SEISMIC RESPONSE ANALYSIS OF CABLE-STAYED BRIDGE TOWERS WITH ISOLATED BASE

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ABSTRACT :

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 control have been growing in acceptance as design alternative for earthquake hazard mitigation. The control systems can be used as base isolators to achieve different objectives or performance goals ranging from a life safety standard to a higher standard that would provide damage control. In this study, an isolated base connection modeling is presented in order to enhance seismic behavior of bridge towers by using viscoelastic material. The material and geometrical nonlinear behavior of the base plate is considered. Nonlinear contact of the bridge towers with the base plate and concrete foundation has been demonstrated and its implementation in a finite element nonlinear analysis is presented. A time domain finite element methodology for nonlinear dynamic analysis problem is developed. That accounts for the isolated bridge towers and the inelastic behavior of base plate through an incremental-iterative procedure. A representative problem of cable-stayed bridge towers subjected to a strong ground motion is analyzed. The study shows how the proposed isolated connection enhances the seismic behavior of the bridge towers.

KEYWORDS: Cable-stayed bridge towers, isolated base, nonlinear response, base connection

1. NTRODUCTION

Nonlinear rocking response analyses of structures have been researched by many researchers [1-6]. It is considered that the base connection between columns and foundation is one of the most important elements in steel frame structures, it has a great influence on the entire structural behavior, and the nonlinearities that arise due to the particular behavior of the connections components make the problem of the analysis of these structures very difficult. Hence many damages have been reported at column bases during past earthquakes. 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 control have been growing in acceptance as design alternative for earthquake hazard mitigation for various structures. The connections are designed to dissipate a large portion of the earthquake input energy in connection details that deform and yield during an earthquake. The most important common features of such connections are a shift in the natural period of the structure to a longer value, and an increase in structural damping. In general, such systems are characterized by a capability to enhance energy dissipation in the structural systems 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.

Consideration of rocking or uplift of the bridge pier foundation introduces other modes of nonlinearity (rocking) and energy dissipation. Limited rocking can reduce demands on the bridge structure, effectively acting as an isolation mechanism. The consideration of rocking as an acceptable mode of response can impact design costs by reducing the required footing size. In addition, the simultaneous rocking of a properly designed foundation and flexural deformation of the supported column is expected to eliminate or substantially reduce damage in the column and residual displacements in the bridge following a major earthquake. Controlled rocking approach to seismic resistance allows uplifting of base plates at the foundation while displacement-based steel yielding devices (anchor bolts) are implemented at the uplifting location to control the rocking response. Allowing uplift effectively increases the structure period of vibration, hence the controlled rocking system has an inherent restoring force that allows for base connection self-centering following a seismic event [5]. Many previous studies have investigated the benefits of allowing a column and footing system to uplift. Analytic studies of bridge column response to one horizontal earthquake component have illustrated the combined effects of rocking and column flexural displacements [4].

The Hyogoken Nanbu earthquake of January 1995 in Japan caused serious damage in a large number of steel structures. One of the most observed and important reasons that have been registered is the failure of the column bases, resulting in such damage. Column bases of steel frames are often designed as base plates welded at the bottom end of the column and fixed by anchor bolts embedded in the concrete [6]. The performance of the base connection depends on the cyclic performance of its components. Much research work is needed in order to better understand the seismic behavior and to formulate improved design procedures. The steel frames behavior could only be accomplished through nonlinear dynamic analyses of complete frame systems with actual support condition. In this study, the nonlinear dynamic behavior and seismic performance of the steel tower of cable-stayed bridge under three-dimensional great earthquake ground motion are studied analytically. The steel tower is studied for three different base connection cases, case of traditional base connection, case of base connection including rocking material and case of base connection with rocking material and anchor bolts. A three dimensional nonlinear dynamic analysis study of the steel tower with the proposed connection is carried out and compared to the response obtained for the tower with its original configuration.

2. EQUATION SOLUTION

Based on the total incremental equilibrium equations, finite displacement three-dimensional beam-column element formulation is carried out. 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 [7-8] can be obtained as:

$$[M]\{\ddot{u}\}^{t+\Delta t}+[C]\{\dot{u}\}^{t+\Delta t}+[K]^{t+\Delta t}\{\Delta u\}^{t+\Delta t} = \{F\}^{t+\Delta t}-\{F\}^{t} \quad (2.1)$$

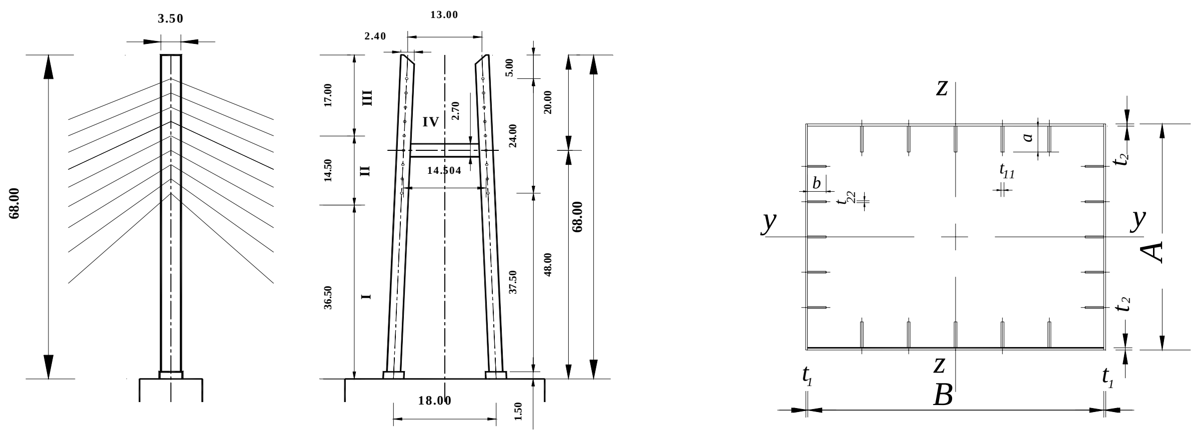
where [M], [C], and $[K]^{t+\Delta t}$ are the system mass, damping and tangent stiffness matrices at time $t+\Delta t$, the tangent stiffness considers the material nonlinearities through bilinear stress strain relation for the beam column element, and the geometrical nonlinearities for the case of 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, $\{F\}^{t+\Delta t}$ - $\{F\}^{t}$ 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 tangent stiffness and internal forces calculations. The implicit Newmark step-by-step integration method is used to directly integrate the equation of motion and then it is solved for the incremental displacement using the Newton Raphson iteration method where the stiffness matrix is updated at each increment to consider the geometrical and material nonlinearities.

3. FINITE ELEMENT OUTLINE

3.1 Tower Structure Model

The steel tower of Tappu cable-stayed bridge located in Hokkaido, Japan is considered. The steel tower is taken out of the bridge and modeled as three-dimensional frame structure. A fiber flexural element is developed for characterization of the tower structure, in which the element incorporates both geometric and material nonlinearities. The stress-strain relationship of the beam element is modeled as bilinear type with kinematic strain hardening rule. The yield stress and the modulus of elasticity are equal to 355 MPa (SM490Y) and 200GPa, respectively, the plastic region strain hardening 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. This tower has nine cables in each side; 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 tower is shown in Fig. 1.



(a) Tower geometry (b) Cross section

Fig. 1 Steel tower of cable-stayed bridge

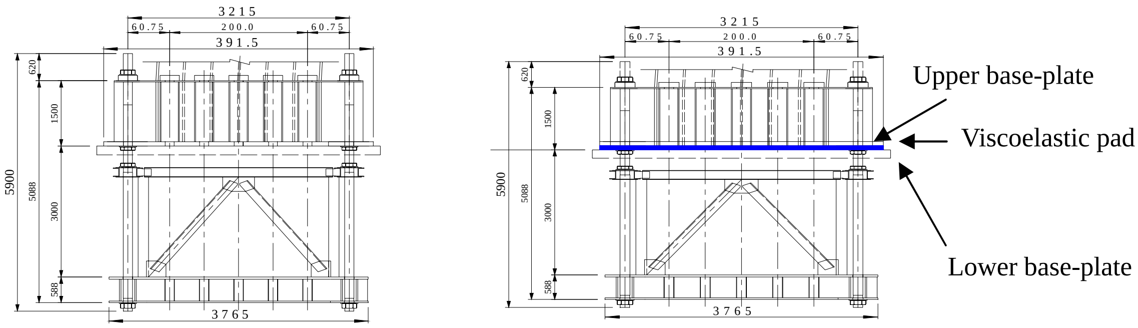
Table 1 Cross section dimensions the tower (cm)

C. S. Dim.		Outer dimension				Stiffener dimension			
		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

This tower has rectangular hollow steel section with internal stiffeners, which has different dimensions along tower height and its horizontal beam. The geometrical properties of the tower are summarized in Table 1. A spectral damping scheme of Rayleigh's damping is used to form damping matrix as a linear combination of mass and stiffness matrices, which effectively captures the tower structures damping and is also computationally efficient. The damping ratio corresponding to the frequencies of the fundamental in-plane and out-plane modes of tower free vibration is set to 2%.

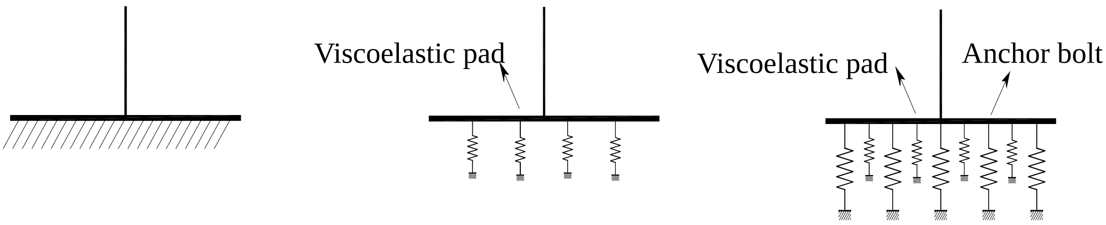
3.2 Proposed Base Connection Modeling

The existing column base connection consists of 24 anchor bolts arranged outside the tower leg flanges. Additional weld is used through welding vertical plates to the flange and lengthening the anchor bolts. A proposed base connection system with viscoelastic material is introduced for the tower foundation connections in the tower frame structure, as shown in Fig. 2. By using this material, it is possible to increase the energy capabilities of the tower structure while reducing its accelerations and inertial forces. The base connection with viscoelastic material behavior is simulated by transitional structural system. The proposed base connection 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. The connection profiles of viscoelastic material is suggested to be installed in-between the two base plates to reduce the structural response, allow the tower rocking and reduce energy dissipation demands. The tower is welded to the upper base plate and the upper base plate is connected to the lower base plate with bolts. The viscoelastic material and the anchor bolts in the connection are model by nonlinear springs. Then each of these springs is added to the system and its stiffness is assembled into the final overall stiffness of the connection. Figure 3 shows mechanical model of the base connection. In this model, the constitutive components of the base connection are represented by means of springs system that includes extensional springs to simulate the anchor bolts tension deformation and extensional springs to simulate the viscoelastic material behavior between the tow base plates.



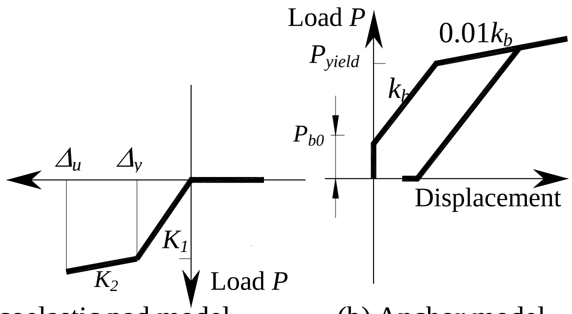
(a) Existing base connection (b) Proposed base connection

Fig. 2 Details of steel tower base connection (mm)



(a) Fixed base model (b) Viscoelastic pad model (c) Pad with anchor model

Fig. 3 Tower base connection mechanical model



(a) Viscoelastic pad model (b) Anchor model

Fig. 4 Base connection components numerical models

Table 2 Base connection components properties

Description	Properties value	
Anchor bolt	E_b	205.80 GPa
	Stiffness k_b	201.2 MN/m
	Yield force / stress	1.93 MN/ 440 MPa
	Pre-tension force	0.81 MN
	Shear stress	251.4 MPa
	Steel / Diameter	S45C / M80
Base plate	Steel	SM490Y

The stiffness and resistance properties of the anchor bolt springs are calculated on the basis of the geometrical characteristics and the mechanical properties of the nonlinear constitutive material as given in Table 2. While the principal parameters of the viscoelastic material are the yield force P_y, the initial stiffness K_1, and maximum displacement, which can be described by the ductility factor $\mu = \Delta u / \Delta y$ and hardening stiffness K_2 as illustrated in Fig. 4. The nonlinear behavior of bolts is accounted for by an iterative numerical procedure. As for the rocking mechanism, the viscoelastic material model is assumed to have no tension resistance. The steel bolt plays little or no part in the compressive behavior of concrete, thus the bolts in compression are neglected.

4. NUMERICAL RESULTS AND DISCUSSION

Structures that are anchored to a base foundation exhibits two distinct capacities to resist uplift and eventual overturning: (a) the restrainers or anchors are strong enough and the structure engages the base foundation into rocking motion; and (b) the anchors pull out or fracture in tension and the structure subsequently rocks as a freestanding block atop its foundation base. The base connection exhibits the most unpredictable semi-rigid behavior, this is due to the fact that their basic components base plate; anchor bolts and concrete foundation are made from different material types, transmitting acting forces through unilateral multi-body contact. Thus, an exact evaluation of the response of such connections can only be achieved through the corresponding analysis of sophisticated 3D finite element numerical simulations. As bridge towers rock with its foundation base due to presence of steel anchors, the anchor bolt stiffness and pre-tension force should be enough to prevent the rocking which lead to overturning, in the other hand the rocking of base foundation keep the bridge towers more flexible to resist earthquake excitation. Hence to achieve an optimum behavior of the present problem, a finite element analysis of steel tower taking into account a more realistic model for the support condition including the rocking of the bridge towers with its base foundation and a comparison to support condition modeling of enhanced rocking of the bridge towers is presented, three different cases are studied as follow:
Case 1: Fixed base connection, Fig. 3 (a).
Case 2: Base connection with viscoelastic pad, Fig. 3 (b).

Case 3: Base connection with viscoelastic bad and engaged by anchor bolts, Fig. 3 (c).
An examination of the bridge tower response under near-fault ground motion records has been carried out. A suite of recorded and simulated standard ground motion records is used for the nonlinear time history analysis. The near-fault ground motion records [9, 10] obtained during the 1995 Hyogoken-Nanbu earthquake (M7.2), including three-component acceleration time histories recorded at JR Takatori is used.

The performance of the presented system technique that proposed for the tower structure is qualitatively examined by comparing the displacement time history of tower top during a strong near-field earthquake as illustrated in Fig. 5. The viscoelastic pad connection effect depends on its ability of natural period elongation and amount of damping through pad material hysteresis. It can be observed that the tower top displacement responses significantly decrease and the tower seismic response is highly damped free vibration response with longer natural vibration, significantly low amplitude and less residual plastic displacement drift. The allowable bridge towers rocking can affect the tower top displacement global response, where its contribution in the maximum displacement reaches about 18 % of that of the rocking with base foundation model. The displacement time history at the tower top with the proposed base connection model shows the response nature to strong excitation. The amplitude of displacement pronounced decrease by using the proposed base connection for different studied cases and decays rapidly after the peak excursions; the proposed base connection has the capabilities of reducing displacement ductility demands. To control such large displacement, the anchor bolts are used. The presence of the anchor bolts in base connection can control the high displacements amplitude, and natural period increasing.

The rocking tower exhibits elastic response due to the redistribution of the seismic forces to the tower elements in accordance to their strength, in addition a large change in the axial forces in the rocking system, which displays pronounced decreasing in the uplift force at the tower base. The axial forces response, Fig. 6, is characterized by long period frequency response with high amplitude and spike at many times, that may be attributed to the effect of the vertically allowed motion due to towers rocking. It can be concluded that the main effect of the bridge towers rocking system is the axial forces damping, which are strongly affected the foundation base plate lift-off. The presence of anchor bolts in the connection has a main objective to give more control to the bridge tower to lift up. That can be observed in reduction of the negative reaction in case of proposed connection model in case of presence of anchor bolt and in case of presence of viscoelastic material only in the model.

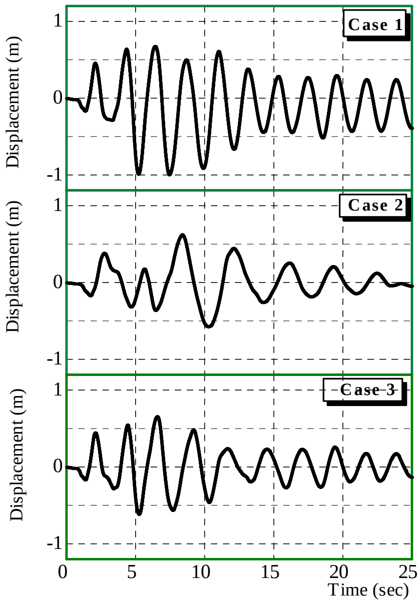


Fig. 5 Displacement time history at tower top

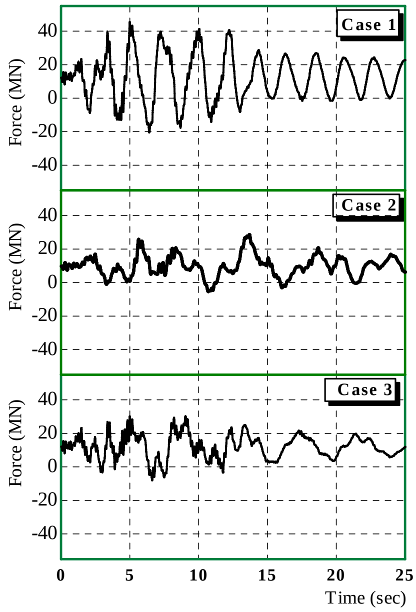


Fig. 6 Vertical force time history at tower base

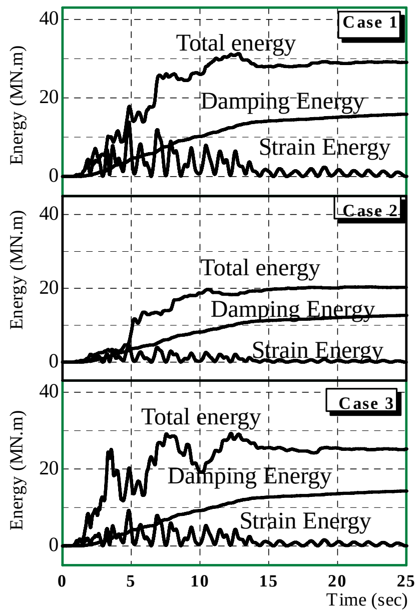


Fig. 7 Input energy to the tower

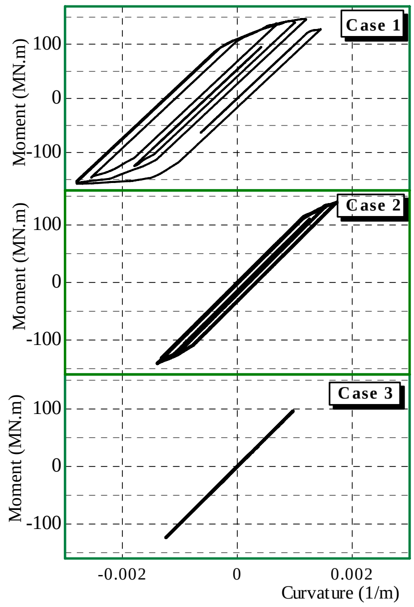


Fig. 8 Moment curvature relationship

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 bridge towers rocking with the viscoelastic material has a great effects on the tower response that 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. The results obtained, Fig. 7, show that the maximum input energy is 32 MN.m, which decrease to 22 MN.m when the bridge towers rocking considered. Hence, the bridge towers rocking contribute by more than 30 % dissipating the total input energy. This can be attributed to that the absorbed energy through the viscoelastic material. The Anchor bolts has an effective in dissipating some energy in the base connection that the energy input to the tower has a little decease in its amount although it finally less than that in fixed base connection . The new model allowing bridge towers rocking is more affective in moderate input ground motion than strong one that the absorption of input energy becomes less in the first case, this is may attributed to the viscoelastic pad material absorb a large part of the input energy through its hysteric loop. If the bridge tower rocking occurs, the plastic deformation of the column elements decreases as a result of the softening of the moment curvature hysteresis relationship. Figure 8 shows the moment curvature hysteresis relationship of the bridge tower at the base element. More concentration of inelastic behavior and ductility at tower base element is attained as the viscoelastic material presence. The base elements of the tower approaches elastic behavior in case of including the anchor bolts in connection, thus there is a possibility of eliminating permanent damage and minimizing the extent of retrofit. It appears the more cycling of hysteric load behavior in case of considering the viscoelastic material only where absorbing more portions in total energy to the tower.

5. CONCLUSIONS

Numerical parametric study of the steel tower cable-stayed bridge has been conducted to study the dynamic behavior considering bridge towers rocking at tower base connection. A base plate connection is proposed with viscoelastic material to provide more rocking for the tower. A finite element program based on the fiber model theory for the nonlinear dynamic analysis of steel tower under static and dynamic loadings of great earthquake ground motion is developed, considering material nonlinearity. Moreover the bending-axial force interaction is automatically considered. An elastic-plastic constitutive model incorporating uni-axial yield surface criteria and kinematic strain hardening flow rule for the material nonlinearity is implemented. The nonlinear finite element dynamic analysis demonstrates how bridge towers rocking influence tower seismic response. From this study, the following conclusions can be drawn as follow:

1) The proposed base connections with viscoelastic material is so effective in providing safety rocking for the bridge tower and elongate its natural frequency.
2) The bridge towers rocking model can affect the tower top displacement response, where its contribution in the maximum displacement reaches about 30 % of that of the bridge towers rocking with its base foundation.
3) The bridge including the bridge towers rocking provides pronounced reduction in the reaction force and moment responses compared to that rocking with its base foundation.

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