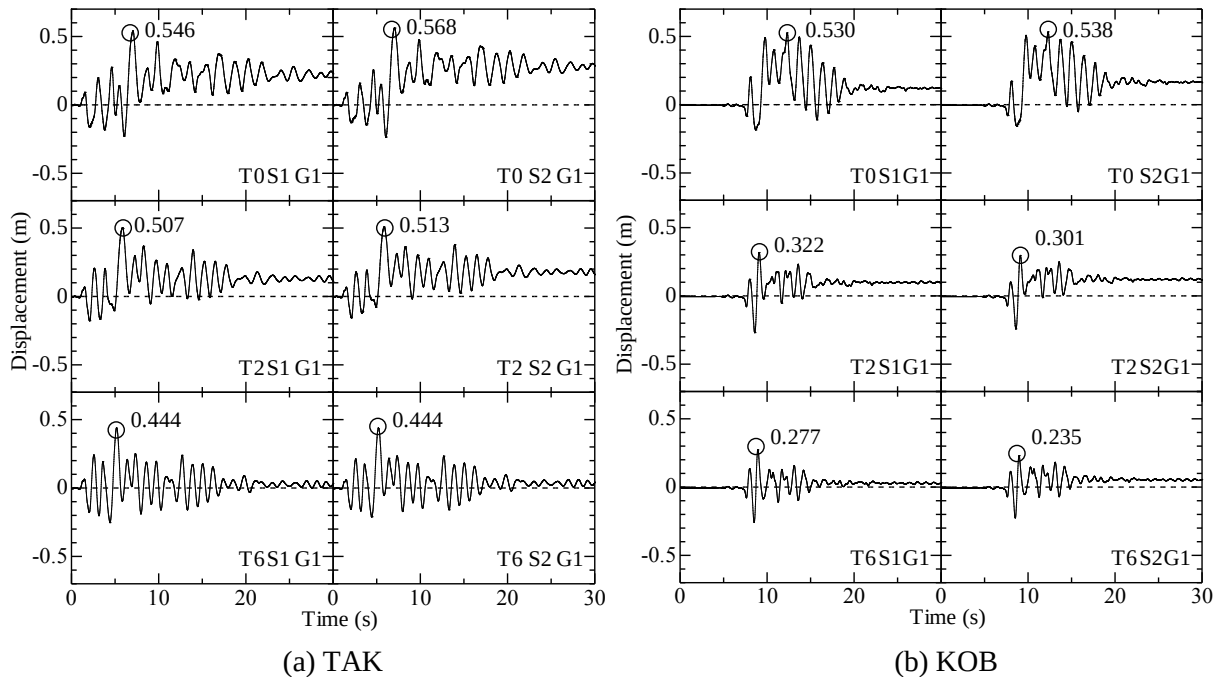


Fig. 7.25 Displacement time-history of simple supported span (S1) and continuous span (S2) for TAK and KOB

top of piers. Maximum and residual displacements of superstructures in positive direction could be reduced by shock absorber device. Maximum and residual displacements of superstructures in positive direction decrease with the increase of thickness of shock absorber device. Large thickness of shock absorber device could be activated earlier during earthquakes, which affects on reducing displacements of superstructures in positive direction.

7.5.5 Relative displacement between two decks

Relative displacements between two decks (in case of T0, T2 and T6) for TAK and KOB inputs are only shown in **Fig. 7.26**. Other cases including T4 and T8 exhibit similar change trend. The dashed red line presents gap between roller bearing and stopper which is 8 cm. A

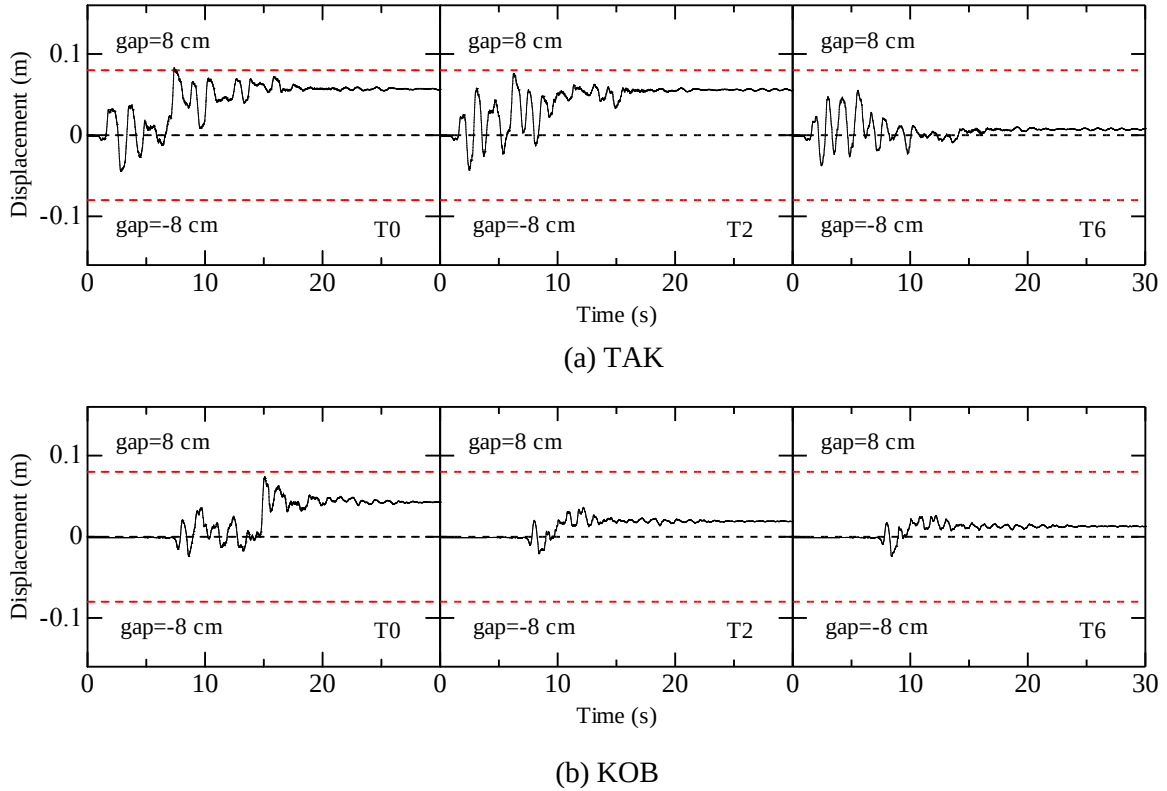


Fig. 7.26 Relative displacement between two decks for TAK and KOB inputs

positive relative displacement between two decks corresponds to an opening (outward) of the expansion joint while a negative relative displacement corresponds to a closing (inward). It can be seen from **Fig. 7.26** that relative displacements present opening trend in all cases for TAK and KOB inputs. However, since positive relative displacement between two decks is larger than the gap between roller bearing and stopper, pounding occurs in case of the viaduct without shock absorber device. Considering viscous damper equipped in the viaduct, pounding in closing direction does not occur. Since viscous damper could afford some external forces that leads to less relative closing value. In addition, viaduct with shock absorber device exhibits less maximum positive displacement. As thickness of shock absorber device increases, maximum and residual positive displacements decrease as it presented in **Fig. 7.26**. Since large thickness of shock absorber device leads to less gap value between roller bearing and steel stopper, shock absorber device could be activated earlier during earthquakes.

7.5.6 Relationship of bending moment and curvature

In this analysis, central pier (P4) of continuous span, supported on roller bearing (B5), is likely subjected to large pounding forces between roller bearing and stopper during earthquakes. Therefore, the relationship between curvature and the ratio of bending moment (M) to yield bending moment (M_y) for P4 in X and Y direction is shown **Fig. 7.27**. The relationship of bending moment and curvature at other piers bases present similar results.

It can be seen from **Fig. 7.27** that the absolute value of M/M_y overpasses 1 in both X and Y directions, which indicates plastic deformation at pier base in these two directions. Maximum curvature at pier base in case of shock absorber devices are less than the case without shock absorber devices. As thickness of shock absorber device increases, maximum curvature gradually decreases. Since shock absorber devices could reduce pounding forces

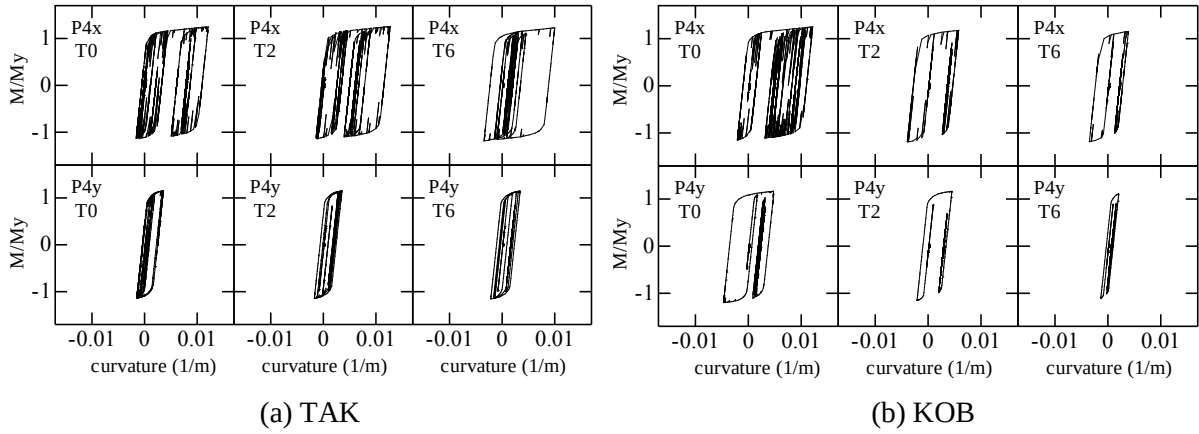


Fig. 7.27 Bending moment ratio-curvature relationship at pier (P4) base for TAK and KOB

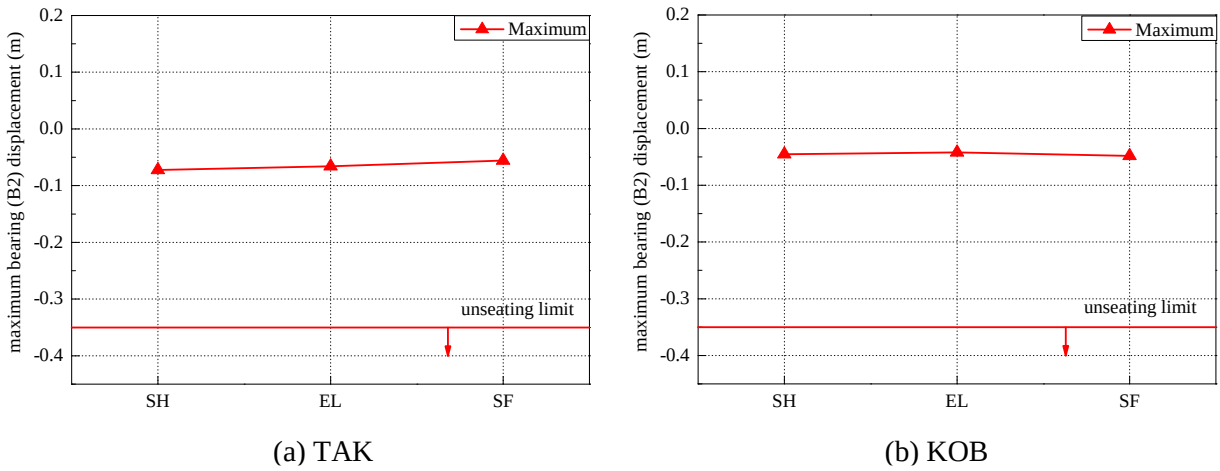


Fig. 7.28 Deck unseating damage for TAK and KOB

between roller bearing and stopper through smooth changes of impact stiffness. Moreover, large thickness of shock absorber device could be activated earlier during earthquakes, which results in less pounding forces between roller bearing and stopper. Besides, maximum bending moment and maximum curvature in X direction is larger than those in Y direction respectively. Since all the bearings are restrained in Y direction that leads to relatively less bearing forces. So, maximum bending moment and maximum curvature at piers bases in X direction play more important effect on the damages of piers bases.

7.6 Effect of SAD's Type

For an easy identification of the different calculation cases, a specific nomenclature is adopted in this research. “SH” refers to strain hardening shock absorber device. “EL” presents elastic shock absorber device. And “SS” indicates strain softening shock absorber device. Two earthquake ground motions were used as seismic excitations. One was recorded at the JR Takatori Station in 1995 Kobe Earthquake. The other one was recorded at the JMA Kobe observatory in 1995 Kobe Earthquake.

7.6.1 Deck unseating damage

It can be seen from **Fig. 7.28** that maximum bearing (B2) displacement in negative direction present almost same value in case of different types of SADs for TAK and KOB

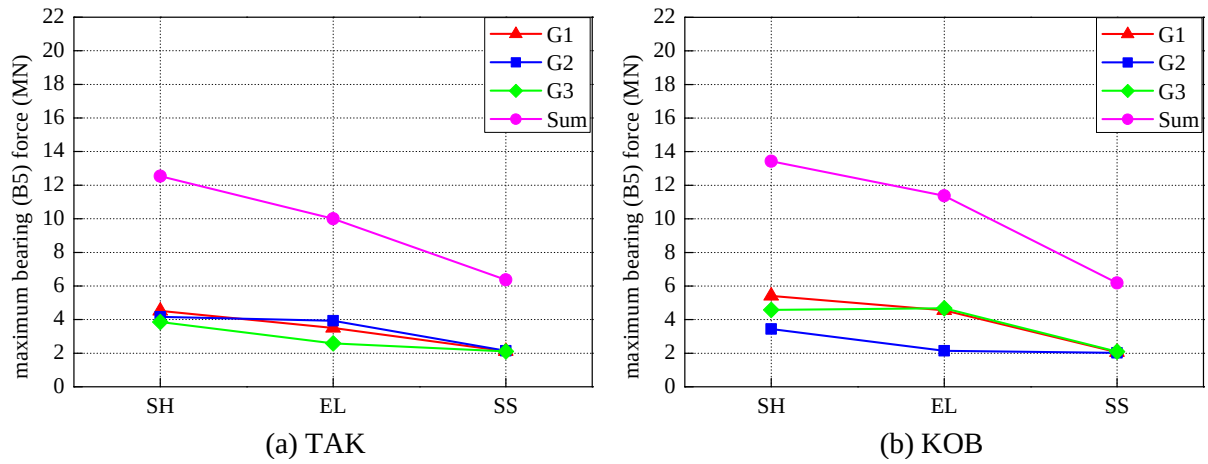


Fig. 7.29 Maximum roller bearing (B5) forces for TAK and KOB

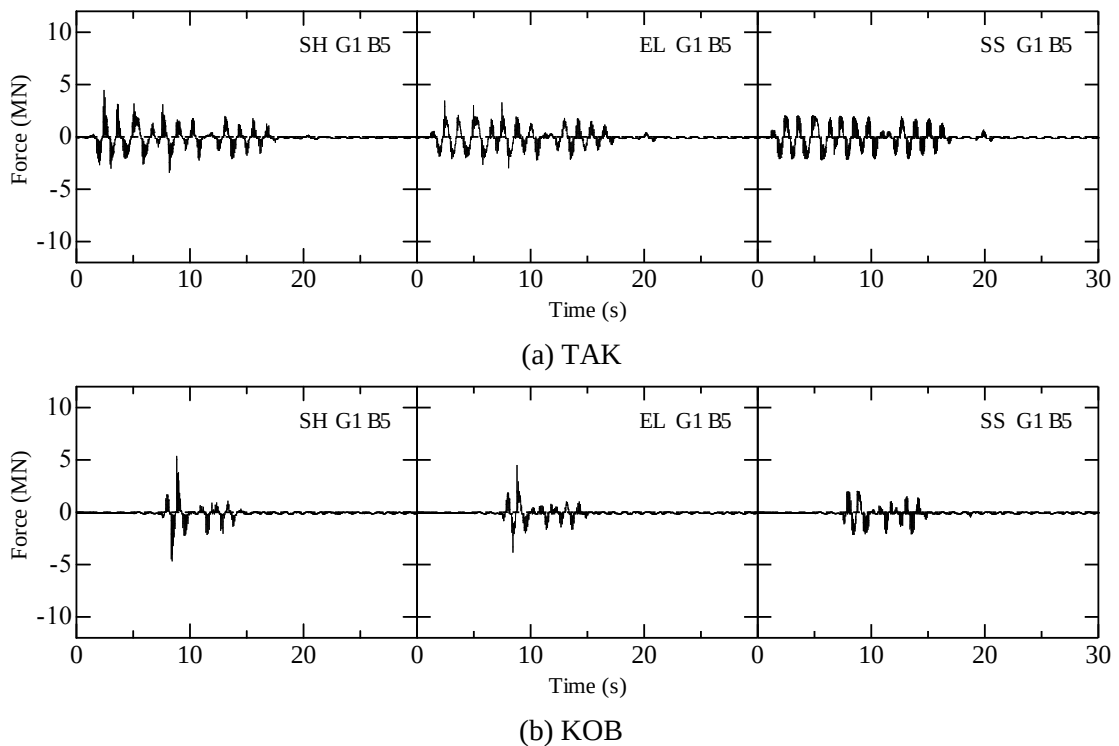


Fig. 7.30 Roller bearing (B5) force time history for TAK and KOB inputs

input earthquakes. Therefore, the type of SADs does not play an important role in the possibility of deck unseating damage.

7.6.2 Bearing force

Roller bearing (B5) forces, supporting continuous span, are shown in **Fig. 7.29**. Other roller bearing forces present the similar change trend. The maximum bearing (B5) force installed of strain softening shock absorber device presents smallest value. However, maximum bearing (B5) force installed of elastic (EL) shock absorber device exhibits the middle between strain hardening (SH) SAD and strain softening (SS) SAD. Since the restoring force at small relative displacement is higher in strain softening SAD than other two types of SADs. Strain softening SAD could afford more external forces at initial phase of

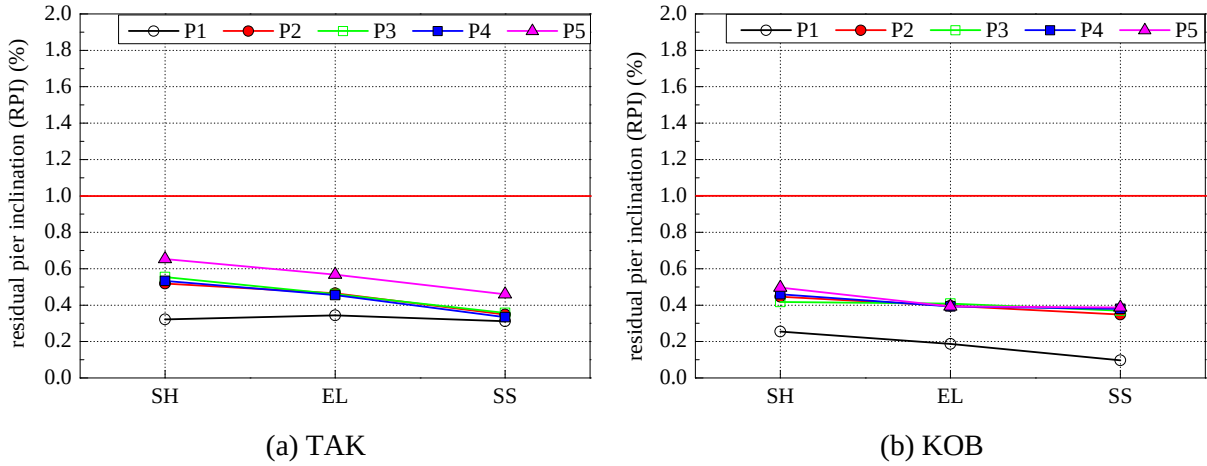


Fig. 7.31 Residual pier inclination for TAK and KOB inputs

pounding between roller bearing and stopper that results in less pounding forces at later phase of pounding phenomenon.

Roller bearing (B5) force time history is shown in **Fig. 7.30**. Pounding between roller bearing and steel stopper occurs in case of strain hardening SAD and elastic SAD, which can be seen from spikes in the bearing force response as shown in **Fig. 7.30**. However, due to large restoring forces at small relative displacement in case of strain softening SAD, impact phenomenon between roller bearing and steel stopper shall not occur that leads to less bearing forces. Since obvious spikes could not be seen in case of strain softening SAD in **Fig. 7.30**.

7.6.3 Pier damage

In this analysis, top of piers presents residual pier inclination in positive direction for TAK and KOB inputs. **Fig. 7.31** shows that residual pier inclination values are less than limit value for all cases. Viaduct installed of strain softening shock absorber devices presents least residual pier inclination. However, viaduct installed of strain hardening shock absorber device exhibits the largest value. Due to higher restoring force at small relative displacement, strain softening SAD could afford more external forces at initial phase of pounding which results in less pounding forces at later phase of pounding phenomenon. Less pounding forces lead to less residual pier inclination. Residual curvature at pier base determines residual pier inclination, so they exhibit almost same change trend as shown in **Fig. 7.31** and **Fig. 7.32**. Since maximum curvature could lead to maximum pier inclination, they also present almost same change trend as shown in **Fig. 7.33** and **Fig. 7.34**. In addition, maximum curvature plays an important role in residual curvature at pier base. Therefore, residual curvature and maximum curvature exhibit similar change trend as shown in **Fig. 32** and **Fig. 7.34**. Residual and maximum curvatures at pier (P1) base present smallest value because of less fixed bearing forces.

Displacement time-history on top of piers (P1 and P4) for TAK and KOB inputs are shown in **Fig. 7.35**. During TAK and KOB input earthquakes, displacements on top of piers present positive direction trend. Viaduct installed of strain softening shock absorber devices presents least maximum and residual displacements on top of piers. However, viaduct equipped with strain hardening SADs exhibits largest displacement. Since strain softening SAD could afford more external forces at initial phase of pounding. In addition, since fixed bearing forces are much less than pounding forces between roller bearing and stopper, maximum and residual displacements on top of pier (P1) in positive direction present less value than the one on top of pier (P4) as shown in **Fig. 7.35**.

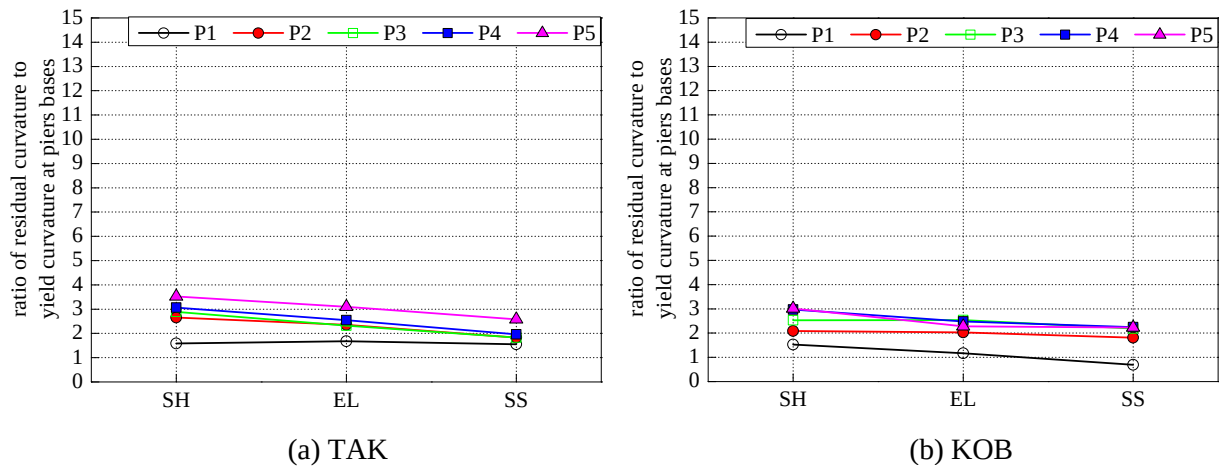


Fig. 7.32 ratio of residual curvature to yield curvature for TAK and KOB inputs

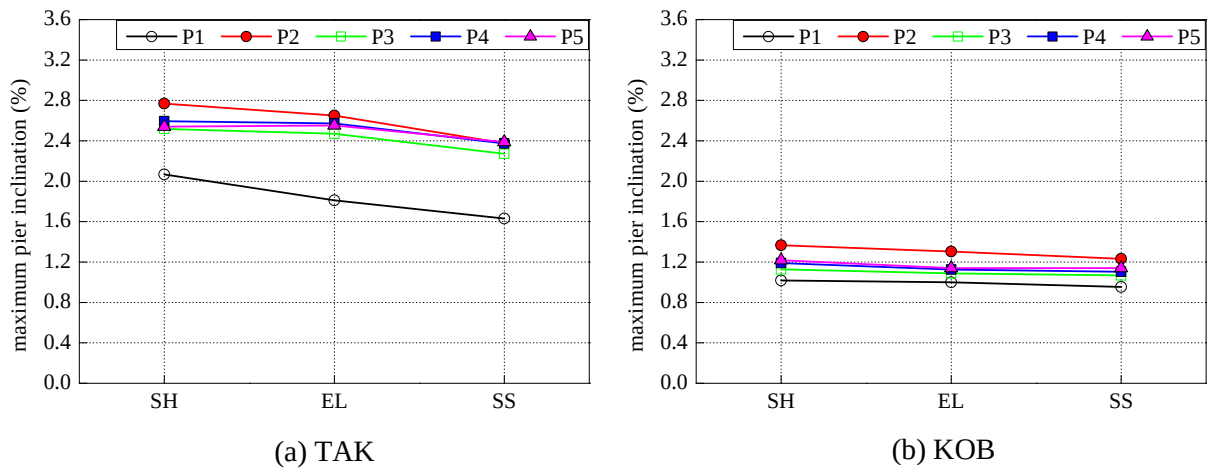


Fig. 7.33 Maximum pier inclination for TAK and KOB inputs

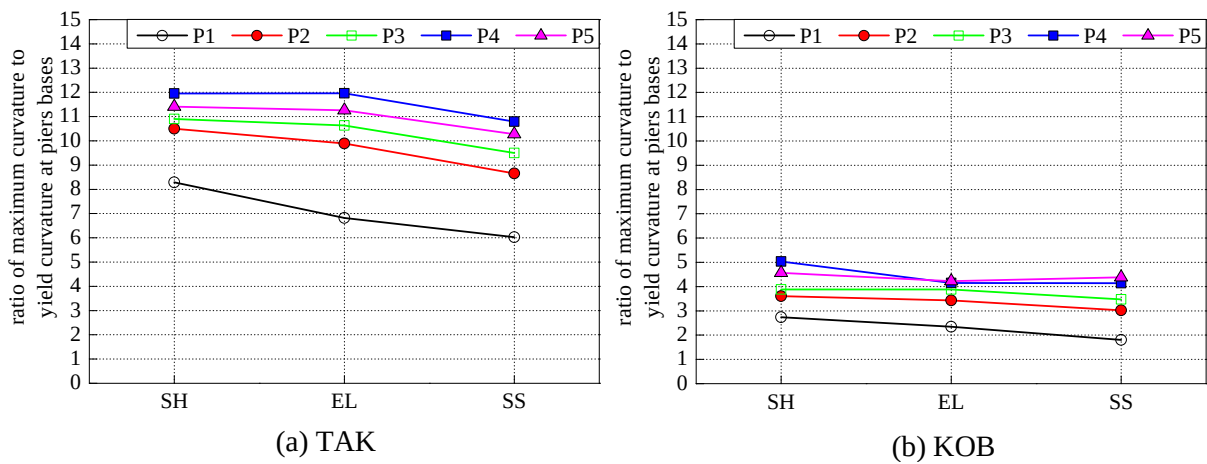


Fig. 7.34 Ratio of maximum curvature to yield curvature for TAK and KOB inputs

7.6.4 Displacement of superstructures

Viaduct with strain softening SADs presents least maximum and residual displacements of superstructures as shown in **Fig. 7.36**. Since maximum and residual displacements on top

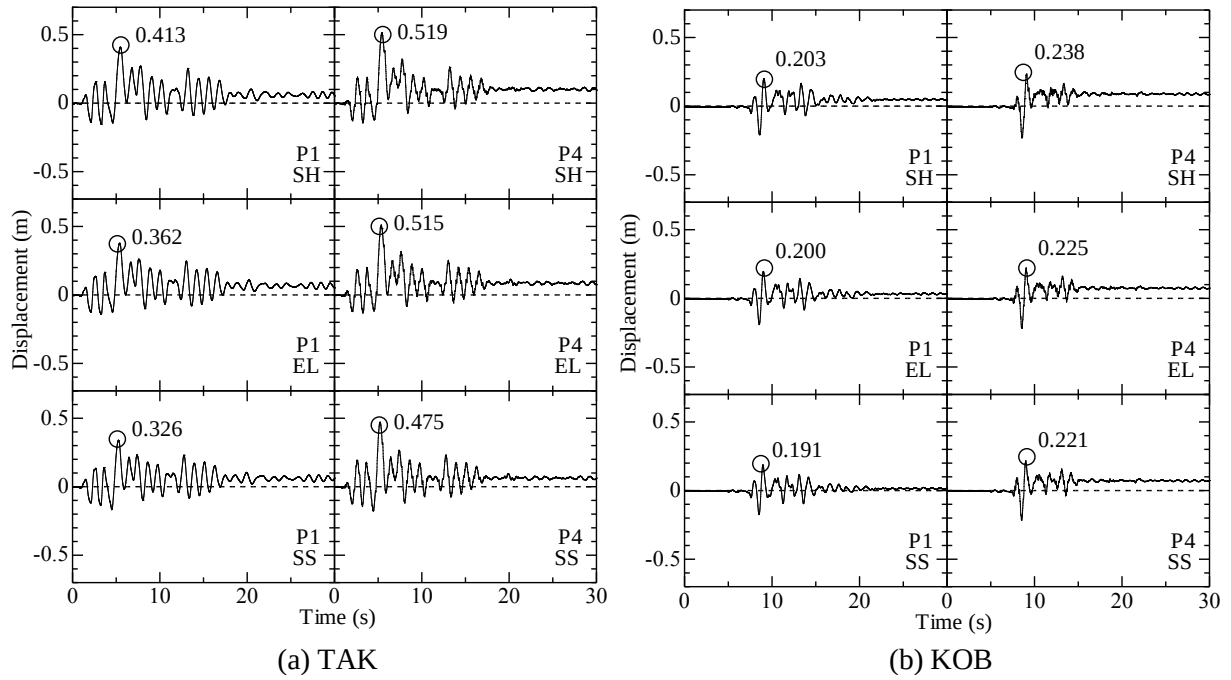


Fig. 7.35 Displacement time-history on top of piers (P1 and P4) for TAK and KOB

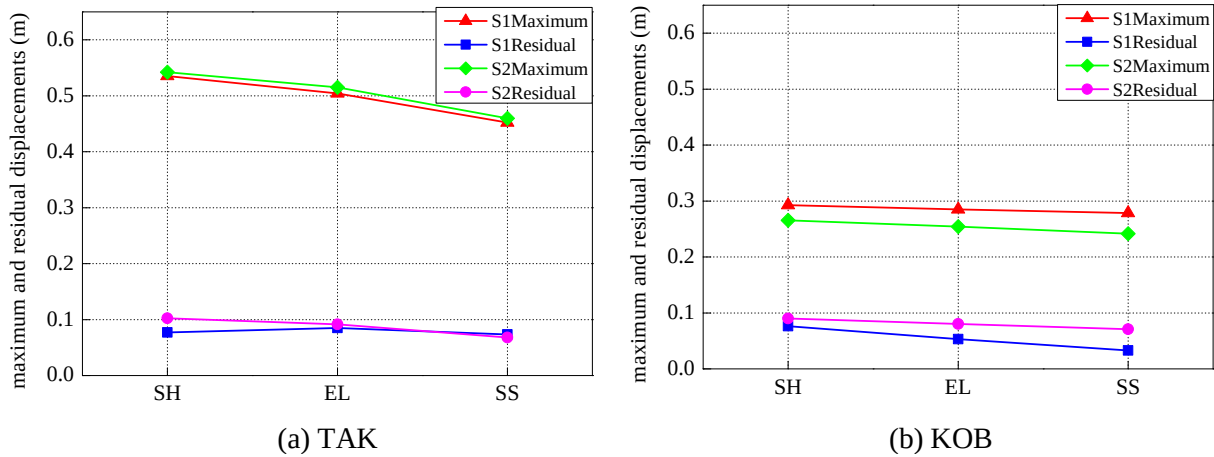


Fig. 7.36 Maximum and residual displacements of simple supported span (S1) and continuous span (S2) for TAK and KOB inputs

of piers play an important effect on maximum and residual displacement of superstructures, **Fig. 7.36** shows the similar change trend with **Fig. 7.31** and **Fig. 7.33** respectively.

During earthquakes, simple supported span (S1) and continuous span (S2) exhibit residual displacement in positive direction in **Fig. 7.37**. Absolute maximum displacement in positive direction is larger than the value in negative direction. In addition, strain softening SADs could greatly reduce maximum and residual displacements of superstructures. Since the restoring force at small relative displacement is higher in strain softening SAD that could afford more external forces at initial phase of pounding.

7.6.5 Relationship of bending moment and curvature

Since central pier (P4) of continuous span, supporting more weight, shall afford more inertial force during earthquakes. So, the relationship between curvature and the ratio of

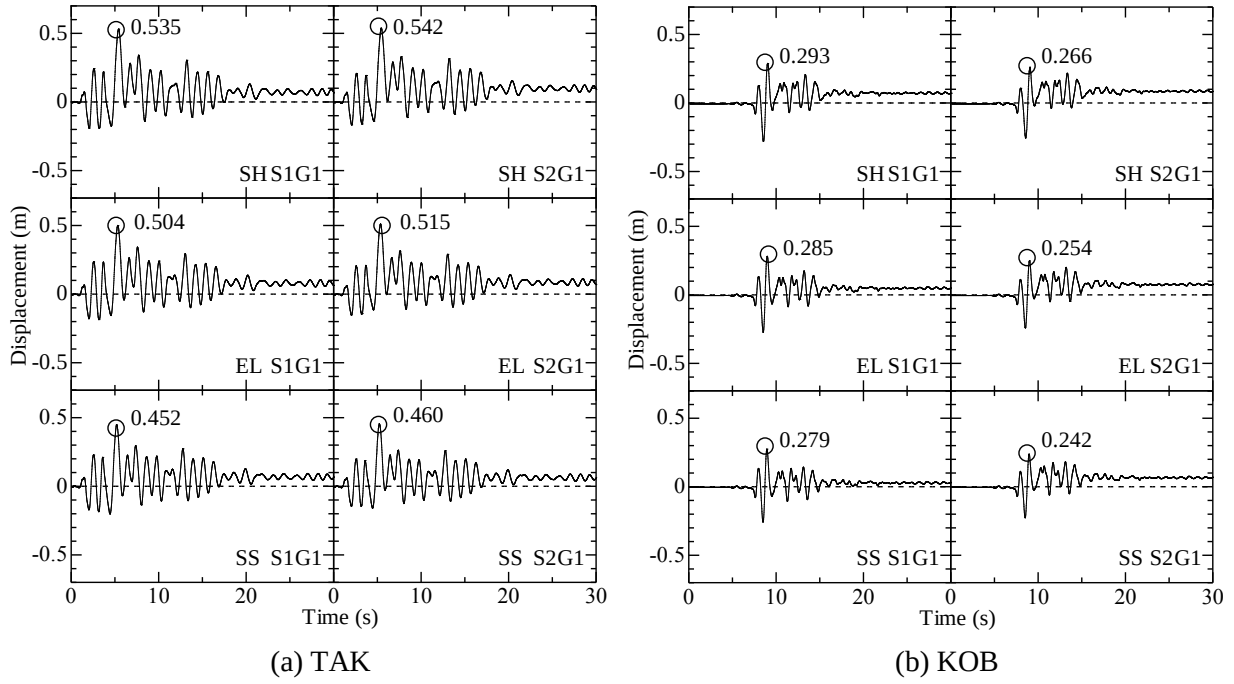


Fig. 7.37 Displacement time-history of simple supported span (S1) and continuous span (S2) for TAK and KOB

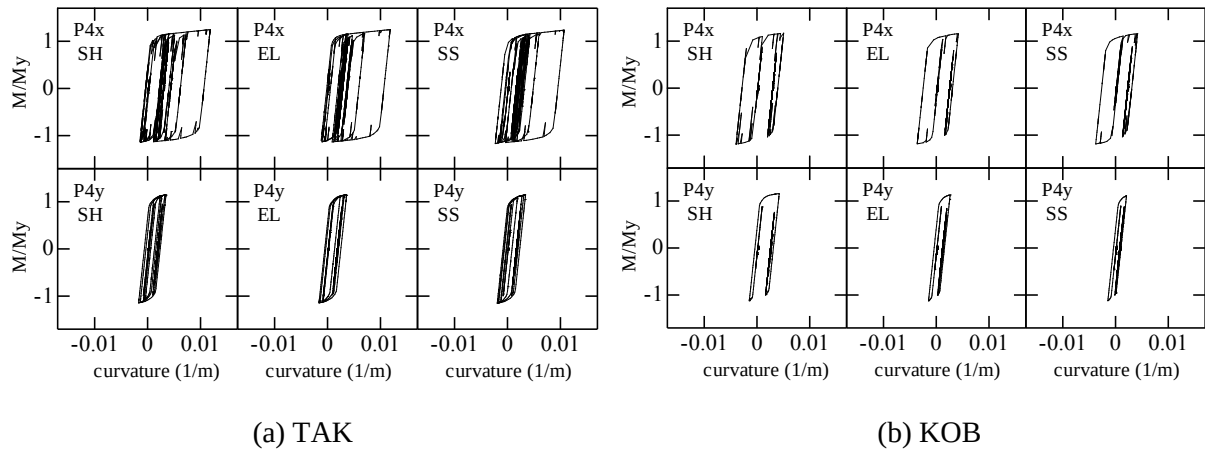


Fig. 7.38 Bending moment ratio-curvature relationship at pier (P4) base for TAK and KOB inputs

bending moment (M) to yield bending moment (M_y) for P4 in X and Y directions is shown **Fig. 7.38**. According to the results for the bending moments in X and Y directions in **Fig. 7.38**, the absolute value of M/M_y overpasses 1 in all cases that indicates plastic deformation at pier base in X and Y directions. Since roller bearings have been restrained in Y direction, maximum bending moment in Y direction is less than the value in X direction. Hence, maximum bending moment and maximum curvature at piers bases in X direction play more important effect on the damages of piers bases. In addition, there are not obvious spikes of pounding forces in case of strain softening SAD. Therefore, viaduct installed of strain softening SADs presents least maximum curvature and least maximum bending moment in X direction. However, maximum curvature and maximum bending moment in X direction present largest value in case of strain hardening SADs.

7.7 Conclusions

In this chapter, the effectiveness of shock absorber devices between roller bearing and stopper to reduce viaduct damage is evaluated through three-dimensional nonlinear finite element response analysis. Moreover, the effect of stiffness and thickness of shock absorber devices on mitigating viaduct damage is evaluated. Finally, the type of shock absorber device also plays an important role in reducing viaduct damages. Strain softening shock absorber device could obviously mitigate viaduct piers damage. However, strain hardening shock absorber device plays least effect on reducing viaduct piers damage. According to previous analysis, the following conclusions can be drawn out.

(1) In case of different stiffness of shock absorber device, curved viaducts without shock absorber devices are more likely subject to deck unseating damage. In addition, as stiffness of shock absorber devices increases, the possibility of deck unseating gradually decreases. So, curved viaduct with large stiffness of shock absorber devices presents less possibility of deck unseating damage. Moreover, shock absorber devices could significantly reduce the pounding forces between roller bearing and stopper. As stiffness of shock absorber devices increases, the pounding forces between roller bearing and stopper gradually decrease. In addition, pounding forces between roller bearing and stopper are much larger than fixed bearing forces in case of with/without shock absorber devices. In other words, pounding forces between roller bearing and stopper play more important influence on bearing damage and piers bases' damages than fixed bearing forces. According to roller bearing force time history, pounding time between roller bearing and stopper last longer in case of shock absorber devices, but the pounding forces are much less. Therefore, viaduct with large stiffness of shock absorber devices presents less pounding forces between roller bearing and stopper in earthquakes.

(2) In case of different stiffness of shock absorber device, viaducts with shock absorber devices present less residual pier inclination than those without shock absorber devices. Moreover, as stiffness of shock absorber device increases, residual pier inclination decreases. Residual pier inclination and curvature at pier base presents almost same change trend. Similarly, maximum pier inclination and curvature at pier base also exhibit almost same change trend. However, similar change trend are shown in residual pier inclination, maximum pier inclination, residual curvature and maximum curvature at piers bases for all cases. Finally, residual pier inclination of pier 1 (P1), supporting fixed bearings, presents less value than the ones of other piers. Pounding forces between roller bearing and stopper have great important effect on residual pier inclination. Conclusively, viaduct with large stiffness of shock absorber devices between roller bearing and stopper shall present less pier damage during earthquakes.

(3) In case of different stiffness of shock absorber device, maximum and residual displacements of superstructures including simple supported span and continuous span present similar change trend as maximum and residual pier inclinations. In other words, maximum and residual displacements on top of piers have significant influence on maximum and residual displacements of superstructures. As stiffness of shock absorber devices increase, maximum and residual displacements of superstructures decrease. So, viaducts with large stiffness of shock absorber devices present less maximum and residual displacements of superstructures. According to the relationship of bending moments and curvatures at piers bases, plastic deformations at piers bases appear in both X and Y directions. However, maximum bending moment, maximum curvature and residual curvature at pier base in X direction is larger than Y direction respectively. Therefore, there will be more serious damages of piers bases in X direction. Maximum bending moment and curvature at pier base in case of shock absorber devices present less value than the case without shock absorber devices. In addition, maximum bending moment and curvature at pier base decreases with stiffness of shock absorber devices increasing.

(4) In case of different thickness of shock absorber device, the possibility of unseating of a deck decreases with thickness of shock absorber device increasing. Moreover, shock absorber devices could significantly reduce the pounding forces between roller bearing and stopper through smooth changes of impact stiffness. As thickness of shock absorber device increases, the pounding forces between roller bearing and stopper obviously decrease. In addition, pounding forces between roller bearing and stopper are much larger than fixed bearing forces. Therefore, pounding forces between roller bearing and stopper play a more important effect on bearing damage and the damages of piers bases than fixed bearing forces. According to roller bearing force time history, contacts time between roller bearing and stopper last longer in case of shock absorber devices, but the pounding forces are much smaller. It is also found that as thickness of shock absorber device increases, residual pier inclination decreases. Residual pier inclination and residual curvature at pier base presents almost same change trend. Similarly, maximum pier inclination and maximum curvature at pier base also present almost same change trend. In addition, maximum curvature at pier base plays an important role in residual curvature. Residual pier inclination of P1, supporting fixed bearings, presents least value. Conclusively, viaduct installed of shock absorber devices with large thickness between roller bearing and stopper shall present less pier damage during earthquakes. Shock absorber device with thickness of half gap between roller bearing and stopper could provide economical and effective design that could obviously reduce residual pier inclination and residual curvature at pier base.

(5) In case of different thickness of shock absorber device, maximum and residual displacements of superstructures present similar change trend as maximum and residual pier inclinations. In other words, maximum and residual displacements on top of piers play an important role in maximum and residual displacements of superstructures. Maximum and residual displacements of superstructures decrease with the increase of thickness of shock absorber device. In addition, maximum and residual relative displacements between two decks in opening trend present less value in case of viaduct with shock absorber devices. As thickness of shock absorber device increases, maximum and residual relative displacements between two decks in opening trend obviously decrease.

(6) In case of different thickness of shock absorber device, plastic deformations at piers bases appear in both X and Y directions during earthquakes. Maximum bending moment, maximum curvature and residual curvature at pier base in X direction is larger than those in Y direction respectively. So, there will be more serious damages at piers bases in X direction. As thickness of shock absorber device increases, maximum bending moment, maximum curvature and residual curvature at pier base in X direction decrease.

(7) In case of different types of shock absorber devices, the type of SAD does not play an important role in the possibility of deck unseating damage. However, strain softening shock absorber device could more effectively reduce the impact forces between roller bearing and stopper than strain hardening SAD and elastic SAD. Strain softening SAD is also more effective in reducing residual pier inclination, maximum pier inclination, residual curvature and maximum curvature at piers bases. Viaduct installed of strain softening SAD also presents least maximum and least residual displacements of superstructures. In case of three types of SADs (including strain hardening, elastic and strain softening SADs), plastic deformations at piers bases appear in both X and Y directions during earthquakes. Maximum bending moment, maximum curvature and residual curvature at pier base in X direction is larger than those in Y direction respectively. Viaduct installed of strain softening SADs presents least maximum curvature and least maximum bending moment in X direction. Viaduct installed of strain hardening SADs presents largest maximum curvature and largest maximum bending moment in X direction. Strain softening shock absorber device could play more important influence on mitigating viaduct damage than other two types of SADs.

References

- [1] Watanabe, E., Sugiura, K., Nagata, K., Kitane, Y.: Performances and damages to steel structures during the 1995 Hyogoken-Nanbu earthquake, *Engineering Structures*, Vol. 20, No. 4-6, pp. 282-90, 1998.
- [2] Scawthorn, C., Yanev, P.I.: Preliminary report 17 January 1995, Hyogo-ken Nambu, Japanese earthquake, *Engineering Structures*, Vol. 17, No. 3, pp. 146-57, 1995.
- [3] Kawashima, K., Unjoh, S.: Impact of Hanshin/Awaji earthquake on seismic design and seismic strengthening of highway bridges, *JSCE Structural Engineering/Earthquake Engineering*, Vol. 13, No. 2, pp. 211-240, 1996.
- [4] Otsuka, H., Unjoh, S., Terayama, T., Hoshikuma, J., Kosa, K.: Damage to highway bridges by the 1995 Hyogoken Nanbu earthquake and the retrofit of highway bridges in Japan, *Proceedings of the 3rd U.S.-Japan Workshop on Seismic Retrofit of Bridges*, Osaka, Japan, pp. 10-11, 1996.
- [5] Hayashikawa, T., Hirooka, T., Ikeda, K., Abdel Raheem, S.E.: Dynamic behaviour of viaducts with PC cables in consideration of pounding of girders and energy absorption, *JSCE Proceedings of Hokkaido*, Vol. 59, Japan Soc Civil Eng, pp. 50-53, 2003.
- [6] Ruangrassamee, A., Kawashima, K.: Relative displacement response spectra with pounding effect, *Earthquake Engineering & Structural Dynamics*, Vol. 30, No. 10, pp. 1511-1538, 2001.
- [7] Jankowski, R., Wilde, K., Fujino, Y.: Reduction of pounding effects in elevated bridges during earthquakes, *Earthquake Engineering & Structural Dynamics*, Vol. 29, No. 2, pp. 195-212, 2000.
- [8] Ruiz Julian, F.D., Hayashikawa, T., Obata, T.: Seismic performance of isolated curved steel viaducts equipped with deck unseating prevention cable restrainers, *Journal of Constructional Steel Research*, Vol. 63, No. 2, pp. 237-253, 2007.
- [9] Zanoardo, G., Hao, H., Modena, C.: Pounding effects in multi-span simply supported bridges induced by spatially varying earthquake ground motions, *Proceedings of the 7th International Seminar on Seismic Isolation, Passive Energy Dissipation and Active Control of Seismic Vibrations of Structures*, Assisi, Italy, 2001.
- [10] DesRoches, R., Muthukumar, S.: Effect of pounding and restrainers on the seismic response of multi-frame bridges, *ASCE Journal of Structural Engineering*, Vol. 128, No. 7, pp. 1-10, 2002.
- [11] Taflanidis, A.A.: Optimal probabilistic design of seismic dampers for the protection of isolated bridges against near-fault seismic excitations, *Engineering Structures*, Vol. 33, No. 12, pp. 3496-3508, 2011.
- [12] Zanoardo, G., Hao, H., Modena, C.: Seismic response of multi-span simply supported bridges to a spatially varying earthquake ground motion, *Earthquake Engineering & Structural Dynamics*, Vol. 31, No. 6, pp. 1325-1345, 2002.
- [13] Mendez Galindo, C., Gil Belda, J., Hayashikawa, T.: Non-linear seismic dynamic response of curved steel bridges equipped with LRB supports, *Steel Construction*, Vol. 3, No. 1, pp. 34-41, 2010.
- [14] Mohamed, W.A., Hayashikawa, T., Aly, A.G.: Hussien MH. Study on seismic response of curved viaduct systems with different isolation conditions under great earthquake ground motion, *JSCE Journal of Structural Engineering*, Vol. 49A, pp. 563-572, 2003.
- [15] Mendez Galindo, C., Hayashikawa, T., Ruiz Julian, D.: Curvature effect on seismic response of curved highway viaducts equipped with unseating cable restrainers, *JSCE Journal of Structural Engineering*, Vol. 54A, pp. 315-323, 2008.
- [16] Abdel Raheem, S.E.: Pounding mitigation and unseating prevention at expansion joints of isolated multi-span bridges, *Engineering Structures*, Vol. 31, No. 10, pp. 2345-2356, 2009.
- [17] Zhu, P., Abe, M., Fujino, Y.: Evaluation of pounding countermeasures and serviceability of elevated bridges during seismic excitation using 3D modeling, *Earthquake Engineering & Structural Dynamics*, Vol. 33, No. 5, pp. 591-609, 2004.
- [18] Ruangrassamee, A., Kawashima, K.: Control of nonlinear bridge response with pounding effect by variable dampers, *Engineering Structures*, Vol. 25, No. 5, pp. 593-606, 2003.
- [19] Hwangm, J-S., Tseng, Y-S.: Design formulations for supplemental viscous dampers to highway bridges, *Earthquake Engineering and Structural Dynamics*, Vol. 34, No. 13, pp. 1627-1642, 2005.
- [20] Kawashima, K., Shoji, G.: Effect of restrainers to mitigate pounding between adjacent decks subjected to a strong ground motion, *12th World Conference on Earthquake Engineering-12WCEE*, Paper no. 1435, 2000.

CHAPTER 8

SUMMARY AND CONCLUSIONS

This chapter presents key summary and conclusions which are drawn from previous chapters. This thesis studies on some technologies for improving behavior of viaduct with steel bearings during serious earthquakes. Stopper installed at both sides of roller bearing in tangential direction can mitigate roller bearing damage by preventing large displacement of roller bearing. As stopper value is less than the gap at expansion joint, the pounding phenomenon between adjacent superstructures can be prevented. The effect of stopper values together with cable restrainer on seismic response of the curved viaduct is carried out. Since cable restrainer can significantly reduce the possibility of deck unseating damage, and stiffness and slack value are the two important parameters of cable restrainer. The effect of stiffness and slack value on the behavior of curved viaduct with steel stopper during serious earthquakes is presented. Due to its large resistance force and energy dissipation capacity, viscous dampers get researchers attention in recent years. The effect of types of viscous damper together with stopper on the behavior of curved viaduct during serious earthquakes is carried out. Finally, shock absorber installed between roller bearing and stopper can obviously mitigate the negative effect of pounding by smooth changes in impact stiffness. The effect of stiffness and thickness of shock absorber device together with viscous damper on the seismic response of the curved viaduct is analyzed. In addition, the types of shock absorber devices also play an important role in reducing viaduct damages. The following shows the most significant summary and conclusions observed from each chapter.

Chapter 1 presents the state of the problem and brief review of previous research. Moreover, the purpose and scope of this thesis is also presented. Finally, organization of the thesis is shown in this chapter.

Chapter 2 illustrates the nonlinear analytical approach for this study. Based on the elasto-plastic finite displacement dynamic response analysis under strong earthquakes, the theoretical approach of the analytical method for determining non-linear dynamic response is used in this study. A non-linear three-dimensional fiber beam element stiffness matrix, derived by using the total Lagrangian formulation, is formulated incorporating both geometric and material non-linearities. The fiber flexural element modeling is adopted to capture the spread of plastic zone in individual element, which is physically motivated from actual uniaxial test data of structural steel bars. Explicit expressions for stress in terms of strain also give the instantaneous stiffness, making the model ideal for iterative non-linear analysis. Newmark's step-by-step method of constant acceleration was formulated for the integration of the equation of motion. The equation of motion was solved for the incremental displacement using the Newton-Raphson iteration scheme. The damping mechanism was introduced into the analysis through the Rayleigh damping matrix. Finally, energetic analysis is formulated under the principle that at any instant the sum of the kinetic energy, strain energy, energy dissipated through normal damping, and energy dissipated through permanent deformation, must be equal to the total input energy.

In **Chapter 3**, the three-dimensional analytical model of the highway viaduct object of this study is presented. Three dimensional analytical model allows for the evaluation of the combined effect of longitudinal, transverse and vertical earthquake excitations on the response of actual viaduct. This fact provides a valuable environment for the non-linear

behavior due to material and geometrical nonlinearities of the relatively large deflection of the structure on the stresses and forces. The analytical curved viaduct, in this thesis, is composed by a single simply-supported span connected to a three-span continuous span. The overall viaduct length of 160 m is divided in equal spans of 40 m. The viaduct alignment is horizontally curved in a circular arc with a radius of curvature of 200 m. The continuous bridge superstructure consists of a concrete deck slab that rests on three I-shape steel girders, modeled with a box cross-section element which represents the composite effect of the continuous steel reinforced concrete deck and the steel girders, identified as a homogeneous material. Besides, pier height is 20 m. Fiber element model can simulate the characterization of viaducts piers. Fixed bearings are simulated by a simple element of high stiffness. Coulomb friction force is taken into account in numerical analysis for roller bearings. Pounding stiffness between roller bearing and steel stopper is a high stiffness slope. Two earthquake ground motions were used as seismic excitations. The first ground motion was recorded at the JR Takatori Station in 1995 Hyogoken-Nanbu Earthquake. The second ground motion was recorded at the JMA Kobe observatory in 1995 Hyogoken-Nanbu Earthquake.

Finally, the definitions and classification parameters of curved viaduct damages is presented respectively in this chapter, including deck unseating damage, pounding damage between adjacent superstructures, tangential and radial joint residual opening damage, and pier damage.

In **Chapter 4**, the purpose is to analyze the effect of stopper values together with cable restrainer on curved viaduct damage. In order to perform a complete investigation of its role on the seismic performance of the viaduct, the dynamic behavior of the structure has been analyzed when the roller bearings are equipped with different stopper values.

The calculated results clearly demonstrate that roller bearing with stopper could provide an effective means for overcoming the potential problems associated with deck unseating. In addition, as stopper value decreases, the possibility of deck unseating damage decreases.

Viaduct, supported on roller bearings with stoppers, does not present tangential and radial joint residual opening damages at expansion joint during earthquakes. The tension of cable restrainer could reduce residual joint tangential displacement for opening direction. According to evaluation of pier inclination damage, the results could be divided into three parts. In first part, as stopper value is less than cable restrainer's slack value of 4 cm, residual pier inclination decreases with stopper value increasing. In second part, as stopper value is larger than cable restrainer's slack value but less than expansion joint gap of 10 cm, residual pier inclination increases with stopper value increasing. In third part, as stopper value is larger than expansion joint gap of 10 cm, residual pier inclination presents fluctuating curve. In addition, residual pier inclination and residual curvature at piers bases present almost same change trend in case of different stopper values.

As stopper value is larger than cable restrainer's value (4 cm) and less than expansion joint gap (10 cm), plastic deformations at piers bases appear in both X and Y directions during earthquakes. Maximum bending moment and maximum curvature at pier base in X direction is larger than those in Y direction respectively. So, maximum bending moment and maximum curvature at piers bases in X direction play more important effect on the damages of piers bases.

Chapter 5 study on the effect of cable restrainer together with stopper on curved viaduct damage. There are two important parameters of cable restrainer including stiffness and slack value. Therefore, there are two main purposes of this chapter study.

One is to analyze the overall performance of highway viaducts with different stiffness of cable restrainers. The effect of cable restrainer's stiffness on deck unseating damage,

tangential and radial joint residual opening damage, and pier damage is analyzed. In addition, a comparison between restrained and unrestrained curved viaduct is also presented. The other 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, and pier damage is analyzed.

The investigation results provide sufficient evidence for the following conclusions.

Curved viaducts without cable restrainers are more vulnerable to deck unseating damage. In addition, the possibility of deck unseating is reduced by increasing the stiffness of cable restrainers. Besides, curved viaducts without cable restrainers are found more possible to tangential joint residual opening damage. Cable restrainers do not obviously affect residual joint radial displacement. According to evaluation of pier inclination damage, viaducts with cable restrainers present less residual pier inclination than those without cable restrainers. Moreover, as stiffness of cable restrainer increases, residual pier inclination decreases. Residual curvature at pier base plays an important role in residual pier inclination. As stiffness of cable restrainer increases, maximum bending moments at piers bases in the X direction decreases. In addition, maximum bending moments at piers bases in the X direction is larger than the ones in the Y direction. So cable restrainers with large stiffness are appropriate to be used to reduce pier damage.

The possibility of deck unseating is reduced by decreasing the slack value of cable restrainers. In addition, curved viaducts with cable restrainers of smaller slack values are found less possible to tangential joint residual opening damage. As slack value of cable restrainer decreases, residual pier inclination decreases. So viaduct with slack value, 2 cm and 3 cm, presents no pier inclination damage. In addition, residual pier inclination and residual curvature at piers bases exhibit similar change trend in all cases. Maximum bending moments at piers bases in the X direction is larger than the ones in the Y direction. Hence cable restrainers with less slack value are appropriate to be used to reduce pier damage.

In **Chapter 6**, the overall performance of highway viaducts with different kinds of viscous dampers and different stopper values is evaluated during serious earthquakes. Application of viscous dampers and stopper reducing some kinds of viaduct damages is also proved in this analysis. Stopper value plays a more important role in piers damage than viscous damper. The investigation results provide sufficient evidence for the following conclusions.

Stopper can effectively reduce deck unseating possibility. However, deck unseating possibility could also be significantly reduced in case of D2000 because of its large resistance force and stiffness. Besides, curved viaduct with D2000 presents smallest residual joint tangential displacement. Maximum closing relative displacement could be significantly reduced in case of viscous dampers. In addition, maximum closing relative displacement obviously decreases as damper's resistance force and stiffness increases. However, stopper could greatly reduce maximum opening relative displacement.

Viaducts with smaller stopper value present less residual pier inclination. As stopper value increases, residual pier inclination increases. Comparing with other piers, P1, fixed bearings on top of it, exhibits smallest residual pier inclination. Viaducts with viscous dampers present less residual pier inclination than those without viscous dampers. But damper's resistance force and stiffness does not obviously affect on the residual pier inclination. Finally, residual pier inclination, maximum pier inclination, residual curvature and maximum curvature at piers bases present similar change trend for all cases.

Plastic deformations at piers bases appear in both X and Y directions during earthquakes. Maximum bending moment and maximum curvature at piers bases in X direction is larger than those in Y direction. So, maximum bending moment and maximum curvature at piers bases in X direction play a more important role in the damages of piers bases.

In **Chapter 7** evaluate the effectiveness of shock absorber devices equipped between roller bearing and stopper on mitigating curved viaduct damage. Shock absorber devices can obviously reduce the negative effect of pounding by smooth changes of impact stiffness. The effect of stiffness and thickness of shock absorber devices on seismic behavior of curved viaduct is carried out respectively. In addition, three types of shock absorber devices including strain hardening, strain softening and elastic shock absorber devices are also evaluated respectively. According to calculation results, the following conclusions can be drawn out.

Curved viaducts without shock absorber devices are more likely subject to deck unseating damage. The possibility of deck unseating gradually decreases with the increase of stiffness of shock absorber devices. In addition, as stiffness of shock absorber devices increases, the pounding forces between roller bearing and stopper gradually decrease. Pounding forces between roller bearing and stopper play more important influence on bearing damage and piers bases' damages than fixed bearing forces. According to roller bearing force time history, pounding time between roller bearing and stopper last longer in case of shock absorber devices, but the pounding forces are much less. Viaducts with shock absorber devices present less residual pier inclination than those without shock absorber devices during serious earthquakes. As stiffness of shock absorber device increases, residual pier inclination decreases. Residual pier inclination of pier 1 (P1), supporting fixed bearings, presents less value than the ones of other piers. Pounding forces between roller bearing and stopper have great important effect on residual pier inclination. Maximum and residual displacements on top of piers have significant influence on maximum and residual displacements of superstructures. Plastic deformations at piers bases appear in both X and Y directions. However, maximum bending moment, maximum curvature and residual curvature at pier base in X direction is larger than Y direction respectively. Curved viaduct with large stiffness of shock absorber device exhibit least viaduct damage during strong earthquakes.

In case of different thickness of shock absorber device, the possibility of unseating of a deck decreases with thickness of shock absorber device increasing. As thickness of shock absorber device increases, the pounding forces between roller bearing and stopper obviously decrease. It is also found that as thickness of shock absorber device increases, residual pier inclination decreases. Residual pier inclination and residual curvature at pier base presents almost same change trend. Similarly, maximum pier inclination and maximum curvature at pier base also present almost same change trend. Shock absorber device with thickness of half gap between roller bearing and stopper could provide economical and effective design. Maximum and residual displacements of superstructures decrease with the increase of thickness of shock absorber device. In addition, maximum and residual relative displacements between two decks in opening trend present less value in case of viaduct with shock absorber devices. As thickness of shock absorber device increases, maximum and residual relative displacements between two decks in opening trend obviously decrease. Curved viaduct with large thickness of shock absorber device presents least viaduct damage during serious earthquakes.

Based on the calculation results about three types of shock absorber devices (SAD), the type of SAD does not play an important role in the possibility of deck unseating damage. But strain softening shock absorber device could more effectively reduce the impact forces between roller bearing and stopper than strain hardening SAD and elastic SAD. Strain softening SAD is also more effective in reducing residual pier inclination, maximum pier inclination, residual curvature and maximum curvature at piers bases. Viaduct installed of strain softening SAD also presents least maximum and least residual displacements of superstructures. Conclusively, strain softening shock absorber device could play more important influence on mitigating viaduct damage than other two types of SADs.