매개변수를 고려한 강도로교의 취약도분석
Parametric Fragility Analysis of Steel Highway Bridges

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ABSTRACT

본 논문의 목적은 스팅베어링의 기존교량과 납-고무베어링(Lead-Rubber Bearing)으로 내진 보강된 교량에 대해서 갭(Gap)의 크기가 교량의 지진 취약도에 미치는 영향에 대해서 평가하였다. 이를 위해서 다경간 단순교(Multi-Span Simply Supported Bridge)와 다경간 연속교(Muti-Span Continuous Bridge)를 대상으로 취약도 분석을 실시하였다. 또한 다양한 크기의 갭사이즈를 도입하여 해석을 실시하였다. 이를 통해서 갭사이즈의 변화가 각 교량의 구성품에 미치는 영향을 확률적으로 평가할 수 있었고, 합성된 취약도 곡선을 이용하여 최적의 갭사이즈를 확정할 수 있었다.

1. Introduction

A seismic fragility analysis for bridges in a region generally includes the uncertainty of ground motion intensity and bridge properties. The uncertainty of the former consists of the characteristics of soils and seismology of the region. The bridge properties consist of material and structural one. The strengths of concrete and reinforcements are the material properties, and the structural properties are the stiffness/strength of foundations, gap size at expansion joints, etc. In a fragility analysis, the statistical characteristics of the material properties such as mean and standard deviation can be utilized for over all bridges. However, the structural properties usually vary so much from bridge to bridge that their mean and standard deviation may not be meaningful. Therefore, for this study, the fragility analysis includes only the material properties as the uncertainty of bridge properties, and the structural properties will be considered as varied parameters. This parametric study can cover the effect of wider range of structural properties and make it easy to capture how a parameter influences on the seismic fragility of bridges. This study picked up the gap size at expansion joints among structural properties to explain the uncertainty in bridges simulation.

The targets of this study are steel highway bridges among the bridges typically found in the Central and Southeastern United State (CSUS). The steel bridges generally have the two types of superstructure such as multi-span simply supported (MSSS) and multi-span continuous (MSC) as shown in Figure 1. The steel girders are usually supported by steel bearings on multi-column bents illustrated in Figure 1.

Lead-rubber bearings were improved as an effective retrofit measure to provide seismic resistance for bridges in Choi's study (2002). The bearings are known to provide good seismic protection to columns and controll deck displacement due to their large damping capacity. Therefore, this study will also inspect the effect of the parameter on the retrofitted bridges by lead-rubber bearings.

2. Analytical Models of Bridges and Retrofitted Measures

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The bridges consist of several components such as columns, abutments, steel bearings, foundations, and superstructures; some of them, particularly columns and bearings, exhibit highly nonlinear behavior. Therefore, 2D nonlinear analytical models of the bridges in longitudinal direction are developed using DRAIN-2DX (Prakash et al., 1992).

The superstructure is modeled using linear beam-column elements, since the superstructure is expected to remain elastic under the seismic loads applied in the longitudinal direction. The stiffness assigned to the superstructure is that of the composite reinforced concrete deck and girder system.

The columns consist of 22 DRAIN-2DX fiber elements for unconfined and confined concrete and reinforcements. Each fiber element has its own stress-strain relationship, and can be used to discretely model the cross-section of the column with its confined and unconfined concrete regions as well as the longitudinal steel reinforcement. A fixed steel bearing composed of one bilinear element and two multi-linear inelastic elements has very short elastic range of 0.3mm which is based on the Manders test (Mander et al., 1996). An expansion steel bearing is governed by frictional force and modeled by a bilinear element with hardening ratio of 0.018 following Manders suggestion (Mander et al., 1996). Abutments are assumed to have 2.4m height, conforming to the AASHTO standard, and show multi-linear inelastic behavior based on recent studies (Maroney et al.,1994). They are much stiffer and stronger for passive (pushing) action than active (pulling) action. The pile foundations are modeled using a combination of linear translational and rotational springs. Pounding between deck slabs can affect the response of the bridge bearings significantly and therefore must be included as part of the bridge model. Pounding is modeled using a tri-linear compression only link element with a gap to represent the expansion joint.

Lead-rubber bearings show bilinear behavior as shown with elastic stiffness of 3.152kN/mm followed by 10.2mm yield deformation for the MSSS bridge and 5.253kN/mm with 12.7mm yield deformation for the MSC bridge. The strain hardening ratio for the bilinear model of lead-rubber bearings is 0.1.

3. Bridge Model Simulation for Fragility Analysis

The simulation of bridges considers the uncertainty of construction materials, namely, concrete and reinforcement. The compressive strength of concrete with nominal strength of 20.7 MPa is taken as random variable of normal distribution with a mean strength of 31.0 MPa and a coefficient of variation (COV) of 0.2 following the recommendation by Hwang and Huo (1998). Hwang et al. (1990) specified the distribution of reinforcement as a lognormal distribution with a mean of 495.2 MPa and a COV of 0.07. Ten bridge samples for each type are generated using Latin Hypercube method based on the above data.

4. Input Ground Motions

Hwang et al. (2000) developed a suite of 100 ground motions for generating fragility curves of bridges in the CSUS region. To produce a synthetic ground motion, Hwang et al. used the uncertainties of seismic source, path attenuation, and local soil conditions. They at first generated a time series of random band-limited white noise and then multiplied an exponential window on the series. The suite of 100 ground motions ranges from weak to strong probabilistically and includes the uncertainties of soil and seismic parameters. The values of PGA of the suite ranges from 0.07g to 0.51g with mean PGA of 0.22g. The range of moment magnitude is Mw=6.0-8.0, and the range of epicentral distance is 40-100km.

5. Methodology for Generating Fragility Curves of Bridges

A fragility curve can be described as the probability of seismic demand reaching or exceeding structural capacity. The probability can be calculated using the following equation:

$$P = \left[\frac{S_d}{S_c} \geq 1 \right] \quad (1)$$

Where P is the probability of exceeding a specific damage state, S_d is the seismic demand, and S_c is the structural capacity. Since the structural capacity and seismic demand can be described by a lognormal distribution, the probability will be lognormally distributed with a lognormal cumulative probability density function as follow:

$$P = \Phi\left[\frac{\ln(S_d/S_c)}{\beta} \right] \quad (2)$$

where [·] is the standard normal distribution function and is the logarithmic standard deviation in which the logarithmic standard deviation for seismic capacity and demand are incorporated. The dispersion, β, is assumed to be 0.6 based on HAZUS (1999) and Mander's Study (Mander and Basoz, 1999) since it is very hard to estimate analytically or empirically. The capacity, S_c, will be defined by damage states of bridge components below. To obtain the demand, S_d, this study uses the probabilistic seismic demand models (PSDM) for each component. A PSDM can be obtained by linear regression analysis of 100 seismic response data through time history analysis in logarithmic scale. Therefore, a PSDM is expressed as below:

$$S_d = x^a \cdot \mathrm{EXP}(b) \cdot \varepsilon \quad (3)$$

where a and b are the regression parameters, x is the ground motion intensity parameter, typically PGA, and ε is the random-error of median=1.0 and standard deviation=$\sigma_{\ln\varepsilon}$.

The damage states for the components of interest are defined in Table 1 in terms of component demand. Following HAZUS (1999), five damage levels are defined: No Damage, Slight, Moderate, Extensive, and Complete. The failure of columns and abutments and unseating of decks can result in collapse of bridges. The damage of bearings results slight or moderate damage for bridges. Therefore, this study considers the damage for three crucial components - columns, abutments, and seating.

Table 1. Definition of Damage States for Bridge Components

Damage State	Slight	Moderate	Extensive	Complete
Column Ductility(μ)	1.0≤μ<2.0	2.0≤μ<4.0	4.0≤μ<7.0	7.0≤μ
Deck Displacement (δ,mm)	50≤δ<100	100≤δ<150	150≤δ<250	250≤δ
Abutment Deformation (δ,mm) (Active Action)	4≤δ<8	8≤δ<25	25≤δ<50	50≤δ
Abutment Deformation (δ,mm) (Passive Action)	7≤δ<15	15≤δ<37	37≤δ<146	146≤δ

The criteria of damage states for columns are based on Duttas Study (Dutta, 1999) using curvature ductility. The damage states for abutment deformation are estimated from an analytical model based on Choi's study (2002). The unseating of decks results in the collapse of bridges and thus it comes under the complete damage-state. Since the size of seat width vary from bridge to bridge, this study selects the seat-widths of 50, 100, 150, 250mm for unseating, and the unseating vulnerabilities for each seat-width are considered. In general, the seat width of bridges in CSUS is not larger than 250mm.

The PGA corresponding to 50% probability exceeding a damage state is Median PGA Value (MPV) of a fragility curve, and the larger MPV means the more resistance for a component to seismic loadings. Therefore, MPV can indicate seismic resistance of bridge components. In the Eq. (2), the probability of 50% for the median value can be expressed as follows:

$$0.5 = \Phi\left[\frac{\ln(S_d/S_c)}{\beta}\right] \tag{4a}$$

$$0.5 = \Phi[0] \tag{4b}$$

Substituting the Sd in the Eq. (3) into the above equation and then rearranging the equation, the equation for a median PGA value (MPV) is obtained as below:

$$x_m = \mathrm{EXP}\left[\frac{\ln(S_c)-b}{a}\right] \tag{5}$$

where x_m is median PGA value (MPV) and S_c, a, and b are predefined.

In this study, the responses of interest are the maximum column ductility, unseating of decks, and the active and passive deformation of abutments. The median PGA values lager than 5.0g are not shown since they have almost zero occurring probability.

6. Parameters of Structural Properties

The structural properties include geometric characteristics and the stiffness of members. Choi's study (2002) indicated that the gap size at expansion joints plays a major role in seismic behavior of highway bridges. Therefore, this study will demonstrate how the variation of the parameter influences on the fragility of the two bridges.

The gap size varies from small to large enough to avoid the pounding between decks or deck and abutment. The range of the variation is from 33% to 400% of the gap size of the as-built bridges of 38mm and 76mm for the MSSS and MSC bridge, respectively.

7. Effect of Gap Size on Fragility Curves

The effect of gap size on the fragility for the two bridges is illustrated in Figure 2 for the MSSS bridge and Figure 3 for the MSC bridge. In the grapes of Figures 2 and 3, MPVs larger than 5.0g are not appeared since they do not have physical meaning.

As-Built Bridges

The median PGA values for column ductility and deck displacement for the MSSS bridge decrease initially with increasing gap size as show Figure 2(a); which means that their vulnerability increases with larger gaps. However, beyond the gap of 50mm, the decrease of MPVs for the two components becomes very small. When gaps (moving tolerance) reach a point that is large enough to avoid poundings between decks or deck and abutment, column ductility and deck displacement show maximum response. Therefore, the responses and their MPVs beyond the gap become constant. Similar to the MPVs for abutments active and passive deformation, small gap permits small deck displacement and pounding force. Thus, the MPVs for abutment active deformation which depends on decks pulling action are larger with smaller gaps, but small with small gaps are the MPVs for passive deformation that is mainly dependent on pounding force. The MPVs for abutment active deformation shown Figure 2(a) are minimum at 38mm gap.

The median profiles for the MSC bridge in Figure 3(a) show more clear trend than those of the MSSS bridge. When the gap for the bridge reaches 100mm, the MPV for abutment passive deformation of slight damage goes upward sharply, which means that pounding does not occur between deck and abutment. The MPVs for active deformation of slight damage is larger than 3.0g for all gap sizes; which means that the probability of all damage states on abutment active deformation is almost zero.

In general, the vulnerabilities of columns, decks, and abutment active deformation are increase with larger gaps although that of abutment passive deformation acts reversely. The fragility of the abutment active deformation for the MSSS bridge is a little deviated from the trend; the component is most vulnerable at the 38mm gap that shows minimum MPVs for all damage states.

Retrofitted Bridge by Lead-Rubber Bearings

The general trend of median profiles for the bridges retrofitted by lead-rubber bearings is similar to those for the as-bulit bridges. However, the noticeable thing is that the MPVs for column ductility are fluctuated with the variation of the gap size as shown in Figure 3(b); the MPVs for column ductility for both bridges are minimum at 50mm gap. The deformation of lead-rubber bearings increases with increasing gap size. This changes the effective stiffness of the bearings and, therefore, the bridge stiffness including columns and bearings stiffness is varying according to the gap size variation; which is guessed at the important reason for the above phenomenon.

The effect for improving fragility by lead-rubber bearings for column ductility and deck displacement is moderate, very good for abutment active deformation, and negative for abutment passive deformation with small gaps in the MSSS bridge. For the MSC bridge, the effect of the bearings is very good for column ductility and slight for deck displacement and abutment passive deformation and negative for abutment active deformation.

8. Optimum Parameter Value

Combined Median PGAValue

The above results show that the MPVs of column ductility, deck displacement, and active abutment deformation decrease with increasing gap, which means they become vulnerable with larger gap. However, the MPVs of passive abutment deformation for two types of the MSSS bridge increase with increasing gap, which means they are less vulnerable with larger gap. The vulnerability of the three components of four increases with larger gap and the other three's vulnerability decreases with increasing gap. Therefore, the thing of importance is how to determine the optimum gap size to obtain the maximum MPV for a bridge system.

For comparison of the seismic fragility for various bridge types, it is essential that the overall bridge fragility be determined. This can be performed through a crude Monte-Carlo simulation but is considerably more expensive computationally than the simulation performed in this study. An alternative would be to combine the component fragility curves to get the system (bridge) fragility curve. This, however, requires information about the stochastic dependence between the damage states of the various bridge components.

Using first-order reliability theory, an upper and lower bound on the system fragility can be easily determined. The lower bound is the maximum component fragility while the upper bound is a combination of all of the component fragilities. These bounds are given in equation 4 where $P(F_i)$ is the probability of failure for each component and $P(F_{sys})$ is the probability of failure for the entire system.

$$\max{}_{i=1}^{m}[P(F_i)] \le P(F_{sys}) \le 1 - \prod_{i=1}^{m}[1-P(F_i)] \qquad (5)$$

These first-order bounds are valid for a series type system which is where a failure of one the components constitutes a failure of the system. When a bridge is modeled in the longitudinal direction as in this study, it in fact behaves like a series system as seen in Figure 1. The lower bound represents the probability of failure for a system whose components are all fully stochastically dependent and provides an un-conservative estimate for the fragilities of the subject bridges. The upper bound assumes that the components are all statistically independent and provides a conservative estimate on the overall bridge fragility. The upper bound estimate to the system fragility gets better as the difference between the upper and lower bound decreases.

Median PGA values (MPV) are reversely proportional to probability of damage. Therefore, the lover and upper bound for a combined MPV can be expressed like below:

$$MPV \text{ for}[1-P(F_i)] \le Combined-MPV \le MPV \text{ for}[\max{}_{i=1}^{m}[P(F_i)] \qquad (6)$$

As previously mentioned, a MPV can represent the resistance for a component to seismic loads and so does a combined MPV for a bridge system. Therefore, combined MPVs for a whole bridge can be calculated with parameter variation and an optimized parameter can be estimated by comparing the combined MPVs for each parameter.

Gap Size Effect

Figures 4 and 5 show the median profiles for slight damage for each component of MSSS and MSC bridge. In Figure 4(a) for the as-built MSSS bridge, the maximum combined MPV occurs at the gap of 12.5mm although the bridge has originally the gap of 38.0 mm. The upper bound of the combined MPV for the gap of 12.8mm is 0.43g that is 23% larger than that for the gap of 38.0mm, 0.35g.

Looking at the Figure 4(b) for the MSSS bridge retrofitted by lead-rubber bearings, the maximum combined MPV is observed at the gap of 38.0mm different from the case of the as-built bridge. Comparing Figures 4(a) and 4(b), the upper bound for the combined MPV at the gap of 38.0mm increase from 0.35g to 0.47g by the retrofitting of lead-rubber bearings, however, at the gap of 12.5mm, the value decrease from 0.43g to 0.32g by the measure. It is noticeable that lead-rubber bearings produce negative effect with the gap of 12.8mm for the MSSS bridge; which means that if the bridge has the gap of 12.8mm originally, the retrofit by lead-rubber bearings is not available.

For the MSC bridge having the gap of 76mm originally, the gap of 35mm is estimated as optimum gap size for all three types of bridges as shown in Figure 5(a). Comparing between the case of as-built and lead-rubber bearings, as shown in Table 4, the MSC bridge retrofitted lead-rubber bearings has similar upper bounds for combined MPVs at the first three gaps to those of the as-built bridge since abutment passive deformation at the gap of 25mm and deck displacement at the gaps of 50mm and 78mm control the upper bounds of the combined MPVs. However, lower bounds of combined MPVs at the three gaps are improved much more by lead-rubber bearings than the upper ones since column ductility demand is improved by the bearings.

Table 4. Combined MPVs for As-Built and Retrofitted MSC Bridge at Three Gaps

		Gap1 (25mm)	Gap2 (50mm)	Gap3 (76mm)
As-Built	Lower (g)	0.248	0.234	0.224
	Upper (g)	0.287	0.284	0.269
Retrofitted	Lower (g)	0.262	0.272	0.268
	Upper (g)	0.297	0.296	0.280

9. Conclusions

The larger gap size increases the vulnerability of column ductility, deck displacement, and active abutment deformation since the deck moving tolerance increases with the larger gap. However, the larger gap decreases the vulnerability of passive abutment deformation since it reduces pounding forces on abutments. When the gap is smaller than 25 mm, the MPVs for the three responses of column ductility, deck displacement, and active abutment deformation are very sensitive to the variation of the gap size.

This study illustrated how to determine optimum gap. As shown in Figure 4(a), the lower bound of MPV for slight damage in the MSSS as-built bridge is improved from 0.345g to 0.426g just by reducing the gap from 38.1 mm to 12.5 mm; which is the increase of 23%. Lead-rubber bearings along with gap size control can improve the vulnerability of the MSSS bridge as shown in Figure 4(b). In the MSC bridge, the gap of 35 mm produced the optimized MPV for the as-built bridge and the retrofitted bridge by lead-rubber bearings.

The results of this study show that retrofit by lead-rubber bearings may provoke negative effect on seismic fragility of bridges under special conditions although lead-rubber bearings are believed to improve seismic vulnerabiliyt of bridges effectively.

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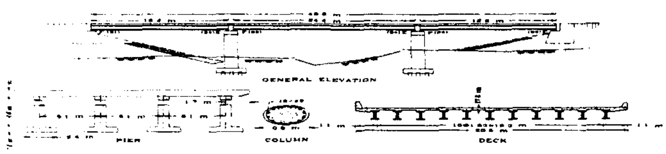
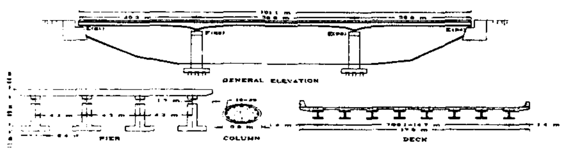


(a) Multi-Span Simply Supported Bridge



(b) Multi-Span Continuous Bridge

Figure 1. Typical Steel Highway Bridges in CSUS

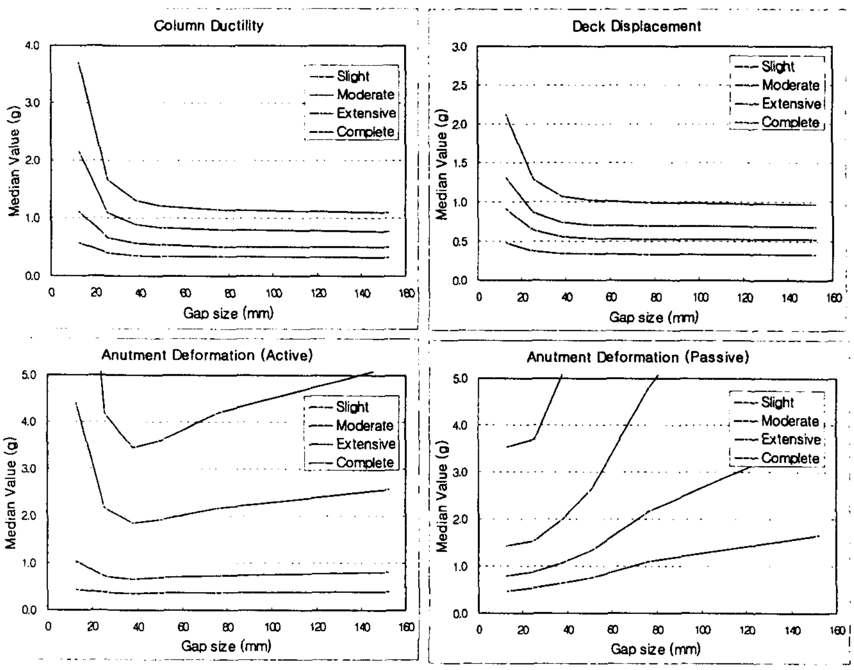


Figure 2(a). MPVs due to Gap Size Variation in MSSS Bridge

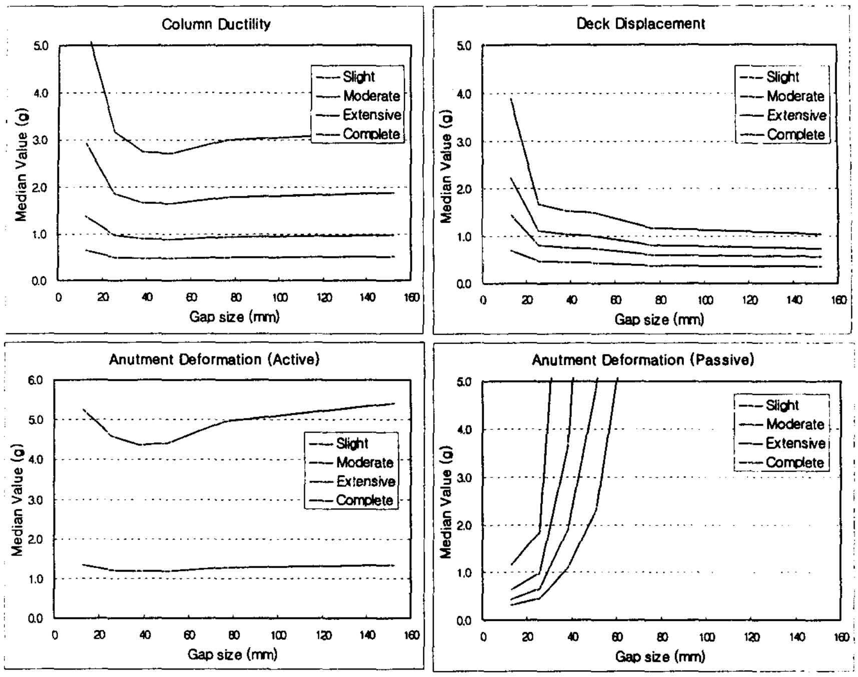


Figure 2(b). MPVs due to Gap Size Variation in MSSS Bridge Retrofitted Lead-Rubber Bearings

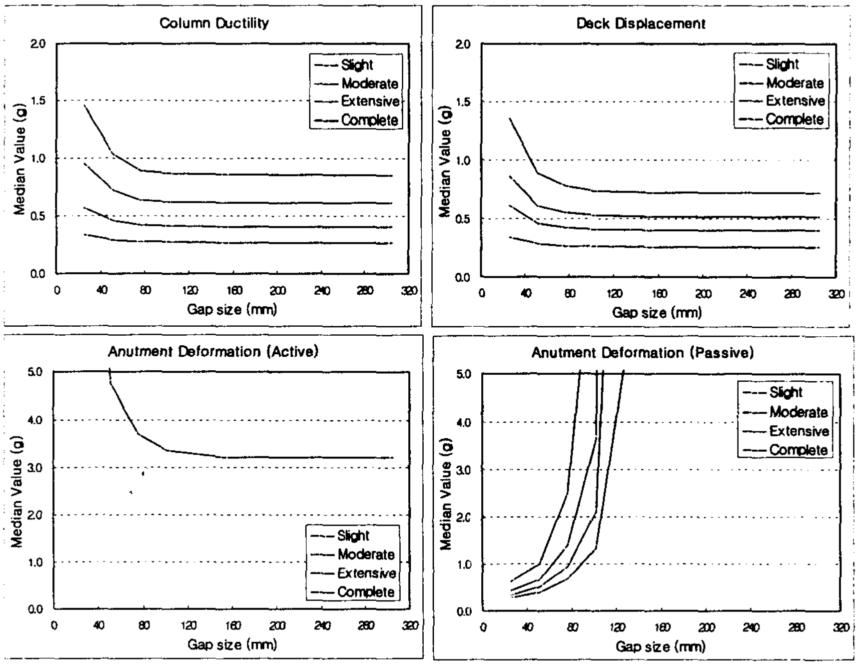


Figure 3(a). MPVs due to Gap Size Variation in MSC Bridge

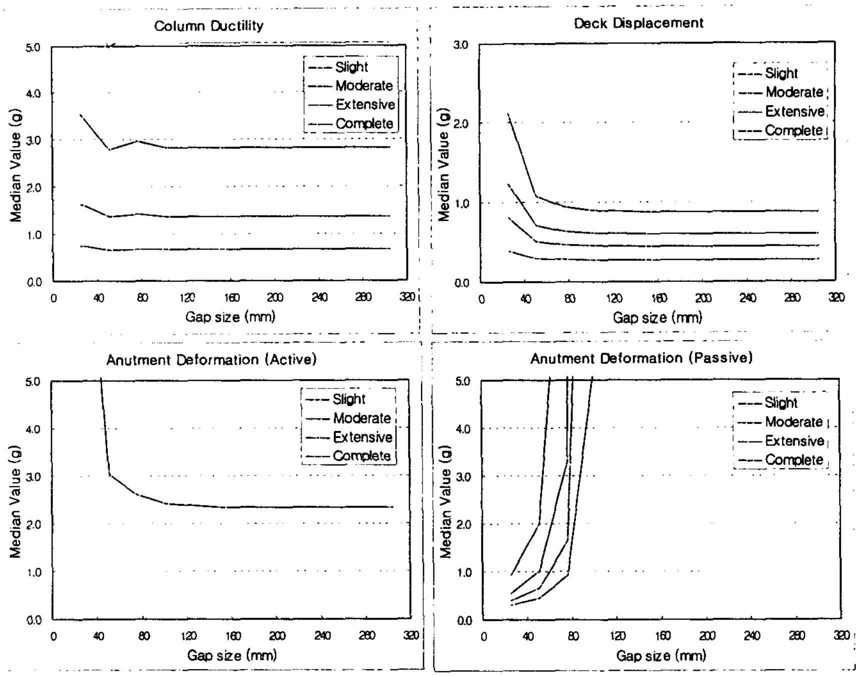


Figure 3(b). MPVs due to Gap Size Variation in MSC Bridge Retrofitted Lead-Rubber Bearings

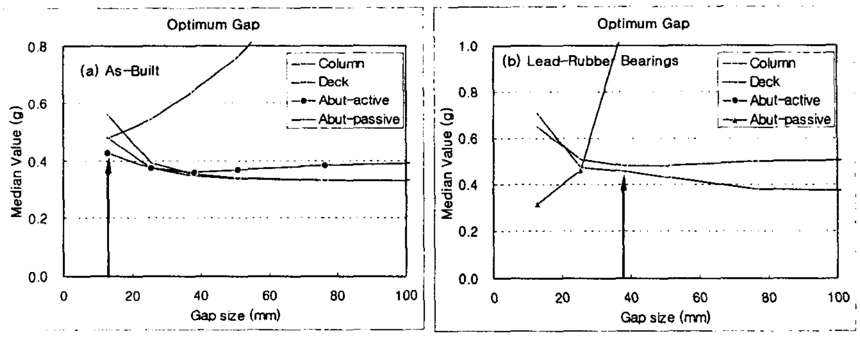


Figure 4. Optimum Gap in As-Built and Retrofitted MSSS Bridges

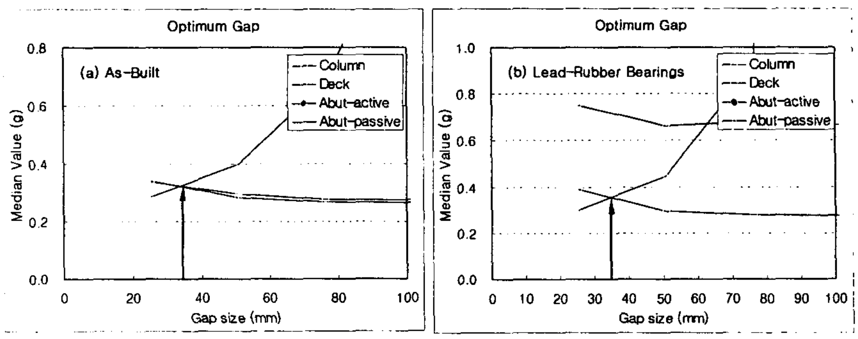


Figure 5. Optimum Gap in As-Built and Retrofitted MSC Bridges