Nonlinear Dynamic Behavior of Steel Tower of Cable-Stayed Bridges with Passive Energy Dissipation System

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For economical earthquake resistant of cable-stayed bridges and its protection from the earthquake hazard, the structures of the bridges must be constructed to dissipate a large amount of seismic energy. This may be achieved by providing effective isolation system at certain locations. This technique has the ability to control and improve their vibration and dynamic behavior. A nonlinear finite element method based on total Lagrangian formulation using linearized finite displacement theory is developed for modeling the steel tower of Iwamizawa cable-stayed bridge under static and dynamic loading of three-dimensional great earthquake ground motion. Both the realistic behavior of material and geometric nonlinearities are considered in the analysis. An incremental iterative numerical scheme based on Newton Raphson is employed for solution of the nonlinear dynamic equilibrium equation. A simplified two-node spring element model is proposed for the energy dissipation device of new viscoelastic type connection isolator is introduced for analysis approach. A comparative study for the steel tower with and without isolation system under a severe ground motion to assess the validity and effectiveness of the proposed isolation system is carried out. The results of this study show that the proposed isolation system provides significant improvement in reducing the tower member forces and increasing the energy dissipation.

Keywords: Steel tower, cable-stayed bridge, isolation, seismic design, great earthquake

1. Introduction

In recent decades, long span bridges such as cable-stayed and suspension bridges have gained much popularity, due to their aesthetic appearance, efficient utilization of structural materials, increase of the horizontal navigation clearances and the economic trade off of span length cost of deep water foundation. From this point view, the trend today for cable-stayed bridges is to use more shallow or slender stiffening girders combined with increasing span lengths. This structural synthesis provides a valuable environment for the nonlinear behavior due to material nonlinearities and geometrical nonlinearities of the relatively large deflection of the structure on the stresses and forces [1]. For that reason, it is highly desirable in bridge engineering to develop accurate procedures that can lead to a through understanding and a realistic prediction of the structural response. After the January 17, 1995, Hyogoken-Nanbu earthquake, the ductility design and dynamic analysis have been reconsidered in Japan[2]. Since the earthquake ground motion have two horizontal directions and up down direction, the necessity has arisen to develop more efficient methods to understand accurately the precise three-dimensional nonlinear dynamic response of the bridge structural systems and to improve its seismic performance precisely.

The traditional approach to seismic hazard mitigation is to design structures with sufficient strength capacity and the ability to deform in a ductile manner. Alternatively, newer concepts of structural passive control have been growing in acceptance as design alternative for earthquake hazard mitigation for various structures. Since, passive control systems can be used in two ways, as connection isolators[3,4] and as base isolators [5,6] to achieve different objectives or performance goals ranging from a life safety standard to a higher standard that would provide damage control and post-earthquake functionality.

The connection isolators are designed to dissipate a large portion of the earthquake input energy in connection details that deform and yield during an earthquake. Since the deformation and yielding are concentrated in the device at selected connections of the structure, damage to other element of the structure may be reduced. Moreover, these devices do not require any additional energy to operate, and are activated by the earthquake input motion. Among the well-known dissipating device are the friction type and the viscoelastic dissipators. The most important common features of such systems are a shift in the natural period of the structure to a longer value, and an increase in structural damping.

In this study, the nonlinear dynamic behavior and seismic performance of the steel tower of Iwamizawa cable-stayed bridge under three-dimensional great earthquake ground motion is studied analytically. The steel tower is studied for four cases, case of the tower original configuration without using isolation system and three cases of implementation with the proposed energy dissipation devices system (sliding and rotational) at different positions (middle and both ends) of the horizontal beam of the tower. A natural vibration and three dimensional nonlinear dynamic analysis study of the steel tower with the proposed isolation devices is carried out and compared to the response obtained for the tower with its original configuration. Moreover, the type and location of this isolation in the tower has been studied to determine the best type and location, which cause more energy dissipation. Since, the most suitable device for isolation of the tower from the seismic input should have variable properties with respect to the external intensity of excitation. So, the isolation device is modeled as tri-linear model to control the displacement of the isolation device at different level of seismic excitation. The property of this isolation model of new viscoelastic type connection isolator made of elastomeric material in reducing the response of the tower to earthquakes is determined.

It is found that the installation of the sliding isolation device at both ends of the horizontal beam of the tower provides more significant improvement in the seismic behavior than that in the middle position. This may be attributed to the frame action of the steel tower under horizontal excitation of earthquake ground motion, which gives high strain action at the tower joint compare with any other position. Moreover, the rotation device is more effective in reducing the tower member forces and increasing the energy dissipation than the sliding device, due to that the flexural effect is more pronounced compared with the shear effect on relative deformation of the isolation device.

2. Nonlinear Dynamic Equilibrium Equation

The governing nonlinear dynamic equation of the tower response can be derived by the principle of energy that the external work is absorbed by the work of internal, inertial and damping for any small admissible motion that satisfies compatibility and boundary condition. By assembling the element dynamic equilibrium equation for the time $t+\Delta t$ over all the elements, the incremental FEM dynamic equilibrium equation[7] can be obtained as:

$$[\mathrm{M}]\{\ddot{u}\}^{t+\Delta t}+[\mathrm{C}]\{\dot{u}\}^{t+\Delta t}+[\mathrm{K}]^{t+\Delta t}\{\Delta u\}^{t+\Delta t} = \{\mathrm{F}\}^{t+\Delta t}-\{\mathrm{F}\}^{t} \quad \text{.............(1)}$$

where [M], [C], and $[\mathrm{K}]^{t+\Delta t}$ are the system mass, damping and tangent stiffness matrices at time $t+\Delta t$, the tangent stiffness considers the material nonlinearities through bilinear stress strain relation for the beam column element, and the geometrical nonlinearities for the case of in-plane, out-plane bending deformations and linear torsional deformations. $\ddot{u}$, $\dot{u}$, and Δu are the accelerations, velocities, and incremental displacements vector at time $t+\Delta t$ respectively, $\{\mathrm{F}\}^{t+\Delta t}$ - $\{\mathrm{F}\}^{t}$ is the unbalanced force vector. It can be noticed that the dynamic equilibrium equation of motion takes into consideration the different sources of nonlinearities both geometrical and material nonlinearities, which affect the calculation of the tangent stiffness and internal forces.

In this study, the Newmark step-by-step integration method is used for the integration of equation of motion, since it has been experienced that the Newmark's β method is the most suitable for nonlinear analysis; it has the lowest period elongation and has no amplitude decay or amplifications. In addition, the stability concern is not a problem with the variable ratio of time increment to natural period. Using the Newmark's integration method, the equation of motion can then be expressed as:

$$[\mathrm{K}]\{\Delta u\}^{t+\Delta t} = \{\mathrm{F}\} \quad \text{............. (2)}$$

in which effective stiffness and force vector can be written as:

$$[\mathrm{K}] = (1/\beta\Delta t^2)\,[\mathrm{M}]+ (\gamma/\beta\Delta t)\,[\mathrm{C}]+ [\mathrm{K}]^{t+\Delta t} \quad \text{.............(3)}$$

$$\{F\}=\{F\}^{t+\Delta t}-\{F\}^{t}-((-1/\beta\Delta t)[M]+(1-\gamma/\beta)[C])\{\dot{u}\}^{t}$$
$$-((1-1/2\beta)[M]+\Delta t\,(1-\gamma/2\beta)\,[C])\,\{\ddot{u}\}^{t} \quad ....(4)$$

where γ and β are Newmark's integration parameters that can be chosen according to the accuracy and stability concerns. The algorithm is unconditionally stable if $\beta \geq (\gamma + 0.5)^2/4$. In this study the Newmark's β of constant acceleration scheme is considered for which γ and β are equal to 0.5 and 0.25 respectively.

The equation of motion is solved for the incremental displacement $\{\Delta u\}^{t+\Delta t}$ using the Newton Raphson iteration method where the stiffness matrix is updated at each increment to consider the geometrical and material nonlinearities and to speed the convergence rate. As the incremental displacement is determined by the solution of Eq. 2, the response acceleration and velocity components at time $t+\Delta t$ can be determined as follows:

$$\{\ddot{u}\}^{t+\Delta t}=(1/\beta\Delta t^2)(\{\Delta u\}^{t+\Delta t}-\Delta t\{\dot{u}\}^{t}-(0.5-\beta)\,\Delta t^2\{\ddot{u}\}^{t}\,)$$

$$\{\dot{u}\}^{t+\Delta t}=(\gamma/\beta\Delta t)(\{\Delta u\}^{t+\Delta t}-(1-\beta/\gamma)\Delta t\{\dot{u}\}^{t}$$
$$-(0.5-\beta/\gamma)\Delta t^2\,\{\ddot{u}\}^{t}\,) \quad ...(5)$$

where $\{\Delta u\}^{t+\Delta t}$ is the displacement increment measured from the previous convergence configuration at time t. In addition, attenuation of the structure adopted the viscous damping of mass proportional type with damping coefficient to the first fundamental natural vibration mode is 5 % as standard.

3. Structural Model

The steel tower of Iwamizawa cable-stayed bridge located in Hokkaido, Japan is considered in this study. The steel tower is taken out of the cable-stayed bridge and modeled as three-dimensional frame structure. A fiber flexural element is developed for characterization of the steel tower, the element incorporates both geometric and material nonlinearities, a cubic displacement field was employed for the transverse displacement of the element and linear displacement field is employed for the axial and torsional displacements. The stress-strain relationship of the beam element is modeled as bilinear type. The yield stress and the modulus of elasticity are equal to 355 MPa and 200GPa, respectively; the strain hardening in the plastic region is 0.01.

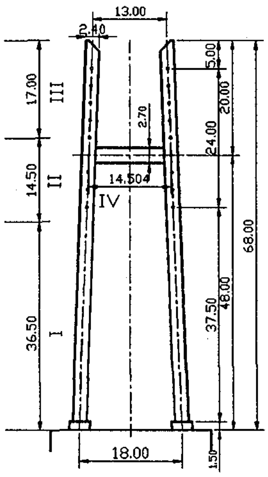
Inelasticity of the flexure element is accounted for by the division of the cross section into a number of fiber zones with uniaxial plasticity defining the normal stress-strain relationship for each zone, the element stress resultants are determined by integration of the fiber zone stresses over the cross section of the element. By tracking the center of the yield region, the evolution of the yield surface is monitored, and a stress update algorithm is implemented to allow accurate integration of the stress-strain constitutive law for strain increments, including full load reversals. To ensure path dependence of the solution, the implementation of the plasticity model for the implicit Newton-Raphson equilibrium iterations employs a stress integration whereby the element stresses are updated from the last fully converged equilibrium state. The transformation between element local and global coordinates is accomplished through a vector translation of element forces and displacements based on the direction cosines of the current updated element coordinate system.

The nonlinear behavior of cable elements is idealized by using the equivalent modulus approach[8], in this approach each cable is replaced by a truss element with equivalent tangential modulus of elasticity (E_{eq}), used to take account of the sag effect, and can be written as:

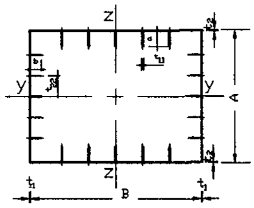
$$E_{eq}=\frac{E}{1+EA(wL)^2/12T^3} \quad ....(6)$$

where E is the material modulus of elasticity, L is the horizontal projected length of the cable, w is the weight per unit length of the cable, A is the cross sectional area of the cable, and T is the tension force in the cable, it can be noticed that increasing the cable tensile force, as the sag decreases, lead to an increase in the apparent axial stiffness of the cable.

This cable-stayed bridge has nine cables in both sides of the tower. The dead load of the stiffening girder is considered to be equivalent to the vertical component of the pretension force of the cables and acted vertically at the joint of cables. The inertia forces acting on the steel tower from the stiffening girders is neglected. For the numerical analysis, the geometry and the structural properties of the steel tower is shown in Fig.1, where this tower has rectangular hollow steel section with internal stiffeners, which has different dimensions along the tower height and its horizontal beam as shown in Table 1.



(a) Tower geometry



(b) Cross section

Fig. 1 Steel tower of Iwamizawa cable-stayed bridge

Table 1 Cross section dimension of different tower region

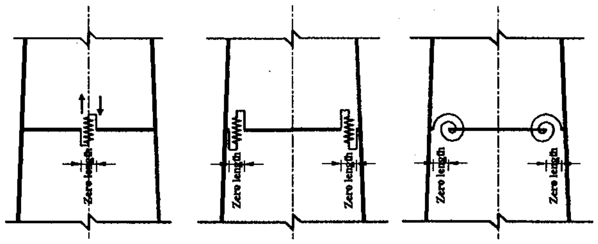
C. S.		Outer dimension				Stiffener dimension			
Dim. (cm)		A	B	t_1	t_2	a	b	t_{11}	t_{22}
Tower parts	I	240	350	2.2	3.2	25	22	3.6	3.0
	II	240	350	2.2	3.2	22	20	3.2	2.8
	III	240	350	2.2	2.8	20	20	2.8	2.2
	IV	270	350	2.2	2.6	31	22	3.5	2.4

4. Passive Energy Dissipation Device Model

The seismic design philosophy depends upon increasing the energy dissipation capacity of a structural frame as opposed to relying upon increased frame stiffness and ductility. The earthquake input energy can be absorbed by the deformation of the isolation devices as opposed to yielding and consequential damage of main structural members, by using these isolation devices, it is possible to increase the energy capabilities of the tower structure while reducing its accelerations and inertial forces. Since the amount of energy transmitted to the tower structure during the input earthquake depends mainly on the ratio of the fundamental period of the steel tower to the predominant period of the ground motion. Then, the proposed isolation system should have two main features, which are a shift in the natural period of the tower to a longer value and an increase in the structural damping due to hysteretic behavior of the inelastic deformation[9].

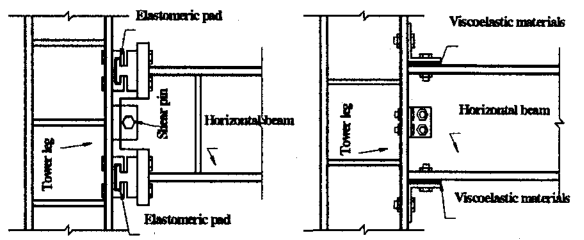
It was suggested for the isolation of the cable-stayed

(a) Location and direction of isolation devices



(1) Case II (2) Case III (3) Case IV

(b) Rotation isolation device design[10] (Case IV).



(1) Elastomeric pad (2) Viscoelastic material

Fig. 2 Passive energy dissipation control system

bridge steel tower that a passive control device by moving vertically is installed in the middle of the horizontal beam of H-type tower, this device is modeled by linear spring (axial and rotational) for each degree of freedom in the global system axes[3]. This linear model depends only on the shift of the frequency of the tower for isolation, and cannot consider the intensity level of external excitation.

In this study, four cases are considered for Iwamizawa cable-stayed bridge tower, three cases of using passive energy dissipation control system, as shown in Fig. 2(a), are compared with other case of the tower original configuration without isolation:

Case I: original configuration without isolation device.

Case II: vertical moving device is installed in the middle of the horizontal beam.

Case III: vertical moving device is installed in both ends of the horizontal beam.

Case IV: in-plane rotational moving device is installed in both ends of the horizontal beam as shown in Fig. 2(b).

Since, the most suitable isolation devices of the bridge structures from seismic input should have variable properties with respect to the intensity of the external excitation[11], these displacement dependent devices (sliding and rotational) are modeled by tri-linear hysteresis as shown in Fig. 3. This modeling of the isolation device is considered to be stiff at small strains; hence they will only undergo insignificant

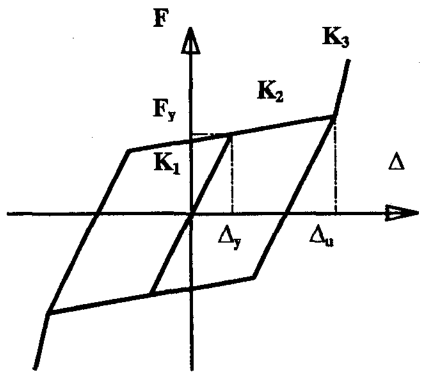


Fig. 3 Tri-linear model of isolation device

displacement under small earthquake loads. Similarly, the isolation system should be flexible of low stiffness and provide adequate damping under moderate excitation loadings. Furthermore, they harden again to be able to limit the relative displacement, providing thus an additional fail-safe action against extreme seismic loads.

The principal parameters that characterize the energy dissipation capacity of the isolation device are the yield force F_y, the initial stiffness K_1, and maximum displacement, which can be described by the ductility factor $\mu = \Delta_u/\Delta_y$. Finally, the isolation device exhibits some strain hardening K_2 as illustrated in Fig. 3. The strain-hardening ratio is typically small (0.01-0.1) and its effect on the structure response using the proposed isolation device is to reduce lateral deformations and increase the forces in the tower elements. The presence of strain hardening has the same effect as increasing the yield force[12], and taken equal to 0.01 in this study.

5. Selected Ground Motions

In the dynamic response analysis, the ground motion that was recorded during the Hanshin/Awaji earthquake 1995 with the largest intensity of ground acceleration is used as an input to assure the seismic safety of bridges. The acceleration time history recoded at JR Takatori Station as shown in Fig. 4 was suggested for analysis of the steel tower of Iwamizawa cable-stayed bridge at type II of soil condition, because it was considered to be capable of exciting of this tower well into its nonlinear range. The selected ground motion has maximum acceleration of its components equal to 641.734 gal (NS), 664.204 gal (EW) and 289.720 gal (UD). From Fourier spectrum analysis, the predominant frequencies for N-S and

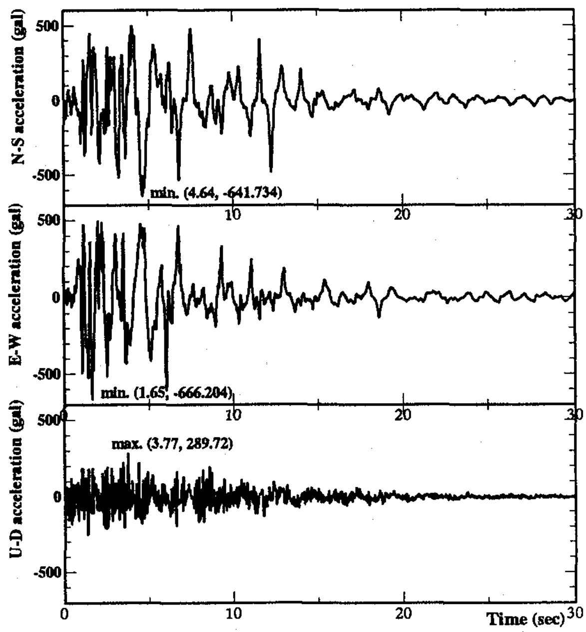


Fig. 4 Strong ground motion measured at JR Takatori observatory

E-W components are 0.830, 0.806 Hz, respectively, which are relatively low, and that for the U-D component is 7.959Hz which includes a high frequency components. The earthquake force of E-W wave was put into the bridge axis direction, and N-S wave to the right angle to the bridge axis.

6. Numerical Results

6.1 Natural Vibration Analysis

Depending on the fundamental frequencies of the steel towers of Iwamizawa cable-stayed bridge in relation to the dominant frequency content of the seismic input, shifting the period of the tower T would significantly reduce accelerations and tower member forces. For the natural vibration analysis, the equation of motion for the steel tower structure can be written in form of frequency equation as follow:

$$\|[K] - \omega^2 [M]\| = 0.0 \qquad \text{.............................(7)}$$

Expanding the frequency equation gives a system of algebraic equations of the Nth degree in the frequency parameter ω^2 for N degrees of freedom of the studied tower.

The N roots of this equation represent the frequencies of the N modes of tower vibration. The natural period T of the fundamental mode of the steel tower with different types and positions of the proposed energy dissipation devices has been

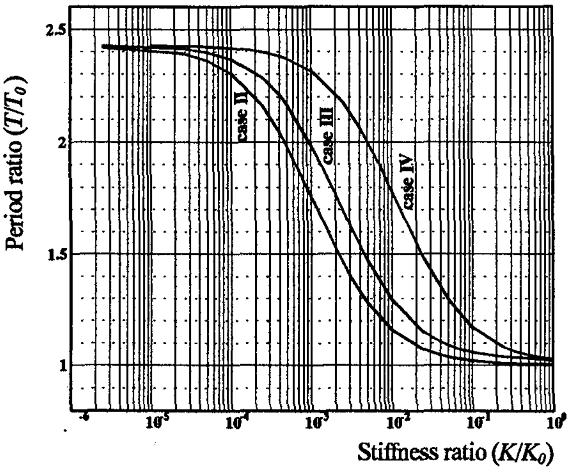


Fig. 5 Natural period ratio (T/T_0) and stiffness ratio (K/K_0) relationship for the isolation device

studied for different levels of isolation device stiffness. A summery of the preliminary natural vibration analysis results for three cases of connection isolators is presented as a relationship of the stiffness ratio (K/K_0) of the isolation device and elongated period ratio (T/T_0) for different cases of study as shown in Fig. 5, where T_0 is the in-plane fundamental vibration mode period of the tower original configuration (case I) equal to 2.072 sec. K_0 is the stiffness of the isolation device for which the tower has the response obtained when the tower is considered with its original configuration ($dT/dK \cong 0$), and its value equal to 19.6 GN/m for the transverse sliding device, 196 GN·m/rad. for rotational device.

Since the effectiveness of the isolation device can be measured by its capabilities in the energy dissipation through the natural vibration period of the fundamental mode shifting and increasing structural damping. It can be concluded from Fig. 5 that the sliding isolation device is effective for stiffness ratio range (0.0001-0.05), and the rotation one is effective for stiffness ratio range (0.0005-0.20). For the same stiffness ratio, the position of the sliding isolation device at both ends (case III) of the horizontal beam of the tower is more suitable and effective than its position at the middle (case II). Moreover, for that position, the effectiveness of the rotation device (case IV) is more pronounced in the natural periods shift than sliding device for the same stiffness ratio.

6.2 Energy Dissipation Device Parameters Determination

The design of a passive energy dissipation system depends on many factors, including the period of original tower configuration, the period of isolated tower, the input response spectrum, and force deformation relationship for isolation device, so the design of isolation system should be able to provide supplemental damping to significantly reduce tower response to ground motion and dissipation a large portion of earthquake input energy through inelastic deformations in the energy dissipation devices. Then the spectral acceleration and structural element forces will be significantly reduced when compared with that of tower original configuration. A numerical parametric study is carried out for the determination of the isolation device parameter through three steps:

(i) First step: linear model (K_1) of the isolation device is used for nonlinear dynamic analysis for the determination of the corresponding maximum displacement Δ_{max} in the force displacement hysteresis loops for the isolation device.

(ii) Second step: a parametric study for different yield displacement (Δ_y) level range (0.2-1.0) Δ_{max} of bilinear model with a strain hardening ($K_2 = 0.01K_1$) is carried for the determination the optimum yield level (Δ_y) of the isolation device model, for which the tower structure response display maximum damping, minimum strain energy dissipated through it and maximum absorbed energy through the isolation system.

(iii) Third step: tri-linear model is used for control the ductility of the isolation device relative deformation by providing rigid behavior ($K_3 = 0.1\ K_0$) after certain design ductility ($\mu = \Delta_u/\Delta_y$), which depend on the device material property.

6.3 Nonlinear Dynamic Response of Tower System

The nonlinear behavior of steel tower of cable-stayed bridge subjected to three-dimensional earthquake ground motions is studied. The aforementioned steel tower is implemented with proposed isolation type for different position of installation (case II, case III), and for different movement types (case III, case IV). The period shift ratio (T/T_0) for the predominant fundamental mode of tower vibration for three levels of stiffness ratio (0.001, 0.01, 0.10) of the energy dissipation system is shown in Table 2. For these isolation stiffness levels, the response of isolated tower with the proposed passive energy dissipation system to a critical ground motion is computed and compared to the response obtained when the steel tower is considered with its original configuration (case I), different value of stiffness can be achieved by using different hardness of used material for device design.

Table 2 Values of T/T_0 of isolated tower for study cases

Isolated tower cases	Stiffness ration (K/K_0)		
	0.001	0.01	0.10
case II	1.76	1.16	1.02
case III	1.99	1.29	1.06
case IV	2.31	1.78	1.17

(1) Case of high stiffness ratio K/K_0 = 0.10

The devices (sliding and rotation) are designed for optimum yield force level, which provides maximum damping energy, minimum strain energy absorbed through the tower itself and maximum dissipated energy through the inelastic deformations in the proposed energy dissipation devices. The designed sliding device has yield displacement equal to 7.8 mm and the ductility factor is equal to 10, 15 for cases II and III, respectively, the rotation device has yield rotation equal to 0.0047radian. The force displacement (cases II, III) and moment rotation (case IV) hysteresis loops for the isolated tower are shown in Fig. 6.

The proposed isolation system is very effective in reducing the large levels of tower acceleration without inducing large displacement, where the isolation devices enters its hysteretic response and activated for energy dissipation once the in-plane input ground motion attain its peak at t = 4.64 sec. The rotation isolators dissipate seismic energy over wider range (4.64 - 23 sec.) of the input ground motion than that of sliding one, which displays short range (4.64 –11 sec.) for energy dissipation as shown in Fig. 7, out of this range, the deformation hysteretic feature of isolation devices almost disappeared, resulting in flat relationship in the dissipation energy time history through the isolation system. Moreover, the sliding isolators exhibit undesirable large ductility, which should be controlled through the rigid stiffness (K_3) tri-linear model after certain ductility factor, but the sharp transition from low strain hardening stiffness (K_2) to rigid stiffness (K_3) of the device model would cause impact effects, leading to high amplification of the tower acceleration response, which are to be avoided by providing smooth transition through gradual stiffness change from K_2 to K_3.

The performance of the proposed energy dissipation control system is analyzed by comparing the energy time history. The input energy is defined as the total energy related to inertia forces induced by the ground motion. It is appeared that the inclusion of these energy dissipation devices system has two effects on the tower response, the first is to change the tower stiffness to be more flexible, resulting in an increase the ability of tower structural system to reflect a portion of earthquake input energy, secondly, it increases the amount of damping and dissipation energy because of the hysteretic properties of the inelastic deformation. The results obtained for this level of isolation devices stiffness show effective energy dissipation for different cases. The position of sliding device at both ends (case III) is more effective for energy dissipation than that in the middle. Moreover, the rotation device type at the end position provides the most effective

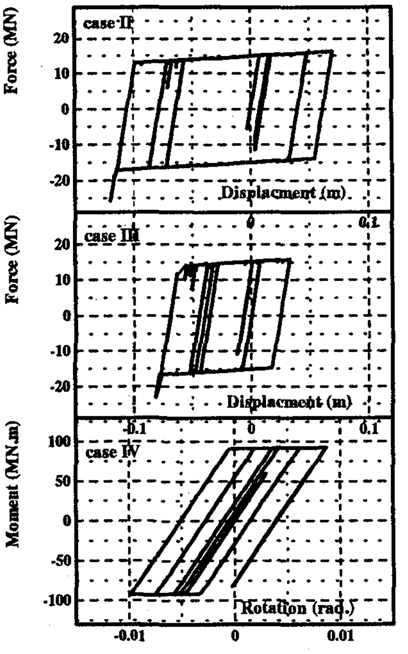


Fig. 6 Force displacement hysteresis loops of high stiffness energy dissipation device (K/K_0 = 0.10)

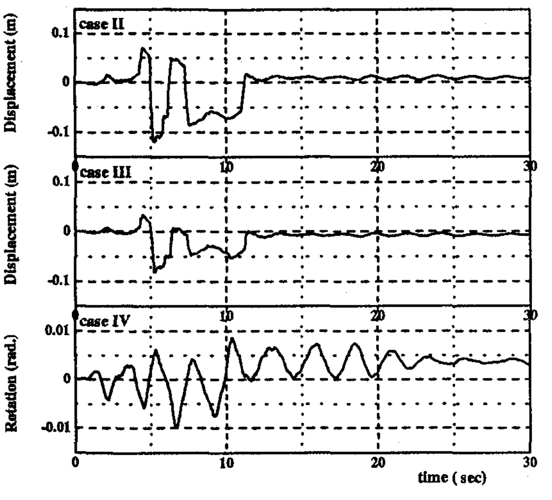


Fig. 7 Displacement time history of high stiffness energy dissipation device system (K/K_0 = 0.10)

energy dissipation and damping relative to the earthquake total input energy as shown in Fig. 8.

The better performance of the isolation proposed isolation system is seen by comparing the reaction force time history at the tower base for different systems of isolation. It is found that the isolated tower provides pronounced reduction in the reaction force response compared to the original tower response (case I), moreover the rotation isolation system displays more effectiveness in reaction force reduction than the sliding one. But the force for the sliding system is quickly damped out after attaining the peak force compared to the rotation system as shown in Figs. 9 and 10. This may be attributed that the sliding isolation device system attains its total dissipation energy within short range; hence the isolated tower structural system gets its full damping quickly affecting the following tower response to quickly damp out. The isolated tower shows no plastic behavior compared with original tower, which shows plastic response with yield moment (M_y) at the tower base and equal to 130 MN·m.

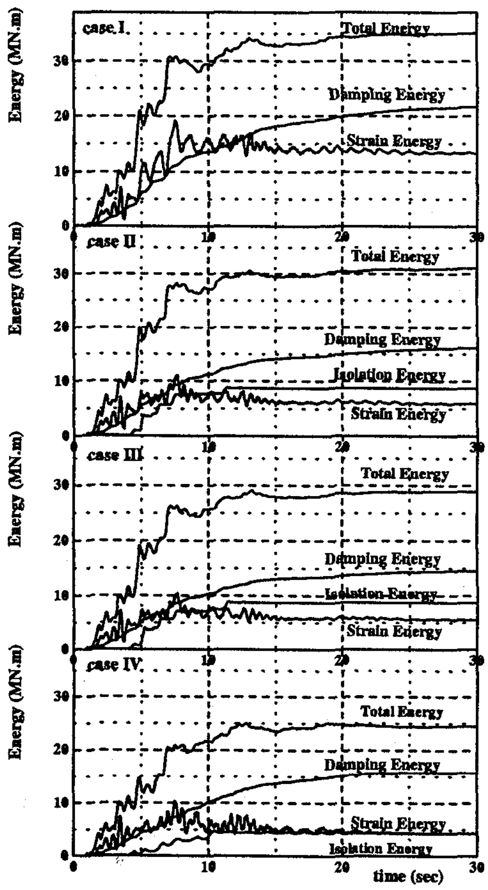


Fig. 8 Energy time history of the whole isolated tower with high stiffness energy dissipation devices system

In general, the isolated tower exhibits elastic response due to the redistribution of the seismic forces to the tower elements in accordance to their strength, but little change in the axial forces except, the sliding isolation system (case III), which displays pronounced reduction in the uplift force at the tower base as in Fig.11. The rotation isolation system (case IV) in characterized by high frequency response that may be attributed to the effect of the U-D motion of the input ground motion is grow up as vibration period has more elongation.

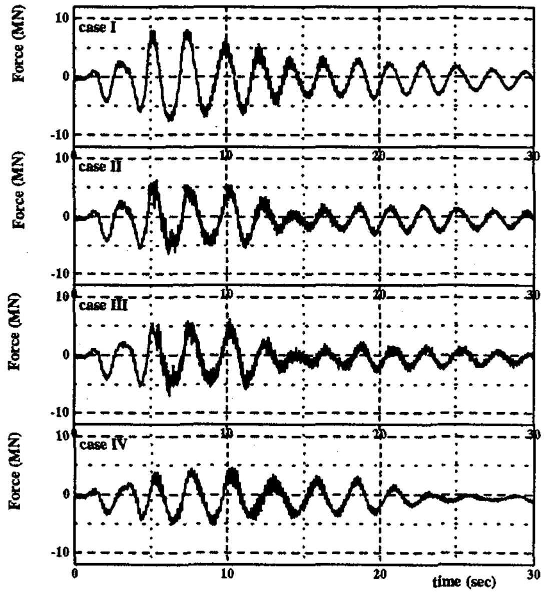


Fig. 9 In-plane force time history at tower base ($K/K_0 = 0.10$)

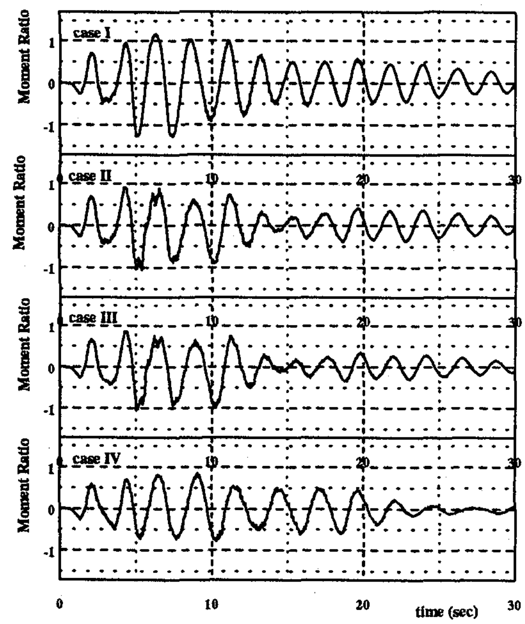


Fig. 10 In-plane moment ratio time history (M/M_y) at tower base ($K/K_0 = 0.10$)

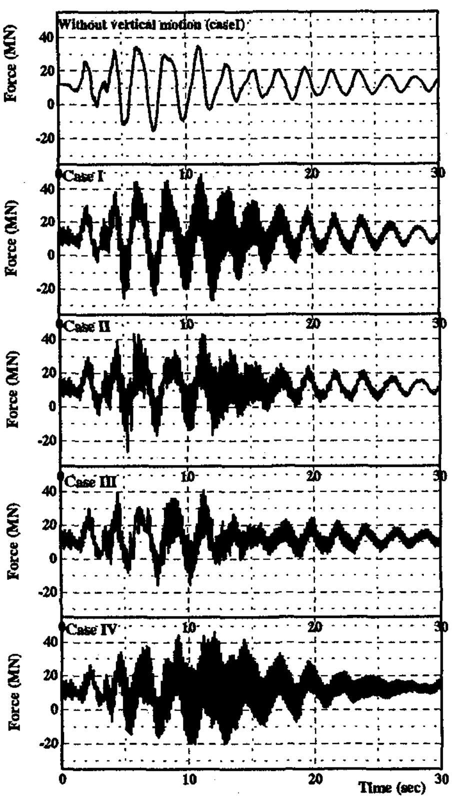


Fig. 11 Vertical force time history at tower base ($K/K_0 = 0.10$)

It can be concluded, that the main effect of the vertical motion is the axial forces generation, which are uncoupled to that due to lateral forces and have a lower vibration period. The axial forces at tower base due to the overturning moments are significant and the contribution of the vertical motion to the total axial force can be comparable, to that of the horizontal motion, as shown in Fig. 11.

(2) Case of medium stiffness ratio $K/K_0 = 0.01$

The designed sliding device has yield displacement level equal to 65, 48 mm for cases II and III respectively, and the rotation device has yield rotation equal to 0.015radian. The hysteresis loops for the energy dissipation system for different cases, as shown in Fig. 12, show bilinear behavior with small ductility factor (less than 2) compared to that of high stiffness device, which provide large ductility (exceed than 20). For this reason, the design of the isolation device as tri-linear model with rigid behavior K_3 is not effective in the isolation system performance. So bilinear model of medium stiffness isolation device is satisfied for efficient enhancement of the tower response. As the isolation system stiffness decreases, the tower structural system exhibits more flexibility, which

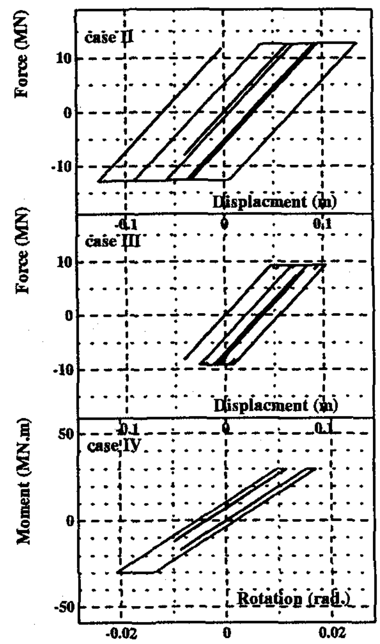


Fig. 12 Force displacement hysteresis loops of medium stiffness energy dissipation device ($K/K_0 = 0.01$)

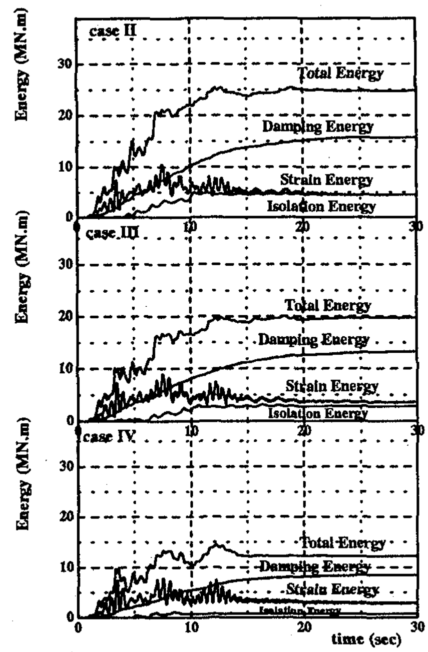


Fig. 13 Energy time history of the whole isolated tower with medium stiffness energy dissipation devices system

modifies the absolute input energy. In effect, the increased flexibility acts as a filter, which reflects a portion of the earthquake energy, as shown in Fig. 13.

The capacity of the energy dissipation system of viscoelastic connections decreases with decrease of its devices stiffness, and also the stress of the tower structure are significantly reduced by the added viscoelastic isolation system during earthquakes. Due to the unsymmetrical configuration of energy dissipation devices for cases III and IV, the compressive and tensile responses of the device are different due tower structural geometric effect; this effect grows up with more flexibility of the isolation system.

The isolation system with medium stiffness level is displays more effective dissipation of the total input energy and minimum strain energy dissipated by the whole tower. The damping energy ratio relative to the input energy increase where its value range about 60% (Case II) and about 75% (case IV), but the efficiency of isolation system for the energy absorption decreases compared with that of high stiffness isolation system. The isolated tower shows elastic behavior and more reduction in the tower base reaction forces and high frequency response due to coupling the U-D component of the input ground motion. Moreover small amplification of tower top displacement through one peak and quickly damped out (case IV), as shown in Figs. 14 and 15.

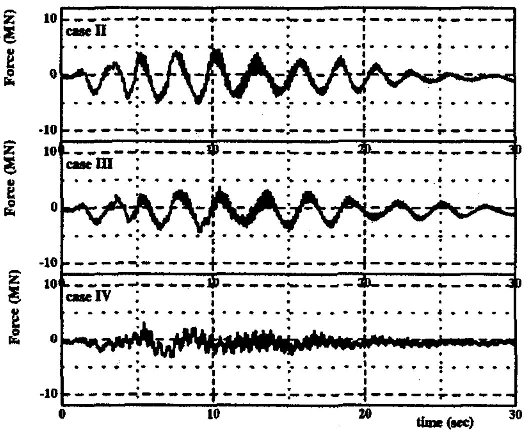


Fig. 14 In-plane force time history at tower base ($K/K_0 = 0.01$)

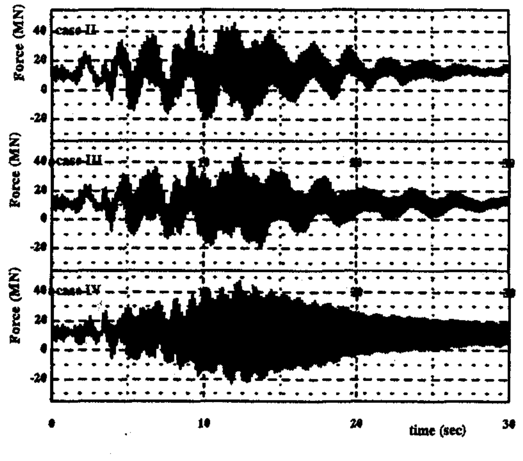


Fig. 15 Vertical force time history at tower base ($K/K_0 = 0.01$)

(3) Case of low stiffness ratio $K/K_0 = 0.001$

It is founded that the isolated tower with bilinear model with different level of yield displacement is not effective for energy dissipation through the isolation system, and for damping through hysteresis of the isolation system. A bilinear model for the isolation device has yield displacement level equal to 140, 80 mm for case II, case III respectively, and yield rotation equal to 0.008 radian is considered for verification of the effectiveness of the isolation system in hysteresis damping and energy absorption. The hysteresis loops for the isolation device for different cases show bilinear behavior of the isolation device, which display low absorbed hysteretic energy, as shown in Fig. 16.

The steel tower with low stiffness isolation system has enough flexibility to reflect most of the absolute earthquake input energy. At this level of flexibility of the tower the three cases of isolation (cases II, III, IV) have the same efficiency for energy dissipation. All cases have minimum strain energy dissipated through the tower structure, maximum damping energy relative to input energy around 80 %, and low dissipated energy through the isolation devices. In this case the passive isolation depends only on the isolated tower flexibility filtering, which reflects the great portion of the earthquake energy as shown in Fig. 17.

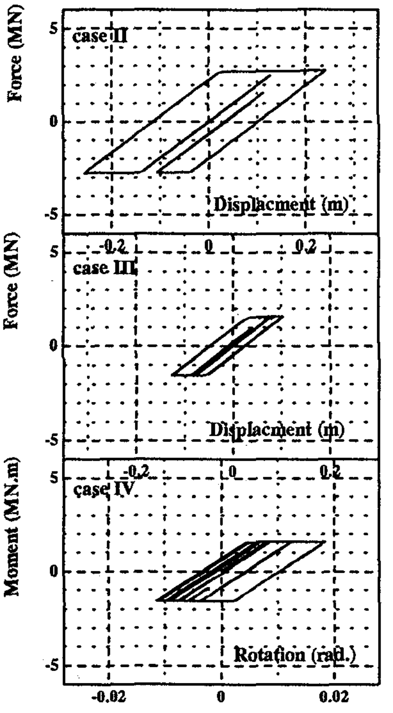


Fig. 16 Force displacement hysteresis loops of low stiffness energy dissipation device ($K/K_0 = 0.001$)

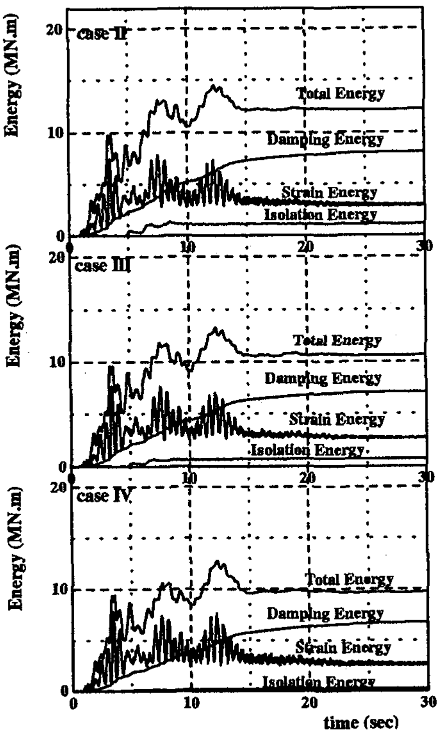


Fig. 17 Energy time history of the whole isolated tower with low stiffness energy dissipation devices system

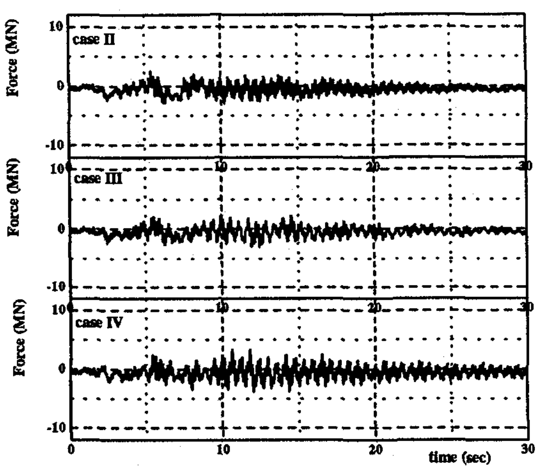


Fig. 18 In-plane force time history at tower base $(K/K_0 = 0.001)$

The isolated tower with more flexible isolation system shows pronounced reduction in the tower base shear reaction, but high frequency response of the tower, this may be due to the effect of vertical ground motion component in generating fluctuating axial forces, which have more pronounced effect as the tower flexibility increases, this effect is appeared as waviness in the force and displacement time history of the isolated tower due to fluctuations in the isolation device force as shown in Figs. 18 and 19.

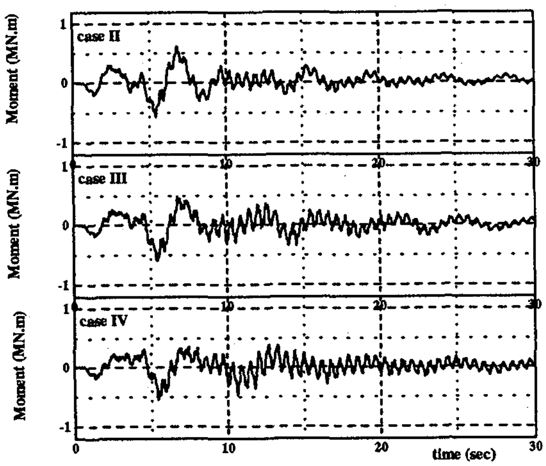


Fig. 19 In-plane moment ratio time history (M/M_y) at tower base $(K/K_0 = 0.001)$

7. Conclusions

Numerical studies of an isolated steel tower of Iwamizawa cable-stayed bridge has been conducted to investigate the dynamic behavior of the isolated tower by the proposed isolation system of viscoelastic connection installed at different positions (middle and both ends) of the horizontal beam of the tower. The isolation device (sliding and rotation) is designed for optimum yield level, for which the isolated tower have maximum damping through the hysteresis loops of the devices, minimum strain energy dissipated through the tower structure and maximum absorbed energy by the proposed isolation devices system.

Nonlinear dynamic behavior of the isolated tower for three case (II, III, IV) of isolation system type and installation position is carried out for three different level of isolation stiffness and compared to non-isolated tower case I. The numerical results demonstrated a substantial reduction of the seismic response of the isolated tower in comparison to the response of the original tower. The following conclusions can be drawn as follow:

(1) It is found that the installation of the sliding isolation device at both ends of the horizontal beam of the tower provides more significant improvement in the seismic behavior than that in the middle position. This may be attributed to the frame action of the steel tower under horizontal excitation of earthquake ground motion, which gives high strain action at the tower joint compare with any other position.

(2) The rotation device is more effective in reducing the tower member forces and increasing the energy dissipation than the sliding device, due to that the flexural effect is more pronounced compared with the shear effect on relative deformation of the isolation device.

(3) For high stiffness ratio (K/K_0 = 0.10) of isolation system, tri-linear model should be provided for energy dissipation through input energy reflecting (K_1), increase damping through hyseresis loops (K_2), and control ductility and relative deformation of the isolation device (K_3). For moderate stiffness ratio (K/K_0 = 0.01) of the isolation system, bilinear in satisfied for energy dissipation, since the ductility factor no exceed 2. For low stiffness ratio (K/K_0 =0.001) of the isolation system, the flexibility of tower structural system acts as filter, which has the ability to reflect most of the absolute earthquake energy.

(4) As the stiffness ratio (K/K_0) of the isolated system decrease, the effect of the high frequency U-D ground motion component upon the tower response became more pronounced, this effect is appeared as waviness in the force and displacement time history of the isolated tower due to fluctuations in the isolation device force.

(5) The efficiency of the isolation system in energy absorption through the isolation devices system decreases with stiffness ratio decrease, on the other side the fundamental natural periods go longer, which has the effect to reduce acceleration response spectra of the tower.

(6) A significant axial force fluctuations as a result of vertical inertia forces of the vertical motion affects the tower behavior, moreover, the axial force tower response has high frequency that related to the frequency content of the vertical motion, which was noticed to be significantly higher than that of the horizontal motion.

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