NONLINEAR DYNAMIC BEHAVIOR AND SEISMIC ISOLATION OF STEEL TOWERS OF CABLE-STAYED BRIDGES UNDER GREAT EARTHQUAKE GROUND MOTION

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SUMMARY

The nonlinear behavior of steel towers of cable-stayed bridges subjected to three-dimensional great earthquake ground motions is studied. Nonlinearities due to geometry changes and material sources of bridge tower elements are considered, and a tangent stiffness iterative procedure is used in the dynamic analysis to obtain the nonlinear seismic response. Numerical examples for three different tower shapes(A-type, H-type and gate-type models) are presented. The effects of various tower models and design parameters of the cable on the dynamic behavior of cable-stayed bridges are investigated. It is shown that the seismic performance of the steel towers with passive control devices is effective to reduce the reaction forces at the tower basements. The adoption of passive control systems can accomplish a significant reduction in seismic-induced forces, as compared to the case with no passive control device added

INTRODUCTION

The Hyogoken-Nanbu Earthquake which occurred on January 1995 in Japan damaged the bridge structures enormously. Then it is compelled to consider a road bridge seismic design method again in Japan. Part V Earthquake Resistant Design was accomplished in December, 1996[Japan Road Association, 1996]. In the present, it has been reconsidered that ductility design method and the dynamic analysis method apply the short span viaducts. So the study on analytical dynamic methods is carried on in many fields[Japan Society of Civil Engineers, 1996, 1997 and 1998]. A cable-stayed bridge used in this study is composed of stiffening girder, cable and tower, it presents a very complicated vibration as the span increases. Especially, it is necessary to understand nonlinear response of the cable-stayed bridge under the great earthquake ground motions correctly and to improve the seismic performance.

The support conditions of stiffening girder of cable-stayed bridge adopt all free type bearings, which is rubber bearing and roller bearing, on all piers in the direction of the bridge axis. That is why, it is expected that the natural periods of the cable-stayed bridge are to be longer and the inertia forces acting on the tower can be reduced. Recently, a fixed bearing is used in the direction of the right angle to the bridge axis. So even if the elasticity bearing is adopted in the direction of the right angle to the bridge axis, it is difficult to extend the natural period and to design the tower for the great earthquake ground motions. The part of the steel tower of cable-stayed bridge becomes plastic. That causes a residual displacement at the top of the tower. Moreover, it is possible that the problem occurs to the service ability of the cable-stayed bridge because the tensility of the cable becomes loose.

After The Hyogoken-Nanbu Earthquake, the study about seismic design of steel and reinforced concrete piers has been carried out in many kind of structures. But there are some papers about the study of cable-stayed bridges. Otsuka et al analyzed the three span continuous cable-stayed bridge(Aratsu Bridge) considered nonlinearlity of steel piers and compared the effects of the model and the performance of the analysis program [Otsuka et al, 1998]. Yoshizawa and Kawakami carried out about two span continuous cable-stayed bridge by the dynamic response analysis considered material and geometrical nonlinearlities[Yoshizawa at el, 1998]. From the latter study, it has little effect to reduce the acceleration of the superstructure because of the material

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nonlinearity. And the displacement of the tower in the direction of the right angle to the bridge axis is large, which is caused by the effect of the geometrical nonlinearity. Ishizaki et al examined the damage of the Higashi Kobe Bridge (bridge length=885m, double deck cable-stayed bridge) and investigated the dynamic response under the earthquake and the mechanism of the damage[Ishizaki et al, 1998]. But there are a few study of the cable-stayed bridge using the nonlinear dynamic response analysis under the three-dimensional great earthquake. The first purposes in this study are to understand the nonlinear response of the cable-stayed bridge under three-dimensional great earthquake ground motions and to examine seismic performance analytically. The steel tower of a cable-stayed bridge modeled by frame structure is used in this analysis and is investigated about the nonlinear dynamic response caused by the tower shape. Second purpose is to examine the effectiveness of the tower shape. Three tower shapes such as A-type, H-type and gate-type tower are used in this study. The elasto-plastic finite displacement analysis for three-dimensional beam elements is carried out. In this analysis, the nonlinearity as the stress-strain relationship of material and geometry is considered. The ground acceleration recorded at JR Takatori Station during the 1995 Hyogoken-Nanbu Earthquake is used (the north-south(NS), east-west(EW) and up-down(UD) components respectively). The influence that the dynamic response of the steel tower is given by the stiffness and shapes of cable is discussed by parametric analysis. Moreover, it is proposed to use the isolation device as a lead rubber bearing acting vertically only at the top of the steel tower and the effect is examined about reducing inertia force.

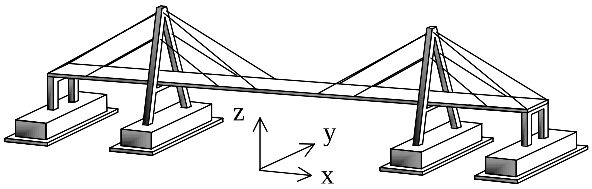
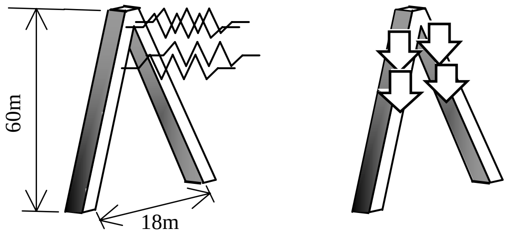


FIG.1: GENERAL VIEW OF CABLE-



a) Model of Cables **b) Dead Loads**

FIG.2: A-TYPE TOWER

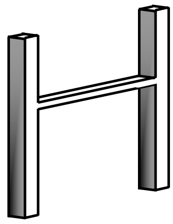


Fig.3: H-Type Tower

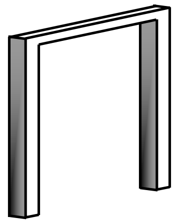


Fig.4: Gate-Type Tower

ANALYTICAL MODEL

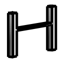
In this study, the nonlinear response analysis is carried out in the shapes of the steel tower of the cable-stayed bridge. Then the steel towers are taken out of the cable-stayed bridge (Fig.1) and modeled (Fig.2). The inertia forces from the stiffening girder acting the steel tower is neglected because the effect of the tower shape is considered in the nonlinear dynamic response analysis. The bridge used in this study is New Iwamizawa Bridge, which is three span continuous cable-stayed bridge : main span=284m; side span=115m; and bridge length=514m. In the model of the tower, the height is 60m, the length between the main tower is 18m and the basement is fixed.

The horizontal spring element, which is considered with the weight of the stiffening girder and the effect of the cable's stiffness, is modeled from the cable as shown in Fig.2(a). At the joint of the cables, the weight of the stiffening girder is acted vertically as shown in Fig.2(b). In the both sides of the tower, this bridge has eight cables and the coefficient of the spring is led from the section and the stress of the cable, and elastic modulus of the cable having sag length[Japan Society of Civil Engineers, 1990]. Dead load is calculated from the cable-stayed bridge, bridge length is 514m and the load (2.5MN) acts at each joint of the cables. In this analysis, the cable has 29.4MN/m as the coefficient of the spring. The shapes(A-type, H-type and gate-type)of the tower are shown in Figs2, 3 and 4, respectively. In those cases, the coefficient of the spring, the height and section of the towers are same structural properties of A-type tower. The section of the tower is the rectangle (3.5m $\times$2.4m,) and the thickness is 25mm.

ANALYTICAL METHOD

In this study, the analytical method is based on the elasto-plastic finite displacement dynamic response analysis composed by finite element method, Newmark β method and Newton-Raphson method. This finite element method considers the element of the frame structure with material yield and geometrical nonlinearity. The analysis using this study adopts the tangent stiffness matrix considering material and geometrical nonlinearities in case of the bending transformation in the in-plane and out-plane[Hayashikawa at el, 1998] and the torsional transformation is linearity. The stress-strain relationship of the member is modeled bilinear type. The yield stress is 235.4MN/m^2, elastic coefficient is 2.0×10^5MN/m^2 and the strain hardening in plastic area is 0.01. The damping of the structure supposed a mass proportion type and the damping coefficient to the 1st natural vibration mode is 5 %. The three dimensional frame structures which uses for the dynamic analysis is composed from the bar element that the number of the nods per element is 2. The number of divisional elements of the steel Tower is 36. This analysis is used the Hyogoken-Nanbu Earthquake recorded at JR Takatori station on January 17, 1995, as a input wave data. The input data composed three accelerations acts on the directions of the bridge axis, the right angle to the bridge axis and vertical at the basement of the tower.

Table.1: Calculated Natural Periods (sec)

Mode			
L1	0.3229	0.3228	0.3719
H1	1.0355	1.2535	1.7832

NUMERICAL RESULTS

Fundamental Response

This study has the purpose of investigating the nonlinear dynamic response of the steel tower. It is shown next about the fundamental natural frequency and dynamic nonlinear response of the analytical model.

Fundamental natural periods

Before executing dynamic response analysis, the natural vibration analysis is carried out by each tower models. The natural periods computed about the fundamental natural mode shape are shown in Table.1. The symbols L1 and H1 in the Table.1 are the 1st natural mode shape in the directions of the bridge axis and of the right angle to the bridge axis, respectively. The 1st natural period in the direction of the bridge axis is smaller than that in the direction of the right angle to the bridge axis with the effect of the spring by the cable. It is seen that the natural period of the gate-type tower has tendency to be getting longer in compared with the A-type and the H-type towers because there is a massive horizontal beam situated on the top of the tower.

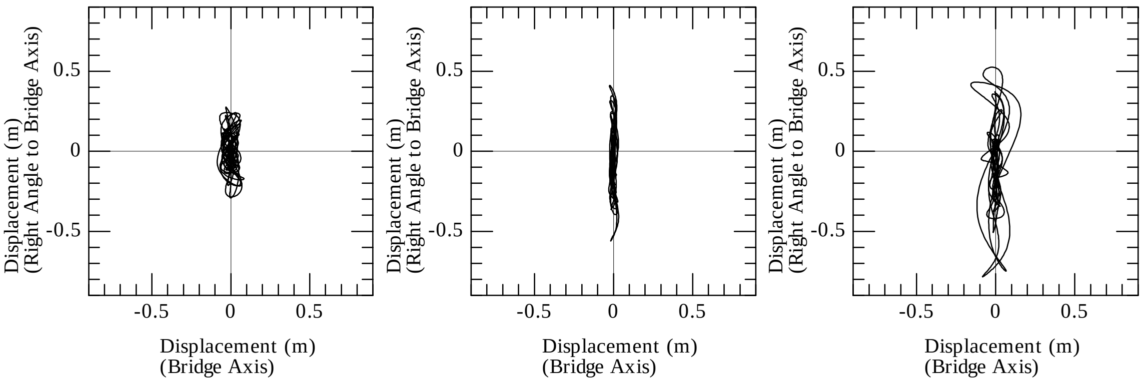
Orbit of response displacement at tower top

It is shown in Fig.5 the orbit of the response displacement at the top of the tower in the horizontal two directions. In the case of H-type tower, the response displacement of the left side at the tower top is used. The vertical axis in Fig.5 is the displacement in the direction of the right angle to the bridge axis and the horizontal axis is the displacement in the direction of the bridge axis. From the results of natural vibration analysis about A-type tower, the response displacement in the direction of the right angle to the bridge axis is larger than that of the bridge axis, because the cables are fixed in the direction of the bridge axis. As for the H-type tower, the displacement in the direction of the bridge axis is small and the displacement in the direction of the right angle to the bridge axis is larger than that in case of the A-type tower. It is seen that the 2nd natural mode shape in the direction of the right angle to the bridge axis excels at the member and so the larger displacement is occurred. Moreover, as for the gate-type tower, the displacement in the two horizontal directions is the largest value in three tower shapes. Due to the transverse vibration of the tower having longer natural period of in-plane (y-z plane), larger displacement is caused in the direction of the right angle to the bridge axis. Also the horizontal beam is situated on the top of the tower, then the displacement in the direction of the bridge axis becomes bigger than that of the A-type and the H-type tower because of the torsional vibration. In respect to the response displacement, the A-type tower is suitable in all tower shapes.

Bending moment and curvature

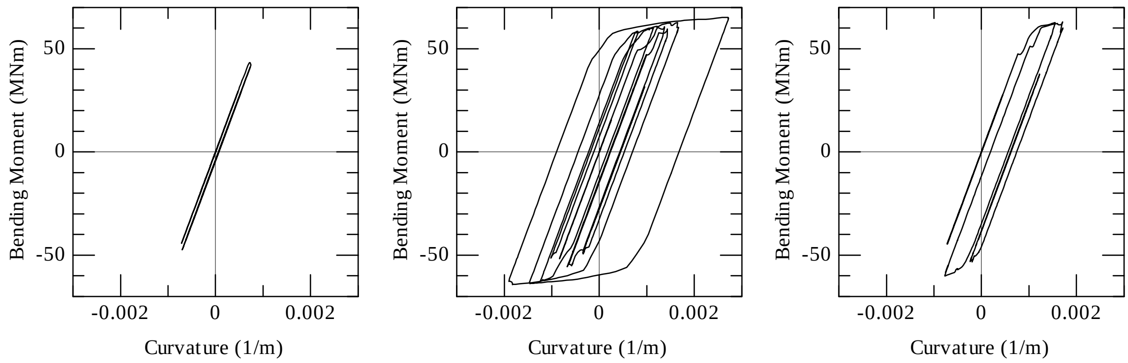
Fig.6 shows the relationship between bending moment and curvature at the tower basement in the direction of the right angle to the bridge axis. The vertical axis is the bending moment and the horizontal axis is the curvature. From Fig.6(a), although the basement of A-type tower is a few in plastic area. the hysteresis curve is small. Fig.6(b) shows that the hysteresis curve becomes the largest of the three towers because of the transverse vibration of in-plane. As the fundamental natural period (1.2535sec) of y-z plane of H-type tower is near the excelling period of the input wave, the large inertia force by the massive horizontal beam acts on the tower.

Fig.6(c) shows that the hysteresis curve is a little smaller than that of H-type tower. It seems that the position of the horizontal beam influences on the relationship between bending moment and curvature. Seen from the results



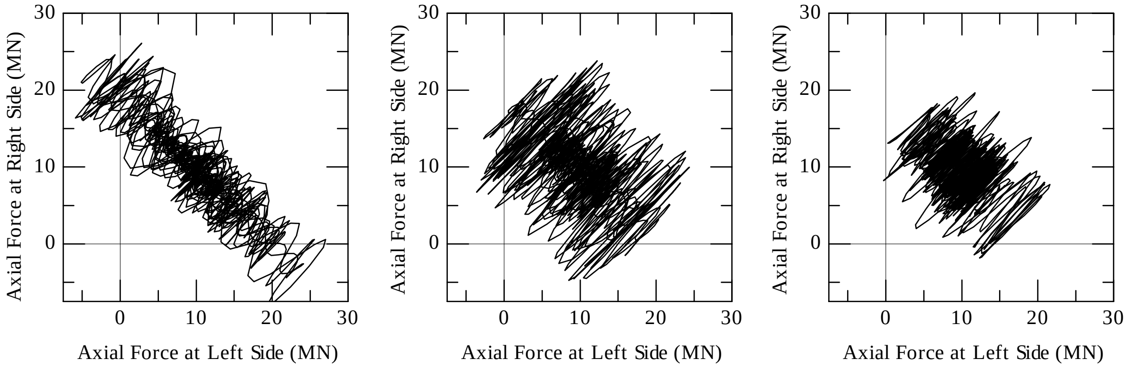
a) A-Type Tower b) H-Type Tower c) Gate-Type Tower

Fig.5: Orbit Of Response Displacement Of Tower Top



a) A-Type Tower b) H-Type Tower c) Gate-Type Tower

Fig.6: Relationship Between Bending Moment And Curvature



a) A-Type Tower b) H-Type Tower c) Gate-Type Tower

Fig.7: Axial Force Of Tower Basement

Axial force of tower basement

The dynamic response of each tower model is examined on axial force of the tower basement. Fig.7 shows the orbit of axial force that occurs at both sides of the tower basement. The vertical axis is the axial force at the right side of the tower basement and the horizontal axis is the axial force at the left side of the tower basement. The deadweight (9.8MN) acts on both sides of the tower basement through cables. The orbit of the axial force computed is the symmetrical form approximately, in which compressive axial force is 9.8MN. So it seems that the axial force at the both sides of the tower basement is same. Fig.7(a) shows larger axial force of A-type tower and the maximum compressive axial force is about 27.0MN. The negative reaction force is caused at the opposite side that the maximum compressive force occurs. The maximum tensile force is about 7.8MN. So that makes the problem that the anchor bolts may fail in the liftup and the safety of the tower basement should be considered. The value of the axial force of the H-type tower is smaller than that of the A-type tower. But the

negative reaction force exists at the tower basement and the anchor bolts may fail in the liftup. The axial force of the gate-type tower vibrates with the axis (9.8MN) at both sides of the tower basement. It seems that the range of the axial force is the smallest of the three tower shapes and that there is little negative reaction force. In respect to the axial force of the tower basement, the A-type tower is unsuitable tower shape because large compressive axial force and negative reaction force occur.

Influence of Spring Coefficient of Cable

The influence caused by the spring coefficient of the cable is examined. About all tower shapes, Fig.8 shows the relationship between the 1st natural period in the direction of the bridge axis and the spring coefficient of the cable. The value of the natural periods of each tower shape decrease as the value of the spring coefficient increases. When the spring coefficient is small, the value of the natural period increases in order of A-type, H-type and gate-type tower. The spring coefficient being 49.0MN/m, the natural periods of each tower model are close. But it being 98.0MN/m, the natural period of A-type tower is the longest value. In this study, the analytical models adopt the spring coefficient, 29.4MN/m, because the natural periods of all tower models are close.

Fig.9 shows the relationship between the spring coefficient and the maximum displacement of the tower top in the direction of the bridge axis. The response displacement of the gate-type tower is the largest and decreases in order of A-type and H-type tower. When the spring coefficient becomes larger than 29.4MN/m, the maximum response displacements of all tower models are less than 0.2m.

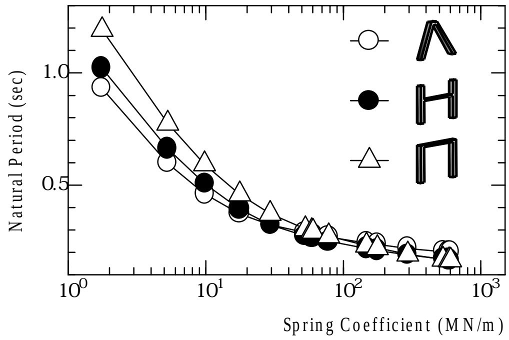


Fig.8: Relationship between Natural Period (L1) and Spring Coefficient

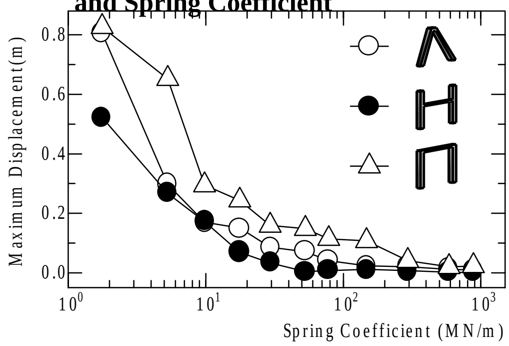


Fig.9: Relationship between Maximum Displacement

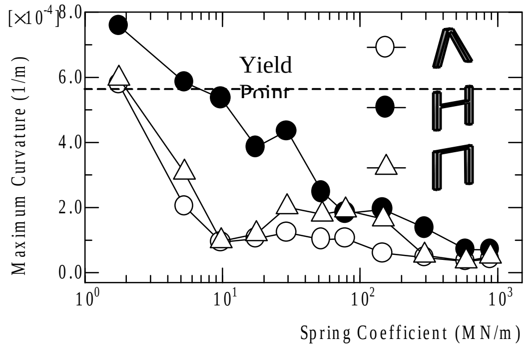


Fig.10: Relationship between Maximum Curvature and Spring Coefficient (Bridge Axis)

The relationship between the spring coefficient and the maximum curvature of the tower basement in the direction of the bridge axis is shown in Fig.10. Compared with A-type, gate-type and H-type tower, the maximum curvature of H-type tower is the largest of the three tower shapes. Especially, when the spring coefficient becomes small, the maximum curvature of H-type tower is much the largest. The broken line as shown in Fig.10 is the yielding point of the maximum curvature at the tower basement in the direction of the bridge axis. The spring coefficient becoming larger than 9.8MN/m, all tower models are in elastic area. So it seems that there is no problem on the seismic design.

Influence of Tower Shapes

The effects of height and length of the horizontal beam on dynamic response are discussed to examine the relationship about three tower shapes. The height of the horizontal beam is raised from the case of H-type tower to the case of gate-tower. Then, the length of the horizontal beam is shortened from the case of gate-type tower to the case of A-type tower as shown in Figs.14 and 15.

Fig.11 shows the relationship between tower shapes and the maximum displacement of the tower top in the directions of the bridge axis and of the right angle to the bridge axis. The horizontal axis is the range of the height and the length of the horizontal beam. The influence by the tower shapes is small about the maximum displacement in the direction of the bridge axis. However, as for in the direction of the right angle to the bridge axis, the influence appears remarkably. The height and the length of the horizontal beam becoming larger, the maximum response displacement is getting larger. Also, the response displacement in the direction of the bridge axis and of the right angle to the bridge axis about the tower shape that is around A-type tower is the smallest of

all tower shapes.

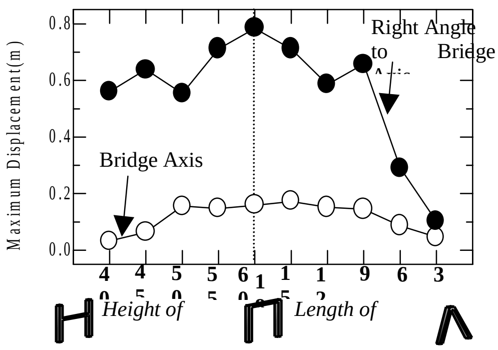


Fig.11: Relationship Between Maximum

Fig.12 shows the relationship between tower shapes and the maximum curvature at the tower basement in the directions of the bridge axis and of the right angle to the bridge axis. The maximum curvature in the direction of the bridge axis is almost constant and is within elastic area. But in case of being near H-type tower, the maximum curvature in the direction of the right angle to the bridge axis has the tendency to become large. On the other hand, as for the tower model that is around A-type, the maximum curvature in the directions of the bridge axis and of the right angle to the bridge axis becomes small.

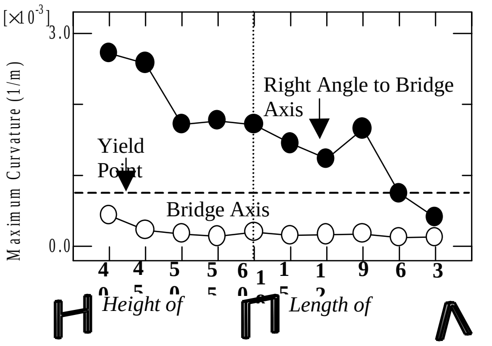


Fig.12: Relationship Between Maximum Curvature And Tower Shape

Above all results of the parametric analysis on tower shapes, the maximum displacement and curvature of A-type tower are the smallest in all tower shapes. It is seen from the results of the parametric analysis that A-type tower is suitable in all tower shapes. In respect to the axial force at the tower basement, however, A-type tower is unsuitable tower shape, because large compressive axial force and negative reaction force occur.

REDUCTION OF SEISMIC –INDUCED FORCES

The dynamic responses of three tower models are examined in the previous chapter. It is suggested that the passive control device by moving vertically is installed in the tower top of the cable-stayed bridge and the effects of seismic isolation device on dynamic response characteristics are examined. The image of the passive control device is shown in Fig.13.

As the analytical model of the passive control device, the spring elements of two horizontal directions, vertical direction and three rotational directions are considered. The spring coefficient of vertical direction is 9.8kN/m as a small value and the others are 9.8×10^{3}MN/m as a large value. In case of H-type and gate-type tower, the passive control device is installed in the center of the horizontal beam. Table.2 shows the calculated results of natural periods on the tower models with the passive control device. The 1st natural period in the direction of the bridge axis (L1) of A-type tower is almost same in comparison with Table.1. The natural period (L1) of H-type and gate-type tower with the passive control device as shown in Table.2 are a little longer than that without passive control device as shown in Table.1. On the other hand, the 1st natural periods of all tower models with passive control device in the direction of the right angle to the bridge axis (H1) are considerably longer than that with no passive control device. The tower can move vertically, so that the H1 becomes longer and it seems that the seismic performance is improved.

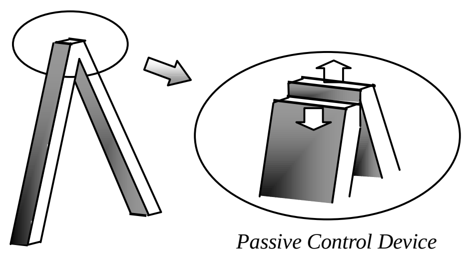
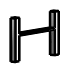
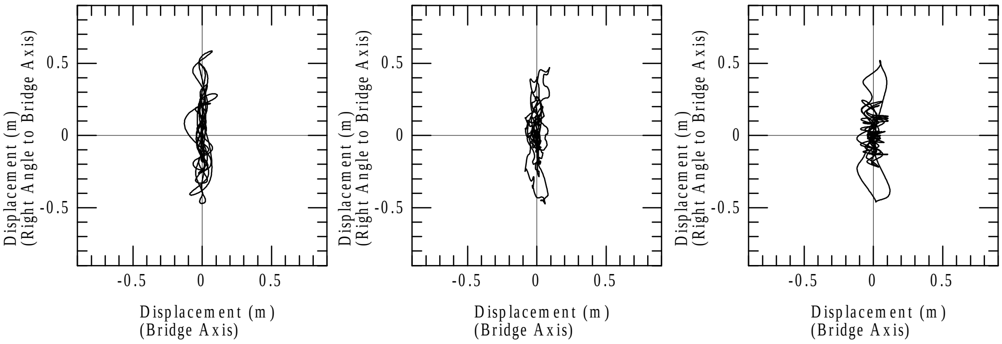


Fig.13: Tower With Passive Control Device

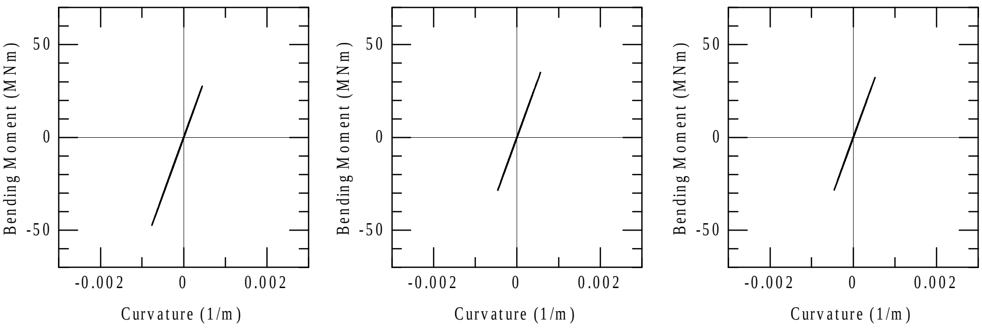
Table.2: Calculated Natural Periods (sec)

Mode			
L1	0.3196	0.3806	0.4820
H1	2.6191	2.5192	2.9818



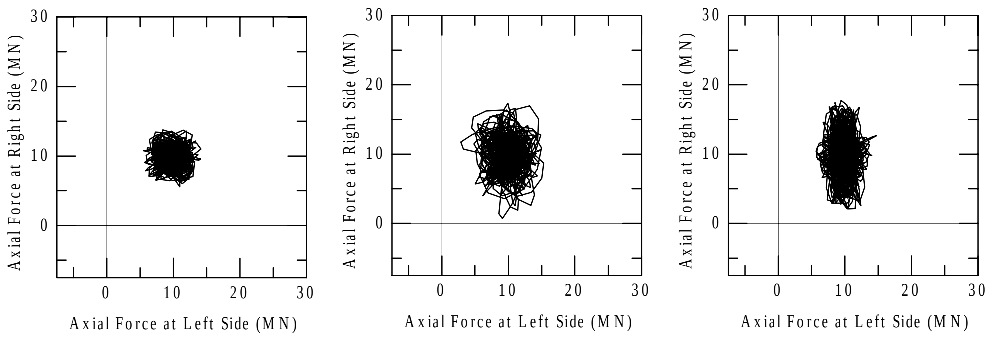
a) A-Type Tower b) H-Type Tower c) Gate-Type

Fig.14: Orbit of Response Displacement of Tower Top (with Passive Control



a) A-Type Tower b) H-Type Tower c) Gate-Type

Fig.15: Relationship between Bending Moment and Curvature (with Passive



a) A-Type Tower b) H-Type Tower c) Gate-Type

Fig.16: Axial Force of Tower Basement (with Passive Control Device)

Figs.17(a), (b) and (c) show the orbits of the response displacement of the tower top of A-type, H-type and gate-type, respectively. The vertical axis is the displacement in the direction of the bridge axis and the horizontal axis is the displacement in the direction of the right angle to the bridge axis. In the cases of A-type and gate-type, the response displacements of the center at the horizontal beam are used and in the case of H-type tower, the response displacement of the left side at the tower top is used. The response displacement of A-type tower in the direction of the right angle to the bridge axis as shown in Fig.14(a) is longer than that as shown in Fig.5(a), because the passive control device is installed in the tower top. Also, it is considered that the natural period (H1) becomes longer by the passive control device. About H-type and gate-type tower, the response displacements as shown in Figs.17(b) and (c) are almost same and become a little smaller in comparison with Figs.8(b) and (c), respectively.

Fig.15 shows the relationship between the bending moment and curvature at the tower basement in the direction of the right angle to the bridge axis. The vertical axis is the bending moment and the horizontal axis is the curvature. As for three tower models, the bending moments are in elastic area and it seems that the seismic performance is improved. Especially, the hysteresis curve of the H-type tower as shown in Fig.15(b) becomes much smaller than that as shown in Fig.6(b).

Fig.16 shows the orbit of axial force that occurs at both sides of the tower basement. As for three tower models, the axial forces as shown in Fig.16 become smaller than those as shown in Fig.7. Especially, the axial force of A-type tower with passive control device is much reduced in comparison with the axial force of A-type tower with no passive control device. It is concluded that the negative reaction forces of three tower models are decreased by the passive control device.

CONCLUSIONS

In this study, the elasto-plastic finite displacement dynamic response analysis considering material and geometrical nonlinearity is carried out to investigate the dynamic behavior of steel towers of the cable-stayed bridge. The effects of the tower shapes on dynamic characteristics under great earthquake ground motion are discussed. The conclusions obtained in this study are summarized as follows.

The dynamic response in the direction of the right angle to the bridge axis has the tendency to be larger in order of A-type, H-type and gate-type tower. Even though the spring coefficients of the cables are same, the response displacements in the direction of the right angle to the bridge axis depend on the tower shapes. From the calculated results of the relationship between bending moment and curvature, it is considered that A-type tower is suitable in three tower shapes. As for H-type and gate-type tower, large inertia force acts on in-plane, so the hysteresis curve depends on the position of the horizontal beam. It is recognized from the axial force of the tower basement that the A-type tower is unsuitable tower shape because large compressive axial force and negative reaction force occur. So the problem that the anchor bolts may fail in the liftup may be happen because of the large negative reaction force. Then, the safety of the tower basement should be considered in seismic design.

The differences of maximum displacements and curvatures in the direction of the bridge axis are not concerned with the tower shapes from the results of the parametric analysis on tower shapes. But, in the direction of the right angle to the bridge axis, the response displacements around the gate-type tower are larger and the maximum curvatures have the tendency to be getting larger around H-type tower. As for A-type tower, the maximum displacement and curvature are the smallest in all tower shapes, so it is seen that A-type tower is suitable in all tower shapes.

It is suggested that the passive control device by moving vertically is installed in the tower top of the cable-stayed bridge to reduce the axial compressive force and the negative reaction force at the tower basement. The response displacement of A-type tower with passive control device in the direction of the right angle to the bridge axis is longer than that with no passive control device. As for H-type and gate-type tower with passive control device, the response displacements are almost same and become a little smaller in comparison with no device towers. As for three tower models, the relationships between the bending moment and curvature are almost in elastic area because of the passive control device and the seismic performance is improved. Especially, the hysteresis curve of the H-type tower with the passive control device become much smaller than that with no passive control device, because the inertia force that acted on in-plane was reduced. About all tower models, the axial forces that occurred at the basement is reduced by the passive control device. Especially, It is concluded that the passive control device is effective to reduce the axial force occurring at the tower basement.

The three-dimensional nonlinear dynamic response analysis shown in this study is effective to investigate the dynamic behavior of complicated frame structures subjected to great earthquakes. It is very important for the seismic design of the cable-stayed bridge that the stiffness of the cables and the tower shape are appropriately selected to improve the seismic performance.

REFERENCES

Hayashikawa, T. and Ohtake, A. and Nakajima, A. (1998.2.), “Nonlinear behavior of space viaduct system under level-2 earthquake ground motions”, *Proceedings of Hokkaido Chapter of the Japan Society of Civil Engineers*, No.54(A), pp146-151.

Ishizaki, H., Kitazawa, M., Nishimori, K. and Noguchi, J. (1998.2.), “Analytical Study of Damage of Higashi-Kobe Bridge by the Hyogoken-nanbu Earthquake”, *Bridge and Foundation Engineering*, pp39-46.

Japan Road Association (1996.12.), *Specifications for Highway Bridges Part V Earthquake Resistant Design*, Japan.

Japan Society of Civil Engineers (1990.9.), *Cable-Stayed Steel Bridges*, Steel Structures Series 5, Japan.

Japan Society of Civil Engineers (1997.5.), *Nonlinear Numerical Analysis and Seismic Design of Steel Bridge Piers*, Japan.

Japan Society of Civil Engineers, Earthquake Engineering Committee (1996.11.), *Proceedings of the 1st Colloquium on Seismic Isolation and Response Control*, Japan.

Japan Society of Civil Engineers, Earthquake Engineering Committee, Ductility Design Subcommittee (1998.1), *Proceedings of the 1st Symposium on Ductility Design Method for Bridges*, Japan.

Otsuka, H. et al (1998.12.), “Nonlinear Dynamic Response Analysis and Modeling of Steel Cable-Stayed Bridge (in Japan)”, *Proceedings of the 2nd Symposium on Ductility Design Method for Bridges*, pp155-162.

Yoshizawa, T. and Kawakami, M. (1998.1), “Nonlinear Dynamic Analysis of Steel Cable-Stayed Bridge under Great Earthquake Ground Motion (in Japan)”, *Proceedings of the 1st Symposium on Ductility Design Method for Bridges*, pp257-260.

NONLINEAR DYNAMIC BEHAVIOR AND SEISMIC ISOLATION OF STEEL TOWERS OF CABLE-STAYED BRIDGES UNDER GREAT EARTHQUAKE GROUND MOTION

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Abstract

In the recent Kobe Earthquake, many bridge structures were seriously damaged. As the earthquake ground motions have two horizontal directions (NS and EW components) and up-down direction, the necessity has arisen to know the precise three dimensional dynamic behavior of the bridge systems. Cable-stayed bridges are composed of three different structural elements, such as stiffening girders, cable and towers. As the main span length of bridges increases, dynamic behavior of cable-stayed bridges under the great earthquake ground motions becomes complicated.

The nonlinear dynamic behavior of steel towers of cable-stayed bridges subjected to three-dimensional great earthquake ground motions is studied. Nonlinearitites due to geometry changes and material sources of steel tower elements are considered, and a tangent stiffness iterative procedure is used in the dynamic analysis to obtain the nonlinear seismic response. Numerical examples for three different towers (A-type, H-type and gate-type models) are presented in this study. The effects of various tower shapes and design parameters of the cable on the dynamic behavior of cable-stayed bridges are investigated.

The major conclusions obtained in this study are summarized in the following. The displacement response at tower top is different due to tower shapes. The displacement response of A-shaped towers in the both directions of bridge axis and perpendicular to the bridge axis is smaller than that of H- and gate shaped towers. It is recognized from calculated resuits between bending moments and curvatures that A-type model is the most appropriate tower shape. However, the negative reaction force at the tower basement occurs in A-type model so that the anchor bolts may fail in the lift-up. It is confirmed that the seismic performance of the steel towers with passive devices can be obtained to reduce the reaction forces at the tower basements. The adaptation passive control system can accomplish a significant reduction in seismic-induced forces, as compared to the case with no passive control devices added.

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STRUCTURAL MATERIALS, ELEMENTS AND SYSTEMS, 3-D nonlinear seismic analysis, Dynamic nonlinear characteristics, Elasto-plastic hysteretic behaviour, Hysteresis loop, Isolated structures, Large deformation, Natural frequency, Nonlinear dynamic analysis., Residual displacement, Strong ground motion, Tower, Cable stayed bridge,