NON-LINEAR EARTHQUAKE RESPONSE ANALYSIS OF PC CABLE STAYED BRIDGE CONSIDERING THE FLUCTUANT AXIAL FORCE

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SUMMARY

In this study, nonlinear earthquake response analysis of the PC cable stayed bridge is carried out using three-dimensional model. Acceleration waves recorded at the Kobe city during the Hyogoken-Nanbu earthquake are used. The nonlinearity of members is prescribed by the relationship between bending moment and curvature. Skeleton of moment-curvature relationship is modeled as tri-linear with breaks at crack and yield points. In most analyses, crack and yield points of skeleton curve are assumed constant. However, when fluctuant axial force is considered, these points vary due to the interrelationship between bending moment and axial force. Crack and yield points increase under large compressive force, but tends to decrease under compressive force less than dead load, or under tensile force. This paper reports the effect of fluctuant axial force to the non-linear earthquake response analysis. In the usual analysis, where fluctuation of axial force is not regarded, Takeda hysteresis model is used to represent the moment-curvature relationship of member. On the other hand, when fluctuant axial force is considered, Edo model is applied. Comparison of these two analyses reveals that remarkable difference of responses occurred when earthquake acts in out-of-plane direction. Numerical analytic results imply the necessity of considering of fluctuant axial force in non-linear analysis.

INTRODUCTION

Minamitabaru Ichi-Gou bridge is a floating PC cable stayed bridge with length of 292.1 m and width of 14.9 m. This was constructed as highway bridge in 1993, designed according to Specifications for Highway Bridges, 1990 edition. After Hyogoken-Nanbu earthquake (January 17, 1995, M=7.2), important bridges that were designed before 1995 must be investigated according to their seismic performance. Nonlinear earthquake response analysis about several cable stayed bridges have been reported. However, to establish the rational seismic design of a cable stayed bridge, it is important to gather various analytical results of cabled stayed bridges. This paper aims to clarify the seismic response characteristics of floating type cable stayed bridge.

Nonlinear analysis was performed with three dimensional FEM model using the observed seismic wave in Hyogoken-Nanbu earthquake. All members of towers and girder has a nonlinearlity that is prescribed by the relationship between bending moment and curvature. In most of past nonlinear earthquake response analysis of bridges, this relationship was obtained by dead load. However, break points in the relationship between bending moment and curvature will be changed by response axial force. This paper also reports the effect of fluctuant axial force to the non-linear earthquake response analysis. In this study, it is assumed that nonlinearity is affected by fluctuant axial force, i.e. crack and yield points of member vary due to the interrelationship between bending moment and axial force. Numerical results with fluctuant axial force is different from the results that is not considered fluctuant axial force. It is confirmed that the fluctuant axial force must be considered in seismic analysis to design the bridge more safely.

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ANALYTICAL MODEL

The bridge treated in this study is Minamitabaru Ichi-Gou bridge, that was constructed as highway bridge in Japan, 1993. Fig. 1 shows the sideview of Minamitabaru Ichi-Gou bridge, this bridge is a floating PC cable stayed bridge with length of 292.1 m and width of 14.9 m, there are two towers 64.5m high. Cross section of girder is separated two box and foundations are pile foundation at P1 and spread foundation at P2. In the analysis, three dimensional FEM model was used as shown in Fig. 1. Boundary conditions of this model are fix at foot of piers, simple support at the ends of girder and the girder is not connected to towers. Initial tensile force of cables and pre-stress force of girder is considered in analysis. Oscillation test was performed when the bridge was just constructed [Uno, 1995]. Natural frequencies obtained from oscillation test and the results of eigen value analysis are shown in Table 1. It is shown that test results and analytical results are cross agreement. Therefore, the efficiency of FEM model used in this study is confirmed.

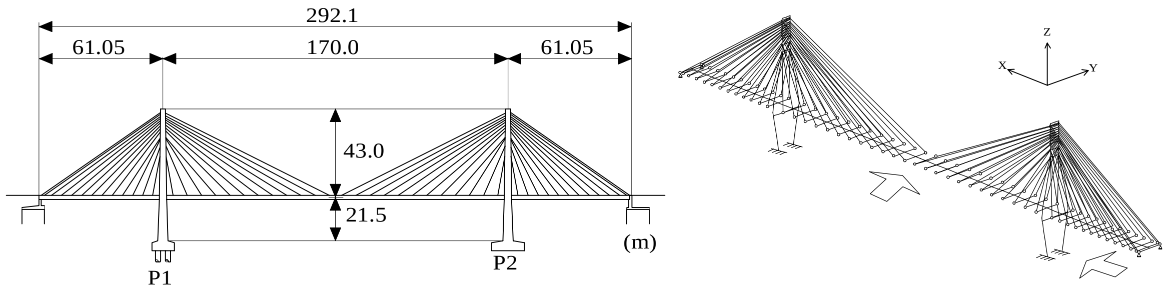


Figure 1 Side view and FEM model of Minamitabaru Ichi-Gou bridge

Table 1 Natural frequencies and effective mass ratio

		1	2	3	4	5	6	7	8	9	10
Test (Hz)		-	0.39	0.65	0.95	1.25	1.49	-	1.64	1.57	1.79
Analysis (Hz)		0.26	0.37	0.87	1.14	1.29	1.42	1.51	1.52	1.54	1.67
Effective mass ratio (%)	X	70	0	0	3	0	0	0	0	0	0
	Y	0	42	0	0	0	0	21	0	11	0
	Z	0	0	9	0	0	17	0	0	0	25

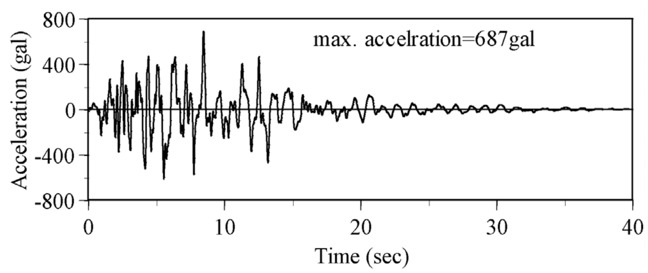


Figure 2 Time history of inputted seismic wave

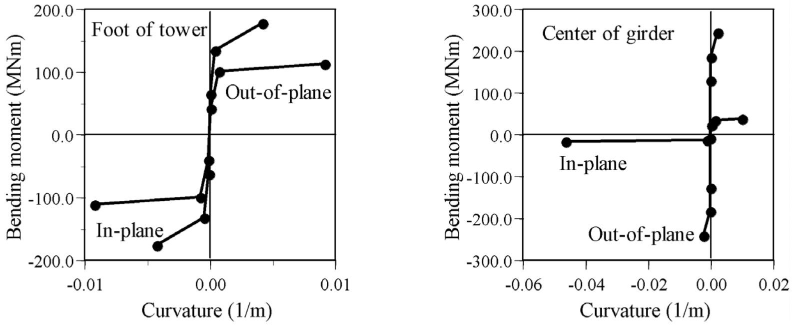


Figure 3 Relationship between bending moment and curvature

Earthquake response analysis was carried out with the N-S component of seismic wave that was observed at Takatori station of West Japan Railway in Hyogoken-Nanbu earthquake. The maximum acceleration is 687gal and recorded time is 40sec with sampling time 0.01sec as shown in Fig. 2. In the analysis, considering the soil condition of the bridge, maximum acceleration was reduced to 0.85 times in accordance with the Japan Highway Specification. The nonlinearity of members is prescribed by the relationship between bending moment and curvature. Skeleton of moment-curvature relationship is modeled as tri-linear with breaks at crack and yield points. For example, skeletons at the center of girder and foot of tower under the initial axial force are shown in Fig. 3. The nonlinearity depends on bending direction, skeletons of in-plane bending and out-of-plane bending are respectively shown in Fig. 3. When fluctuation of axial force is considered, break points of skeleton is re-calculated by relationship between axial force and bending moment.
Takeda hysteresis model is used to represent the moment-curvature relationship of member when fluctuant axial force is not considered. On the other hand, when fluctuant axial force is considered, Edo model is applied. Earthquake responses were computed using Newmark's β method (β =0.25), of time interval is 0.01sec. The damping coefficients of members of girder and towers are set to 0.03.

ANALYTICAL RESULTS

CASE 1:The case of fluctuant axial force is not considered

Time histories of horizontal response displacements at the center of girder and that of top of the tower under in-plane seismic motion are shown in Fig. 4. Figure 5 shows same figures as Fig.4 when seismic wave inputted to out-of-plane direction. In these figures, fluctuant axial force is not considered. Under in-plane oscillation, both of time history of response displacements in Fig. 4 are almost the same. Since this bridge is a floating type, the tower will be oscillated by in-plane vibration of girder. The maximum response displacement of top of the tower is 71.8cm and that of girder is 73.4cm. On the other hand, response characteristics under out-of-plane oscillation is different from that of in-plane oscillation. Girder is vibrated with long period because it is not connected to the tower. Vibration period of tower is shorter than girder. Stiffness of tower along out-of-plane

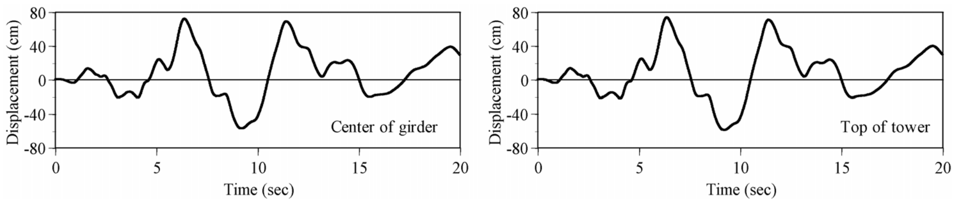


Figure 4 Response displacement under in-plane oscillation

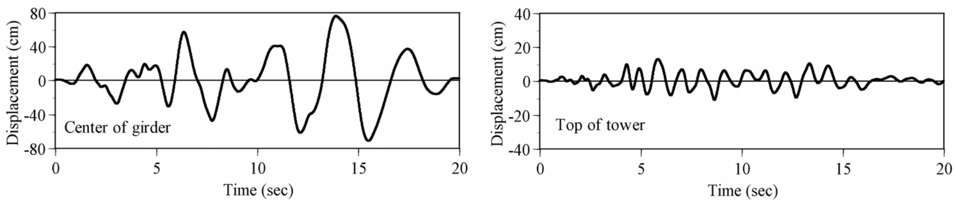


Figure 5 Response displacement under out-of-plane oscillation

motion is greater than that of in-plane motion due to the shape of tower. In this case, maximum response displacement is 75.6cm at girder and 12.7cm at the top of the tower.

Figure 6 shows the distribution of maximum response displacement at each node points of girder and Fig. 7 indicate that of tower. Figure 6 presents the vertical displacement of girder. It is clear from Fig. 6 that large vertical displacement is created with in-plane oscillation. Distributions of displacement of tower are different at the upper part. Cables are connected to tower at this area, thus due to the effects of cables and girder, displacement along out-of-plane oscillation are reduced. Time history of response axial force at the foot of tower is indicated in Fig. 8. The variation of axial force under in-plane oscillation is 2MN from initial axial force. However, it shows a large variation of axial force with time when seismic wave is inputted to out-of-plane direction. The variation in this case is 20MN from initial value. Towers of this bridge consists of two columns which line up in out-of-plane direction, large variation of axial force will be caused by assignment of response axial force mutually under the out-of-plane oscillation. The relationship between bending moment and curvature, which describe the nonlinearity of member, depends on axial force level. These results indicate the necessity of considering the fluctuant axial force in nonlinear seismic response analysis.

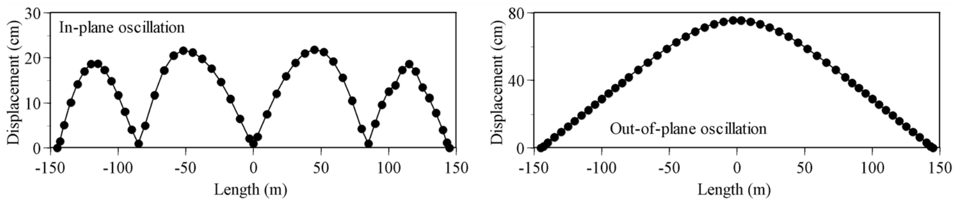


Figure 6 Distribution of maximum response displacement of girder

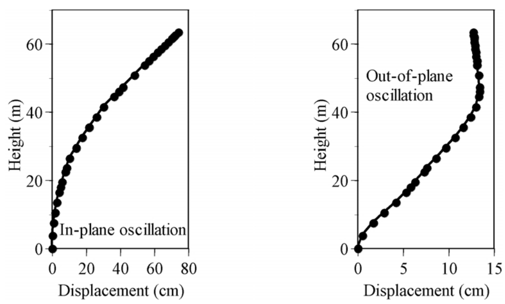


Figure 7 Distribution of maximum response displacement of tower

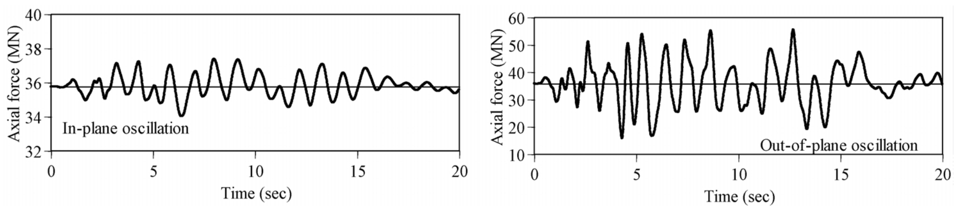
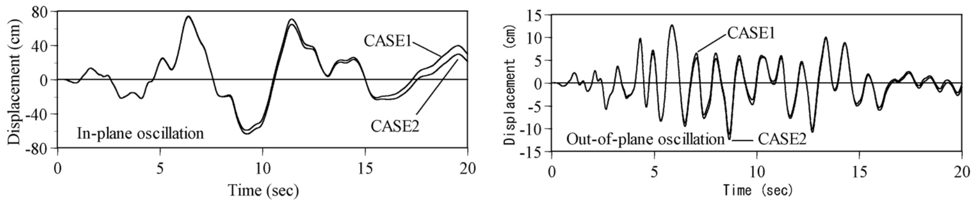


Figure 8 Time history of response axial force

CASE 2:The case of fluctuant axial force is considered

In this case, nonlinearity of members of tower is recalculated in considering with fluctuant axial force. However, nonlinearity of members of girder is not considered with fluctuant axial force, because variation of response axial force in girder is small. Time histories of response displacement at the top of the tower are shown in Fig. 9 compared with the result of CASE 1. Response displacement of this case indicate almost same tendency with that of CASE 1 in both exciting direction. Figure 10 indicates time history of response bending moment at the foot of pier. When oscillated in in-plane direction maximum response bending moment of foot of pier is



107.0MNm in both cases. On the other hand, considering fluctuant axial force ,maximum value of bending

Figure 9 Comparison of response displacements at the top of tower

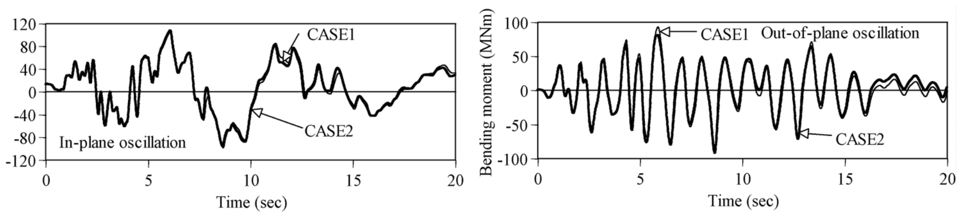


Figure 10 Comparison of bending moments at the foot of tower

moment under out-of-plane oscillation is 91.8MNm. This value is smaller than that of CASE 1 where the maximum value is 93.2MNm.

The relationship between response bending moment with curvature under in-plane oscillation is shown in Fig.11 and that of out-of-plane oscillation in Fig. 12. When fluctuant axial force is not considered (CASE1), the skeleton curve is not change from initial shape, bending moment-curvature hysteresis are symmetrical regardless of fluctuant axial force in both oscillated direction. Results of CASE2 under in-plane oscillation shows almost the same tendency that of CASE1 because variation of axial force is small as shown in Fig. 8. The difference in hysteresis pass of in-plane oscillation is effected by the hysterisis model. Bending moment-curvature hysteresis of CASE2 for out-of-plane excitation is asymmetrical. This asymmetrical phenomina is effected by widely varied axial force. When considering the fluctuant axial force, bending strength is increased with increasing of axial force and decrease of axial force is caused low bending strength.

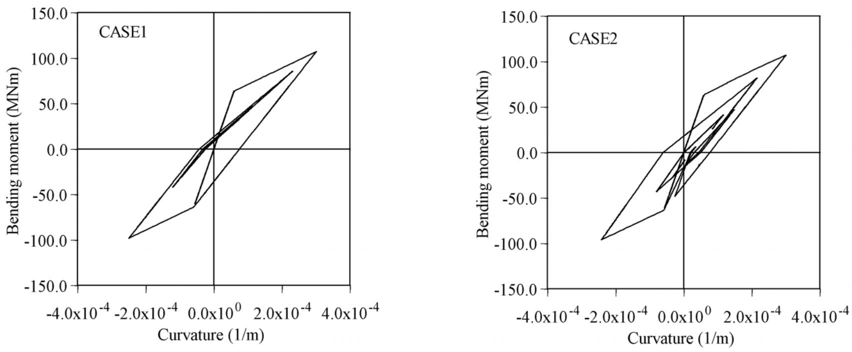


Figure 11 Relationship between bending moment and curvature under in-plane oscillation

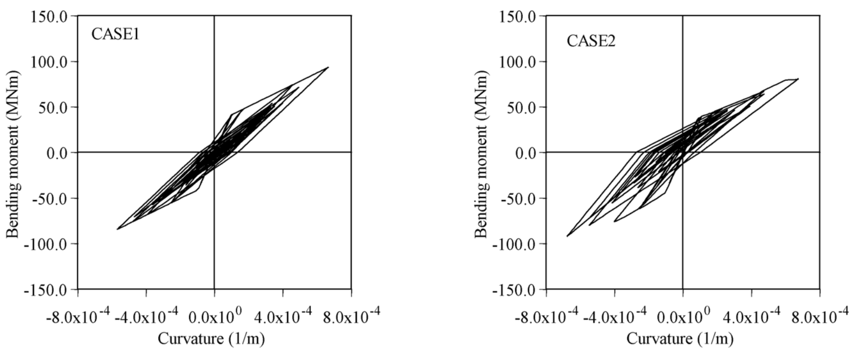


Figure 12 Relationship between bending moment and curvature under out-of-plane oscillation

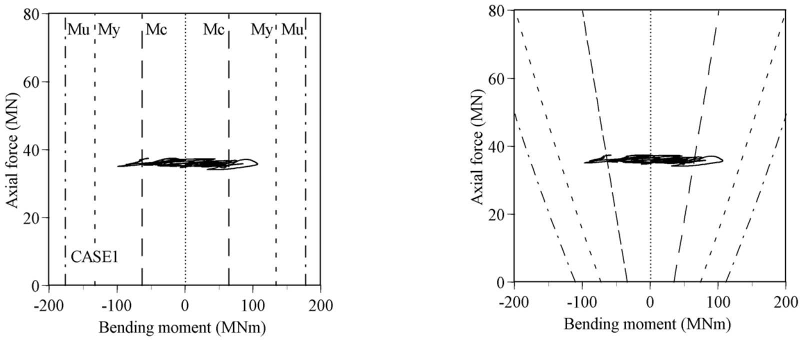


Figure 13 Relationship between bending moment and axial force under in-plane oscillation

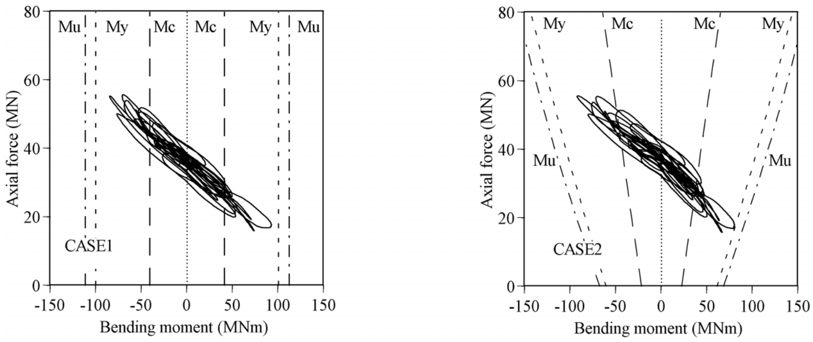


Figure 14 Relationship between bending moment and axial force under out-of-plane oscillation

Figures13 and 14 indicate the relationship between response bending moment and axial force of each exciting direction. Figure 13 shows low fluctuation of axial force under in-plane seismic motion. Response bending moments over the crack moments, however it does not reach to yield moment in both cases. In this oscillation, seismic response is not effected by fluctuant axial force. On the other hand, strength of tower is effected by fluctuation of axial force due to the axial force varied to about 40MN. When considering the fluctuant axial force, crack moment Mc=53MNm, yield moment My=123MNm and ultimate moment Mu=137MNm under axial force is 60MN. Axial force is reduced to 20MN, these moments are reduced to Mc=32MNm, My=83MNm and Mu=93MNm. From this reduction of strength, bending moment reached yield moment when the fluctuant axial force is considered in seismic analysis.

CONCLUSIONS

1. Seismic response behavior of floating type PC cable stayed bridge is effected by direction of seismic motion. Under in-plane oscillation, response motion of towers are dominated by the motion of girder.
2. The bridge which was analyzed in this paper has a tower that consists of two columns. Remarkable fluctuant axial force occurred when oscillated in out -of-plane direction. The fluctuation of axial force is 2MN under in-plane oscillation and 20MN under out-of-plane excitation.
3. Response displacements of tower is not related with fluctuant axial force. Thus time histories are almost the same. Response bending moment at the foot of tower is slightly reduced when fluctuant axial force is considered.
4. Along in-plane direction, relationship of bending moment and curvature is the same irrespective to fluction of axial force. The same tendency for relationship of axial force and bending moment.
5. When considering the fluctuant axial force, strength of towers are decreased due to value of response axial force. Response bending moment of some menders reached yield level under out-of-plane oscillation.

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