13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 338

# DETERMINATION OF PARTIAL FACTORS FOR MODEL UNCERTAINTIES ADOPTED IN EARTHQUAKE PROOF DESIGN CODE FOR HIGHWAY ARCH BRIDGES

Toshiyuki SUGIYAMA[1]

## SUMMARY

Determination of partial factors for model uncertainties prescribed in earthquake proof design code for highway arch bridges has been attempted. In this study, model uncertainties mean the scatter of the numerical results obtained from dynamic response analysis of arch bridge under strong earthquake motion. The dynamic response analysis with respect to one typical steel arch bridge is carried out by several bridge researchers and engineers. Inverse-Lohse type arch deck bridge whose arch span length and bridge length are 114 meters and 173 meters, respectively, is adopted as the subject of study. Hyogoken-Nanbu Earthquake (Kobe Earthquake) wave is used as an input earthquake motion. Computer software for structural analysis such as ABAQUS Ver.5.8, Y-fiber3D Ver.3.2, TDAP-III Ver.2.11, and NON-PIER are applied in numerical analysis. Either Fiber model or Moment-Curvature model is used as a structural element model. About damping, either Rayleigh damping model or mass-proportional one is adopted in numerical analysis. Furthermore, it is treated in this study whether geometric nonlinearity is considered or not. The value of partial factor for model uncertainties is determined by considering the scatter of maximum axial force of arch rib, maximum bending moment of stiffened girder, maximum horizontal displacement of stiffened girder, and so on obtained from dynamic response analysis. The result shows that the value of 1.15 may be suitable to the partial factor in case that Moment-Curvature model is adopted instead of Fiber model. It has been also revealed that the value of 1.10 may be appropriate to the case that mass-proportional damping model is applied instead of Rayleigh damping model or the case that geometric nonlinearity is not taken into account in dynamic response analysis of highway arch bridges. Finally, the following value of partial factor $\gamma_A$ is proposed according to the response analysis; that is, $\gamma_A$ = 1.0 in case of static analysis and $\gamma_A$ = 1.05 in case of dynamic analysis.

## INTRODUCTION

In a structural design, the kind, shape and size of structural member may be determined so as to ensure the required performance of structure. In other words, structure or structural member may be designed so that the probability that the structural member will reach the predicted limit state(s) may become lower than the a priori prescribed target value. However, it is considerably difficult to introduce the above mentioned

[1] Structural Engineering Division, Interdisciplinary Graduate School of Medicine and Engineering, University of Yamanashi

probability into the structural design code directly at present. Therefore, the structural design format given by $R_d/S_d$ >= SF is commonly adopted in most of design codes and specifications in the world. Here, $R_d$, $S_d$ and SF correspond to the design value of strength, design value of load effect, and safety factor, respectively. As well known, this design format is also adopted in ISO 2394[1]. In ISO 2394, it is strongly recommended that the characteristic values of material strength and actions included in the calculation procedure of $R_d$ and $S_d$ are determined based on the reliability theory. However, the determination methodology with respect to the partial factors such as the one for model uncertainties, for load combinations, for geometric uncertainties, and so on included in the calculation procedure of $R_d$ and $S_d$ has not been explained in ISO 2394. In Standard Specification for Design and Construction of Concrete structures in Japan [2], it is not described how the values of these partial factors have been determined.

The purpose of this study is to attempt to determine the partial factors for model uncertainties prescribed in earthquake proof design code for highway arch bridges. In this study, model uncertainties mean the scatter of the numerical results obtained from dynamic response analysis of arch bridge under strong earthquake motion. The dynamic response analysis with respect to one typical steel arch bridge is carried out by several bridge researchers and engineers. Inverse-Lohse type arch deck bridge whose arch span length and bridge length are 114 meters and 173 meters, respectively, is adopted as the subject of study. Hyogoken-Nanbu Earthquake (Kobe Earthquake) wave is used as an input earthquake motion. Computer software for structural analysis such as ABAQUS Ver.5.8, Y-fiber3D Ver.3.2, TDAP-III Ver.2.11, and NON-PIER are applied in numerical analysis. Either Fiber model or Moment-Curvature model is used as a structural element model. About damping, either Rayleigh damping model or mass-proportional one is adopted in numerical analysis. Furthermore, it is treated in this study whether geometric nonlinearity is considered or not. The value of partial factor for model uncertainties is determined by considering the scatter of maximum axial force of arch rib, maximum bending moment of stiffened girder, maximum horizontal displacement of stiffened girder, and so on obtained from dynamic response analysis.

## ADOPTED DESIGN FORMAT

The design format considered in this study is as follows [3].

$$\gamma_I \frac{S_d}{R_d} \leq 1 \tag{1}$$

where

$R_d$: design value of limit strength

$$= \frac{1}{\gamma_F \cdot \gamma_{B1} \cdot \gamma_{B2}} R\left( \frac{f_{k,1}}{\gamma_{M,1}}, \frac{f_{k,2}}{\gamma_{M,2}}, \cdots, \frac{f_{k,n}}{\gamma_{M,n}} \right) \tag{2}$$

$f_{k,i}$: characteristic value of material strength

$\gamma_F$: partial factor for the scatter associated with fabrication and/or construction

$\gamma_{B1}$: partial factor related to the accuracy of analytical model for structural strength

$\gamma_{B2}$: partial factor in order to consider the feature of the limit state(s) that should be taken into account at design stage

$\gamma_{M,i}$: partial factor in order to consider the effect of material property on the limit state of structure

$R(\cdot)$: function to calculate structural limit strength based on material strength

$S_d$: design value of load effect

$$= \gamma_A \cdot \gamma_C \cdot \sum S\left(\gamma_{1,j} \cdot \gamma_{2,j} \cdot \gamma_{3,j} \cdot F_{k,j}\right) \quad (3)$$

$F_{k,j}$: design value of j-th action/load

$\gamma_A$: partial factor related to the accuracy of analytical model for structural response

$\gamma_C$: partial factor in order to consider the occurrence frequency of load combination

$\gamma_{1,j}$: partial factor in order to consider the feature of limit state(s) that may depend on how j-th action/load acts on the structure

$\gamma_{2,j}$: partial factor that indicates whether the j-th action/load is dominant in consideration of load combination

$\gamma_{3,j}$: partial factor in order to take account of the uncertainties of j-th action/load itself

$S(\cdot)$: function to calculate structural response in case that design actions/loads act on a designed structure

$\gamma_{I}$_: safety factor that depends on the importance of structure

In this study, only the partial factor $\gamma_A$ is selected as the subject of study.

## DETERMINATION METHODOLOGT OF PARTIAL FACTOR $\gamma_A$

### Partial factor $\gamma_A$

The partial factor $\gamma_A$ is the factor that is related to the accuracy of analytical model for structural response. In other words, $\gamma_A$ is the partial factor for model uncertainties adopted in structural response analysis. As the dynamic response characteristics of steel arch bridge under strong earthquake motions here, $\gamma_A$ is the partial factor with respect to the scatter arising when maximum response displacement, maximum response stress resultant, residual displacement, and so on are calculated by using some kind of computer software for structural analysis.

### Determination methodology of partial factor $\gamma_A$

As mentioned in the previous section, model uncertainties mean the scatter of the numerical results obtained from dynamic response analysis of arch bridge under strong earthquake motion. The dynamic response analysis with respect to one typical steel arch bridge is carried out by several bridge researchers and engineers (described as “calculation institute” hereafter). Inverse-Lohse type arch deck bridge whose arch span length and bridge length are 114 meters and 173 meters, respectively, is adopted as the subject of study (see Fig. 1).

Hyogoken-Nanbu Earthquake (Kobe Earthquake) wave is used as an input earthquake motion. Computer software for structural analysis such as ABAQUS Ver.5.8, Y-fiber3D Ver.3.2, TDAP-III Ver.2.11, and NON-PIER are applied in numerical analysis. Either Fiber model or Moment-Curvature model is used as a structural element model. About damping, either Rayleigh damping model or mass-proportional one is adopted in numerical analysis. Furthermore, it is treated in this study whether geometric nonlinearity is considered or not. The value of partial factor for model uncertainties is determined by considering the scatter of maximum axial force of arch rib, maximum bending moment of stiffened girder, maximum horizontal displacement of stiffened girder, and so on obtained from dynamic response analysis

The following procedure is adopted in order to determine the value of partial factor $\gamma_A$.

1) The average of each of maximum response displacement, maximum response stress resultant, residual displacement, and so on calculated by several calculation institutes is obtained.

2) Each of the maximum response values calculated by several calculation institutes is divided by the average value obtained in step 1), and then, non-dimensional response value is obtained.
3) By investigating the scatter of this non-dimensional response value, the value of partial factor is determined.

.

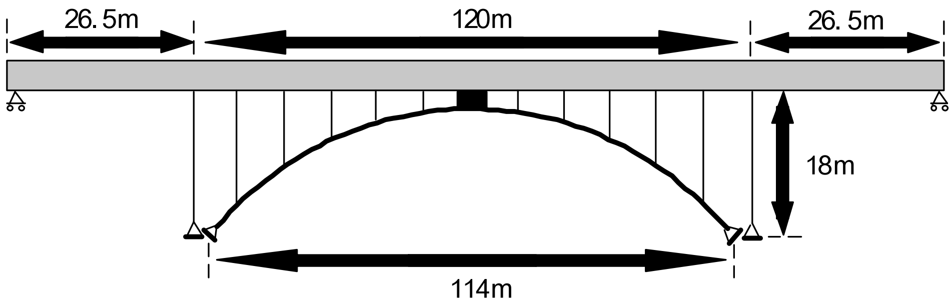


Fig. 1 Steel inverse-Lohse arch deck bridge

## CALCULATION RESULTS AND DISCUSSION

### Static Analysis

Before the discussion about dynamic response analysis results, static response analysis results are introduced first. The response values of displacement, reaction force, bending moment and axial force of seven positions shown in Fig. 2 are calculated. Figs. 3 (a)-(d) show the scatter of non-dimensional response values. In each figure, vertical axis corresponds to the non-dimensional response value and horizontal axis corresponds to the calculation institute that conducted the numerical analysis by using their own computer software for structural analysis. From Figs. 3 (a)-(d), it is found that non-dimensional response values except in case of bending moment of arch rib take the values between 0.97 and 1.03. That is, the scatter of the static responses is considerably small. This fact indicates that significant model uncertainties with respect to static response analysis of arch bridge can not be recognized. Therefore, it may be concluded that the value of partial factor $\gamma_A$ for static response analysis except in case of bending moment is between 1.00 and 1.05. It should be necessary to reconsider about the reason why non-dimensional response value of bending moment becomes large.

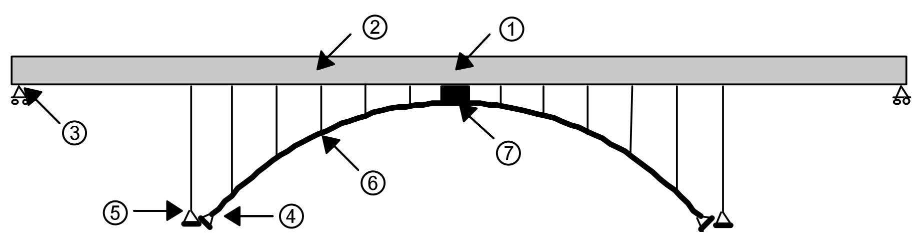


Fig. 2 Positions where static response is calculated

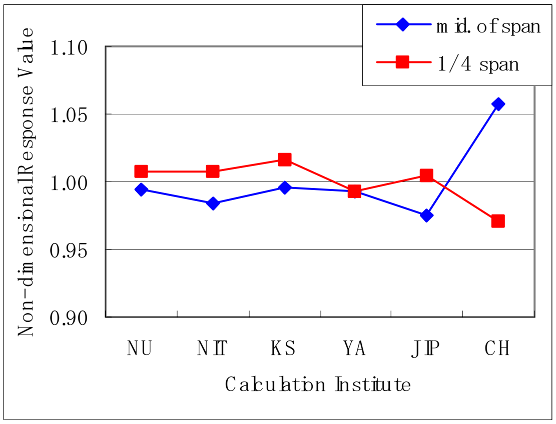


Fig. 3 (a) Static response value of vertical displacement of stiffened girder

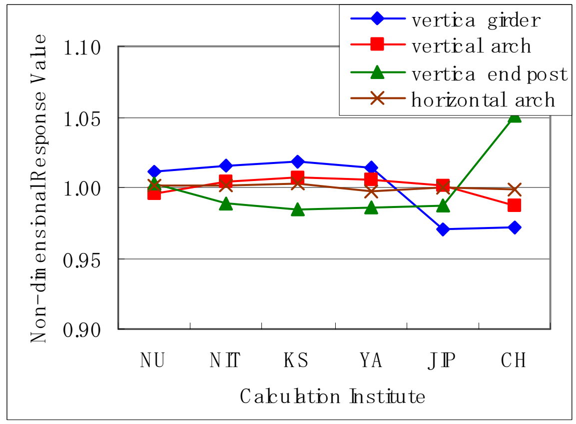


Fig. 3 (b) Static response value of reaction force

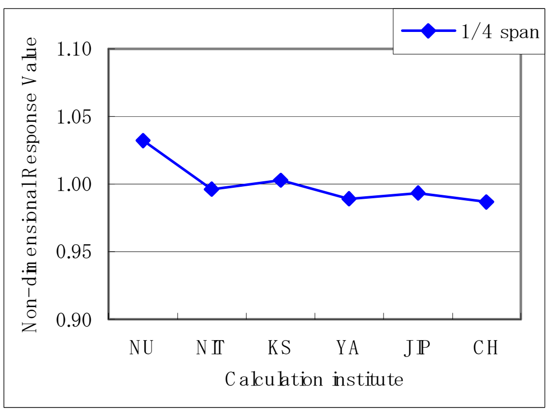


Fig. 3 (c) Static response value of bending moment of stiffened girder

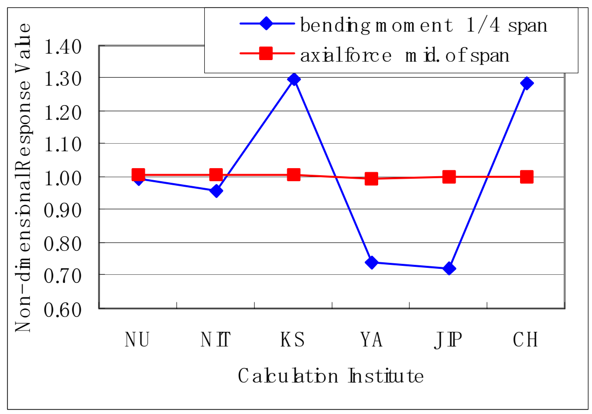


Fig. 3 (d) Static response value of bending moment and axial force of arc rib

**Dynamic Analysis**

The following seven dynamic response non-dimensional values at eleven positions illustrated in Fig. 4 are calculated here.

1) horizontal displacement (middle of stiffened girder, middle and 1/4 span length of arch rib )
2) vertical displacement (1/4 and 3/4 span length of stiffened girder)
3) bending moment of stiffened girder (1/4 and 3/4 span length)
4) bending moment of arch rib (1/4 and 3/4 span length)
5) axial force of arch rib (at the both ends of arch rib)
6) axial force of end post (at the bottom of end post)
7) shear force of center post (at the top of center post)

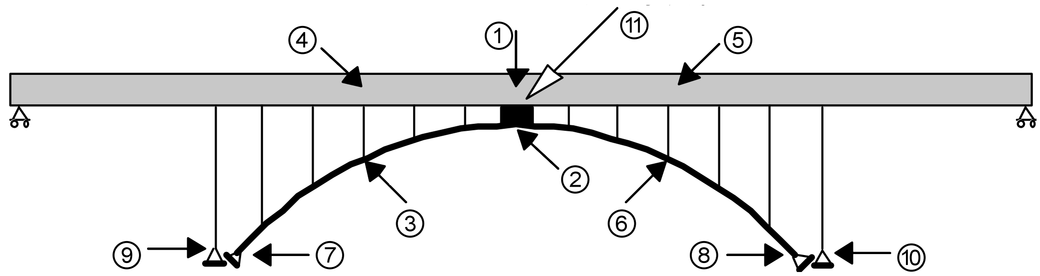


Fig. 4 Positions where dynamic response value is calculated

The scatters of dynamic response non-dimensional values are shown in Figs. 5 (a)-(g) in which horizontal axis corresponds to the calculation institute that conducted the numerical analysis by using their own computer software for structural analysis.

From Fig. 5 (a), the fact that the non-dimensional value with respect to the horizontal displacement takes the value between 0.95 and 1.05 is found. On the other hand, non-dimensional vertical displacement varies from 0.90 to 1.10 as expressed in Fig. 5 (b).

It is found from Figs. 5 (c) and (d) that the non-dimensional response values of bending moments of both stiffened girder and arch rib are approximately from 0.80 to 1.20. Furthermore, it can be also recognized that the scatter of arch rib is larger than that of stiffened girder when non-dimensional value at 1/4 span length position is compared with that at 3/4 span length one.

Fig. 5 (e) shows the scatter of non-dimensional response value of axial force of arch rib. About the axial force of arch rib, as only the axial compression force denoted as $N_{min}$ in this figure is important in the design, $N_{min}$ is taken out here as the subject of consideration. From Fig. 5 (e), the fact that the scatter of $N_{min}$ is small is obtained.

About the non-dimensional response value of axial force of end post, the calculation results obtained by two institutes are extremely large as shown in Fig. 5 (f). It may be considered that human error has caused this significant difference. On the other hand, the non-dimensional value of shear force of center post takes the value from 0.85 to 1.15 as presented in Fig. 5 (g).

Summarizing the facts obtained so far, the scatter of non-dimensional response values in case of dynamic analysis is larger than that in case of static analysis. The reason may be as follows. Table 1 shows all of the non-dimensional response values obtained by every institute. In this table, the value that is smaller than 0.80 or larger than 1.20 is expressed by red-colored boldface. And if the time when maximum response occurs is explicitly different from the other analytical results, its non-dimensional value is encircled with bold frame in Table 1. From this table, it is found that nine non-dimensional values among 66 ones (the ratio is about 0.14) take considerably different values regarding to not only maximum response value but also its occurrence time. Accordingly it is necessary to pay attention to not only maximum response value but also its occurrence time when dynamic response analysis of arch bridge under strong earthquake motion is conducted. These facts indicate that the value of partial factor $\gamma_A$ is to be determined separately for dynamic analysis and static one.

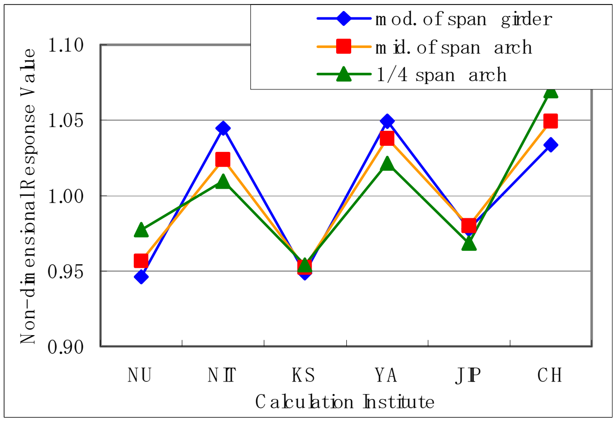


Fig. 5 (a) Dynamic non-dimensional response value of horizontal displacement

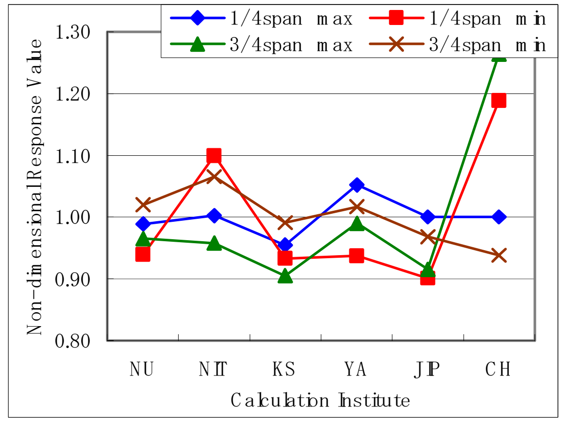


Fig. 5 (b) Dynamic non-dimensional response value of vertical displacement of stiffened girder

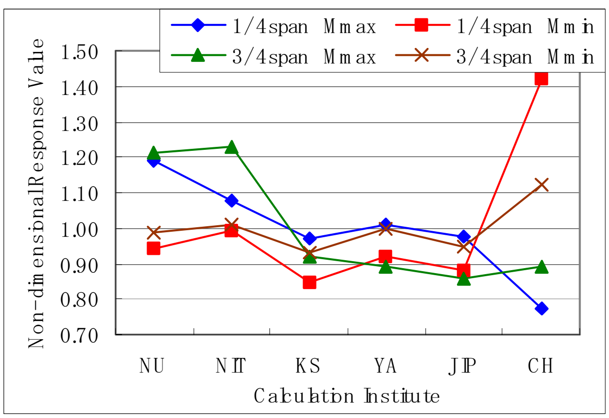


Fig. 5 (c) Dynamic response non-dimensional value of bending moment of stiffened girder

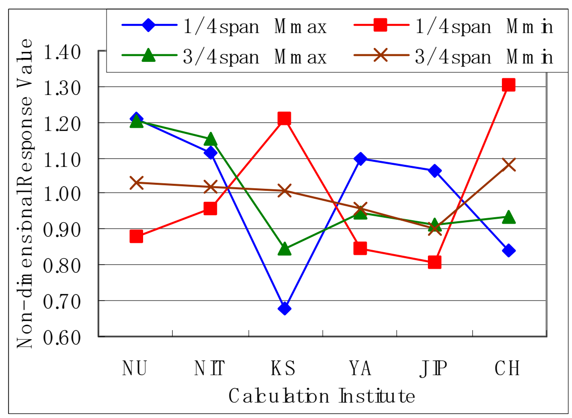


Fig. 5 (d) Dynamic non dimensional response value of bending moment of arch rib

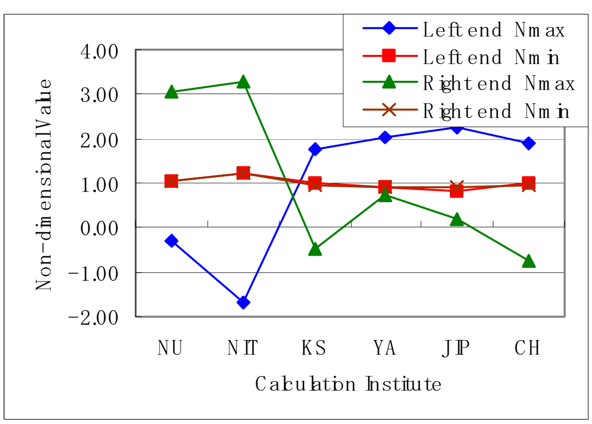


Fig. 5 (e) Dynamic non-dimensional response value of axial force of arch rib

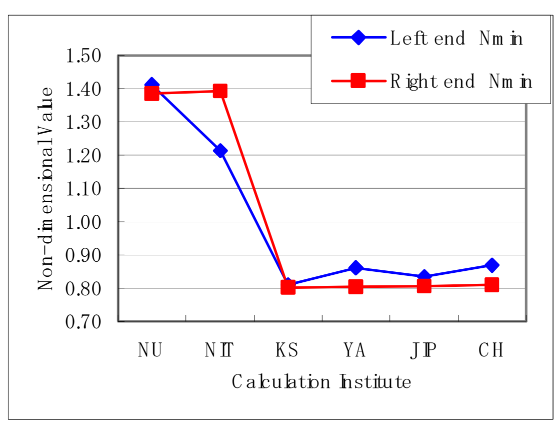


Fig. 5 (f) Dynamic non-dimensional response value of axial force of end post

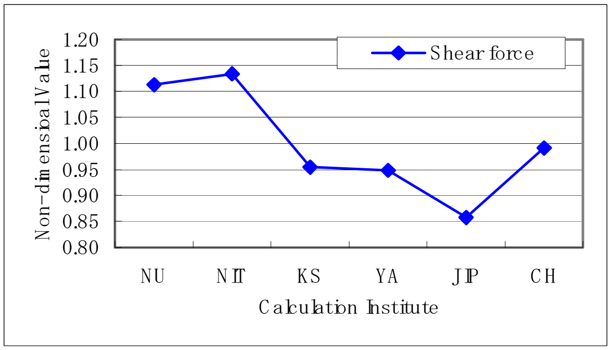


Fig. 5 (g) Dynamic non-dimensional response value of shear force of center post

Table 1 Dynamic non-dimensional response value and its occurrence time

| Response quantity | | | NU | NIT | KS | YA | JIP | CH | remarks |
|---|---|---|---|---|---|---|---|---|---|
| Horizontal displacement | mid. of girder | | 0.95 | 1.04 | 0.95 | 1.05 | 0.98 | 1.03 | (1) |
| | mid. of arch | | 0.96 | 1.02 | 0.95 | 1.04 | 0.98 | 1.05 | (2) |
| | 1/4 span of arch | | 0.98 | 1.01 | 0.95 | 1.02 | 0.97 | 1.07 | (3) |
| Vertical displacement of Stiffened girder | 1/4 span | max | 0.99 | 1.00 | 0.95 | 1.05 | 1.00 | 1.00 | (4) |
| | | Min | 0.94 | 1.10 | 0.93 | 0.94 | 0.90 | 1.19 | |
| | 3/4 span | max | 0.97 | 0.96 | 0.91 | 0.99 | 0.92 | **1.26** | (5) |
| | | Min | 1.02 | 1.07 | 0.99 | 1.02 | 0.97 | 0.94 | |
| Bending Moment of Stiffened girder | 1/4 span | Mmax | 1.19 | 1.08 | 0.97 | 1.01 | 0.97 | **0.78** | (6) |
| | | Mmin | 0.94 | 1.00 | 0.84 | 0.92 | 0.88 | **1.42** | |
| | 3/4 span | Mmax | **1.21** | **1.23** | 0.92 | 0.89 | 0.86 | 0.89 | (7) |
| | | Mmin | 0.99 | 1.01 | 0.93 | 1.00 | 0.95 | 1.12 | |
| Bending Moment of Arch rib | 1/4 span | Mmax | **1.21** | 1.11 | **0.68** | 1.10 | 1.06 | 0.84 | (8) |
| | | Mmin | 0.88 | 0.96 | **1.21** | 0.84 | **0.80** | **1.30** | |
| | 3/4 span | Mmax | 1.20 | 1.16 | 0.85 | 0.95 | 0.91 | 0.94 | (9) |
| | | Mmin | 1.03 | 1.02 | 1.01 | 0.96 | 0.90 | 1.08 | |
| Axial Force of Arch rib | left end | Nmin | 1.04 | **1.21** | 0.99 | 0.90 | 0.84 | 1.01 | (10) |
| | right end | Nmin | 1.04 | **1.21** | 0.94 | 0.93 | 0.92 | 0.95 | (11) |
| Axial Force of End post | left bottom | Nmin | **1.41** | **1.21** | 0.81 | 0.86 | 0.84 | 0.87 | (12) |
| | right bottom | Nmin | **1.38** | **1.39** | **0.80** | **0.80** | 0.81 | 0.81 | (13) |
| Shear Force at the top of center post | | | 1.11 | 1.13 | 0.96 | 0.95 | 0.86 | 0.99 | (14) |

*Effect of adoption of Fiber model or Moment-Curvature model on response analysis result*

The effect of adoption of Fiber model or Moment-Curvature model as the structural element model on the dynamic response analysis results is discussed here. Non-dimensional response values resulted from dynamic response analysis where analytical conditions except only structural element model are completely the same are shown in Fig. 6. In this figure, non-dimensional response value is calculated by dividing the response value obtained through Moment-Curvature model by that obtained through Fiber model. The horizontal axis represents the response quantities listed in remarks column in Table 1.

From Fig. 6, it is found that the non-dimensional value in case of Moment-Curvature model generally becomes larger than those in case of Fiber model and its ratio varies from 0.95 to 1.12.

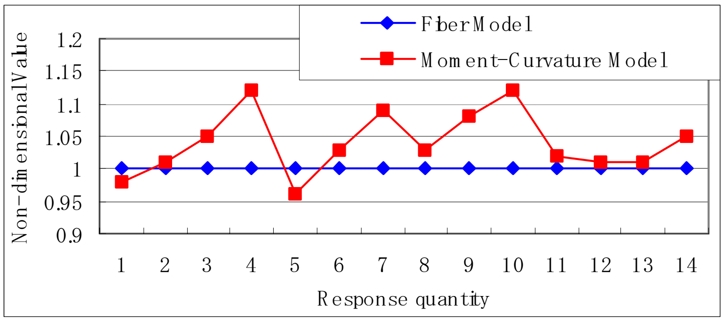


Fig. 6 Comparison between Fiber model and Moment-Curvature model

*Effect of adoption of Rayleigh damping model or mass-proportional one on response analysis result*

The effect of adoption of Rayleigh damping model or mass-proportional one as the structural damping model on the dynamic response analysis results is investigated here. Non-dimensional response value resulted from dynamic response analysis where analytical conditions except only damping model are completely the same is shown in Fig. 7. In this figure, non-dimensional response value is calculated by dividing the response value obtained through mass-proportional model by that obtained through Rayleigh damping model. The horizontal axis represents the response quantities listed in remarks column in Table 1.

The facts that the non-dimensional value in case of mass-proportional damping model generally becomes smaller than that in case of Rayleigh damping model and its ratio takes the value between 0.90 and 1.10 are found.

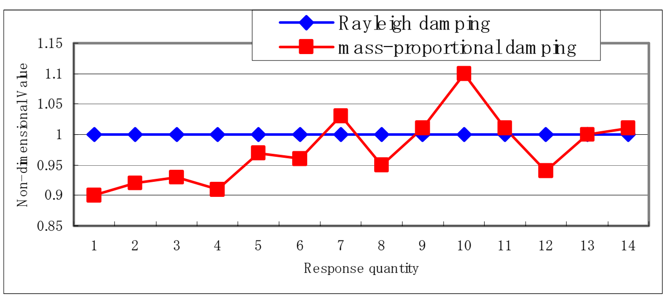


Fig. 7 Comparison between Rayleigh damping and mass-proportional damping

*Effect of consideration of geometric nonlinearity on response analysis result*

The effect of consideration of geometric nonlinearity on the dynamic response analysis results is discussed here. Non-dimensional response value resulted from dynamic response analysis where analytical conditions except only consideration of geometric nonlinearity are completely the same is shown in Fig. 8. In this figure, non-dimensional response value is calculated by dividing the response value obtained through infinitesimal displacement analysis by that obtained through finite displacement analysis. The horizontal axis represents the response quantities listed in remarks column in Table 1.

From Fig. 8, it is found that the non-dimensional value in case of infinitesimal displacement analysis is smaller than that in case of finite displacement one and its ratio changes from 0.90 to 1.0.

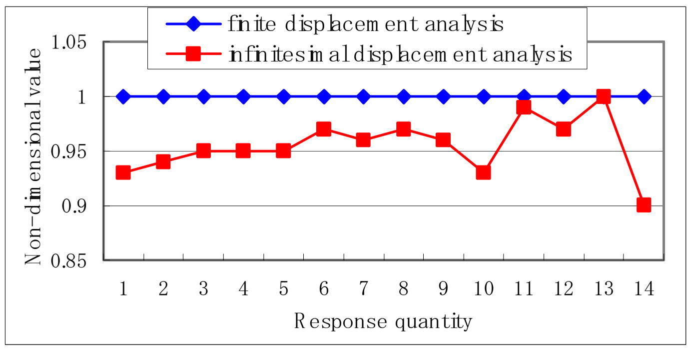


Fig. 8 Comparison between finite displacement analysis and infinitesimal one

*Comparison in case that two analytical conditions are different*

In the previous three sections, the case that only one analytical condition is different was discussed. However, it is a rare case in general. Then how the consideration of two different analytical conditions may influence on the dynamic response analysis results is discussed here. Non-dimensional response values calculated from dynamic response analysis where analytical conditions except both structural element and damping models are completely the same are shown in Fig. 9. In this figure, non-dimensional response value is calculated by dividing the response value obtained through Moment-Curvature and Rayleigh damping models by that obtained through Fiber and mass-proportional damping model. The horizontal axis represents the response quantities listed in remarks column in Table 1.

The fact that the results in case of Moment-Curvature and Rayleigh damping models generally become larger than those in case of Fiber and mass-proportional damping models and its ratio takes the value between 1.0 and 1.10. In other words, non-dimensional response values do not necessarily become so larger even if more than two analytical conditions are different in the dynamic response analysis.

Fig. 10 shows the ratio calculated by dividing the response value obtained through Moment-Curvature and finite displacement analysis models by that obtained through Fiber and infinitesimal displacement analysis models. From this figure, approximately the same fact as recognized from Fig. 9 is also found.

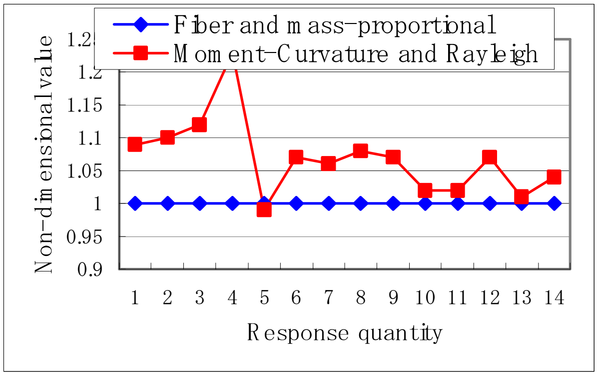


Fig. 9 Comparison of Fiber and mass-proportional damping models with Moment-Curvature and Rayleigh damping models

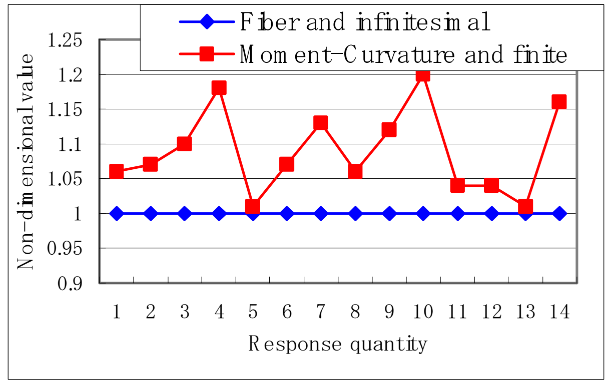


Fig. 10 Comparison of Fiber and infinitesimal models with Moment-Curvature and finite models

## PROPOSAL OF VALUE OF PARTIAL FACTOR $\gamma_A$

As discussed in the former chapter, it may be desirable to determine the value of partial factor $\gamma_A$ separately for dynamic analysis and static one. Furthermore, it is also desirable to determine it by considering what kind of model(s) is/are adopted in the analysis. Although it is a bit rough estimation, the following value is proposed as partial factor $\gamma_A$ for model uncertainties in earthquake proof design code for highway arch bridges.

1) $\gamma_A$ = 1.0 in case of static analysis

If the relatively low accurate analysis models such as Moment-Curvature model, infinitesimal displacement analysis model, and so on may be adopted in the design, $\gamma_A$ should be changed from 1.05 to 1.10 according to the accuracy of adopted analysis model.

2) $\gamma_A$ = 1.05 in case of dynamic analysis

If the relatively low accurate analysis models such as Moment-Curvature model, infinitesimal displacement analysis model, and so on may be adopted in the design, $\gamma_A$ should be changed from 1.15 to 1.20 according to the accuracy of adopted analysis model.

3) It is strongly recommended that at least two kinds of computer software may be applied to the dynamic response analysis of highway arch bridges under strong earthquake motions. At this stage, it is necessary to pay attention to not only maximum response value but also its occurrence time obtained from more than two kinds of computer software for dynamic response analysis. If the scatter of these results is small, the value of $\gamma_A$ can be reduced to 1.0 or 1.05.

## CONCLUDING REMARKS

Determination of the value of partial factor for model uncertainties prescribed in earthquake proof design code for highway arch bridges has been attempted. The value is determined by considering the scatter of maximum axial force of arch rib, maximum bending moment of stiffened girder, maximum horizontal displacement of stiffened girder, and so on obtained from dynamic response analysis. The result shows that the value 1.15 may be suitable to the partial factor in case that Moment-curvature model is adopted instead of Fiber model. It has been also revealed that the value of 1.10 may be appropriate to the case that mass-proportional damping is applied instead of Rayleigh damping or the case that geometric nonlinearity is not taken into account in dynamic response analysis of highway arch bridges. Finally, the following value of partial factor $\gamma_A$ is proposed according to the response analysis; that is, $\gamma_A$ = 1.0 in case of static analysis and $\gamma_A$ = 1.05 in case of dynamic analysis.


## ACKNOWLEDGEMENT

This study is supported financially by The Japan Iron and Steel Federation. And the author is grateful to Dr. Tsutomu USAMI, Prof. of Nagoya University, for his valuable academic suggestion to this paper. Cordial thanks are also extended to Mr. Tomoharu YAMAZAWA, former student of Department of Civil Engineering, University of Yamanashi, for his assistance for numerical calculations.


## REFERENCES


1. ISO/TC98/SC2: ISO 2394 General Principles on Reliability for Structures, June, 1998.
2. Japan Society of Civil Engineers; Standard Specification for Design and Construction of Concrete structures in Japan, 2002.
3. Japanese Society of Steel Construction: Guideline for Performance based Design of Steel Civil Engineering Structures, JSSC Technical Report No. 49, October, 2001.