第12回地震時保有耐力法に基づく橋梁等構造の
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A4－6

Seismic Performance of Columns Designed Using High Response Accelerations under Near-Field Ground Motions

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1. Introduction

To live in a country where earthquakes occur frequently, it is very important for us to have a seismic performance criteria which can maintain the safety of human's lives when earthquakes happen.

In the seismic design of bridges, the basic concept of seismic design philosophy and the performance criteria is more or less similar among the current codes worldwide. When bridges are subjected to small to moderate earthquakes, the structure should be resistd within the elastic range so that function can be maintained, and should not collapse when they are exposed to significant earthquakes. **Table 1** shows the seismic performance goals of highway bridges in Japan (JRA 2002). Here, function evaluation ground motions and safty evaluation ground motions are considered under 2 level seismic design approach. Middle-field ground motions generated by earthquakes with magnitude of 8 (Type I ground motions) and near-field ground motions generated by earthquakes with magnitude of 7 (Type II ground motions) are used. The seismis performance is classified in terms of safety, function and reparability.

Table 1 Seismic performance goals of highway bridges

(a) Seismic performance goals

Design Ground Motion		Standard Bridges	Important Bridges
Function Evaluation		SP 1: Functional	
Safty Evaluation	Type I	SP 2: Limited damage	SP 3: Prevent critical damage
	Type II		

(b) Seismic performance levels

Performance Levels		SP 1	SP 2	SP 3
Safty		Maintain safty for collapse	Maintain safty for collapse	Maintain safty for collapse
Function		Keep function	Regain function in a short time after earthquakes	
Repara-bility	Short term	No emergent repair is required	Emergent repair can regain function	
	Long term	Only minor repair	Permanent repair without difficulty	

2. Damage-free bridge

Based on current seismic performance goal, the bridge should maintain function during small to moderate earthquakes, and prevent collapse when it is exposed to extreme events. However, the downtime due to the damage of bridges result in significant inconvenience for firefighting, evacuation, and transportation of medical and restoring equipments, especially for those important bridges. For those reasons, when we design a bridge, we should consider a damage-free bridge. The seismic performance goals for damage-free bridges is to secure their function immediately after an extreme event so that the bridges can maintain their service.

However, the seismic performance demanded by public is far different from current code. A questionnaire survey on expected seismis performance goals of the public was conducted in 2004[1]. **Fig. 1** shows the accepted downtime for the public. 89.3% people replied that the bridges should be repaired within one week after an extreme event.

However, the real downtime of bridges is much longer than expect by public. For example, the downtime of Jyuso viaduct after 1995 Kobe, Japan earthquake was more than one year.

Although we can build a bridge which can maintain function immediately after a significant earthquake, technical and econominal constraints are the main difficulties to achieve this goal. But except special conditions, it is feasible to build bridges which can maintain function immediately after an extreme event if the economical constraint is mitigated.

Therefore, the other question of the questionnaire survey is the acceptable increase of cost of the bridges for enhancing the seismic safety. **Fig. 2** shows how public

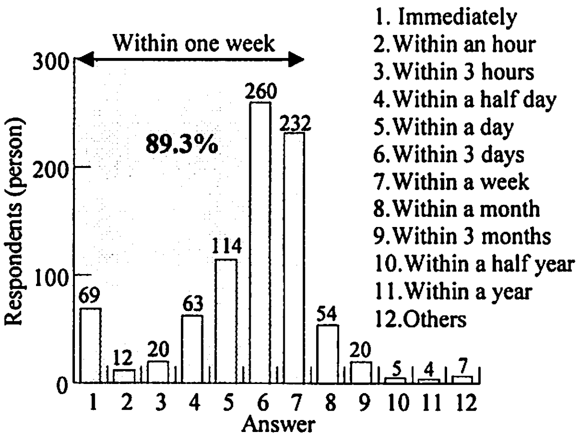


Fig. 1 Accepted downtime by public

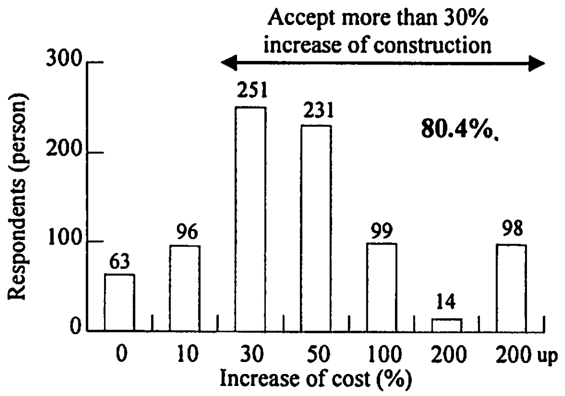


Fig. 2 Acceptable of cost increase of the bridges

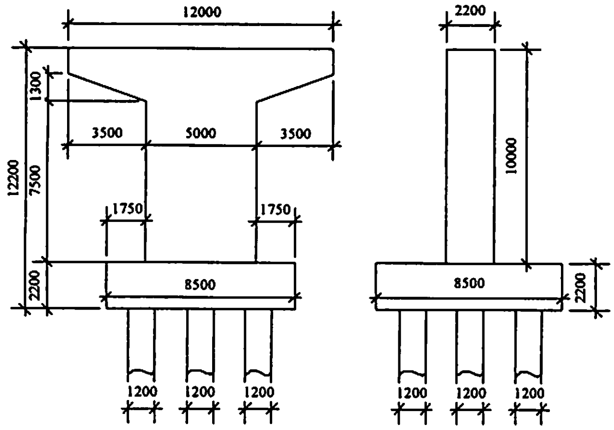
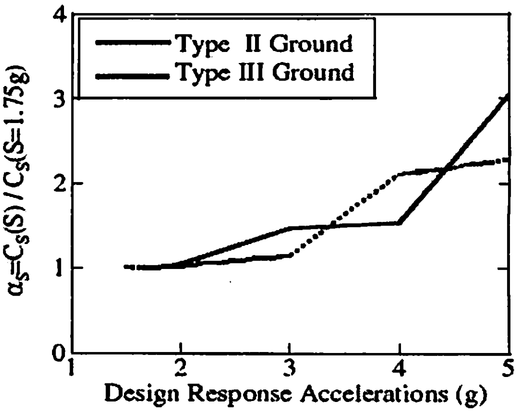


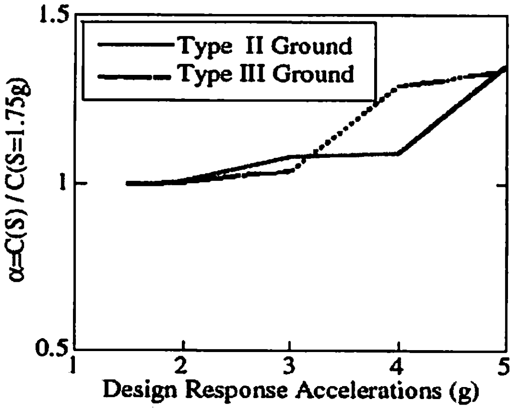
Fig. 4 The column

considers the cost increase of the construction that is validated to build bridges which do not suffer damage. Most people can accept the increase of construction cost up to 30%, followed by up to 50%. 80.4% people replied that the cost increase more than 30% of current level can can be validated.

Fig. 3 shows the increase of cost. The cost of substructure (C_S) designed based on 3g and 5g response acceleration are 1.47 and 3.06 times current level, but the cost of a total bridge (C) is only 1.08 and 1.36 times current level. Based on **Fig. 2**, such a cost increase may be accepted level by public.



(a) Substructure



(b) The cost of total bridge

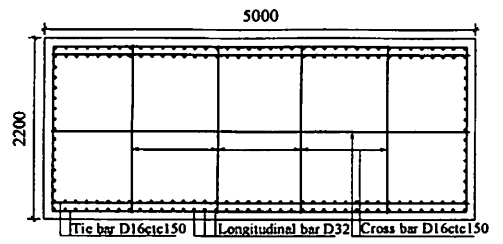
Fig. 3 Cost increase ration

3 . Target bridges and analytical idealization

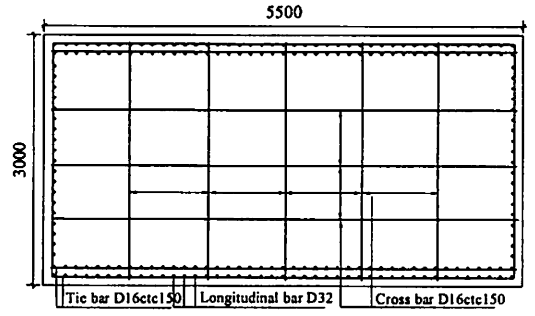
The target bridge is a 5 span continuous steel deck bridge supported by 12m tall rectangular reinforced concrete columns. The deck is supported by elastomeric bearings. The footings are supported by nine 30m long cast-in-place reinforced concrete piles with a diameter of 1.2m. The soil consists of 16.2m thick clay with the N value around 10, underlain by thick gravel layer with N larger than 50. Liquefaction does not occur in this site. This is type II (moderate) soil based on JRA Design Specifications.

The bridge is designed based on Design Specifications of Highway Bridges (JRA 2002). The design response acceleration S_A is assumed as 1.75g, and then it was increased from the standard design response acceleration value to 2g, 3g and 5g. Design concrete strength of columns, footings and the piles are 24MPa, 22MPa and 24MPa respectively.

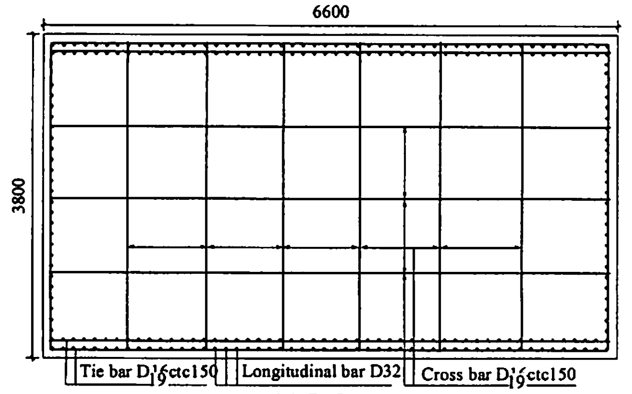
Figs. 4 and 5 show the designed columns. Because columns have the same height and soil condition along the bridge is almost uniform, a column-substructure-deck system is analyzed here.



(a) S=1.75g



(b) S=3g



(c) S=5g

Fig. 5 Section and reinforcement of the columns

A column and the footing idealized by beam elements and fiber elements at the plastic hinge zone. Plastic length was assumed to be a half width of the column. Soil-pile and footing interaction was idealized in terms of a set of translation and rotational springs.

4. Input ground motions

In the analysis, three typical strong near-field ground motions recorded at JR Takatori (1995 Kobe earthquake, Japan), Kawaguchi (2004 Chuetsu earthquake, Japan) and Shikhkang (1999 Chi-Chi earthquake, Taiwan) were used. **Fig. 6** shows the acceleration response spectra of three ground motions.

5. Response of 1.75g bridge

According to the dynamic analysis, as shown in **Fig. 7**, we can see that the peak deck response accelerations under JR Takatori ground motion and Kawaguchi ground motion are 19-20m/s^2 in the longitudinal direction, and 6-20m/s^2 in the transverse direction. As shown in **Fig. 8**, peak deck response displacements are 0.7-0.8m in the longitudinal direction and 0.2-0.65m in the transverse

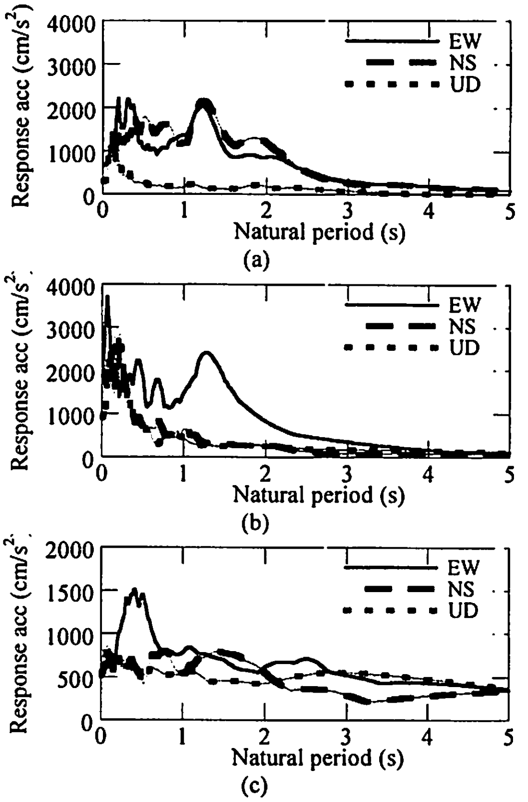
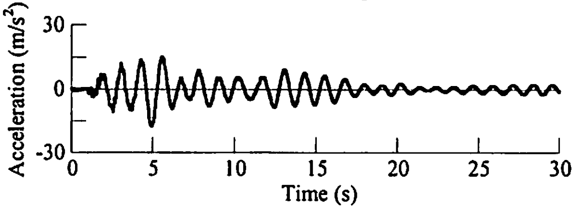
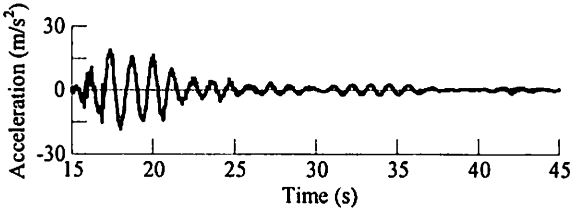


Fig. 6 Acceleration response spectrum ((a): JR Takatori, (b): Kawaguchi, and (c): Shikhkang)

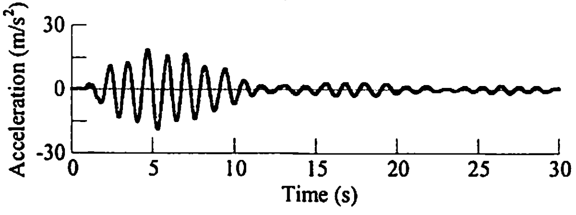


(a) JR Takatori

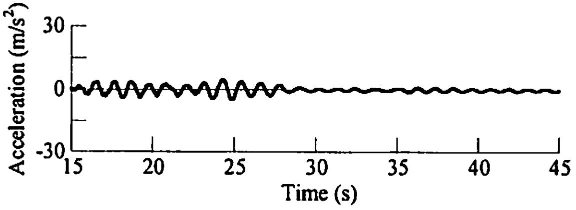


(b) Kawaguchi

(1) Longitudinal



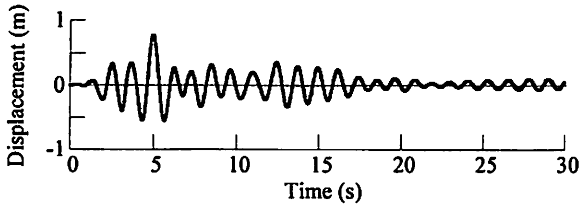
(a) JR Takatori



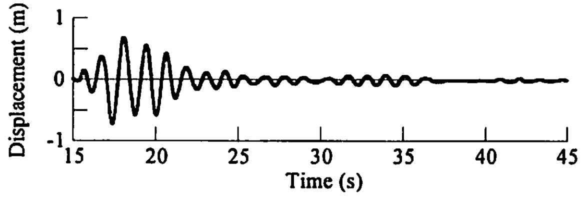
(b) Kawaguchi

(2) Transverse

Fig. 7 Response acceleration at the deck of 1.75g column

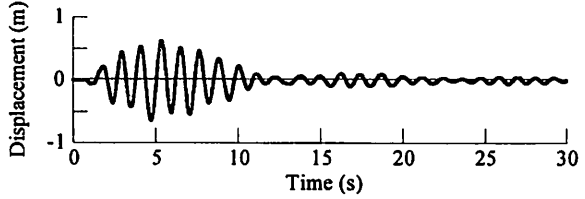


(a) JR Takatori

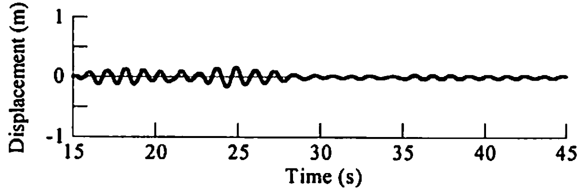


(b) Kawaguchi

(1) Longitudinal



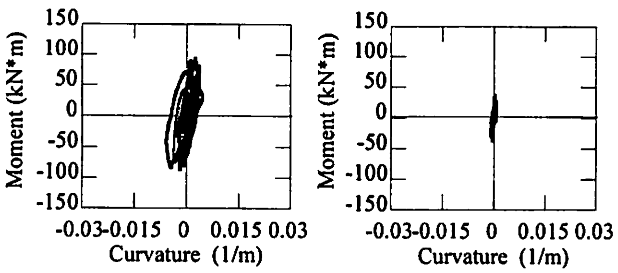
(a) JR Takatori



(b) Kawaguchi

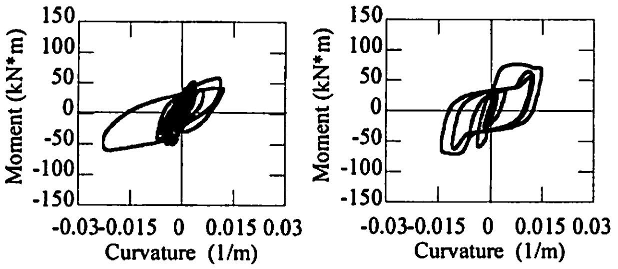
(2) Transverse

Fig. 8 Response displacement at the deck of 1.75g column



(a) JR Takatori (b) Kawaguchi

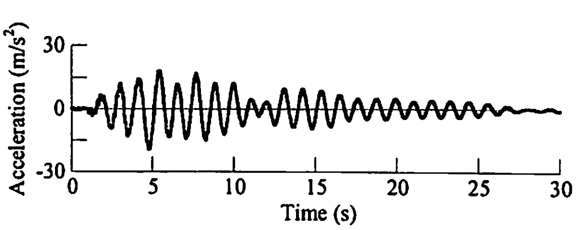
(1) Longitudinal



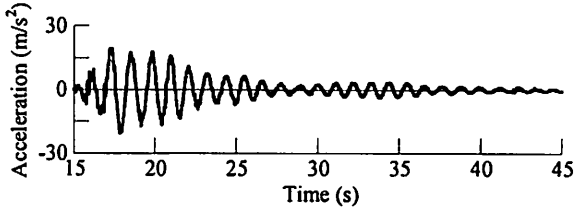
(a) JR Takatori (b) Kawaguchi

(2) Transverse

Fig.9 Moment-curvature hysteresis at the plastic hinge of 1.75g column

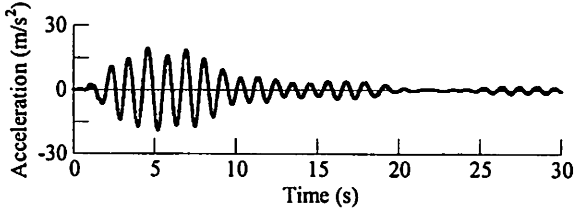


(a) JR Takatori

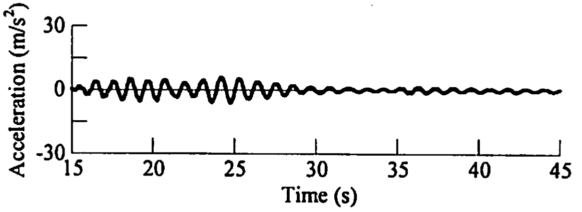


(b) Kawaguchi

(1) Longitudinal



(a) JR Takatori



(b) Kawaguchi

(2) Transverse

Fig. 10 Response acceleration at the deck of 3g column

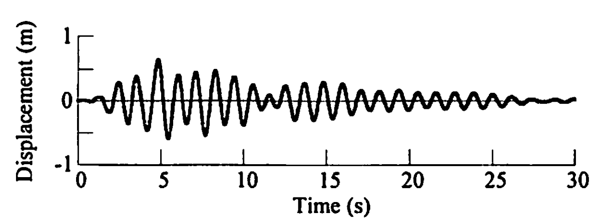
direction.

The ultimate curvature of the column is 0.0228 1/m in the longitudinal direction and 0.00536 1/m in the transverse direction. **Fig. 9** shows the curvatures at plastic hinge, they are about 0.006 1/m in the longitudinal direction and 0.022 1/m in the transverse direction under JR Takatori ground motion, and about 0.002 1/m in the longitudinal direction and 0.015 1/m in the transverse direction under Kawaguchi ground motion. The curvature is larger than ultimate curvature in the transverse direction under those two ground motions.

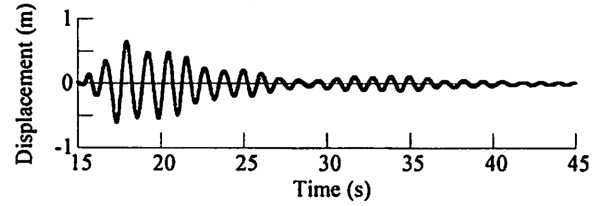
6. Response of 3g and 5g bridge

Response accelerations of the bridge designed based on S_A=3g are shown in **Fig. 10.** Deck response accelerations are 17-20m/s^2 in the longitudinal direction, and 7-18m/s^2 in the transverse direction under JR Takatori and Kawaguchi ground motions. **Fig. 11** shows that the deck response displacements are 0.68-0.7m in the longitudinal direction and 0.25-0.6m in the transverse direction.

Fig. 12 shows the moment-curvature relationship at plastic hinge of 3g column, the curvature is much smaller than the curvature of 1.75g column.

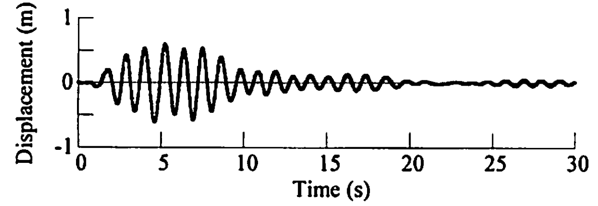


(a) JR Takatori

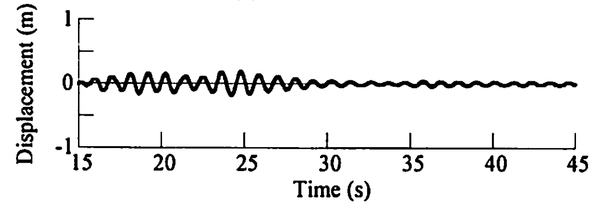


(b) Kawaguchi

(1) Longitudinal



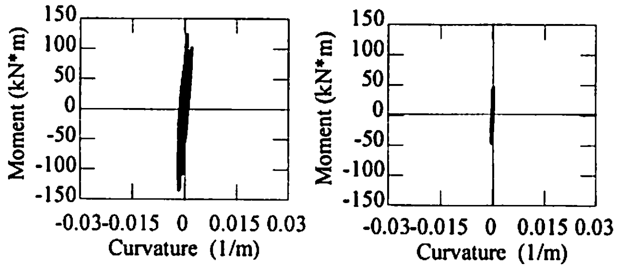
(a) JR Takatori



(b) Kawaguchi

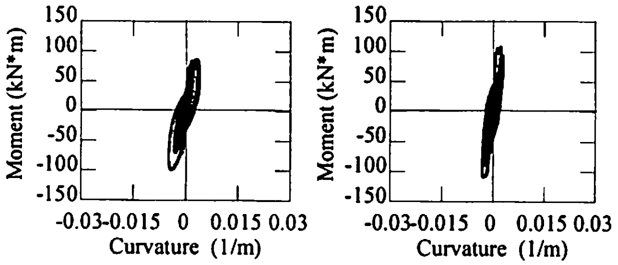
(2) Transverse

Fig. 11 Response displacement at the deck of 3g column



(a) JR Takatori (b) Kawaguchi

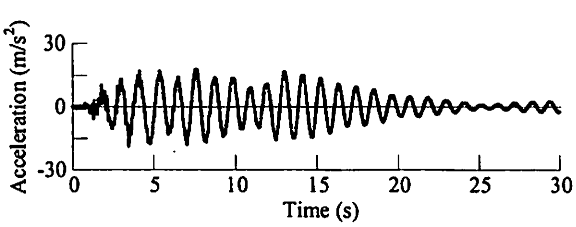
(1) Longitudinal



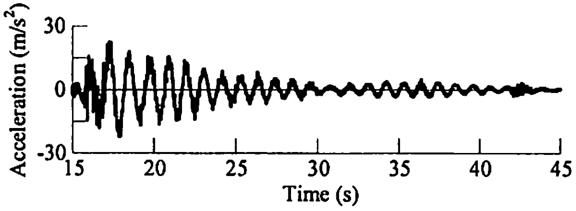
(a) JR Takatori (b) Kawaguchi

(2) Transverse

Fig.12 Moment-curvature hysteresis at the plastic hinge of 3g column

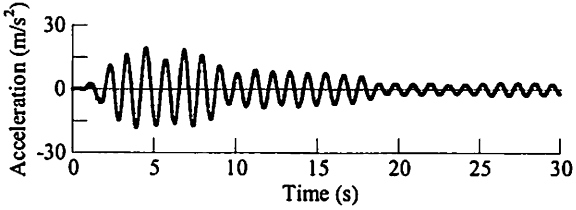


(a) JR Takatori

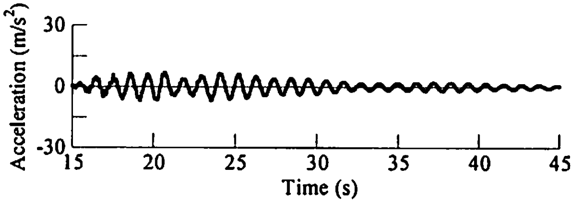


(b) Kawaguchi

(1) Longitudinal



(a) JR Takatori



(b) Kawaguchi

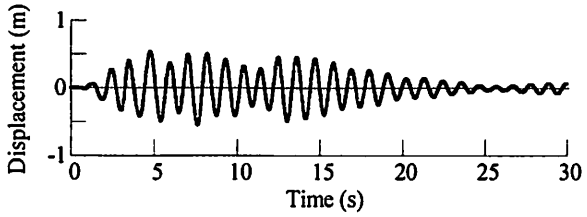
(2) Transverse

Fig. 13 Response acceleration at the deck of 5g column

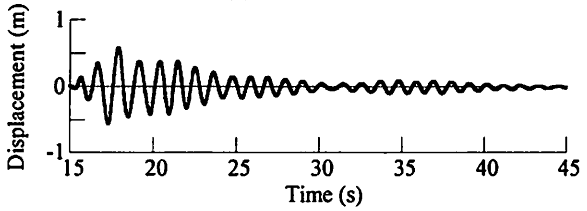
Deck response accelerations of the bridge designed based on S_A=5g are shown in **Fig. 13**. The peak deck accelerations are 20-22.5m/s^2 in the longitudinal direction and 7.5-20m/s^2 in the transverse direction. **Fig. 14** shows the deck response displacements. They are 0.55-0.6m in the longitudinal direction and 0.2-0.6m in the transverse direction. The displacement is slightly smaller than the displacement of the bridge designed based on S_A=1.75g. **Fig. 15** shows the curvatures of plastic hinge of 5g column. The yielding curvatures of the 5g column are 6.26×10^{-4} and 4.37×10^{-4} in the longitudinal and transverse direction, respectively. The response curvatures of 5g column are in the limited response that the response curvatures are slightly larger than the yielding curvature. Therefore, 5g column behaves almost elastically.

7. Conclusions

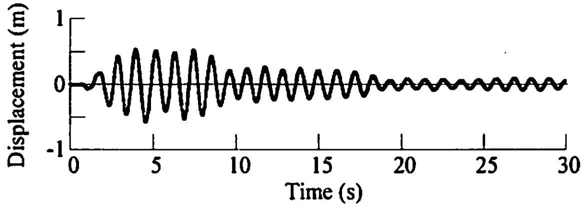
For evaluating the seismic response of a bridge designed based on 1.75g, 3g and 5g design response acceleration under two near-field ground motions, a nonlinear dynamic response analysis was conducted. Based on the results presented herein, the following conclusions may be deduced:



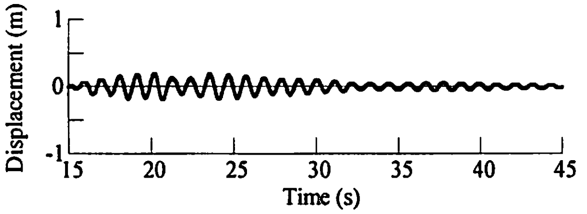
(a) JR Takatori



(b) Kawaguchi
(1) Longitudinal



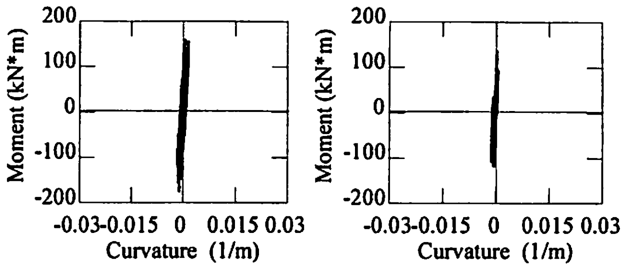
(a) JR Takatori



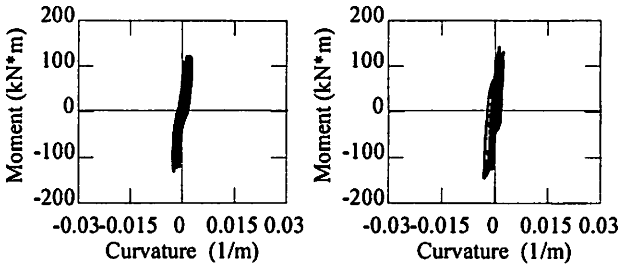
(b) Kawaguchi
(2) Transverse

Fig. 14 Response displacement at the deck of 5g column

(1) Bridge designed based on 3g design response acceleration behaves almost linearly with an exception of the column response in the transverse direction under JR Takatori ground motion.



(a) JR Takatori (b) Kawaguchi
(1) Longitudinal



(a) JR Takatori (b) Kawaguchi
(2) Transverse

Fig.15 Moment-curvature hysteresis at the plastic hinge of 5g column

(2) Bridge designed based on 5g design response acceleration behaves almost linearly, and it suffers no damage under the near-field ground motion.

(3) Considering that the cost increase of the 3g and 5g bridges is 1.04 and 1.36 times the current level, it is feasible to construct "damage-free bridge" even based on the current technology. However, it is expected to develop technology while enables us to construct damage-free bridges without any cost increase.

References

1) Kawashima, K. and Miyaji, K. : Seismic Performance Requirement of Highway Bridges, *Proc. 8th U.S. National Conference on Earthquake Engineering*, Paper No. 565, 2006.