SEISMIC PERFORMANCE OF A SKEWED BRIDGE CONSIDERING FLEXURE AND TORSION INTERACTION

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ABSTRACT :

This study shows an analysis on the effect of seismic torsion for the performance of a skew bridge. A nonlinear torsional hysteretic model developed by Tirasit and Kawashima (2007) is implemented to analyze pier torsion due to combined torsion and bending interaction. Pounding and failure of bearings are also included in analysis under a near-field ground motion. The analysis shows that nonlinear torsion of the pier is important on the seismic response of the bridge.

KEYWORDS: Skew bridge, Torsion, Pounding, Seismic performance

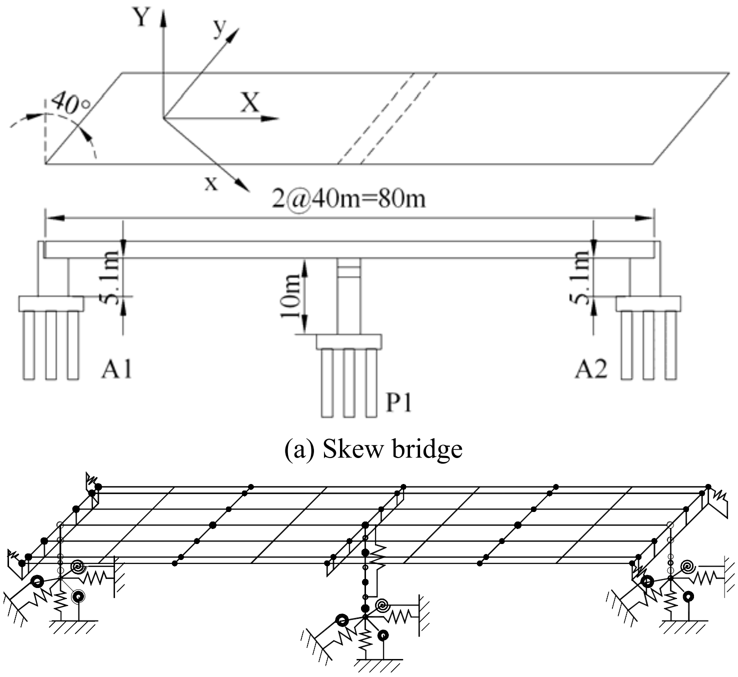
1. INTRODUCTION

It is well known in past earthquakes that skewed bridge are susceptible to in-plane rotation which caused extensive damage such as Gavin Canyon and Mission–Gothic Undercrossings during the 1994 Northridge, USA earthquake and Kawaraginishi viaduct in Meishin expressway during the 1995 Kobe, Japan earthquake. Based on the past studies, in-plane rigid body response is dominant in bridges supported by short and stiff columns. Rotation of a deck is induced primarily as a result of the impact between the deck and the abutment (for example, Maragakis and Jennings (1987), Watanabe and Kawashima (2004 a)) and is likely to develop rotation in the column. This effect can be extensive when the columns are rigidly connected to the deck. Lock of bearings and expansion joints due to their break can result in large transfer of rotation from a deck to a column, which can induce larger torsion in the colum. Therefore, combined action of bending and torsion can result in more extensive damage in the columns (Tirasit and Kawashima (2008)). This paper presents an analysis on a skewed bridge by taking account of the torsion and flexure interaction based on Tirasit and Kawashima (2007) to clarify the seismic response of a skewed bridge.

2. REPRESENTATIVE SKEW BRIDGE

A 40-degree 2-span continuous skewed bridge as shown in Fig. 1(a) is selected for analysis. The deck is supported by a 10 m tall reinforced concrete pier with a 3 m by 3 m section and two 5.1 m tall reinforced concrete abutments as shown in Fig. 2. Five steel movable bearings are set on each abutment to support the deck. Because steel bearings were vulnerable in past earthquakes suffering to extensive damage resulting in deterioration of the lateral capacity, it is assumed in this analysis that the deck can be free in the lateral directions with the vertical offset fixed on the abutments. Bridge axis direction and the direction perpendicular to the bridge axis direction are denoted here as X and Y directions, respectively. The directions which are parallel and perpendicular to the edge of skewed direction are denoted as x and y directions, respectively (refer to Fig. 1).

The strength of concrete of the pier and the abutments are 21 MPa. Longitudinal and tie reinforcements are deformed 29 mm and 16 mm bars respectively with the yield strength of 295 MPa. Tie bars are provided at 150 mm interval as shown in Fig. 3. The longitudinal reinforcement ratio p_l and the volumetric tie reinforcement ratio ρ_s is 1.26% and 0.87 %, respectively.



(b) Analytical idealization

Fig.1 Bridge model

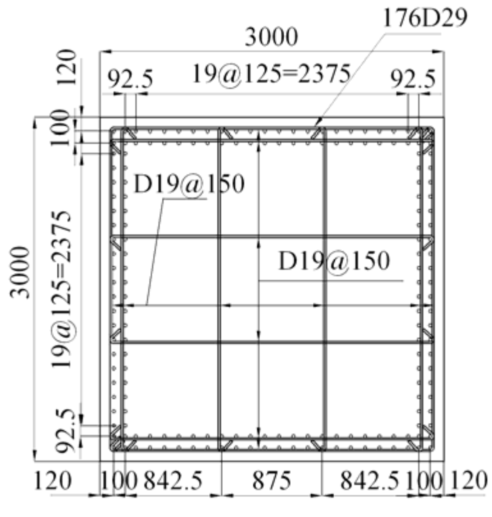


Fig. 2 Cross section of column

3. ANALYTICAL MODELING AND INPUT GROUND MOTION

Analytical model is shown in Fig. 1(b). Superstructure and substructure are idealized by beam elements. Fiber elements are used at the plastic hinge of the pier to take account of the hysteretic behavior under 3D excitation. The stress-strain constitutive model of confined concrete including unloading and reloading paths is used based on Hoshikuma et al (1997) and Sakai and Kawashima (2006). The longitudinal reinforcements are idealized based on Menegotto-Pinto (1973).

A key parameter in the torsion and flexure interaction is a ratio of the rotation θ_z and lateral drift Δ defined as

$$r \equiv \frac{\theta_z}{\Delta} \tag{1}$$

in which, θ_z is rotation of a column (rad) and Δ is drift ratio. From the definition, $r=0$ and $r=\infty$ imply the pure flexure and pure torsion, respectively.

Fig. 3 shows an example of the torsion and flexure interaction for the column specimens which were subjected to different combinations of torsion and flexure under a constant axial load. The columns had a 400 mm x 400 mm square cross section and they were 1,750 mm tall with a 1,350 mm effective height. The longitudinal reinforcement and the tie volumetric reinforcement ratios were 1.27% and 0.75%, respectively. The design concrete strength was 30 MPa. The axial compression due to the deck dead weight of 160 KN was applied to the columns so that the axial stress at the plastic hinge zone was 1Mpa, which is typical for Japanese bridge columns.

Based on the experiment results, the primary curve and unloading and reloading rules were developed as shown in Figs. 4 and 5, respectively. The flexural strength of column under combined actions $\widetilde{F}_c$ deteriorates as r increases, while the torsional strength of column under combined action $\widetilde{T}_c$ increases and approaches to the torsional strength of column under pure cyclic torsion T_c as r increases. They are expressed as

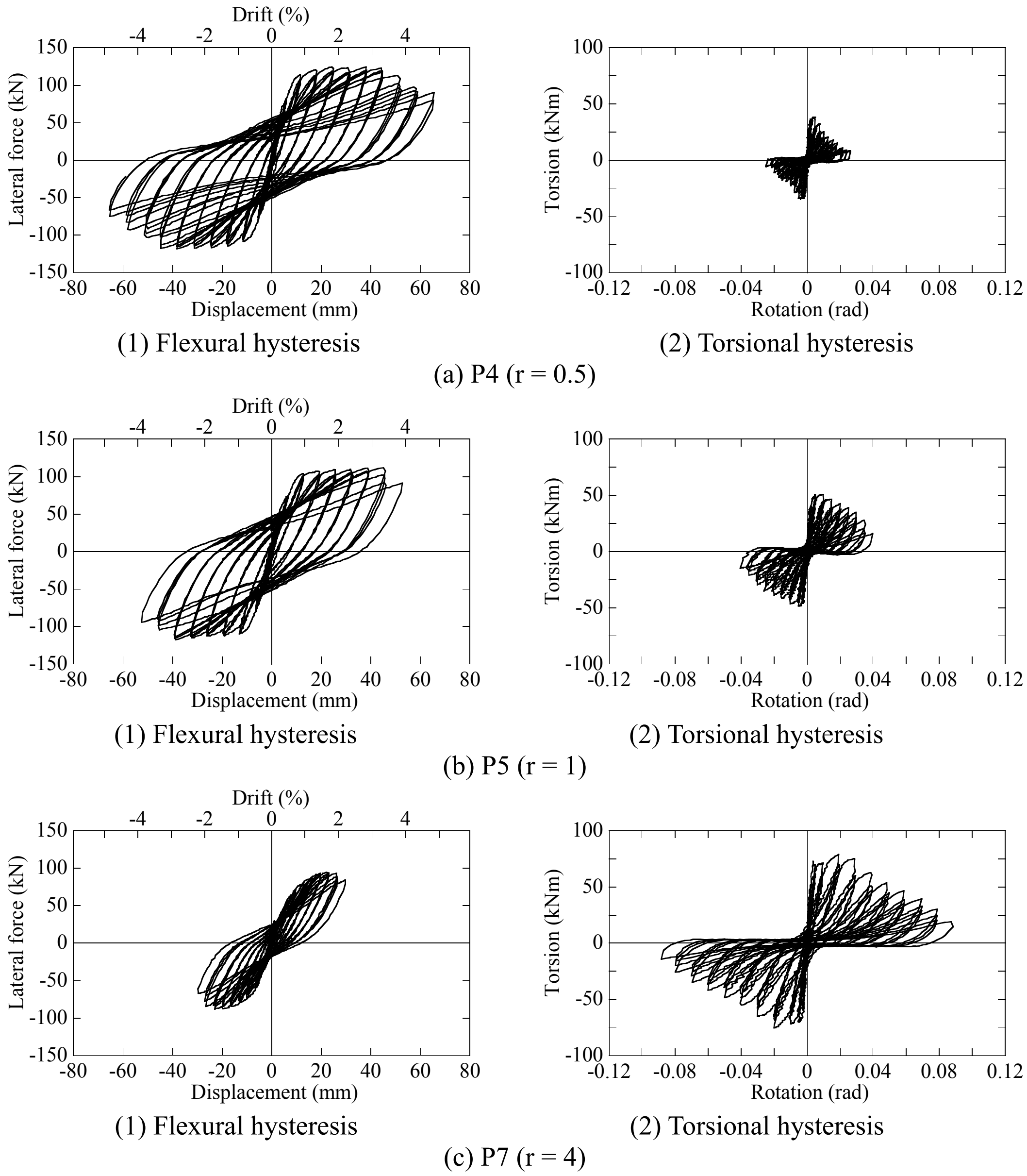
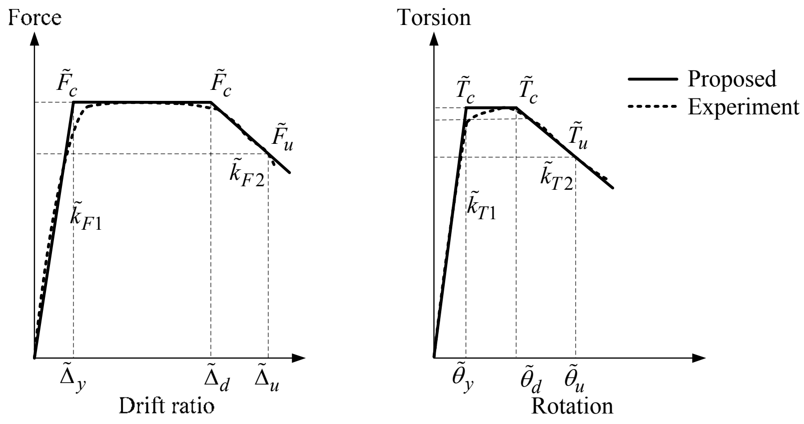
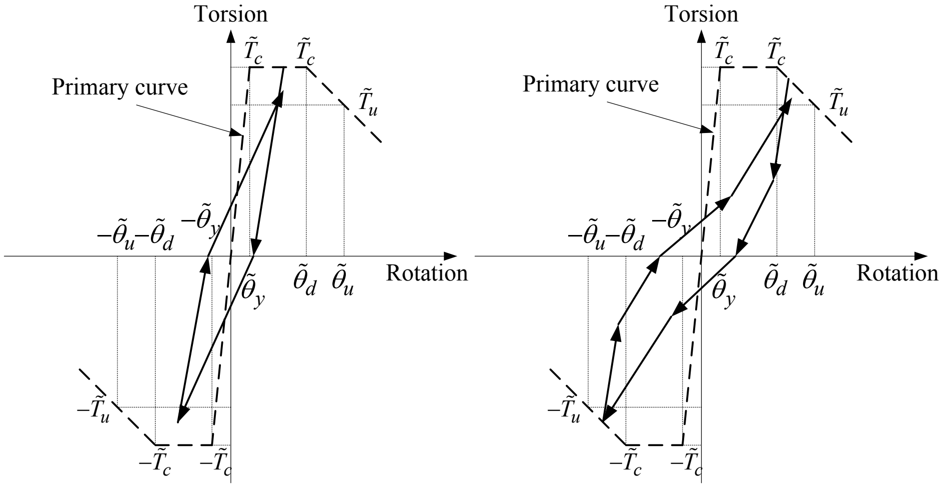


Fig. 3 Flexural and torsional hystereses of columns under combined bending and torsion.



(a) Flexural hysteresis (b) Torsional hysteresis

Fig. 4 Proposed hysteretic model for torsion and flexure.



(a) Less significant pinching (b) Significant pinching

Fig. 5 Nonlinear torsional hysteretic model

$$\tilde{F}_c = F_c.e^{-0.088r} \tag{2}$$

$$\tilde{T}_c = T_c.e^{-0.42/r} \tag{3}$$

in which, F_c represents the flexural strength of a column under pure cyclic bending.

The yield drift $\tilde{\Delta}_y$, the drift when the lateral restoring force starts to deteriorate $\tilde{\Delta}_d$ and the ultimate drift $\tilde{\Delta}_u$ under combined action are provided as

$$\tilde{\Delta}_y = (1-0.035r)\Delta_y ; \tilde{\Delta}_d = \left(5.03e^{-0.18r}\right)\Delta_y ; \ \tilde{\Delta}_u = \left(6.37e^{-0.18r}\right)\Delta_y \tag{4}$$

in which Δ_y is the yield drift of column under cyclic bending.

In this analysis, only the deterioration of torsion in accordance with an increase of flexural moment is considered by disregarding the deterioration of flexural strength due to an increase of the torsion. An initial value of r is first assumed to evaluate the bridge response. After determining r by equation (1) based on the computed rotation and drift, the analysis is repeated using the estimated r if the estimated r is not close enough to the originally assigned r. This iteration is repeated until r satisfies the following requirement

$$\left|r_j - r_{j-1}\right| / r_{j-1} \le 0.02 \tag{5}$$

in which, j is the trial number of current analysis. r can be an any value between 0 and ∞, however, because the deterioration of the flexural strength occurs at r much larger than 0.5, r is assumed to be larger than 0.5 in this analysis.

The initial tosional stiffness is evaluated as

$$K = \gamma\left(G_c \beta_c a^3 b / L_{eff}\right) \tag{6}$$

in which, E_c and G_c are the elastic modulus and shear modulus of the concrete, respectively, β is St Venant's torsional constant, a and b are smaller and larger dimension of rectangular cross section, respectively, L_{eff} is the column effective height, and γ is the modification factor for the initial stiffness. γ is assumed as

0.681 based on the experiment results.

In order to consider the pounding between the deck and the abutments, impact springs are provided between the ends of the deck and the tops of the abutments. Stiffness of the impact spring k_I is evaluated as (Watanabe and Kawashima (2004 b))

$$k_I = \frac{nEA}{L} \tag{7}$$

in which, E, L and A are the elastic modulus, length and section area of the deck and n is the number of elements of the deck.

NS, EW and UD components of the ground acceleration at JMA Kobe Observatory during 1995 Kobe, Japan earthquake are imposed to the bridge in the longitudinal, transverse and vertical directions, respectively.

The natural periods and mode shapes assuming the linear properties are evaluated. The fundamental natural period with the predominant rotation mode of the deck is 2.2 s, however, the effective mass is nearly 0. Therefore, 2nd, 3rd and 6th modes with the natural periods of 0.86 s, 0.62 s and 0.26 s, respectively are predominant.

4. RESPONSE OF THE BRIDGE USING LINEAR COLUMN TORSION

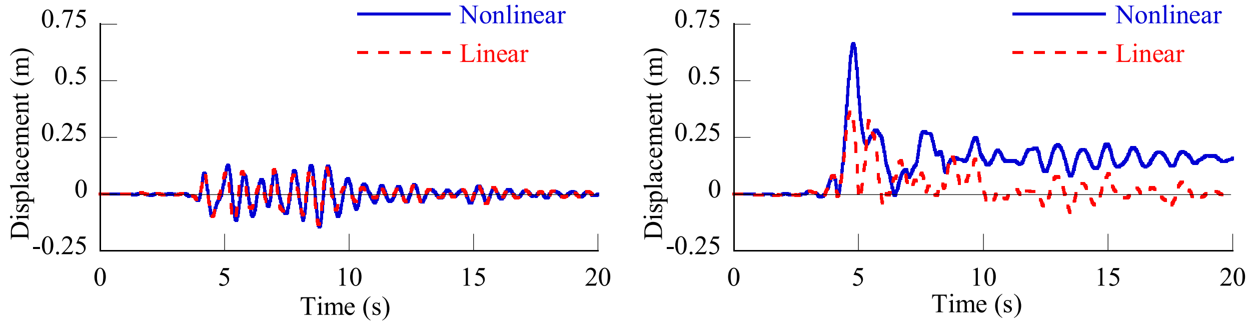
Fig. 6 shows the deck response displacements and accelerations when the column P1 is assumed to be linear with the gross section torsional stiffness. Responses considering the combined torsion and flexure interaction which will be described later are also shown here for comparison. It was assumed here that gaps Δ_G between the ends of deck and two abutments is 50 mm. It is important to note that deck displacement in Y direction is larger in the positive direction than the negative direction. This is resulted from rotation of the deck in the anti-clock wise direction (refer to Fig. 7) due to poundings which occur at the deck ends with the abutments. As it is well known, this is the unique response of skew bridges. The peak rotation of the deck is 0.0068 rad . It is noted that the residual displacement of the deck in Y direction is very limited, i.e., no residual rotation occurs in the deck because torsional column stiffness is assumed to be linear. High pulse accelerations over 60g occur at the acute and obtuse corner of the deck. As shown in Fig.8, poundings between the deck and the abutments occur many times. The maximum impact force is 27.4 MN and 33 MN at acute and obtuse corner, respectively, which correspond 4.586 times and 5.524 times the one-span deck weight

Fig. 9 shows the torsion vs. rotation hysteresis of P1. The hysteresis computed by assuming nonlinear torsion and flexure interaction which will be described later is also shown here for comparison. The peak rotation is 0.0068 rad with the peak torsion of 79.8 MNm, while the torsion capacity based on the space truss analogy is 26.6 MNm.

Fig. 10 shows the moment vs. curvature hystereses of P1 around x and y axis. The hystereses computed by assuming nonlinear torsion are also shown here for comparison.The peak flexure curvature is 0.0078 1/m around y axis while it is 0.0039 rad around x axis. This is because the column response due to the flexure is larger in x direction than y direction. It should be noted here that deterioration of the flexural strength due to the torsion is not included in this analysis. The ultimate flexural curvature is 0.00566 1/m around x and y axes when the torsion and flexure interaction is disregarded. The computed curvature around y axis is 1.38 times the ultimate curvature.

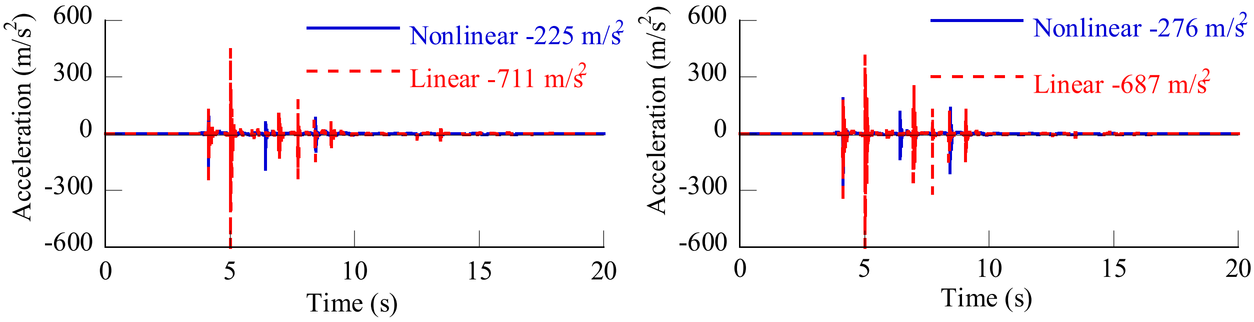
5. EFFECT OF NONLINEAR TORSION AND FLEXURE INTERACTION MODEL

Fig. 6 shows the deck responses at the abutment A2 when nonlinear torsion and flexure interaction is considered.



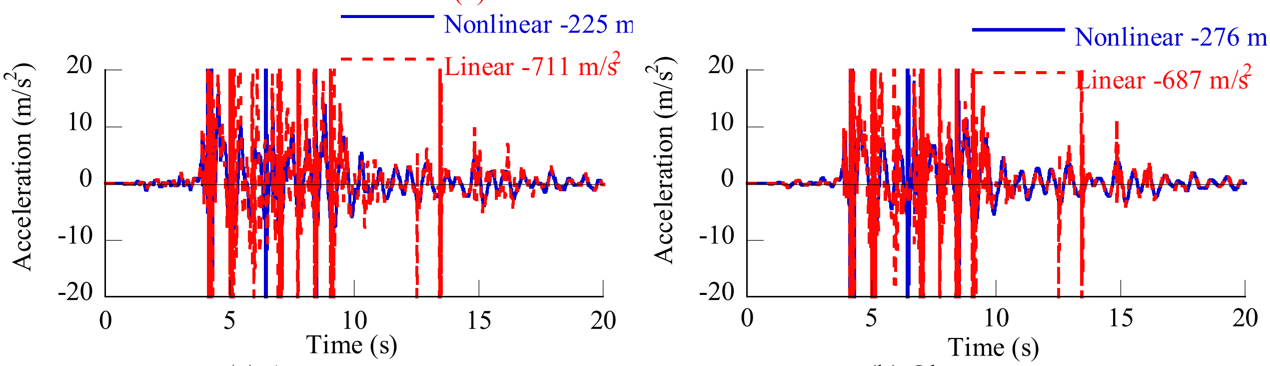
(a) X direction (b) Y direction

(1) Deck displacement



(a) Acute corner (b) Obtuse corner

(2) Deck acceleration in x direction



(a) Acute corner (b) Obtuse corner

(3) Deck acceleration in x direction (with focusing on responses less than 2g)

Fig. 6 Deck response at abutment A2

The deck displacement in X direction is virtually the same between the case which takes account of the nonlinear torsion and flexure interaction and the other case which disregards it. This is because the column torsion stiffness has the limited contribution to the deck displacement in X direction. On the other hand, the deck displacement in Y direction when the nonlinear torsion and flexure interaction is taken into account is 1.8 times larger than the deck response computed assuming the linear column torsion stiffness. The peak deck rotation is 0.0107 rad when the torsion and flexure interaction is included as shown in Fig. 7. This is 47% larger than the deck rotation evaluated by assuming the gross torsional section stiffness. It is important to note that 0.15 m residual displacement occurs in Y direction resulted from residual deck rotation of 0.004 rad. The 0.004 rad residual rotation corresponds 40% the peak rotation.

It is interesting that impact force computed by taking the torsion and flexure interaction into account is 12 MN and 22 MN at the acute and obtuse edges of the deck, respectively. They are only 43.8% and 67% the response which is computed by assuming the linear torsion stiffness. Impact force at the first pounding at 4.09 s is nearly the same between the response which takes account of torsion and flexure interaction and the response which

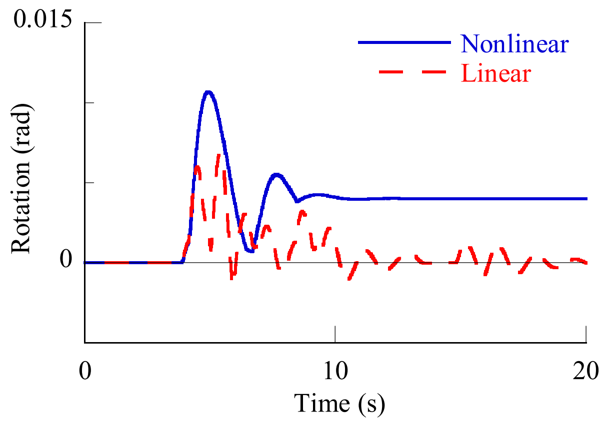
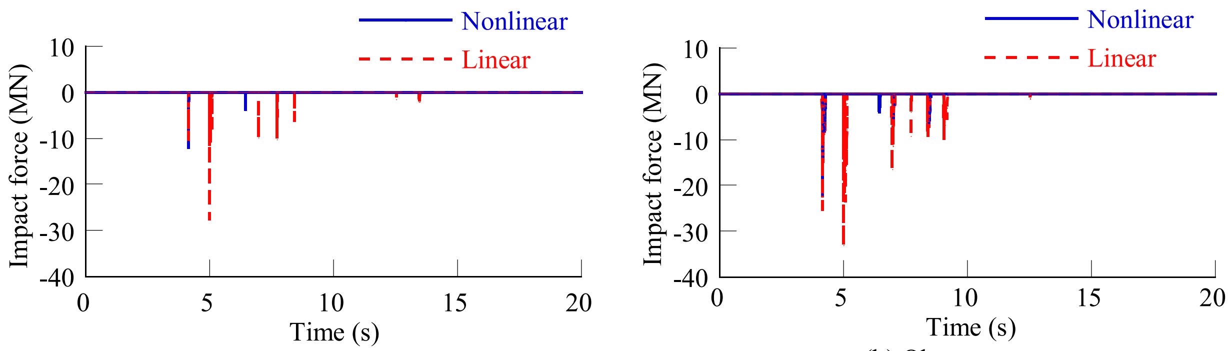


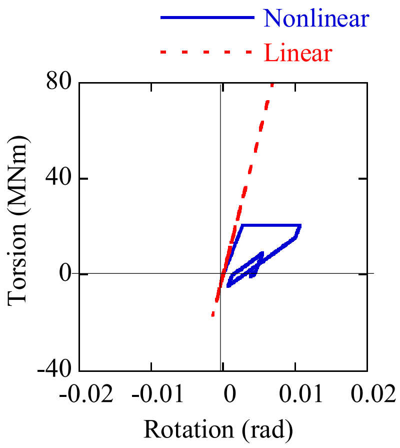
Fig. 7 Rotation of deck

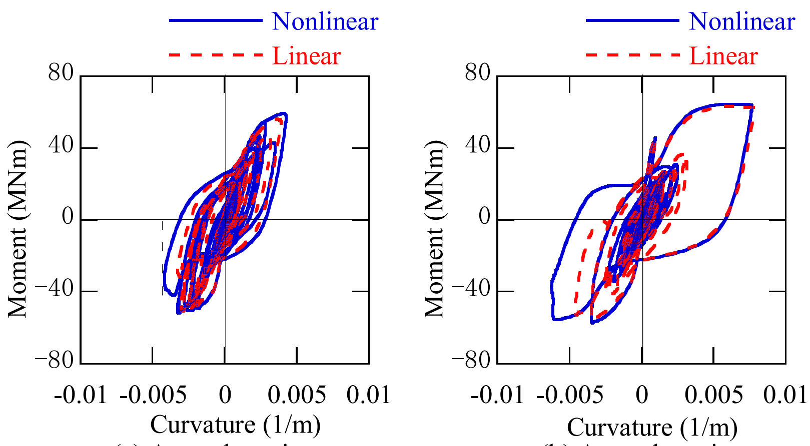


(a) Acute corner

(b) Obtuse corner

Fig. 8 Impact force between deck and abutment A2 (x direction)





(a) Around x axis

(b) Around y axis

Fig. 9 Tosion vs. rotation hystereses

Fig. 10 Moment vs. curvature hystereses

ignores the interaction. However, the impact force at the subsequent poundings becomes much smaller in the response which takes account of the torsion and flexure interaction as shown in Fig.9. This indicates that the deteriorated torsion resulted in smaller impact force. The torsional strength of P1 is 20.5MNm which is 77% the pure torsion capacity based on the space truss analogy of 26.6MNm. This is resulted from the torsion and flexure interaction.

The peak curvature of P1 is 0.0043 1/m and 0.0076 1/m around x and y axes, respectively. They are close to the curvatures evaluated by disregarding the torsion and flexure interaction.

The rotation vs. drift ratio r converged to 1.14. Fig. 11 shows crack modes of a specimen loaded under

$r=1$ (Tirasit and Kawashima (2007)). It is anticipated that P1 would suffer lightly extensive failure than that shown in Fig. 11.

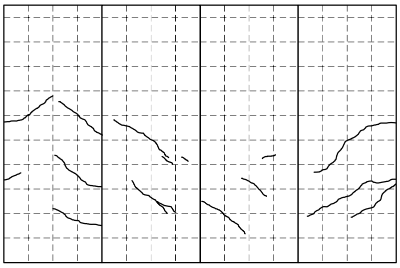


Fig. 11 Crack modes of a column loaded under $r=1.0$ ($\theta=0.005$ rad and $\Delta=0.5\%$)

6. CONCLUSIONS

The seismic response of a skewed bridge was analyzed with/without the torsion and flexure interaction of a column. Based on the results presented herein, the following conclusions may be deduced:

1. The column and deck rotation evaluated by taking the torsion and flexure interaction becomes 47% larger than that evaluated by assuming the linear gross torsional section stiffness. Use of the linear gross torsional section stiffness is likely to underestimate the rotation of decks and column.

2. A residual rotation of 40% the peak rotation is developed when the torsion and flexure interaction is considered. This residual rotation of the column results in 0.15 m lateral residual drift of the deck.

3. By taking the torsion and flexure interaction, impact forces which take place between the end of deck and abutments decrease to 43.8% and 67% the value evaluated by assuming the linear gross torsional section stiffness.

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