A CONSIDERATION ON BI-AXIAL BENDING EFFECTS FOR CONTINUOUS CURVED BRIDGES

Naoyuki TAMEHIRO[1] And Atsushi MORI[2]

SUMMARY

The seismic investigations by means of three dimensional non-linear dynamic analysis have been performed on curved bridges. However, “Bi-axial bending effects” of bridge piers are rarely considered in these numerical investigations. These effects can make the conventional procedure of seismic investigations unreasonable. We have focused on a practical estimation method for these effects which had been proposed for the columns in high rise buildings[5] and have performed investigations and improvements on it in terms of adopting in bridge piers[6][7]. This paper describes the reasonability of this method and the characteristics of curved bridges’ behavior under earthquake excitation. One single column pier model and two curved bridge overall models were arranged, and a lot of dynamic analyses were carried out with them.

The results of almost every analysis case showed that it was necessary to pay attention to bi-axial bending effects in three dimensional non-linear analysis. The results of the single column pier model analyses showed that the method we have focused on could provide approximate response to the fiber model analysis recognized as a valid way to estimate these effects. The results of the curved bridge overall model analyses gave us some suggestions about the structural configuration of a curved bridge not to lead to complex seismic behavior and to reduce the influence of bi-axial bending effects in terms of design works. However, there are some problems left to be considered. The reports about the shaking table tests concerned with these effects have been expected.

INTRODUCTION

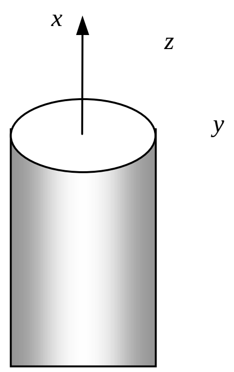
Curved bridges and bridges with eccentric piers are often required the seismic investigation by means of three dimensional non-linear dynamic analysis because of their unpredictable behavior under earthquake excitation. Recently, numerical estimation methods for “Bi-axial bending effects” of bridge piers have interested to researchers and bridge designers, and some new studies concerned with them have been reported in the past few years[2][4][6][7][8]. However, Only few attempts have so far been made to estimate these effects clearly on the seismic design stage of actual bridges.

At first, considering of the above, we thought we needed a practical estimation method of bi-axial bending effects. So we found out one method which was an expansion of conventional frame analysis and have studied on it in terms of its availability for bridge piers. Secondly, we thought it was very important to understand how bi-axial bending affects the seismic response of highway bridges both qualitatively and quantitatively. So we aimed at curved bridges by way of illustration and prepared three-dimensional frame analysis models idealized from the actual curved bridges which had different radiuses of curvature. We have carried out a lot of non-linear dynamic analyses with these models subjected to large base motion waves in various directions. In this paper we are concerned with the consideration on: (1) the necessity to estimate bi-axial bending effects in three dimensional non-linear analysis; (2) qualitative and quantitative interpretation of curved bridges’ behavior under earthquake excitation; (3) the reasonability of a practical estimation method we have focused on.

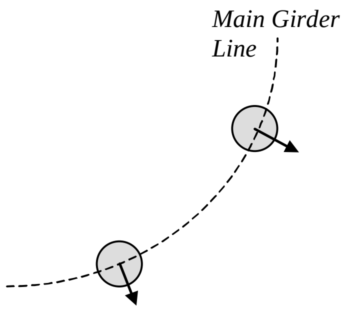
Bi-axial bending issues can be recognized with a rein-forced concrete column constructed with a circular cross-section as shown in Figure 1 which is common to the piers in curved bridges. Though such a column has an

[1] *Numerical Engineering Group, Kozo Keikaku Engineering, Inc., Japan, :Email: tamehiro@kke.co.jp*
[2] *Earthquake Disaster Mitigation Engineering Grp., Japan Engineering Consultants Co., Ltd., Japan, Email: moria@jecc.co.jp*

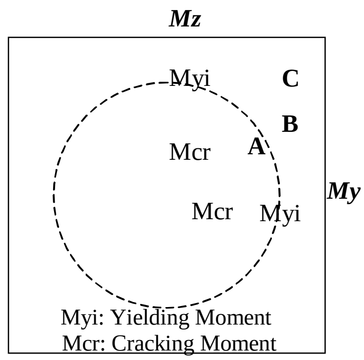
isotropic bending deformation ability in all horizontal directions, when a column is modeled with three-dimensional beam elements, this ability has to be represented by only two uni-axial bending deformation abilities about perpendicular local axes (y and z axis) which are often putted along the longitudinal and the transverse direction of the bridge it belong to as shown in Figure 1(b). Bending moments about two local axes of a conventional beam element are estimated separately and yielding judgement is performed respectively without the consideration of the interaction between them. When a bi-axial bending moment is seen as shown in Figure 1(c), if it is estimated in the direction it appears, it will be judged as yielding at point A. But in conventional frame analysis it won't be judged until one of the bending moment components (My or Mz) reaches yielding moment, that is to say point B or C. It is obvious that this judgement is too late and it is possible that the yielding time at each pier alters depending on the fashion of beam elements arrangement, and that another overall inelastic response is provided. This is a bi-axial bending issue of the columns which have circular cross-sections. It is needless to say that there is the same issue in the other shaped sections such as rectangular, too.



(a) Local Axes of a column



(b) Arrangement of columns



(c) Yielding Judgement

Figure 1: Bi-Axial Bending in Circular Section Columns

ESTIMATION METHODS

Numerical methods to estimate bi-axial bending effects can be categorized into two main groups. The first consists of approaches with stress-strain based methods, such as fiber model analysis and finite element analysis, in which yielding of materials starts from the part concentrated with stress in each critical cross-section. Therefore, they have already been recognized as the valid ways for the estimation of these effects, while it is also recognized that these methods are bothersome to treat for designers in practical design works. The second refers to the frame analysis approach employing the bi-axial interaction curves on the two dimensional section-force space (My-Mz plan) in each critical cross-section of beam elements to judge cracking and yielding with estimating bi-axial bending effects. This method was proposed and has been studied as the estimation method for these effects of the columns in high rise buildings[5][1]. We focused on it as a practical method and have performed improvements and investigations on it in terms of adopting in bridge piers[6][7] with a non-linear frame analysis program named RESP-T which one of the authors have been involved in the development. What has to be noticed about this method is that the input procedure to the program and the assessment procedure of output from the program are almost the same as conventional frame analysis.

According to this method, two closed curves supposed tri-linear envelope (interaction curves for cracking and yielding criteria) as shown in Figure 1(c) have to be established in each critical section of a three-dimensional beam element. These curves are provided by the following equation:

$$\left(\frac{my}{Mcy}\right)^a + \left(\frac{mz}{Mcz}\right)^a = 1.0 \qquad \text{(1)}$$

where Mcy is the criterion (cracking or yielding) moment about local y axis; Mcz is the criterion moment about local z axis; and (my,mz) is the section-force state, in the other words bending moments about two perpendicular local axes at a time step. In accordance with the locative relationship between these curves and the section-force state at each time step, yielding status is judged. When the section-force state is within the cracking curve the element behaves elastically, and when it reaches the cracking or yielding curve plastic deformations occur.

Depending on the judgement the element stiffness is modified. After cracking or yielding, accompanying with the progression of the section-force state, each interaction curve is modified by translating or expanding or both of them according to the hardening rules established in classical plasticity theory. This is a procedure to trace hysteretic rules estimating bi-axial bending effects.

ANALYTICAL MODELS

One single column pier model as shown in Figure 2 and two curved bridge overall models as shown in Figure 3 were arranged. Single column pier model was idealized from a simple and standard R/C pier. The column with a rectangular cross-section was modeled with non-linear three dimensional beam elements. The square pile foundation with soil was modeled as rigid beam and linear rocking-sway springs. The rubber bearing shoe which is fixed in the transverse direction was modeled as a linear shear spring and a rigid spring. The beam elements as the pier column were established uni-axial moment-curvature relationships about two local axes with tri-linear envelope according to [3]. A fiber model (60×30 confined concrete fibers, 3 layers of unconfined concrete fibers and main steel bar fibers on a section) was also prepared to compare with the framed model.

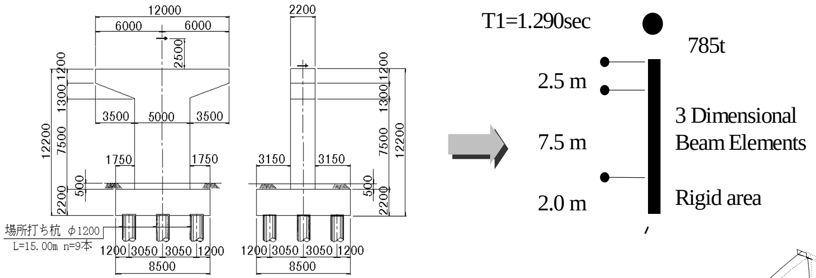


Figure 2: Single Column Pier Model

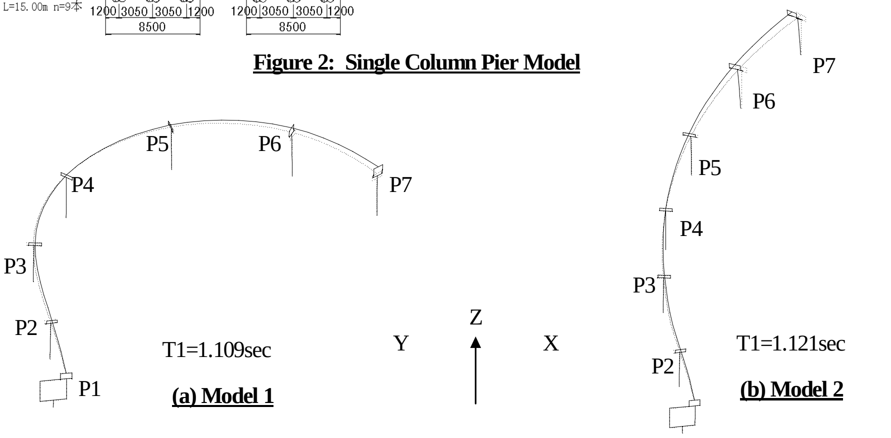


Figure 3: Curved Bridge Models and the First Vibration Modes

Turning now to the curved bridge models. Curved bridge Model 1 as shown in Figure 3(a) was modeled from an actual concrete bridge with a length of 220m. The part from P1 to P3 was nearly straight and the other part P3-P7 with a length of 150m was the curved part which had a radius of curvature with a length of 60m. Curved bridge Model 2 as shown in Figure 3(b) was the same as Model 1 excepted the radius of curvature of the curved part modified into 200m length. In both of the two models all piers(R/C) had circular cross-sections and isotropic rubber bearings on the tops of them. The piers were modeled as three dimensional beam elements established uni-axial moment-curvature relationships about two local axes with tri-linear envelope according to [3]. The local axes of beam elements were putted in the longitudinal and the transverse direction at each pier's arranging point in the bridge they belonged to. The continuous girder was modeled as linear beam elements including the estimation of torsional stiffness. The pile and caisson foundations with soil were modeled as rigid beams and linear rocking-sway springs. The rubber bearings were modeled as linear shear springs.

CONDITIONS OF DYNAMIC ANALYSES

The first is the common condition in all cases: (1) Newmark-β method was used to integrate the equation of motion with $\beta = 0.25$, $\Delta t = 0.002\,sec$; (2) '1995 OGAS FUKIAI N27W (the maximum acceleration is 736.3cm/sec^2)' which is one of the standard base waves for design of highway bridges was used as horizontal input base wave; (3) Rayleigh damping with the damping ratios of the appropriate vibration modes of each model was used; (4) Degrading tri-linear hysteretic model was used on the moment-curvature relationships.

The analysis conditions for single column pier model were given as below: (1) The input base wave was used in the direction of 45 degrees from the transverse direction corresponded to the global X axis; (2) Three main conditions were given: frame analysis without estimating bi-axial bending effects; frame analysis with estimating them by the method as mentioned earlier; and the fiber model analysis; (3) The interaction curves had been established as ellipses (a=2.0 in Eq.1) in the past studies[5][1]. However, it had been seen from our pre-investigation with static analysis that the curves on the condition of a=3.0 (Figure 4) provided more approximate results to the fiber model analysis than ellipses for the rectangular section[7][8]. Though it was a debatable point[4], Eq.1 with a=3.0 was used on single column pier model.

The analysis conditions for curved bridge models were given as below: (1) Two main conditions were given: not estimating bi-axial bending effects (henceforth to be referred to as Type A) and estimating them with the method mentioned earlier (Type B); (2) The interaction curves of all piers were established as circles (a=2.0 and Mcy=Mcz in Eq.1), because all of the piers in these bridges had designed with circular cross-sections. We thought it left no room for doubt; (3) The input base wave was used in six directions as shown in Figure 5. Dir 3 (53 degrees from global X axis) was almost corresponded to the direction of the straight part (P1-P3) of each model. Consequently, 24 dynamic analyses were carried out (2 models, 2 types, and 6 directions).

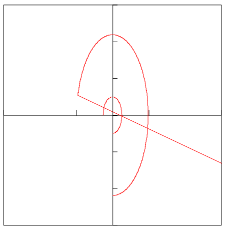


Figure 4: Interaction Curves of a Rectangular Section

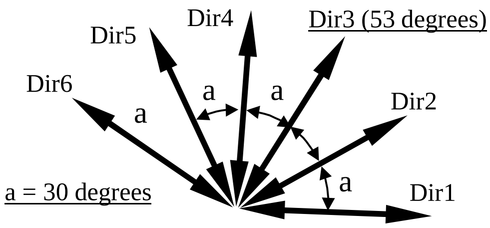


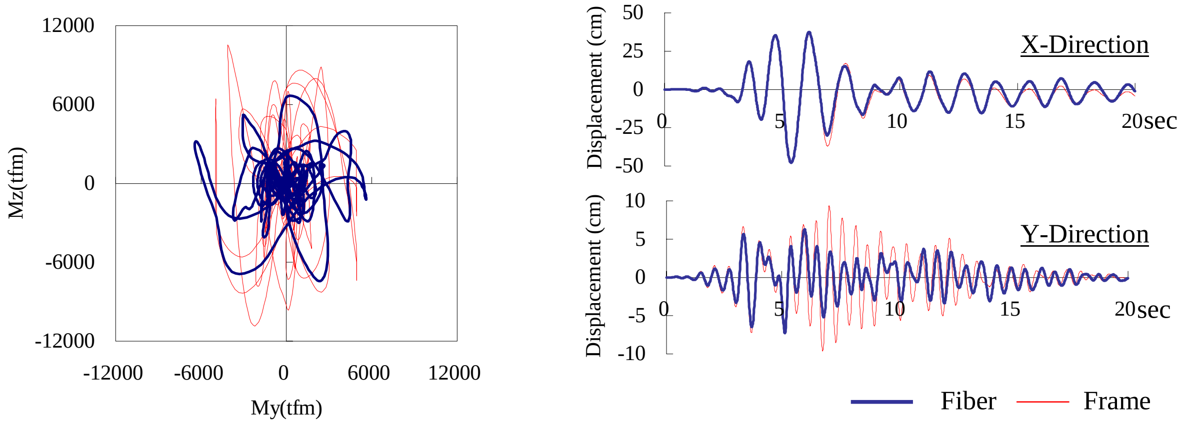
Figure 5: Directions of the Input Base Wave

RESULTS OF ANALYSES

Single Column Pier Model

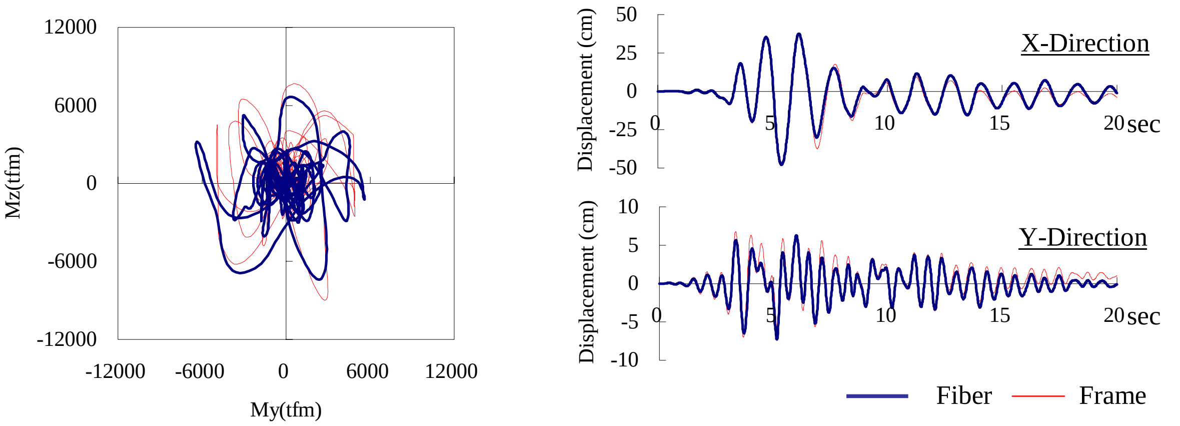
Figure 6 shows the results of the dynamic analysis with single column pier model on condition that the yielding judgements of bending moments about two local axes were performed separately without the consideration of bi-axial bending effects, comparing with the results of the fiber model analysis. The first point to notice was the complexity of the My-Mz response curves on the lower end of the pier’s column. It showed that a pier could behave complexly in spite of its structural simplicity when the direction of base wave wasn’t corresponded to any local axis of its cross-section. Figure 6 also indicates that there was the significant qualitative difference on seismic response between the fiber model analysis which was recognized as a valid way for the estimation of bi-axial bending effects and the frame analysis which did not estimate them. It was a significant issue that how complex a pier would behave and what kind of differences would appear between estimating and not estimating them had been unpredictable. Meanwhile Figure 7 shows the results of the frame analysis with the estimation of these effects by the practical method mentioned earlier, comparing with the results of the fiber model analysis. They indicated that this method could provide the approximate response to the fiber model analysis.

Though this method’s potential could be seen, it should be noted that there has been some issues left to be considered. First, the studies and the debating haven’t been done enough about how bi-axial interaction curves are established for various shaped cross-sections. Secondly, it hasn’t been explained why the difference appeared on the moment-curvature response envelope between the fiber model analysis and the frame analysis as shown in Figure 7(c). The bending moment about local y axis with the fiber model analysis increased in spite of after yielding. So far in this paper, fiber model approach has been regarded as a valid way, but there might be the necessity to confirm the validity of it in terms of dynamic response. Some papers have reported the adaptability of fiber model analysis in tracing of cyclic loading experiments[2], but the adaptability in tracing of shaking table tests concerned with bi-axial bending effects have never been reported as far as we know.

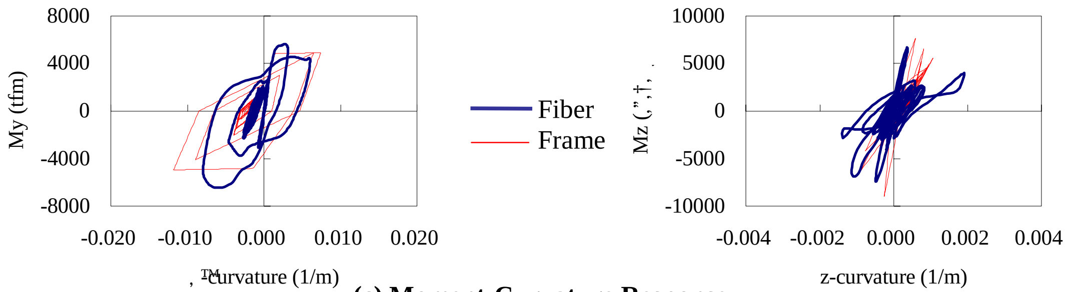


(a) My-Mz Response in the Base of the Pier **(b) Displacement Time History of the Superstructure**

Figure 6: Frame Analysis (not estimating B.A.B.E.) and Fiber Model Analysis



(a) My-Mz Response in the Base of the Pier **(b) Displacement Time History of the Superstructure**

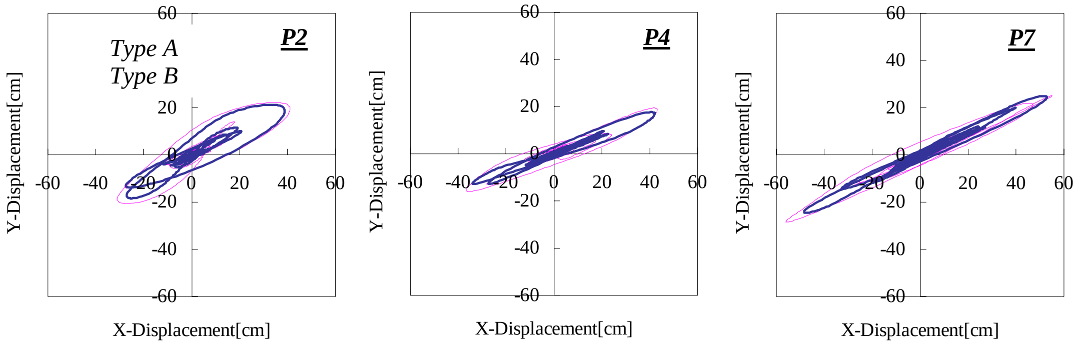


(c) Moment-Curvature Response

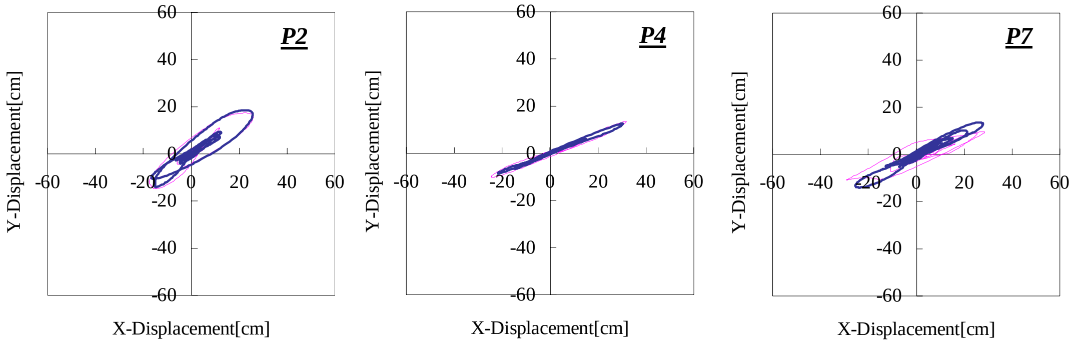
Figure 7: Frame Analysis (estimating B.A.B.E.) and Fiber Model Analysis

Curved bridge models

What should be focused on first was the observation that both the piers and the girders in curved bridge models had a tendency to draw regular displacement orbits which were almost along the direction of the input base wave as shown in Figure 8. This tendency could be seen in all analysis cases with curved bridge models. It was thought that the structural configurations composed of the isotropic elements such as piers with a circular section, square pile foundations and isotropic rubber bearings led to it. This fact suggested two things. For one thing, a structural configuration as above was very effective not to lead complex seismic behavior as shown in single column pier model. What was more, so far as such models were concerned, almost reasonable seismic response could be provided by arranging piers on condition that one of the local axes of each column was corresponded to the direction of input base wave, instead of employing any special method to estimate bi-axial bending effects. This new condition was established as Type C and 12 additional dynamic analyses (2 models × 6 directions) were carried out.



(a) Girder at just above Each Pier



(b) Top of Each Pier

Figure 8: Displacement Orbits of *Model1-TypeA-Dir2* and *Model1-TypeB-Dir2*

Table 1: Yielding Progressions (*Model 1*)

Model 1	Type	1st	2nd	3rd	4th	5th	6th
Dir 1	A	P5 (3.414)	P4 (3.440)	P2 (3.452)	P6 (3.520)	P3 (3.520)	P7 (3.540)
	B	P4 (3.406)	P5 (3.414)	P2 (3.430)	P3 (3.438)	P6 (3.460)	P7 (3.540)
	C	P5 (3.416)	P4 (3.418)	P2 (3.440)	P3 (3.454)	P6 (3.474)	P7 (3.540)
Dir 2	A	P4 (3.416)	P5 (3.456)	P6 (3.474)	P3 (3.480)	P2 (3.514)	P7 (4.000)
	B	P5 (3.410)	P4 (3.416)	P2 (3.434)	P3 (3.460)	P6 (3.476)	P7 (3.518)
	C	P4 (3.416)	P5 (3.422)	P2 (3.450)	P3 (3.468)	P6 (3.476)	P7 (3.538)
Dir 3	A	P4 (3.442)	P5 (3.450)	P2 (3.460)	P3 (3.472)	P6 (3.506)	P7 (4.432)
	B	P4 (3.404)	P5 (3.416)	P3 (3.456)	P6 (3.458)	P2 (3.460)	P7 (3.546)
	C	P4 (3.418)	P5 (3.428)	P2 (3.460)	P3 (3.466)	P6 (3.474)	P7 (3.996)
Dir 4	A	P5 (3.428)	P4 (3.446)	P2 (3.480)	P6 (3.504)	P2 (3.538)	P7 (3.984)
	B	P4 (3.406)	P5 (3.428)	P3 (3.434)	P2 (3.444)	P6 (3.454)	P7 (3.992)
	C	P4 (3.420)	P5 (3.428)	P3 (3.450)	P2 (3.458)	P6 (3.468)	P7 (3.994)
Dir 5	A	P4 (3.420)	P3 (3.448)	P6 (3.464)	P5 (3.466)	P2 (3.476)	P7 (4.032)
	B	P5 (3.408)	P4 (3.420)	P2 (3.434)	P3 (3.436)	P6 (3.466)	P7 (3.940)
	C	P5 (3.422)	P4 (3.422)	P3 (3.448)	P2 (3.448)	P6 (3.464)	P7 (3.968)
Dir 6	A	P5 (3.432)	P2 (3.440)	P3 (3.446)	P4 (3.448)	P6 (3.506)	P7 (3.986)
	B	P5 (3.406)	P4 (3.406)	P2 (3.440)	P3 (3.446)	P6 (3.454)	P7 (3.918)
	C	P5 (3.416)	P4 (3.420)	P2 (3.442)	P3 (3.444)	P6 (3.470)	P7 (3.944)

Pier's No. (the first Yielding time: sec)

Secondly, Some comparisons in Type A, B and C have to be focused on. Table 1 shows the yielding progressions of the piers and the yielding times of each pier of all analysis cases with Model 1. The similar yielding progressions could be seen in Type B and C analyses excepted some events which were close in time, while the yielding progressions of analysis cases on Type A which was the only condition not paying attention to bi-axial bending effects showed different tendency from the other types. The fastest yielding progressions were seen in the analyses on Type B because yielding judgement had been performed in the direction that bending moment had appeared actually. The slowest progressions were seen in Type A analyses because of the procedure of yielding judgement which could be too late as mentioned in chapter 1. The same observation applied to the comparisons in the maximum accelerations of the girder at the points just above each pier and in the maximum displacements of the top of each pier as shown in Figure 9. Type A analyses showed different tendency from Type B and C in this comparison, too. Especially, the maximum accelerations of Type A analyses had tendency to be higher than Type B and C. One explanation for it was that the difference in the yielding progressions altered the energy-absorbing capacity of each model

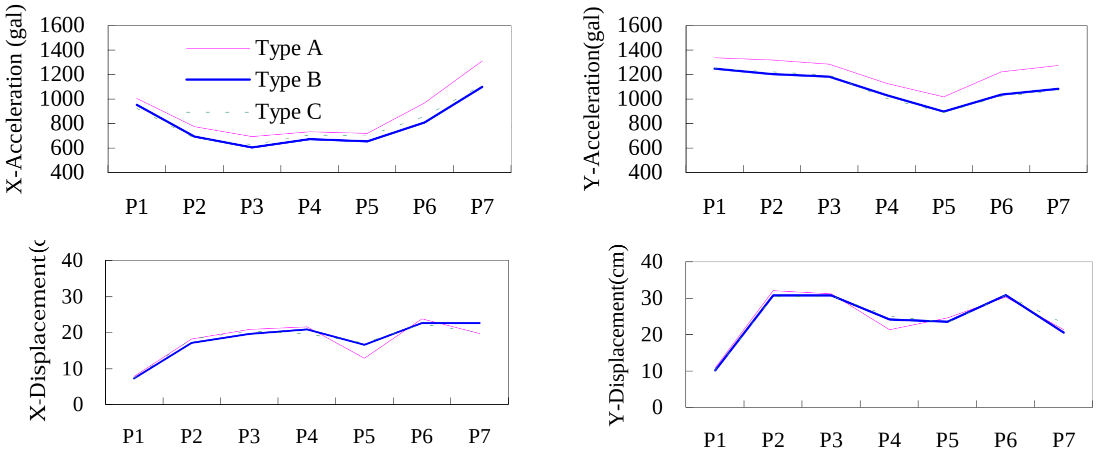


Figure 9: Maximum Response (*Model 1-Dir 3*)

Then, the ductility factors (with curvature) of each pier were figured out as the yielding achievement indicators.

Type A … $\mu A^{*} = \dfrac{\textit{the maximum curvature}}{\textit{yielding curvature}}$. *about each local axis of a section.

Type B … $\mu B1^{*} = \dfrac{\textit{the maximum curvature}}{\textit{yielding curvature(from the first yielding point)}}$, to compare with Type A.

$\mu B2 = \dfrac{OM}{OY}$, regarding the curvature orbits as nearly straight (Figure 10), to compare with Type C.

Type C … $\mu C = \dfrac{\textit{the maximum curvature}}{\textit{yielding curvature}}$, about one local axis which is valuable.

Table 2 shows the ductility factors of the typical analysis case. Though there wasn't any significant difference in the absolute values of the maximum curvatures, the ductility factors on Type B which provided the fastest yielding progressions were higher than the other conditions.

Table 2: Ductility factors on the Lower End of *P5's column* in *Model 1-Dir 3*

Type	Local Axis	The Maximum Carvature [1/m]	The Yielding Carvature [1/m]	Ductility Factor
A	y	0.003608	0.001204	2.996
	z	0.009951	0.001204	8.265
B	y	0.004796	0.000377	12.715
	z	0.010642	0.000725	14.678
	Bi	0.011673	0.000817	14.282
C	z	0.010917	0.001204	9.068

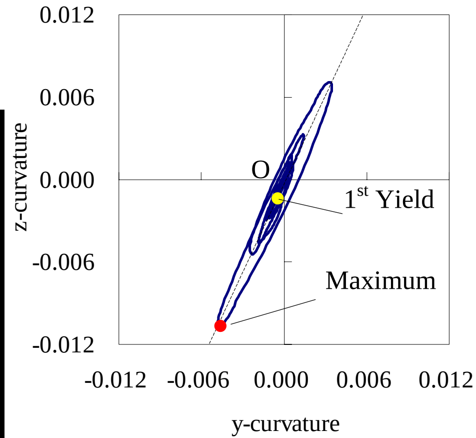
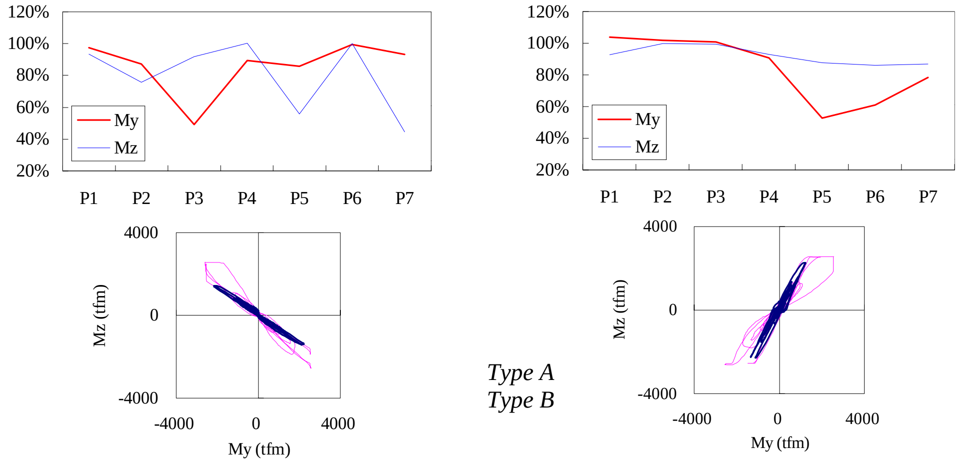


Figure 10: A Curvature Response

Figure 11 shows the percentages of each pier's maximum bending moments of Type B to Type A. The piers which weren't affected by bi-axial bending were given nearly 100% about both of two local axes such as P6 in Model1-Dir3. Meanwhile in the piers which were affected by it significantly these percentages were reduced to about 50% such as P5 in Model1-Dir3. It was because of the unreasonable yielding judgements procedure on Type A including the delay of the judgement as mentioned earlier and the time lag between two uni-axial yielding judgements in a section which could be seen as unnatural figures on My-Mz response curve (* in Figure11). It should also be added that affected and not affected piers were seen alternately in Model 1 which had a shorter radius of curvature, because they had been arranged putting their local axes in the various directions along the bridge's sharp curve. This inclination couldn't be seen in Model 2 which had a gentle curve.



<u>Figure 11: Percentages of the Maximum Moments of *Type B* to *Type A* and My-Mz Response</u>

CONCLUSIONS

It should be recognized that a three dimensional dynamic analysis without paying attention to bi-axial bending effects might not be the most severe seismic investigation. The results of the dynamic analysis of a curved bridge with circular section piers alter depending on the local axes' directions of each pier.

A bridge composed of isotropic elements provides simple seismic behavior whether it is a single column pier model or a curved bridge overall model. And such a structure can be hard to be affected by bi-axial bending in terms of bridge design works as shown in our results which didn't show significant difference in deformations and curvatures between estimating and not estimating them.

The practical estimation method for these effects we focused on could provide the approximate response to the fiber model analysis and showed some qualitative agreements. However, its reasonability hasn't been confirmed enough yet. Neither has fiber model analysis, we think. The comparison with the shaking table tests concerned with these effects will prove their reasonability or suggest some additional improvements.

Finally, We are indebted to Mr.Yoshihide Okimi (Kajima Corporation) for providing us the results of the fiber model analysis.

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