Earthquakes and Structures, *Vol. 5, No. 6 (2013) 657-678*
DOI: http://dx.doi.org/10.12989/eas.2013.5.6.657

Energy dissipation system for earthquake protection of cable-stayed bridge towers

Shehata E. Abdel Raheem[*1,2] and Toshiro Hayashikawa[3]

[1]Taibah University, Madinah Munawarh, KSA
[2]Faculty of Engineering, Assiut University, Egypt
[3]Graduate School of Engineering, Hokkaido University, Sapporo, Japan

(Received February 17, 2013, Revised September 4, 2013, Accepted September 10, 2013)

Abstract. For economical earthquake resistant design of cable-stayed bridge tower, the use of energy dissipation systems for the earthquake protection of steel structures represents an alternative seismic design method where the tower structure could be constructed to dissipate a large amount of earthquake input energy through inelastic deformations in certain positions, which could be easily retrofitted after damage. The design of energy dissipation systems for bridges could be achieved as the result of two conflicting requirements: no damage under serviceability limit state load condition and maximum dissipation under ultimate limit state load condition. A new concept for cable-stayed bridge tower seismic design that incorporates sacrificial link scheme of low yield point steel horizontal beam is introduced to enable the tower frame structure to remain elastic under large seismic excitation. A nonlinear dynamic analysis for the tower model with the proposed energy dissipation systems is carried out and compared to the response obtained for the tower with its original configuration. The improvement in seismic performance of the tower with supplemental passive energy dissipation system has been measured in terms of the reduction achieved in different response quantities. Obtained results show that the proposed energy dissipation system of low yield point steel seismic link could strongly enhance the seismic performance of the tower structure where the tower and the overall bridge demands are significantly reduced. Low yield point steel seismic link effectively reduces the damage of main structural members under earthquake loading as seismic link yield level decreases due their exceptional behavior as well as its ability to undergo early plastic deformations achieving the concentration of inelastic deformation at tower horizontal beam.

Keywords: cable-stayed bridge towers; energy dissipation damper; seismic response; time history analysis; passive control; low yield steel

1. Introduction

The recent trend for cable-stayed bridges is to use high strength materials and, therefore, shallower or more slender stiffening girders combined with the rapid increase in span length. This structural synthesis provides a valuable environment for the nonlinear behavior due to material nonlinearity and geometrical nonlinearity of the structure relatively large deflection on the stresses and forces (Ali and Abdel-Ghaffar 1995, Hayashikawa and Abdel Raheem 2002). The Northridge

*Corresponding author, Associate Professor, E-mail: shehataraheem@yahoo.com

http://www.techno-press.org/?journal=eas&subpage=7 ISSN: 2092-7614 (Print), 2092-7622 (Online)

1994 and Hyogoken-Nanbu 1995 earthquakes led to an increased awareness concerning the response of bridge structures subjected to earthquake ground motions (Moehle 1995, Committee of Earthquake Engineering 1996, Japan Road Association 1996). The necessity has arisen to develop more efficient analysis procedures that can lead to a comprehensive understanding and a realistic prediction of the seismic response of bridge structural systems to improve the bridges seismic performance. The traditional approach to seismic hazard mitigation is to design structures with sufficient strength capacity and able 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.

The development of capacity design principles in New Zealand in the 1970's (Park and Paulay, 1976) was an expression of the realization that the distribution of strength through a structure is more important than the absolute value of the design base shear. It was recognized that a frame building would perform better under seismic attack if it could be assured that plastic hinges would occur in the beams rather than in the columns (weak beam/strong column mechanism). Drawing on the experience with the repair of buildings after the 1995 Kobe earthquake, Otani (1997) noted that although frame buildings designed in accordance with the weak-beam/strong-column philosophy survived the earthquake without collapse, the cost of repairing the many locations of inelastic action, and hence localized damage, was often excessive and uneconomic. Alternative structural systems with fewer locations of inelastic action as might occur in structural buildings were more economical in terms of repair costs. The formulation of design guidelines and building code requirements for structural implementation of energy dissipation devices has been significant in promoting the use of this emerging technology (Whittaker *et al.* 1993, FEMA 1997). Passive energy dissipation systems encompass a range of materials for enhancing damping, stiffness, and strength can be used for both natural hazard mitigation and upgrading structural performances. Such systems are characterized by a capability to enhance energy dissipation in the structural systems to which they are installed to achieve different performance goals ranging from a life safety standard to a higher standard that would provide damage control and post-earthquake functionality.

The design of bridge energy dissipation system is achieved as the result of two conflicting requirements: no damage under serviceability limit state load condition and maximum dissipation under ultimate limit state load condition. Extensive research has been conducted to develop robust steel energy dissipation device system for energy absorption in the structure during an earthquake through the inelastic deformation of metallic devices (Soong and Dargush 1997, Soong and Spencer 2002, Sahoo and Rai 2010). The ADAS (Added Damping and Stiffness) and TPEA (Triangular Plate Energy Absorbers) devices incorporate X-shaped or triangular steel plates respectively to spread the yielding uniformly throughout the material, where Flexure yielding of steel damper of triangular shapes has been used to maximize energy dissipation potential (Bergman and Goel 1987, Whittaker *et al.* 1991, Tsai *et al.* 1993, Ghabraie *et al.* 2010, Benavent-Climent *et al.* 2011, Karavasilis *et al.* 2012). Solid and slit webs of steel sections have also yielded in shear and act as a damper under lateral loads (Chan and Albermani 2008, Chan *et al.* 2009, Zhengying *et al*. 2011). A few studies have also been carried out on low yielding steel shear panels utilizing their shear deformations as means to dissipate the energy (Nakashima *et al.* 1996, Chou and Tsai 2002). Dusicka *et al.* (2002, 2004) investigated built-up shear links constructed using plates of different grades of steel including high performance steels (HPS) as well as Japanese Low Yield Point steels (LYP). These different grades of steel provided a range of nominal yield strengths from 100 MPa to 485 MPa. The LYP steel in particular allowed for innovative designs of compact shear links without stiffeners. The shear yielding of low yielding alloy metals, such as aluminum, has been found to be very ductile and can undergo large inelastic deformations without tearing or buckling. The yielding in shear mode maximizes the material participating in plastic deformation without excessive localized strains. Regarding this, I-shaped shear-links made of low yield ductile Aluminum alloys have been found to be excellent energy dissipative devices limiting the energy dissipation demand on structural members of the primary structure (Rai and Wallace 1998, Matteis *et al*. 2007, Sahoo and Rai 2009). Shaking table tests were conducted to evaluate the load resistance mechanism, failure/damage pattern, and the hysteretic behavior of shear-link systems and to provide data for developing suitable design procedures for proportioning various elements of the overall system (Rai *et al.* 2013). Elasto-plastic characteristics of shear panel dampers as passive energy dissipation and the applicability of the shear panel dampers to achieve a rational seismic retrofit of long span bridges against large-scale Level 2 earthquakes were investigated experimentally and analytically (Vargas and Bruneau 2007, Sugioka *et al.* 2011, Zhengying *et al.* 2011).

Recently, some applications based on the use of special LYP steel have been proposed as hysteretic dampers (Nakashima *et al.* 1994, Matteis *et al.* 2003, McDaniel and Seible 2004) due to their exceptional hysteretic behavior as well as their ability to undergo early plastic deformations. A design strategy of passive control technique is adopted. In this strategy, an effective energy dissipation concept is suggested by providing a typical concentration of inelastic behavior at tower horizontal beam using low relative strength and stiffness through insertion of low yield point steel material instead of cross section dimensions reduction. Since the horizontal beam is easy to inspect and repair if necessary, the rest of the structure will remain elastic, thus; eliminating permanent damage and minimizing the extent of retrofit. The main objective of this research is, thus, to formulate a general framework for the optimal design of passively steel energy dissipation system for seismic structural applications. The structural performance of the proposed energy dissipation system is investigated to quantify the effectiveness of the design strategy of passive control technique at reducing, possibly eliminating plastic deformations of tower primary structures under strong earthquakes. For this purpose, a nonlinear dynamic analysis investigation of the tower model with the proposed energy dissipation systems is carried out and compared to the response obtained for the reference tower design with its original configuration. The seismic performance improvement of the tower with supplemental passive energy dissipation system has been measured in terms of the reduction achieved in different response quantities. The calculated results clarify the effectiveness of the proposed energy dissipation system in reducing structural elements forces and control tower ductility demand to the primary structure for economical earthquake resistant design. The considered design procedure is really effective and convenient; low-yield steel panel provides an apparent reduction of tower drift and damage level of the primary structure.

2. Finite element analysis procedure

2.1 Three-dimensional beam element tangent stiffness

A nonlinear dynamic finite element technique is developed to analyze the elasto-plastic dynamic response of frame structures under strong earthquake excitation. A nonlinear beam element of six degrees of freedom (6-dofs) at each node is formulated according to the geometrical nonlinear beam theory, where all couplings among bending, twisting and stretching deformations for beam element are considered. The element nodal displacement vector in local coordinate system is given by

$$\mathbf{d} = \left\{ \bar{u}_i \quad \bar{v}_i \quad \bar{w}_i \quad \bar{\theta}_{xi} \quad \bar{\theta}_{yi} \quad \bar{\theta}_{zi} \quad \bar{u}_j \quad \bar{v}_j \quad \bar{w}_j \quad \bar{\theta}_{xj} \quad \bar{\theta}_{yj} \quad \bar{\theta}_{zj} \right\}^T \tag{1}$$

The displacement u, v, and w of a general point (x, y and z) in the beam can be written as

$$u = \bar{u} - y \cdot d\bar{v}/dx - z \cdot d\bar{w}/dx, \quad v = \bar{v}, \quad w = \bar{w} \tag{2}$$

where $\bar{u}$, $\bar{v}$ **,** and $\bar{w}$ are the displacements of a point in the centroid axis of a beam corresponding to x-, y- and z- axes, respectively. $\bar{\theta}$ is the cross section rotation about x axis. The displacement transformation matrix of a beam element relates the element internal displacement to the nodal point displacement

$$\bar{\mathbf{u}} = \left\{ \bar{u} \;\; \bar{v} \;\; \bar{w} \;\; \bar{\theta} \right\}^T = \left[\bar{\mathbf{N}}_x \; \bar{\mathbf{N}}_y \; \bar{\mathbf{N}}_z \; \bar{\mathbf{N}}_\theta \right]^T \mathbf{d} \tag{3}$$

$$\begin{aligned}
\bar{\mathbf{N}}_x &= [N_1\, 0 \;\; 0 \;\; 0 \quad 0 \;\; 0 \;\; N_2 \;\; 0 \;\; 0 \;\; 0 \quad 0 \;\; 0] \\
\bar{\mathbf{N}}_y &= [0 \;\; N_3\, 0 \;\; 0 \quad 0 \;\; N_4\, 0 \;\; N_5\, 0 \;\; 0 \quad 0 \;\; N_6] \\
\bar{\mathbf{N}}_z &= [0 \;\; 0 \;\; N_3 \;\; 0 \; -N_4\, 0 \;\; 0 \quad 0 \;\; N_5\, 0 \; -N_6 \;\; 0] \\
\bar{\mathbf{N}}_\theta &= [0 \;\; 0 \;\; 0 \;\; N_1 \quad 0 \;\; 0 \;\; 0 \quad 0 \;\; 0 \;\; N_2 \quad 0 \;\; 0]
\end{aligned} \tag{4}$$

The shape functions N_i (i = 1-6) for the local element displacements are given as follow

$$\begin{aligned}
&N_1 = 1 - \zeta, \quad N_2 = \zeta \;, \quad N_3 = 1 - 3\zeta^2 + 2\zeta^3, \quad \zeta = x/l \\
&N_3 = (\zeta - 2\zeta^2 + \zeta^3)l, \quad N_5 = 3\zeta^2 - 2\zeta^3, \quad N_6 = (-\zeta^2 + \zeta^3)l
\end{aligned} \tag{5}$$

in which l is the original length of the beam element.

$$\frac{d\bar{u}}{dx} = \mathbf{B}_x\mathbf{d} \;, \; \frac{d^2\bar{v}}{dx} = \mathbf{B}_2\mathbf{d} , \; \frac{d^2\bar{w}}{dx} = \mathbf{B}_3\mathbf{d} , \; \frac{d\bar{v}}{dx} = \mathbf{B}_4\mathbf{d} , \; \frac{d\bar{w}}{dx} = \mathbf{B}_5\mathbf{d} \;, \; \frac{d\bar{\theta}}{dx} = \mathbf{B}_6\mathbf{d} \tag{6}$$

The strain in the deformed configuration ε_t can be expressed in terms of the displacement at the equilibrium state at any specified point in the beam cross section (x, y and z) that define the state of strain at any arbitrary point. The strain displacement equation (Green-Lagrange representation of axial strain ε_t) is given in the following

$$\varepsilon_t = \frac{d\,u_t}{dx} + \frac{1}{2}\left(\frac{d\,u_t}{dx}\right)^2 + \frac{1}{2}\left(\frac{d\,v_t}{dx}\right)^2 + \frac{1}{2}\left(\frac{d\,w_t}{dx}\right)^2 \tag{7}$$

where u_t , v_t and w_t are the element displacement vectors in x-, y- and z-axis directions, respectively. It is assumed that $d\,u_t/dx$ << 1, thus; $(d\,u_t/dx)^2$ is ignored compared with its linear term. Then the normal strain ε_t is calculated. The unknown strain in t +1 configuration can be written incrementally from configuration t as follows

$$\varepsilon_{t+1} = \frac{d\,u_{t+1}}{dx} + \frac{1}{2}(\frac{d\,v_{t+1}}{dx})^2 + \frac{1}{2}(\frac{d\,w_{t+1}}{dx})^2 \tag{8}$$

$$\delta\,\varepsilon_{t+1} = \frac{d\delta\,u}{dx} + \frac{d\,v_t}{dx}\cdot\frac{d\,\delta\,v}{dx} + \frac{d\,\mathrm{w}_t}{dx}\cdot\frac{d\,\delta\,\mathrm{w}}{dx} + \frac{1}{2}\delta(\frac{d\,\mathrm{v}}{dx})^2 + \frac{1}{2}\delta(\frac{d\,w}{dx})^2 = (\mathbf{B}_{\mathrm{L}} + \mathbf{B}_{\mathrm{NL}})\delta\mathbf{d} \tag{9}$$

$$\mathbf{B}_{\mathrm{L}} = \mathbf{B}_1 - y\mathbf{B}_2 - z\mathbf{B}_3 \quad , \quad \mathbf{B}_{\mathrm{NL}} = (\mathbf{B}_4^T\mathbf{B}_4 + \mathbf{B}_5^T\mathbf{B}_5)/2 \quad , \quad \mathbf{B}_1 = \mathbf{B}_x + \frac{dv_t}{dx}\mathbf{B}_4 + \frac{dw_t}{dx}\mathbf{B}_5 \tag{10}$$

From the principle of energy, the external work is equivalent to the internal work. The equilibrium equation in the deformed configuration $t+1$ can be expressed in terms of the principle of virtual displacements

$$\delta U - \delta W = 0 \tag{11}$$

$$\int_V \sigma_{t+1}\,\delta\,\varepsilon_{t+1}\,dV + \int_l T_{sv}\cdot(d\theta/dx)dx + \frac{1}{2}\int_l GJ(d\theta/dx)^2\,dx - \int_S f_{t+1}^T\,\delta\,\mathbf{d}_{t+1}\,dS = 0 \tag{12}$$

where U = internal work, W = external virtual work, σ_{t+1} = axial Couchy stress, V = volume, δ = virtual quantity, f = external applied force, and θ is the cross section rotation about x axis. $t+1$ is unknown configuration which can be written incrementally from configuration t consistently with a nonlinear Lagrangian scheme as

$$\int_V (\sigma_t + \sigma)\,\delta\,\varepsilon_{t+1}\,dV + \int_l T_{sv}\,\frac{d\theta}{dx}dx + \frac{1}{2}\int_l GJ(\frac{d\theta}{dx})^2\,dx - \int_S (f_t^T + f^T)\;\delta(\mathbf{d}_t + \mathbf{d})\,dS = 0 \tag{13}$$

where σ = second Piola-Kirchhoff axial stress

$$\int_V \{\sigma\,\mathbf{B}_{\mathrm{L}} + \sigma_t\mathbf{B}_{\mathrm{NL}}\}dV + \frac{1}{2}\int_l GJ(\frac{d\theta}{dx})^2\,dx - \int_S f^T\delta\mathbf{d}\,dS = \int_S f_t^T\delta\mathbf{d}\,dS - \int_V \sigma_t\mathbf{B}_{\mathrm{L}}\,dV - \int_l T_{sv}\,\frac{d\theta}{dx}\,dx \tag{14}$$

$$\begin{aligned} &\int_V \{\sigma\,\mathbf{B}_{\mathrm{L}} + \sigma_t\mathbf{B}_{\mathrm{NL}}\}dV + 0.5\int_l GJ(d\theta/dx)^2\,dx \\ &= \delta\mathbf{d}^T\int\{E_1\mathbf{B}_1^{\mathrm{T}}\mathbf{B}_1 - E_2(\mathbf{B}_2^{\mathrm{T}}\mathbf{B}_1 + \mathbf{B}_1^{\mathrm{T}}\mathbf{B}_2) + E_3\mathbf{B}_2^{\mathrm{T}}\mathbf{B}_2 - E_4(\mathbf{B}_3^{\mathrm{T}}\mathbf{B}_1 + \mathbf{B}_1^{\mathrm{T}}\mathbf{B}_3) + E_5\mathbf{B}_3^{\mathrm{T}}\mathbf{B}_3 + E_6(\mathbf{B}_2^{\mathrm{T}}\mathbf{B}_3 + \mathbf{B}_3^{\mathrm{T}}\mathbf{B}_2)\}\mathbf{d}dx \\ &\quad + \delta\mathbf{d}^T\int_l F_x(\mathbf{B}_4^{\mathrm{T}}\mathbf{B}_4 + \mathbf{B}_5^{\mathrm{T}}\mathbf{B}_5)dx\mathbf{d} + \delta\mathbf{d}^T\int_l GJ(\mathbf{B}_6^{\mathrm{T}}\mathbf{B}_6)\mathbf{d}dx \end{aligned} \tag{15}$$

$$\begin{aligned} \int_V \sigma_t\,\mathbf{B}_{\mathrm{L}}dV + \int_l T_{sv}(d\theta/dx)dx &= \delta\mathbf{d}^T\int_V \sigma_t(\mathbf{B}_1^T - y\mathbf{B}_2^T - z\mathbf{B}_3^T)dV + \delta\mathbf{d}^T\int_l T_{sv}\mathbf{B}_6^T\,dx \\ &= \delta\mathbf{d}^T\int_l (F_x\mathbf{B}_1^T - M_z\mathbf{B}_2^T - M_y\mathbf{B}_3^T + T_{sv}\mathbf{B}_6^T)dx \end{aligned} \tag{16}$$

$$\begin{aligned} &E_1 = E_T\int dydz,\quad E_2 = E_T\int ydydz,\quad E_3 = E_T\int y^2dydz,\quad E_4 = E_T\int zdydz,\quad E_5 = E_T\int z^2ydydz, \\ &E_6 = E_T\int zydydz,\quad F_x = \int\sigma_t dydz,\qquad \mathrm{M}_z = \int\sigma_t ydydz,\qquad \mathrm{M}_y = \int\sigma_t zdydz \end{aligned} \tag{17}$$

That can be determined through numerical integration over the cross section fiber segments. The tangent stiffness can be written as follows

$$\mathbf{k}_t = \int \{E_1\mathbf{B}_1^T\mathbf{B}_1 - E_2(\mathbf{B}_2^T\mathbf{B}_1 + \mathbf{B}_1^T\mathbf{B}_2) + E_3\mathbf{B}_2^T\mathbf{B}_2 - E_4(\mathbf{B}_3^T\mathbf{B}_1 + \mathbf{B}_1^T\mathbf{B}_3) + E_5\mathbf{B}_3^T\mathbf{B}_3 + E_6(\mathbf{B}_2^T\mathbf{B}_3 + \mathbf{B}_3^T\mathbf{B}_2) + F_x(\mathbf{B}_4^T\mathbf{B}_4 + \mathbf{B}_5^T\mathbf{B}_5) + GJ(\mathbf{B}_6^T\mathbf{B}_6)\}dx \tag{18}$$

The tower including horizontal beam is modeled with fiber element where the beam element is divided along its length and over its cross section directions. The stiffness quantities of the section are calculated based on the stress states of integration points over the cross section. Then, the element stiffness quantities are obtained by integrating along the element length where the plasticity development and the axial force effect on the structural seismic responses are automatically considered. The stiffness matrix calculations of the elements are completed by numerical integration procedure. In the nonlinear incremental analysis, the structure tangent stiffness matrix, which is assembled from the element tangent stiffness matrices, is used to predict the next incremental displacements under a loading increment.

2.2 Equation of motion

The governing nonlinear dynamic equation of the tower structure response can be derived using the principle of energy, i.e. the external work is absorbed by the internal one, inertial and damping energy for any small admissible motion that satisfies compatibility and boundary condition. By assembling the element dynamic equilibrium equation at time $t+\Delta t$ over all the elements, the incremental FEM dynamic equilibrium equation (Chen 2000, Abdel Raheem and Hayashikawa 2003, Abdel Raheem 2009) can be obtained as

$$[\mathbf{M}]\{\ddot{u}\}^{t+\Delta t} + [\mathbf{C}]\{\dot{u}\}^{t+\Delta t} + [\mathbf{K}]^{t}\{\Delta u\}^{t+\Delta t} = \{\mathbf{F}\}^{t+\Delta t} - \{\mathbf{r}\}^{t} \tag{19}$$

where [**M**], [**C**], and [**K**] $^{t+\Delta t}$ are the system mass, damping, and tangent stiffness matrices at time t, respectively. $\ddot{u}$, $\dot{u}$ and Δu are the accelerations, velocities, and incremental displacement vectors at time $t+\Delta t$, respectively. $\{\mathbf{F}\}^{t+\Delta t} - \{\mathbf{r}\}^{t}$ is the unbalanced force vector. It can be noticed that the dynamic equilibrium equation of motion takes into consideration the different sources of nonlinearities which affect the calculation of the tangent stiffness and internal forces. The implicit Newmark's 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. In this method, the stiffness matrix is updated at each increment to consider the geometrical and material nonlinearities and to speed up the convergence rate. A spectral damping scheme of Rayleigh's damping is used to form damping matrix (Abdel Raheem and Hayashikawa 2007 & 2008). The damping ratio corresponding to the frequencies of the in-plane and out-plane fundamental modes of tower is set to 2%. A nonlinear dynamic analysis computer program is developed based on the above mentioned formulation to predict the vibration behavior of framed structures as well as the nonlinear response under earthquake loadings. The program has been validated through a comparison with different commercial software EDYNA, DYNA2E and DYNAS (Japanese software).

2.3 Input earthquake ground motions

The Hyogoken Nanbu Earthquake of January 17, 1995 caused severe damage to buildings, highway bridges, railways, lifeline systems, and port facilities. This event is the first instance of engineering structures designed for the highest seismic forces in the world to be subjected to such destructive ground motions. Following the 1995 Hyogoken Nanbu earthquake, Japan Society of Civil Engineers issued "Proposal on Earthquake Resistance for Civil Engineering Structures". According to this proposal, two types of earthquake ground motions should be taken into account in earthquake resistant design of the structures. One is Level I motion of moderate intensity; which is likely experienced by the structures once or twice during their life time, and the other is Level II motion of extreme intensity which is rarely experienced during their life time. One of the most important decisions in carrying out proper design is to select a design earthquake that adequately represents the ground motion expected at a particular site and in particular the motion that would drive the bridge structure to its critical response, resulting in the highest damage potential. Wide ranges of peak ground accelerations; frequency contents and energy or duration for the records, vertical ground motion, and near source ground motion are potentially important to bridge facilities design (Abdel Raheem 2003).

A suite of recorded and simulated standard ground motion records are used for the nonlinear time history analysis: four near-fault ground motion records (Elgamal 1999) obtained during the 1995 Hyogoken-Nanbu earthquake (M7.2) and the 1994 Northridge Earthquake (Mw = 6.7), including three-components acceleration time histories recorded at JMA, JR Takatori, Sylmar-Converter Sta., and Rinaldi Receiving Sta. The Input ground motions characterization is introduced in Table 1 where the calculated responses for different records are compared. Furthermore, together with the standard ground motions (Committee of Earthquake Engineering 1996, Japan Road Association 1996), level II was introduced through Japan Highway Specification

Table 1 Input ground motions characterization

Ground Motion Characteristics			a_{max} (g)	v_{max} (cm/sec)	d_{max} (cm)	Predominant period (sec)
The 1995 Kobe "Hyogoken Nanbu" Earthquake	KJMA station “KJM”	KJM090	0.5985	74.32	19.95	0.706
		KJM000	0.8213	81.26	17.68	0.683
		KJM-UP	0.3428	38.29	10.29	1.024
	Takatori station “TAK”	TAK090	0.616	120.7	32.72	1.205
		TAK000	0.611	127.1	35.77	1.280
		TAK-UP	0.272	16.0	4.47	0.126
The 1994 Northridge Earthquake	Rinaldi Receiving Sta. “RRS”	RRS318	0.472	73.0	19.76	0.602
		RRS228	0.838	166.1	28.78	1.707
		RRS-UP	0.852	50.7	11.65	0.250
	Sylmar Converter Sta. “SCS”	SCS142	0.8972	102.8	46.99	3.413
		SCS052	0.6125	117.45	53.43	2.275
		SCS-UP	0.5862	34.60	25.20	0.297
Japanese Standard Ground motion Level 2 Type II		T2-II-m1	0.701	76.98	81.42	1.033
		T2-II-m2	0.686	137.1	77.63	1.078
		T2-II-m3	0.751	113.4	118.8	1.280

1996 for different types of soil condition (Type I, II and III) to reflect a more realistic ground motion. Level II ground motions occur at a very short distance with a magnitude of about 7-7.2.

3. Finite element modeling of tower structure

3.1 Tower structure model

The steel tower of a three continuous spans cable-stayed bridge located in Hokkaido, Japan is considered. The main span of the bridge is 284m. Since the cable-stayed bridges are not structurally homogeneous, it is concluded from previous studies (Ali and Abdel-Ghaffar 1995, Abdel Raheem and Hayashikawa 2003, McDaniel and Seible 2004, Endo *et al.* 2004, chi-yu *et al.* 2008, Xie *et al.* 2012) that the tower, deck, and cable stays affect the structural response in a wide range of vibration modes. Since the modes of the deck, cable stays, and the tower are fairly well decoupled, the steel tower is taken out of the cable-stayed bridge and modeled as a three-dimensional frame structure (Ali and Abdel-Ghaffar 1995, Abdel Raheem and Hayashikawa 2003 & 2007 & 2008, McDaniel and Seible 2004, Endo *et al.* 2004, chi-yu *et al.* 2008, Xie *et al.* 2012). For the numerical analysis, the geometry and the structural properties of the bridge structure and steel tower are shown in Figs. 1 and 2. The tower structure has a rectangular hollow steel section with internal stiffeners which have different dimensions along the tower height and its horizontal beam as shown in Table 2. The stress-strain relationship of the steel is modeled as bilinear stress strain relation where the yield stress and the modulus of elasticity are 355 MPa (SM490Y) and 200GPa, respectively. The strain hardening in the plastic region equals 0.01. The nonlinearity of stayed cable is idealized by using the equivalent modulus approach. In this approach, each cable is replaced by a truss element with equivalent tangential modulus of elasticity E_{eq} that is calculated by Ernst equation (Ernst 1965) as

$$E_{eq} = E \,/\, \{1 + EA\,(wL\,)^2 \,/\, 12T^{\,3}\,\} \tag{20}$$

where E = material modulus of elasticity; L = horizontal projected length of the cable; w = weight per unit length of the cable; A = cross sectional area of the cable; and T = tension force in the cable. The stayed-cable is represented by an equivalent straight cable element with relative axial deformation (Δl) where the stiffness matrix of the cable element $\mathbf{K}$ has a value of $E_{eq}A/l$ for $\Delta l > 0$, and the cable stiffness vanishes and no element force exist when shortening occurs, i.e. $\Delta l < 0$. This cable-stayed bridge has nine cables in each tower side. The dead load of the stiffening girder is

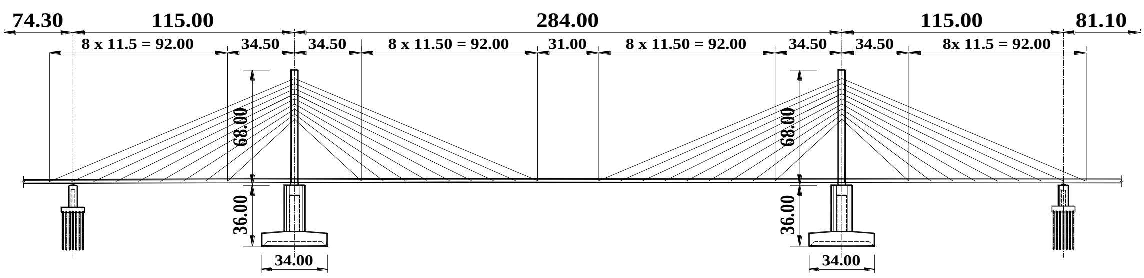
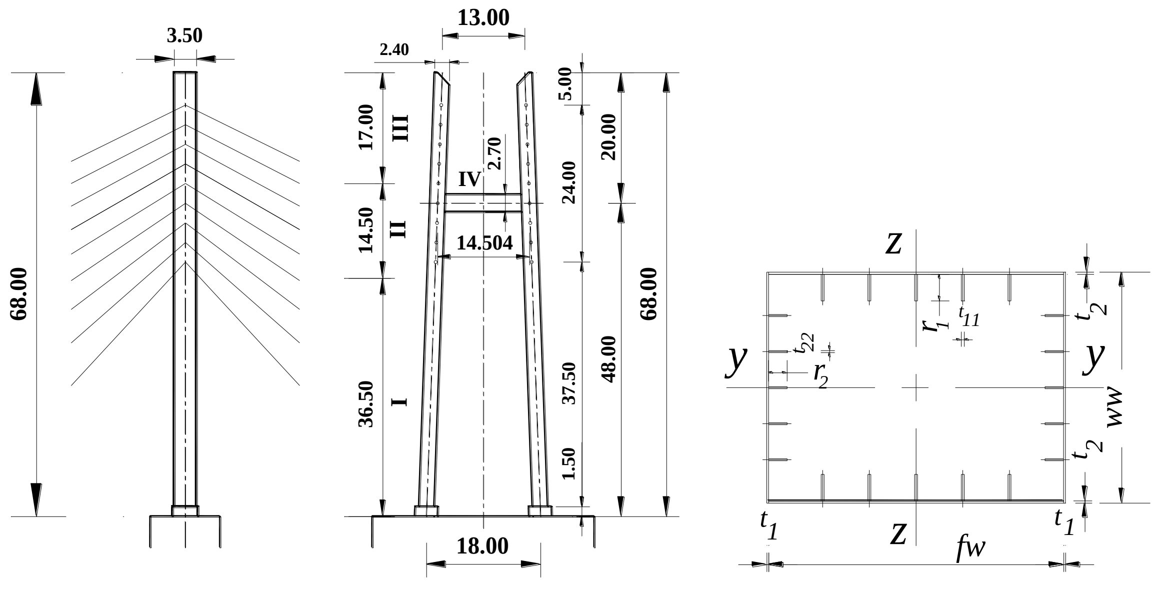


Fig. 1 General view of the cable-stayed bridge (m)

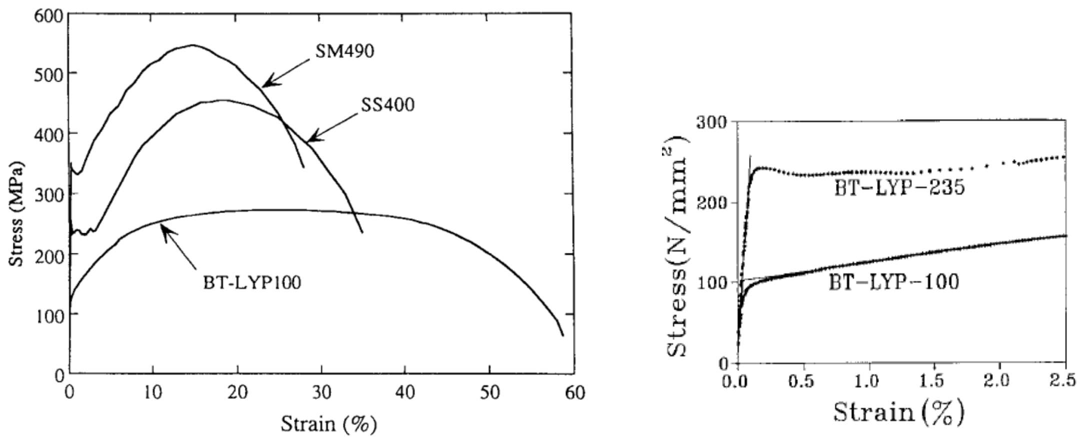


(a) (b)

Fig. 2 Steel tower of cable-stayed bridge: (a) tower geometry (*m*) and (b) cross section

Table 2 Cross section dimension of different tower region (*cm*)

Tower parts	Outer dimension				Stiffener dimension			
	fw	*ww*	t_1	t_2	r_1	r_2	t_{11}	t_{22}
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



(a) (b)

Fig. 3 Tensile stress- strain relationship for LYP-235 and LYP-100 steel considered to be equivalent to the vertical component of the pretension force of the cables and acted vertically at their joints.

3.2 Low yield material model

The low yield point steel has been introduced for the hysteretic damper concept; the energy dissipation of hysteretic damper system through their materials could be used for structure damage control under large earthquake excitations. Finally, when damaged due to earthquake loading, seismic link could be simply replaced without major involvement of the primary structure. A design strategy for maximizing the benefits of LYP hysteretic damper system should be based on yielding of damping mechanism at very low level of external forces for triggering energy dissipation as early as possible and late yielding of the main frame and minimization of tower drift for retarding serious damage to the primary structure. Thus, the spectral acceleration and structural element forces could be significantly reduced when compared to that of original tower seismic demands. Fig. 3 illustrates examples of stress strain relationships of conventional and low yield steels where the Low Yield Point (LYP) steels have the same young’s modulus as conventional steels with low yield stresses and high ductility. This ensures the damper to undergo large inelastic deformations at the first stages of the loading process, thus; enhancing the energy dissipation capability of the whole system in a wide range of deformation demand.

4. Numerical results and discussion

4.1 Natural vibration analysis

The natural vibration analysis is carried out for the previous described steel tower modal. The Eigen values (natural periods) of the tower with the description of the mode shapes for the first eight modes and the corresponding effective modal mass and the damping coefficient obtained from the analysis are listed in Table 3.

Table 3 Summary of principal vibration modes for global model

Mode order	Period T (sec)	Effective mass as a fraction of total mass	Viscous damping percent	Mode shape
1	2.0723	33.195	2.00	H_1
2	0.9335	30.330	2.00	L_1
3	0.7726	0.000	2.18	T_1
4	0.5235	0.034	2.81	V_1
5	0.3751	1.735	3.68	L_2
6	0.3625	0.080	3.79	H_2
7	0.3296	0.000	4.12	T_2
8	0.1559	34.079	8.35	V_2
Sum	--	99.423	--	--

H: transverse vibration, T: torsional vibration, L: longitudinal vibration, V: vertical vibration

4.2 Nonlinear time history analysis

A design strategy for the cable-stayed bridge towers using LYP steel energy dissipation system for maximizing hysteretic damping through their materials is adopted by setting the first yielding low for the purpose of triggering energy dissipation as early as possible and to set maximum resistance of the main element large for the purpose of retarding serious structural damage to the main frame as much as possible. A metallic hysteretic Energy dissipation system is used in bridge design against ultimate limit states to serve as high stiffness components below a force threshold and to undergo large hysteretic cycles when the threshold is crossed.

The nonlinear dynamic behavior and seismic performance of the steel tower under earthquake excitations are studied for three different cases: a reference case of original tower (Yield level = 1) and other two cases of proposed energy dissipation system. Two yield level models (Yield level = 0.67 and 0.28) relative to that tower primary structure; corresponding to using two grades of low yield point steel with nominal yield strength equal to 235 MPa (LYP - 235) and 100 MPa (LYP - 100), respectively. The concentration of inelastic behavior along the tower horizontal beam by reduction of its strength is considered, this reduction is done by using low yield point steel material instead of cross section dimensions reduction.

To verify the effectiveness of the presented seismic design with proposed energy dissipation strategy, numerical simulation based on nonlinear time history analysis for the four recorded and three standard earthquakes specified in Japanese seismic code is used to evaluate the seismic behavior of steel tower with inserted low yield energy dissipation system. Since the success of the structural system is largely dependent on the ability of the seismic link to function most effectively as an energy dissipater, energy dissipation capacity is the primary means of measuring the performance of the seismic link. Several forms of performance indices were presented and different responses quantities were used in their evaluation. In particular, tower drifts were used as a measure of the deformations and possible damage of structural members and non-structural components. The acceleration response is alternatively employed as a measure of the shear forces and stresses developed in the main structural members. In this regard, it is interesting to examine the effectiveness of the yielding metallic devices in reducing these response quantities.

4.2.1 Evaluation criteria for peak and normed responses

To evaluate the capability of LYP steel energy dissipation system for reducing the peak responses and the normalized responses over the entire time record, thirteen criteria for either peak or normed responses each have been defined to evaluate the capabilities of proposed energy dissipation strategy.

Thirteen evaluation criteria $JP_1 - JP_{13}$ are considered in this study, the first three evaluation criteria consider the ability of the energy dissipation system to reduce peak responses. Evaluation criteria $JP_1 - JP_{13}$ are related to peak response quantities where: JP_1 = the peak total input energy, JP_2 = the peak of the ratio of the strain energy to the total input energy, JP_3 = the peak of the ratio of the damping energy to the total input energy, JP_4 = the peak curvature of tower at base level, JP_5 = the peak curvature of tower leg below the horizontal beam level, JP_6 = the peak curvature at the horizontal beam end, JP_7 = the peak overturning moment of tower leg at base level, JP_8 = the peak overturning moment of tower leg below the horizontal beam level, JP_9 = the peak overturning moment at the horizontal beam end, JP_{10} = the peak shear force of tower at base level, JP_{11} = the peak shear force of tower leg below the horizontal beam level, JP_{12} = the peak axial force of tower leg at base level, and JP_{13} = the peak displacement of the tower top. Evaluation criteria $JN_1 – JN_{13}$

are related to normed response quantities corresponding to response quantities for $JP_1 - JP_{13}$. The evaluation criteria for the peak response, JP, are defined as the ratio of the maximum absolute value "peak" of the measured seismic response of the tower with different yield level to that of the Reference case of original tower (Yield level = 1).

$$JP = \frac{\max\limits_t \left| measured\ response\,(t)_{\text{Yield level}} \right|}{\max\limits_t \left| measured\ response\,(t)_{\text{Yield level}=1.0} \right|}\ ,\ JN = \frac{\max\limits_t \left\| measured\ response\,(t)_{\text{Yield level}} \right\|}{\max\limits_t \left\| measured\ response\,(t)_{\text{Yield level}=1.0} \right\|}\ ,$$

$$\left\| . \right\| \equiv \sqrt{\frac{1}{t_f}\int_0^{t_f} (.)^2\, dt} \qquad (21)$$

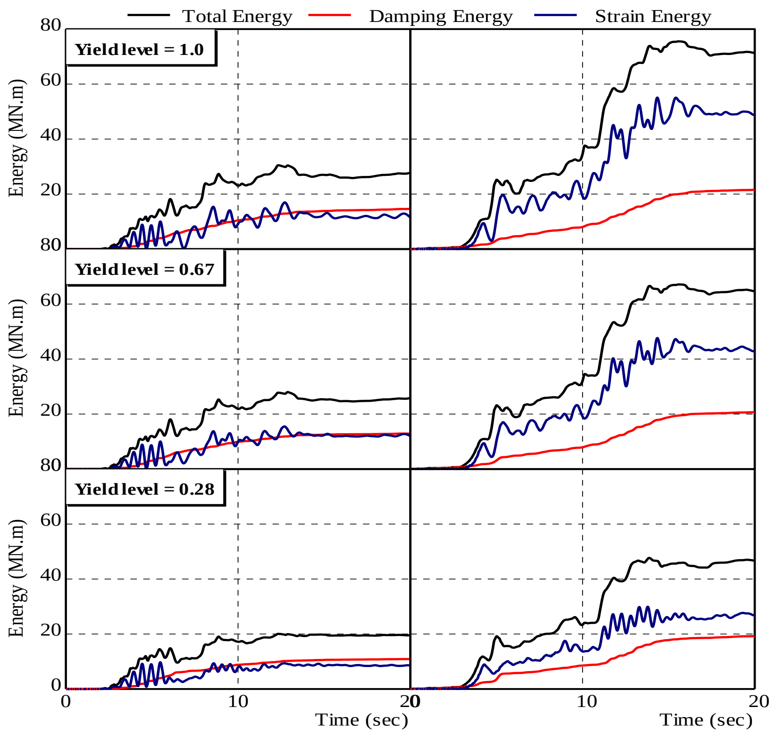
In general, dynamic peak and normed responses to various earthquake records resulted in lower values for the tower with the proposed LYP steel energy dissipation system. In addition to these advantages, the tower with the proposed LYP steel energy dissipation system has a reduced base shear, an overturning moment along tower height, a more uniform distribution of tower drift, and a larger energy dissipation capacity per unit drift. Other advantages of tower with the proposed LYP steel energy dissipation system include easy link replacement after an extreme earthquake and the ease of tailoring link strengths to by adjusting link yield level. The LYP steel energy dissipation strategy with different yield levels is very effective in reducing the force and displacement response especially for ultimate limit states as shown in Tables 4 and 5.

Table 4 Evaluation criteria for peak responses

	Evaluation criteria	JP_1	JP_2	JP_3	JP_4	JP_5	JP_6	JP_7	JP_8	JP_9	JP_{10}	JP_{11}	JP_{12}	JP_{13}
Yield Level = 0.67	KJM-AT2	1.00	1.00	1.00	1.00	1.00	1.02	1.00	1.00	0.96	1.00	1.00	1.00	1.00
	TAK-AT2	0.92	0.91	0.88	0.60	0.32	3.64	0.96	0.86	0.75	0.85	0.96	0.99	1.12
	RRS-AT2	0.95	0.86	1.03	0.63	0.21	4.04	0.97	0.92	0.83	0.98	1.18	0.97	1.04
	SCS-AT2	0.89	0.86	0.96	0.46	0.12	4.25	0.97	0.83	0.78	0.92	1.01	0.93	1.02
	T2-II-m1	0.87	0.91	0.83	0.71	0.51	3.21	0.90	0.83	0.74	0.92	0.91	0.86	0.97
	T2-II-m2	0.91	0.94	0.76	0.74	0.59	2.98	0.92	0.85	0.74	0.89	0.93	0.87	0.92
	T2-II-m3	0.92	0.92	0.76	0.64	0.39	3.36	0.95	0.84	0.74	0.85	0.94	0.87	1.11
	Average	**0.92**	**0.92**	**0.89**	**0.68**	**0.45**	**3.21**	**0.95**	**0.87**	**0.79**	**0.92**	**0.99**	**0.93**	**1.03**
Yield Level = 0.28	KJM-AT2	1.01	1.05	0.96	0.71	0.69	4.29	0.71	0.69	0.51	0.65	0.82	0.88	0.97
	TAK-AT2	0.66	0.57	0.74	0.27	0.18	3.37	0.53	0.49	0.36	0.55	0.65	0.81	0.75
	RRS-AT2	0.83	0.64	1.14	0.30	0.07	4.24	0.94	0.65	0.47	0.81	1.03	0.82	0.89
	SCS-AT2	0.63	0.54	0.89	0.23	0.06	5.39	0.87	0.48	0.44	0.65	0.66	0.77	0.95
	T2-II-m1	0.61	0.67	0.57	0.45	0.29	5.98	0.66	0.50	0.39	0.65	0.77	0.65	1.00
	T2-II-m2	0.65	0.69	0.46	0.43	0.30	5.35	0.57	0.46	0.38	0.51	0.56	0.60	0.94
	T2-II-m3	0.60	0.57	0.54	0.34	0.20	3.05	0.55	0.44	0.35	0.54	0.58	0.65	0.77
	Average	**0.71**	**0.68**	**0.76**	**0.39**	**0.26**	**4.52**	**0.69**	**0.53**	**0.41**	**0.62**	**0.73**	**0.74**	**0.90**

Table 5 Evaluation criteria for normed responses

Evaluation criteria		JN_1	JN_2	JN_3	JN_4	JN_5	JN_6	JN_7	JN_8	JN_9	JN_{10}	JN_{11}	JN_{12}	JN_{13}
Yield Level = 0.67	KJM-AT2	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.01	1.00	1.00	1.00	1.00
	TAK-AT2	0.94	1.00	0.91	0.56	0.28	3.96	0.83	0.80	0.74	0.88	0.84	0.96	1.01
	RRS-AT2	0.95	0.91	1.04	0.62	0.16	4.33	0.92	0.88	0.83	0.95	1.02	0.99	0.98
	SCS-AT2	0.91	0.89	0.97	0.41	0.13	4.09	0.91	0.81	0.75	0.97	0.94	0.90	0.97
	T2-II-m1	0.93	1.04	0.83	0.76	0.60	3.21	0.86	0.83	0.79	0.90	0.87	0.93	0.90
	T2-II-m2	0.93	1.10	0.81	0.75	0.68	2.34	0.81	0.79	0.76	0.85	0.83	0.90	0.83
	T2-II-m3	1.00	1.23	0.81	0.61	0.32	4.33	0.78	0.76	0.72	0.80	0.81	0.98	1.06
	Average	**0.95**	**1.02**	**0.91**	**0.67**	**0.45**	**3.32**	**0.87**	**0.84**	**0.80**	**0.91**	**0.90**	**0.95**	**0.96**
Yield Level = 0.28	KJM-AT2	1.01	1.14	0.97	0.62	0.61	3.34	0.62	0.61	0.58	0.65	0.73	0.94	0.70
	TAK-AT2	0.73	0.71	0.79	0.28	0.15	2.74	0.47	0.44	0.38	0.50	0.50	0.83	0.62
	RRS-AT2	0.79	0.65	1.16	0.23	0.04	3.03	0.74	0.52	0.46	0.70	0.77	0.92	0.61
	SCS-AT2	0.65	0.56	0.92	0.17	0.07	4.45	0.67	0.45	0.40	0.63	0.58	0.72	0.82
	T2-II-m1	0.70	0.85	0.59	0.42	0.33	4.29	0.49	0.46	0.41	0.51	0.52	0.79	0.66
	T2-II-m2	0.69	0.93	0.54	0.41	0.36	4.67	0.45	0.42	0.37	0.46	0.48	0.81	0.67
	T2-II-m3	0.74	0.93	0.59	0.34	0.18	2.62	0.47	0.44	0.40	0.49	0.53	0.88	0.62
	Average	**0.76**	**0.82**	**0.79**	**0.35**	**0.25**	**3.59**	**0.56**	**0.48**	**0.43**	**0.56**	**0.59**	**0.84**	**0.67**

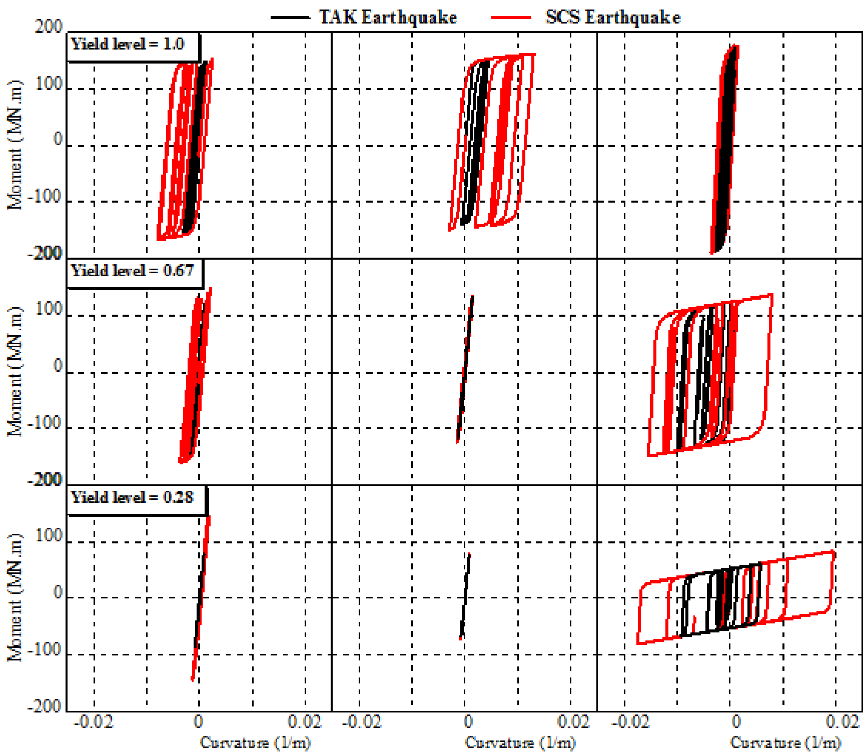


(a) TAK earthquake (b) SCS earthquake

Fig. 4 Energies time history for LYP system

4.2.2 Energy dissipation analysis

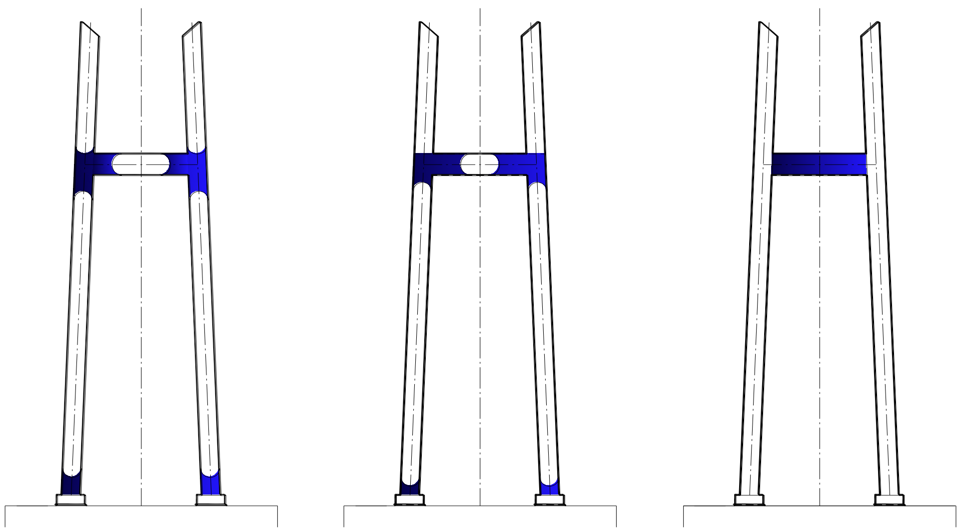
Since the success of a structural system is largely dependent on the ability of the energy sunk link to function most effectively as an energy dissipater, energy dissipation capacity is the primary mean of measuring the performance of these links. The strain energy is composed of recoverable elastic strain energy and irrecoverable hysteretic energy due to plastic yielding. The performance of the proposed energy dissipation system is analyzed by comparing the energies time history. In order to investigate the beneficial effect provided by LYP steel horizontal beam seismic link on the seismic response of the aforementioned cable-stayed bridge tower, different energy dissipation links have been considered. In particular, three different nominal strength ratios have been assumed (Yield level equals 1.0, 0.67 and 0.28). The calculated results of different yield levels show that the application of LYP steel link allows a large amount of earthquake energy to be dissipated by complementary elements of the horizontal beam which serve as hysteretic dampers and, thus, enhancing the energy dissipation capability of the whole system. As the yield level horizontal beam material decreases, the energy dissipation is concentrated as much as possible in the horizontal beam, rather than allowing it in the primary structural element, and thereby reducing damage in the main structure. LYP steel system becomes more effective in energy absorption and damage control as shown in Fig. 4, thus the load capacity of tower horizontal beam decreases and forces redistribution in tower structural elements occurs. Comparisons in energy dissipation between the original tower and the tower sacrificial link damper of low yield point steel through tower horizontal beam justified the applicability of the proposed method.



(a) Tower leg base (b) Tower leg below Hz beam level (c) Hz beam end

Fig. 5 Moment curvature relations for LYP system

It can be seen from Fig. 5 that more concentration of inelastic behavior and ductility at tower horizontal beam is attained as yield level change for low values which is easy to inspect and repair if necessary. The rest of the tower structure approaches elastic behavior as yield level decreases, thus there is a possibility of eliminating permanent damage and minimizing the extent of retrofit. The main tower structure parts attain almost perfect elastic behavior at yield level equal to 0.28. Fig. 6 shows the distribution of plastic regions (blue filled parts) at the end of the input earthquake. Under ground motion excitation, the damages of the original tower "reference model" are mainly concentrated on three zones: the tower legs' bottom, the tower legs' below horizontal beam level, and the end zones of the horizontal beam.



(a) Yield Level = 1.0 (b) Yield Level = 0.67 (c) Yield Level = 0.28

Fig. 6 Distribution of plastic regions with different yield levels at the end of input earthquake

4.2.3 Shear and axial force demands

The better performance of the control proposed energy dissipation system is indicated by comparing the reaction force time history at the tower base for different levels of yield strength of the horizontal beam. The proposed low yield hysteretic dampers lead to effective reduction of vertical force and base shear force response as yield level decreases, as a result; buckling demand decreases as shown in Figs. 7 and 8. The controlled tower exhibits elastic response due to the redistribution of the seismic forces to the tower elements in accordance to their strength.

The LYP hysteretic damper system can provide relatively large energy dissipation through their materials that are strained beyond their yield limits based on yielding of damping mechanism at very low level of external forces for triggering energy dissipation as early as possible and late yielding of the main frame and minimization of tower drift for retarding serious damage to the primary structure. Thus, the spectral acceleration and structural element forces could be significantly reduced when compared to that of original tower seismic demands. However the hysteretic dampers cannot be activated as dampers unless their materials receive inelastic excursions, so the hysteretic dampers are effective only for larger earthquake excitation, hence fail

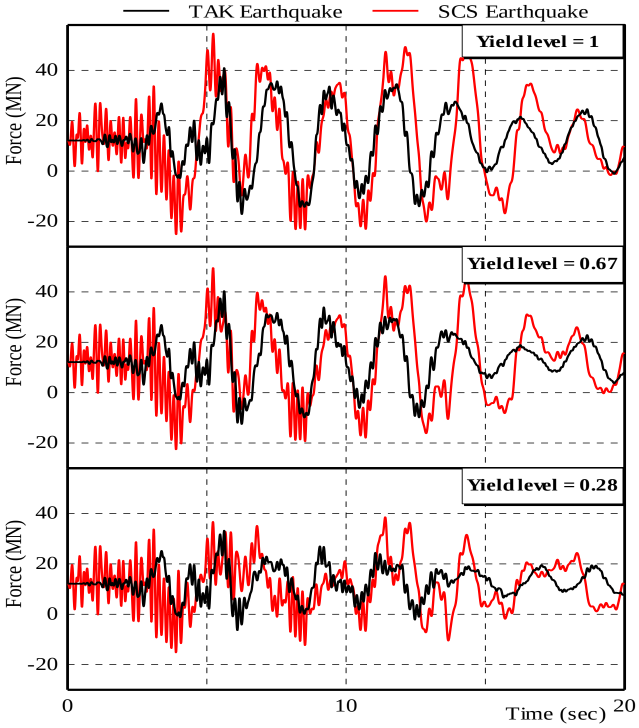


Fig. 7 Tower base axial force time history

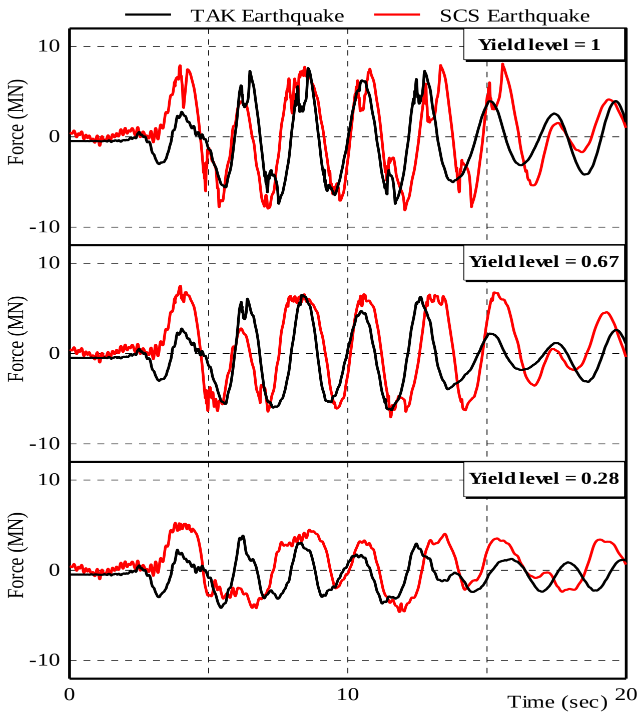
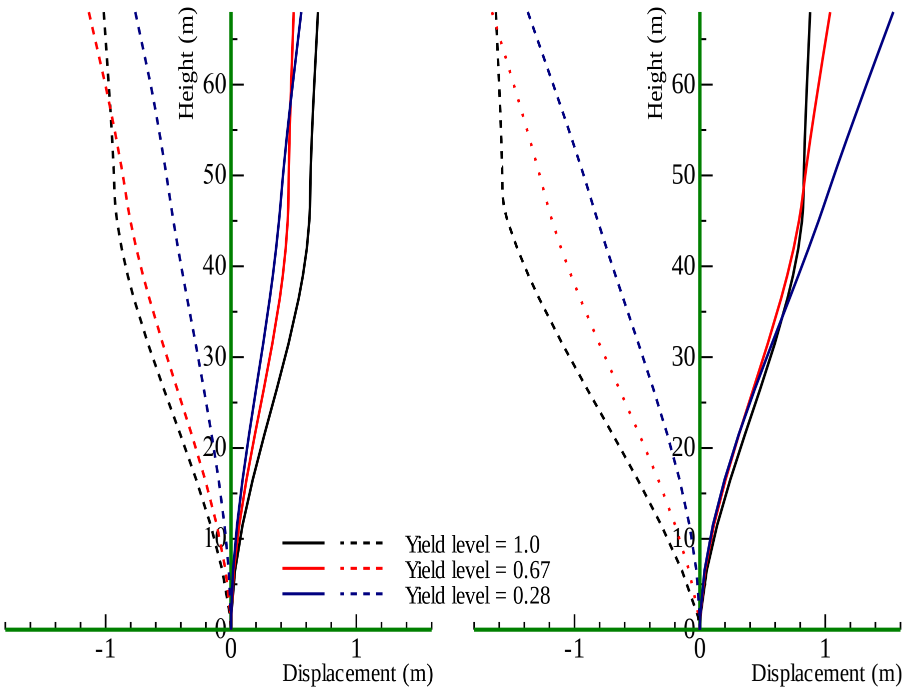


Fig. 8 Tower base shear time history

in providing the required damping for smaller vibrations. Another aspect of post-yield buckling capacity should be considered in the design. In this case of study, as the horizontal beam receives inelastic excursions; the buckling effective length could get larger and, as a result, the load carrying capacity decreases. The buckling load carrying capacity is calculated for the worst case of no contribution of horizontal beam to the tower legs stiffening, it is found to be equal to 49.5 MN, which is much greater than the corresponding tower response.

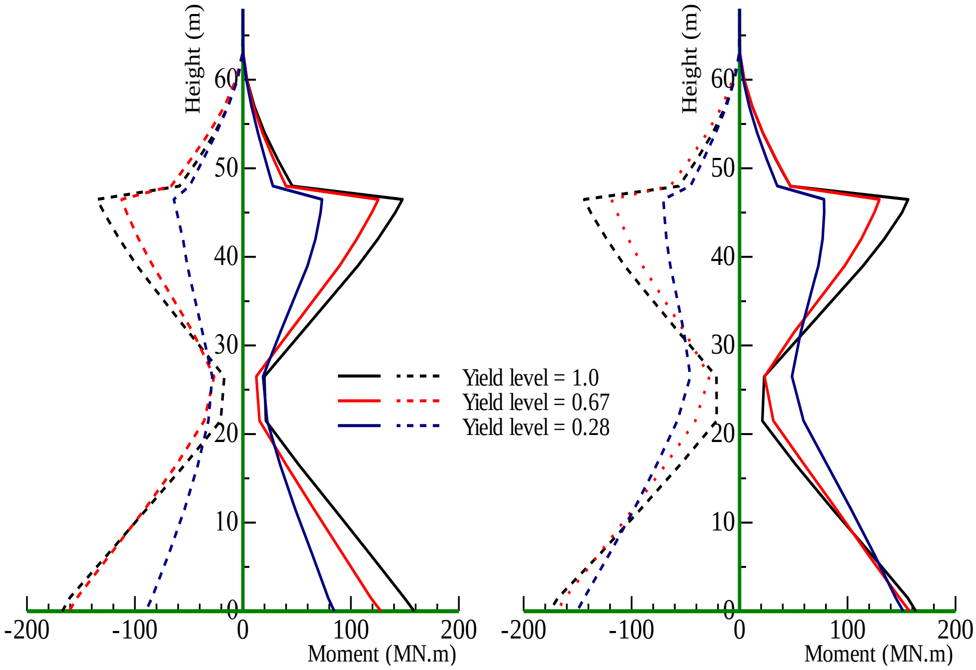
4.2.4 Response envelopes

Inter-storey drift and storey shear response envelopes for both systems are compared for all ground motions. Tower shaft drift profile envelopes for different yield levels are compared for different input excitations as illustrated in Fig. 9. The use of LYP steel seismic link reduces the drift demands along the tower height. On the other hand, the tower top displacement demand is increased approximately 12% and significantly decreased 25% for yield level 0.67 and 0.28, respectively. Although the link has inelastic response, the residual tower drift is negligible due to elastic behavior of primary structural elements. The flexural capacity of tower leg decreases as tower cross section gets smaller in size from the base to top of tower leg, while the moment response demand along the tower leg height, as illustrated in Fig. 10 attains peak values at base and horizontal beam level of tower leg, hence higher moment demand/capacity ratios at tower horizontal beam level are achieved. The use of LYP steel link significantly reduces the moment demands along the tower height, especially, at tower base and horizontal beam level. This reduction reaches about 51 % and 30% for LYP-100, respectively.



(a) TAK earthquake (b) SCS earthquake

Fig. 9 Tower drifts profile envelopes



(a) TAK earthquake (b) SCS earthquake

Fig. 10 Tower shaft moment demand envelopes

4.2.5 Design guidelines

The motivation to use passive energy dissipation devices in a structure is to limit damaging deformations in structural components, where the hysteretic energy dissipation demand on critical components of the structure can be reduced by transferring the energy dissipation demand to the metallic energy dissipation system. The degree to which LYP energy dissipation system is able to accomplish this goal depends on the inherent properties of the basic structure, the properties of the LYP energy dissipation system, the characteristics of the ground motion, and the limit state being investigated. Given the large variations in each of these parameters, it is usually necessary to perform an extensive suite of nonlinear response-history analyses to optimize the energy dissipation system design parameters "stiffness and yield strength" with consideration of several meaningful performance indices. The damage in the tower frame structure can be quantified via a certain damage measure index to achieve different performance goals ranging from a life safety standard to a higher standard that would provide damage control and post-earthquake functionality. The intent of the authors of that study is to direct the dissipation of earthquake induced energy into the hysteretic damping system and away from components of the gravity load resisting system, thereby reducing repair costs and business interruption following severe earthquake shaking. Depending on the performance desired, different design solutions could be obtained. Several forms of performance indices are presented and different responses quantities are used in their evaluation. In particular, tower drifts were used as a measure of the deformations and possible damage of structural members and non-structural components. The acceleration response is alternatively employed as a measure of the shear forces and stresses developed in the main structural members.

5. Conclusions

Systematic design procedures for optimal design parameter of the protective systems in structural systems to obtain the desired reduction in the optimum performance function value are needed. The main objective of this study is, therefore, to formulate a general framework for the optimal design of passive energy dissipation systems for seismic structural applications with consideration of several meaningful performance indices. The current study aims to recognize the feasibility to adopt LYP steel energy dissipation link for enhancing the seismic performance of the cable stayed bridge tower and to investigate the influence of yield strength ratio on the tower dynamic response. A design strategy of passive control technique is adopted. In which, an effective energy dissipation concepts are suggested by a typically concentration of inelastic behavior at tower horizontal beam using low relative strength and stiffness through insertion of low yield steel material instead of cross section dimensions reduction. Since the horizontal beam is easy to inspect and repair if necessary, the rest of the structure will remain elastic, thus eliminating permanent damage and minimizing the extent of retrofit. The efficiency of low yield material dissipative mechanisms to protect seismically tower structure from the near-source ground motions is examined. Based on the nonlinear dynamic analysis of the proposed energy dissipation systems, the following conclusions can be drawn as follow: in general, dynamic peak and normed responses to various earthquake records resulted in lower values for the tower with the proposed LYP steel energy dissipation system. In addition to these advantages, the tower with the proposed LYP steel energy dissipation system has a reduced base shear, an overturning moment along tower height, a more uniform distribution of tower drift, and a larger energy dissipation capacity per unit drift. Low-yield point steel energy dissipation link strongly enhance the seismic performance of cable stayed bridge tower, while acting as hysteretic dampers, they supply a large source of energy dissipation, which results in a limitation of plastic deformation demand to the primary structure. Beneficial effect of LYP steel seismic link appears to be significantly dependent on yield strength ratio. The implementation of the proposed energy dissipation systems in tower structures enables a predominant elastic behavior of the main structure under severe earthquakes that depends upon yield level, as the induced energy is mainly dissipated through plastic hysteresis in horizontal beam low yield material. The use LYP steel link effectively reduces the displacement and moment demands along the tower shaft height and also is beneficial in further reducing the demand/capacity ratios, moreover enable the tower to tune out potential resonant response. Although the link has inelastic response, the residual tower drift is negligible due to elastic behavior of primary structural elements. Comparisons in energy dissipation between the original tower and the tower sacrificial link damper of low yield point steel through tower horizontal beam justified the proposed method applicability.

The low yield material technique could add supplemental damping primarily by material hysteresis and increase structure flexibility as the horizontal beam yield early attains, in terms tower structural system ability to reflect a portion of earthquake input ground motion energy. The energy dissipation system becomes more effective in energy absorption through the horizontal beam region. As a result, the current constructional technology should be effective not only in reducing the vibration and drift of the whole structure at the time of an earthquake, but also in minimizing damage to primary structural elements. The tower demands are reduced, including the tower drift and moments as well as axial loads. It was shown that the inelastic tower links could be used to tune the dynamic response of bridge towers in regions of high seismicity. The proposed LYP steel energy dissipation system is quite effective in reducing the structural dynamic response.

This energy dissipation system can be optimally designed to reduce certain response quantities such as story deformations, base shear and floor accelerations or to achieve a desired structural performance objective.

References

Abdel Raheem, S.E. and Hayashikawa, T. (2003), “Nonlinear behavior of cable-stayed bridges tower under level II earthquake ground motion”, *International Workshop on Structural Health Monitoring of Bridges/Colloquium on Bridge Vibration*, Kitami, 1-2 September.

Abdel Raheem, S.E. and Hayashikawa, T. (2007), “Damping characteristics in soil foundation superstructure interaction model of cable-stayed bridges tower”, *J. Constr.Steel - JSSC*, **15**, 261-268.

Abdel Raheem, S.E. and Hayashikawa, T. (2007), “Low yield steel energy dissipation link for seismic protection of cable-stayed bridge towers”, *GJBS07 - 7th German-Japanese Bridge Symposium*, Osaka, Japan, 25-41.

Abdel Raheem, S.E. and Hayashikawa, T. (2008), “Seismic response control of cable-stayed bridge tower with low yield steel hysteretic damper”, *EASEC-11 - Eleventh East Asia-Pacific Conference on Structural Engineering and Construction*, Taipei, Taiwan, Paper ID 413.

Abdel Raheem, S.E. and Hayashikawa, T. (2008), “Vibration and damping characteristics of cable-stayed bridges tower”, *IABSE Conference, Information and Communication Technology (ICT) for Bridges, Buildings and Construction Practice*, Helsinki, Finland, Paper ID. F15.

Abdel Raheem, S.E. (2009), “Pounding mitigation and unseating prevention at expansion joint of isolated multi-span bridges”, *Eng. Struct.*, **31**(10), 2345-2356.

Abdel Raheem, S.E. and Hayashikawa, T. (2003), “Parametric study on steel tower seismic response of cable-stayed bridges under great earthquake ground motion”, *Struct. Eng. Earthq. Eng. - JSCE*, **20**(1), 25-41.

Ali, H.M. and Abdel-Ghaffar, A.M. (1995), “Modeling the nonlinear seismic behavior of cable-stayed bridges with passive control bearings”, *Comput. Struct.*, **54**(3), 461-492.

Benavent-Climent, A., Morillas, L. and Vico, J.M. (2011), “A study on using wide-flange section web under out-of-plane flexure for passive energy dissipation”, *Earthq. Eng. Struct. Dyn.*, **40**(5), 473-490.

Bergman, D.M. and Goel, S.C. (1987), “Evaluation of cyclic testing of steel plate devices for added damping and stiffness”, Report no. UMCE87-10, Ann Arbor (MI, USA), The University of Michigan.

Chan, R.W.K. and Albermani, F. (2008), “Experimental study of steel slit damper for passive energy dissipation”, *Eng. Struct.*, **30**(4), 1058-1066.

Chan, R.W.K., Albermani, F. and Williams, M.S. (2009), “Evaluation of yielding shear panel device for passive energy dissipation”, *J. Constr. Steel Res.*, **65**(2), 260-268.

Chen, C.N. (2000), “Efficient and reliable solutions of static and dynamic nonlinear structural mechanics problems by an integrated numerical approach using DQFEM and direct time integration with accelerated equilibrium iteration schemes”, *Appl. Math. Model.*, **24**(8-9), 637-655.

Chou, C.C. and Tsai, K.C. (2002), “Plasticity fibre model for steel triangular plate energy dissipating devices”, *Earthq. Eng. Struct. Dyn.*, **31**(9), 1643-1655.

Committee of Earthquake Engineering (1996), “The 1995 Hyogoken-Nanbu Earthquake, Investigation into Damage to Civil Engineering Structures”, Japan Society of Civil Engineering, JSCE, Japan.

Dusicka, P., Itani, A.M. and Buckle, I.G. (2002), “Cyclic behavior of shear links and tower shaft assembly of San Francisco-Oakland bay bridge tower”, Report No. CCEER 02-06, Center for Civil Engineering Earthquake Research, University of Nevada, Reno, NV.

Dusicka, P., Itani, A.M. and Buckle, I.G. (2004), “Evaluation of conventional and specialty steels in shear link hysteretic energy dissipators”, *13th World Conference on Earthquake Engineering*, Vancouver, B.C., Canada, August 1-6, 2004, Paper No. 522.

Elgamal, A., Ashford, S. and Kramer, S. (1999), Eds: 1st PEER workshop on characterization of special source effects, Workshop Report, UCSD, July 20-21, 1998, Pacific Center for Earthquake Engineering Research, UC, Berkeley.

Endo, K., Kawatoh, C. and Unjoh, S. (2004), "Analytical study on seismic performance evaluation of long-span suspension bridge steel tower", *the 13th World Conference on Earthquake Engineering*, Vancouver, B.C., Canada, August 1-6, 2004, Paper ID 944.

Ernst, J.H. (1965), "Der e-modul von seilen unter berucksichtigung des durchhanges", *Der Bauingenieur*, Berlin, **40**(2), 52-55. (in German)

Federal Emergency Management Agency - FEMA (1997), "NEHRP guidelines and commentary for the seismic rehabilitation of buildings", Reports No. 273 and 274, Building Seismic Council, Washington, DC.

Ghabraie, K., Chan, R.W.K., Huang, X. and Xie, Y.M. (2010), "Shape optimization of metallic yielding devices for passive mitigation of seismic energy", *Eng.Struct.*, **32**(8), 2258-2267.

Hayashikawa, T. and Abdel Raheem, S.E. (2002), "Nonlinear dynamic behavior of steel tower of cable-stayed bridges with passive energy dissipation system", *J. Struct. Eng. - JSCE*, **48A**, 863-874.

Hayashikawa, T. and Abdel Raheem, S.E. (2002), "Steel tower seismic response of cable-stayed bridges with passive energy dissipation system", *Proceeding of 7th International Conference on STEEL & SPACE STRUCTURES*, Singapore, 191-198, 2-3 October.

Hsu, H.L., Juang, J.L. and Chou, C.H. (2011), "Experimental evaluation on the seismic performance of steel knee braced frame structures with energy dissipation mechanism", *Steel Compos. Struct*., **11**(1), 77-91.

Japan Road Association (1996), "Reference for highway bridge design, Specification for highway bridges-part IV substructures", Chapter 7-9.

Japan Road Association (1996), "Specification for highway bridges-Part V Seismic design", Maruzen, Tokyo, Japan.

Jiao, C.Y., Li, J.Z. and Fan, L.C. (2008), "Nonlinear seismic response analysis and seismic performance evaluation of a reinforced-concrete cable-stayed bridge tower using IDA method", The 14th World Conference on Earthquake Engineering, Beijing, China, October 12-17, Paper ID 05-02-0067.

Karavasilis, T.L., Kerawala, S. and Hale, E. (2012), "Hysteretic model for steel energy dissipation devices and evaluation of a minimal-damage seismic design approach for steel buildings", *J. Constr. Steel Res.*, **70**, 358-367.

Matteis, G.D., Landolofo, R. and Mazzolani, F.M. (2003), "Seismic response of MR steel frames with low-yield steel shear panels", *Eng. Struct.*, **25**(2), 155-168.

Matteis, G.D., Mazzolani, F.M. and Panico, S. (2007), "Pure aluminum shear panels as dissipative devices in moment resisting steel frames", *Earthq. Eng. Struct. Dyn.*, **36**(7), 841-859.

McDaniel, C.C. and Seible, F. (2004), "Influence of inelastic tower links on cable-supported bridge response", *J. Bridge Eng. - ASCE*, **10**(3), 272-280.

Moehle, J.P. (1995), "Northridge earthquake of January 17, 1994: reconnaissance report, volume 1 - highway bridges and traffic management", *Earthq. Spectra*, **11**, 287-372.

Nakashima, M., Saburi, K. and Tsuji, B. (1996), "Energy input and dissipation behavior of structures with hysteretic dampers", *Earthq. Eng.Struct. Dyn.*, **25**(5), 483-496.

Nakashma, M., Iwai, S., Iwata, M., Takeuchi, T., Konomi, S., Akazawa, T. and Saburi, K. (1994), "Energy dissipation behavior of shear panels made of low yield steel", *Earthq. Eng. Struct. Dyn.*, **23**(12), 1299-1313.

Otani, S. (1997), "Development of performance-based design methodology in Japan" in Fajfar P. and Krawinkler, H (Eds) (1997), "Seismic design methodologies for the next generation of codes", *Proceedings of International Conference at Bled*, Slovenia. A.A. Balkema, Rotterdam/Brookfield, 59-68.

Park, R. and Paulay, T. (1975), *Reinforced concrete structures*, John Wiley & Sons, New York, 769.

Rai, D.C., Annam, P.K. and Pradhan, T. (2013), "Seismic testing of steel braced frames with aluminum shear yielding dampers", *Eng. Struct.*, **46**, 737-747.

Rai, D.C. and Wallace, B.J. (1998), "Aluminum shear link for enhanced seismic resistance", *Earthq. Eng. Struct. Dyn.*, **27**, 315-342.

Sahoo, D.R. and Rai, D.C. (2009), "A novel technique seismic strengthening of RC frame using steel caging and aluminum shear yielding device", *Earthq. Spectra*, **25**(2), 415-437.

Sahoo, D.R. and Rai, D.C. (2010), "Seismic strengthening of non-ductile reinforced concrete frames using aluminum shear links as energy-dissipation devices", *Eng.Struct.*, **32**(11), 3548-3557.

Soong, T.T. and Dargush, G.F. (1997), *Passive energy dissipation systems in structural engineering*, John Wiley & Sons, New York.

Soong, T.T. and Spencer, Jr.B.F. (2002), "Supplemental energy dissipation: state-of-the-art and state-of-the-practice", *Eng. Struct.*, **24**(3), 243-259.

Sugioka, K., Hamada, N., Kobayashi, H., Nishioka, T. and Sugiyama, N. (2011), "Experimental study on elasto-plasticity characteristics of shear panel dampers for long span bridges", *J. Struct. Eng. - JCSE*, **57A**, 528-541.

Tsai, K., Chen, H., Hong, C. and Su, Y. (1993), "Design of steel plate energy absorbers for seismic resistant construction", *Earthq. Spectra*, **9**(3), 505-528.

Vargas, R. and Bruneau, M. (2007), "Experimental response of buildings designed with metallic structural fuses", *J. Struct. Eng. - ASCE*, **135**(4), 394-403.

Whittaker, A., Aiken, I., Bergman, D.M., Clark, P., Cohen, J., Kelly, J.M. and Scholl, R. (1993), "Code requirements for the design and implementation of passive energy dissipation", *ATC-17-1 Seminar on Seismic Isolation, Passive Energy Dissipation, and Active Control*, San Francisco, CA, 497-508.

Whittaker, A.S., Bertero, V.V., Thomson, C.L. and Alonso, L.J. (1991), "Seismic testing of steel plate energy dissipation devices", *Earthq. Spectra*, 7(4), 563-604.

Xie, X., Lin, G., Duan, Y.F., Zhao, J.L. and Wang, R.Z. (2012), "Seismic damage of long span steel tower suspension bridge considering strong aftershocks", *Earthq. Struct.*, **3**(5), 767-781.

Zhengying, L., Albermani, F., Chan, R.W.K. and Kitipornchai, S. (2011), "Pinching hysteretic response of yielding shear panel device", *Eng. Struct.*, **33**(3), 993-1000.

IT