Elastic Stiffness of EDCB and Its Influence on Seismic Behavior of Steel Frame Structures

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Abstract

Based on continuous medium method, theoretical calculations were derived for the elastic lateral stiffness of energy dissipation lattice column with dense battens (EDCB) under the horizontal concentrated force at its top surface. A practical design process was thus developed, utilizing the two-stage method from the seismic design code of buildings (GB 50011-2010). In this paper, EDCB was applied to the seismic design of a five-story steel frame with weak stiffness in Y-direction to verify effects of these columns on the improvement of seismic behaviors of steel frames with a single-direction stiffness relatively weak. This hypothesis was proved theoretically by the elastic calculation, and enough to meet with standard requirements of inter-story drift ratio of the ground floor. From the dynamic elasto-plastic time history analysis, the deformation and stress status of steel frames were improved by EDCB, inducing a more uniform damage distribution. Compared with steel frames without EDCB, the deformation of plastic hinges of bottom steel frame columns with those were evidently superior as the first yielding occurs in battens under earthquakes.

Keywords:
Steel frame structure; continuous medium method; Seismic performance; Energy dissipation lattice column with dense battens; Dynamic elasto-plastic time history analysis

Introduction

Previous earthquakes, such as 1985 Mexican Earthquake (oberto Villa Verde.1991), 1995 Osaka-Kobe Earthquake (EQE.1995), 2008 Wenchuan Earthquake (Zhao.2008), have demonstrated that conventional lateral resistance systems (i.e. frames or frame-brace systems) with a weak layer are susceptible to severe damage (Fig. 1). The reparation of damage structures with
above mentioned systems is with high cost but low efficiency, the majority of which thus require large-scale reconstruction in the post-disaster recovery process.

Over the past decades, increasing attention has been drawn on the performance-based seismic design (Fema, 2012, 2006) with constant development of earthquake engineering. Various novel energy dissipation devices were invented by scholars worldwide, including buckling-restrained braces (Sabelli R. 2003, Saeki E. 1996), post-tensioned pre-stressed steel frames (Rojas P. 2005) and dissipative connections (Plumier A. 2006). Those can focus damage on non-load bearing members in structures to achieve design purposes of convenient replacement, rapid functional recovery and damage control (Condor J. J. 1997).

However, structure systems with advanced equipment have much lower capability in competition of flexibility in architectural function and convenience in construction to traditional steel frames. For instance, buckling-restrained braces are undesirable for designers due to constant conflicts with doors or windows and contour of buildings under demands for architectural functions. Therefore, MacRae et al. (MacRae, 2004) investigated frame-brace systems with rock columns, the stiffness of which affected the concentration degree of stratified deformation. Results from C. Grigorian and M. Grigorian (Grigorian, 2016) showed that the ultimate bearing capacity of structures was affected by auxiliary components, followed by a design proposal for frame-rocking wall structures. Research on frame-rocking wall structures with buckling-restrained braces at bottom was conducted by Feng et al. (Feng, 2016). Jiang et al. (Jiang, 2019) studied seismic performance of hinged truss frame structures, with superior lateral stiffness and similar lateral deformation to frame-rocking wall structures. Results from Jia et al. (Jia, 2018) suggested that rocking truss could improve and control failure modes of steel frame structures under earthquakes.

An innovative light energy dissipative rocking frame was studied by Du and Wu (Du, 2014) with control effects on structural deformation. New continuous energy-dissipative columns (CEDC) system proposed by Li et al. (Li, 2018, 2019) consists of two replaceable steel columned connected by a steel damper that can transfer moment and shear force to boundary columns uniformly under horizontal loading. By analyzing seismic performance of RC frame structures with this system, various kinds of functions were verified, including energy dissipation, resistance to lateral loading and reduction in stratified deformation. Effects of CEDC system on seismic behaviors of steel frames were also investigated as an effective improvement (Li, 2019, 2017, 2018, 2019).

An energy-dissipative rock substructure would decrease internal forces of structures, subjected to earthquake, by releasing the corresponding restraint (Housner, 1963), the application of which is based on increased deformation due to rock (Wu, 2016). A portion of input seismic energy can thus be consumed by energy-dissipative members yielded for large deformation to control damage. Note that fabricated steel structures have merits of accelerated construction and standardization. This paper deals with an energy dissipation lattice column with dense battens (EDCB) incorporating common steel to enhance seismic performances of steel frame structures. Through utilising the continuous medium method, theoretical calculations were derived for the elastic lateral stiffness of EDCB under concentrated loading at top. A practical design process was thus proposed, according to the two-stage method from the seismic design code of buildings (GB 50011-2010). At last, effects of EDCB on seismic behaviours has been illustrated by a practical project of a
five-story steel frame structure. Results showed that plastic hinges first occurred at battens of EDCB, the quantity and damage of which were significantly improved in comparison to those of pure steel frame structures at the column bottom. Hence, a simple and effective seismic strengthening method is proposed for existing steel frames in this paper.

![Fig. 1. Steel frame with box column collapse in weak layer on the first floor in 1995 Osaka-Kobe Earthquake](image)

1. **Lateral stiffness of EDCB**

1.1. **Continuous medium method**

With reference to a calculation method for dual shear walls (Stafford, 1991) with a calculation diagram shown in Fig. 2, three assumptions regarding continuous medium method are adopted in this chapter including: (i) dense battens connect two lattice columns along the height, transferring limited connections to infinity; (ii) axial deformation of battens are neglected, referring to identical horizontal deformation for both two lattice columns with same elevation, rotation and curvature, as the point of inflection is in battens’ mid-span; (iii) vertical spacing $h$, moment of inertia $I_b$ and area $A_b$ of battens and the latter two ($I_1$, $I_2$, $A_1$ and $A_2$) of two lattice columns are constants along the height.

Based on these assumptions and force-method equation, two lattice columns can be cut along dense battens to create cross sections, where the internal force $\tau(x)$ is considered as a redundant unknown force together with the axial force $\sigma(x)$ between columns. Hence, displacement along $\tau(x)$ would be zero in this basic system under the combination of external loading, shear force $\tau(x)$ and axial force $\sigma(x)$.  

1.2. Establishment of force-method equation

For the basic system with combined actions of external force, shear force $\tau(x)$ and axial force $\sigma(x)$ at the cross section, the displacement along $\tau(x)$ can be obtained from three parts separately.

1.2.1. Displacement due to bending deformation of lattice columns

The relative displacement along $\tau(x)$, due to bending deformation of lattice columns under combined actions, is depicted in Fig. 3 and Eq.(1).

\begin{equation}
\delta_l = -2c \theta = 2c \frac{dy_m}{dx}
\end{equation}

where $\theta$ is rotation generated from bending deformation of lattice columns, assuming clockwise rotation as positive.

1.2.2. Displacement due to axial deformation of lattice columns

As shown in Fig. 4, the difference in axial deformation of two lattice columns from bottom to
section $x$ can produce relative displacement Eq. (2) at the cross section under combined actions.

$$
\delta_2 = \frac{1}{E} \left( \frac{1}{A_1} + \frac{1}{A_2} \right) \int_0^H \int_0^x \tau(x) dx dx
$$

where $E$ is elastic modulus of steel lattice columns; $A_1, A_2$ are cross-sectional areas of lattice columns; and $H$ is the total height of EDCB.

1.2.3. Displacement due to bending and shear deformation of battens

Under $\tau(x)h$, bending and shear deformation at the cross section of batten plate can produce relative deformation accordingly Eq. (3) and Eq. (4). The sum of relative deformation is illustrated from Eq. (5) to Eq. (7).

$$
\delta_{3M} = 2 \frac{\tau(x)h a^3}{3 EI_b}
$$

$$
\delta_{3V} = 2 \frac{\mu \tau(x)h a}{A_b G}
$$

$$
\delta_3 = \delta_{3M} + \delta_{3V} = \frac{2 \tau(x)h a^3}{3 EI_b} \left( 1 + \frac{3 \mu EI_b}{A_b Ga^2} \right)
$$

$$
\delta_3 = \delta_{3M} + \delta_{3V} = \frac{2 \tau(x)h a^3}{3 EI_b}
$$

where $\mu$ is the coefficient of non-uniform shear stress distribution; $G$ is shear modulus; $I_b$ is moment of inertia of batten plate and $\tilde{I}_b$ is the transformed moments of inertia of batten plate considering shear deformation as $\tilde{I}_b = \frac{I_b}{1 + \frac{3 \mu EI_b}{A_b Ga^2}}$.
1.2.4. Basic differential equation of EDCB

Total displacement along \( \tau(x) \) under combined actions (external forces, shear force \( \tau(x) \) and axial force \( \sigma(x) \)) can be derived in Eq. (7) as sum of Eq. (1), Eq.(2) and Eq. (6). Then, Eq. (8) and Eq. (9) can be obtained from the first and second partial derivative with respect to \( x \) accordingly.

\[
\delta = \delta_1 + \delta_2 + \delta_3 = -2c\theta + \frac{1}{E} \left( \frac{1}{A_1} + \frac{1}{A_2} \right) \int_{x_0}^{x} \tau(x) dx dx + \frac{2\tau(x)ha^3}{3EI_b} = 0
\]  
(7)

\[
-2c\theta - \frac{1}{E} \left( \frac{1}{A_1} + \frac{1}{A_2} \right) \int_{x_0}^{x} \tau(x) dx + \frac{2ha^3}{3EI_b} \tau'(x) = 0
\]  
(8)

\[
-2c\theta' - \frac{\tau(x)}{E} \left( \frac{1}{A_1} + \frac{1}{A_2} \right) + \frac{2ha^3}{3EI_b} \tau''(x) = 0
\]  
(9)

As shown in Fig. 6, an equilibrium equation Eq. (10) can be derived for EDCB at section \( x \) as:

\[
M_1 + M_2 = M_p - 2cN(x)
\]  
(10)

where \( M_1 \) and \( M_2 \) are moment of lattice columns at section \( x \) and \( M_p \) is the external moment at section \( x \).

By introducing Eq. (12) to Eq. (11) obtained from bending of lattice columns, Eq.(13) can be thus derived and substituted to Eq.(9) to propose Eq.(14). That is the basic differential equation for EDCB under a concentrated force at top, utilizing force method and continuous deformation at cross sections of dense battens.

\[
E(I_1 + I_2) \frac{d^2 y_m}{dx^2} = M_p - \int_0^x 2c \tau(\lambda) d\lambda
\]  
(11)

\[
m(x) = 2c \tau(x)
\]  
(12)
\[
\theta' = -\frac{1}{E(I_1 + I_2)} = V_p - m
\]  

(13)

\[
m''(x) - \frac{\alpha^2}{H^2} m(x) = -\frac{\alpha^2}{H^2} V_0
\]  

(14)

where \( m(x) \) is the sum of moment in lattice columns induced by shear force of battens as restrain moment; \( V_p \) is the total shear force at section \( x \) produced by external forces; \( V_0 \) is the base shear and

\[
\alpha^2 = \alpha_1^2 + \frac{3H^2 D}{hcS}, \quad D = \frac{I_k c^2}{a^3}, \quad \alpha_1^2 = \frac{6H^2}{h \sum I_i} D, \quad S = \frac{2cA_1A_2}{A_1 + A_2}.
\]

1.3. Solution to force-method equation

Through introduction of \( \xi = \frac{x}{H} \) and \( m(x) = \Phi(x)V_0 \frac{\alpha_1^2}{\alpha^2} \), Eq. (14) can be simplified as Eq.(15) with special solution of 1 and generalized solution in Eq. (16).

\[
\Phi''(\xi) - \alpha^2 \Phi(\xi) = -\alpha^2
\]  

(15)

\[
\Phi(\xi) = C_1 ch(\alpha \xi) + C_2 sh(\alpha \xi) + 1
\]  

(16)

where \( C_1 \) and \( C_2 \) are arbitrary constants that can be determined from two boundary conditions:

(i) when \( \xi = 0 \) and \( x = 0 \), moment at the top of lattice columns is zero.

\[
\theta' = -\frac{d^2 y_m}{dx^2} = 0
\]  

(17)

(ii) when \( \xi = 1 \) and \( x = H \), moment at the bottom of lattice columns is zero due to hinge joint.

\[
\theta' = -\frac{d^2 y_m}{dx^2} = 0
\]  

(18)
Hence, $C_1$ and $C_2$ can be determined as:

$$
\begin{align*}
C_2 &= 0 \\
C_1 &= \frac{\alpha(\alpha^2 - \alpha_0^2)}{\alpha_0^2 \sinh \alpha}
\end{align*}
$$

(19)

Based on relationship between shear deformation and shear force of lattice columns, the horizontal displacement $y_v$ due to shear deformation of EDCB can be derived from the integral equations in Eq.(20). Combining with horizontal displacement $y_m$ due to bending deformation of EDCB obtained from Eq. (11), the horizontal displacement of EDCB can be determined from Eq. (21).

$$
\frac{dy_v}{dx} = -\frac{\mu V_p}{G(A_1 + A_2)}
$$

(20)

$$
y = y_v + y_m = \frac{1}{E \sum I_i} \int_{m_0}^{m_1} \int_{n_0}^{n_1} M_p dx dx - \frac{\mu}{G \sum A_i} \int_{m_0}^{m_1} V_p dx
$$

(21)

Substituting external force $V_0$ and $m(x)$ into Eq. (21), $y$ can be determined as:

$$
y = \frac{V_0 H^3}{6E \sum I_i} (1 - T)(2 - 3\xi + \xi^2) + \frac{\mu V_p H}{G \sum A_i} (1 - \xi) -
$$

$$
V_0 H^3 \left\{ C_1 \frac{1}{\alpha_0^3} [\sinh(\alpha \xi) + (1 - \xi) \cosh(\alpha) - \sinh(\alpha)] +
$$

$$
C_2 \frac{1}{\alpha_0^3} [\cosh(\alpha \xi) + (1 - \xi) \sinh(\alpha) - \cosh(\alpha)
$$

$$
- \frac{1}{2} \alpha_0^2 \xi^2 + \alpha_0^2 \xi - \frac{1}{2} \alpha_0^2]
$$

(22)

When $\zeta = 0$, the displacement at the top of EDCB can be determined in Eq.(23) that can be rearranged as Eq. (24) by introducing $C_1$.

$$
\Delta = \frac{V_0 H^3}{3E \sum I_i} (1 - T) + \frac{\mu V_p H}{G \sum A_i} -
$$

$$
\frac{V_0 H^3}{E \sum I_i} C_1 \frac{1}{\alpha_0^3} [\sinh(\alpha \xi) + (1 - \xi) \cosh(\alpha) - \sinh(\alpha)]
$$

(23)

$$
\Delta = \frac{V_0 H^3}{3E \sum I_i} [1 + 3\gamma^2 - T + \psi \tau]
$$

(24)

where

$$
T = \frac{\alpha \gamma^2}{\alpha^2}, \quad \gamma = \frac{\mu E \sum I_i}{H^2 G \sum A_i}, \quad \psi = \frac{1}{\alpha \gamma^2} - \frac{1}{\alpha^2} \frac{1}{(1 - \frac{\alpha \cosh}{\sinh})}.
$$

Therefore, the equivalent stiffness of EDCB, under a concentrated force at the top, can be determined from Eq.(25). Based on the derivation process above, equivalent stiffness of EDCB under triangle and uniform load distribution can also be obtained with constants $C_1$ and $C_2$ in
Eq. (26) and Eq. (27).

\[
K = \frac{1}{\Delta} = \frac{3E \sum I_i}{[1+3\gamma^2 - T + \psi_i T]H^3} \quad (25)
\]

\[
\begin{align*}
C_2 &= -\frac{2}{\alpha} \\
C_1 &= \frac{2(\alpha^4 + \alpha_i^2 \sinh \alpha - \alpha_i^2 \alpha^2)}{3\alpha_i^2 \sinh \alpha}
\end{align*}
\]

\[
\begin{align*}
C_2 &= -\frac{1}{\alpha} \\
C_1 &= \frac{\alpha^4 + \alpha_i \cosh \alpha - 1 - \alpha_i^2 \alpha^2}{2\alpha \alpha_i^2 \sinh \alpha}
\end{align*}
\]

(26) \quad (27)

2. Design method of steel frames with EDCB

Common steel frames with a weak layer are susceptible to collapse shown in Fig. 1 under rare earthquakes. This paper utilized EDCB to avoid this phenomenon and improve seismic performance of multistory frames, as lattice columns and dense battens are made by steel with high and low yield strength respectively. Due to relatively short length and dense distribution, battens between two lattice columns will yield first to dissipate seismic energy and coordinate co-work of layers to induce support effects from stories with higher stiffness to those with lower stiffness. High uniformity can be thus obtained for the inter-story drift ratio for this simple and effective seismic strengthening method.

For convenience of engineering practice, detailed design steps are proposed in this paper incorporating the two-stage method from the seismic design code of buildings (GB 50011-2010) . Stiffness ratio \( \beta_k \) of steel frames to dense battens is introduced in Eq.(28) for the sake of elastic design.

\[
\begin{align*}
\beta_k &= \frac{k_{EDCB}}{k_{frame}} \\
k_{frame} &= 1/(\sum k_i) \\
k_{EDCB} &= \frac{3E \sum I_i}{[1+3\gamma^2 - T + \psi_i T]H^3}
\end{align*}
\]

(28)

where \( k_{frame} \) is lateral stiffness of the top of steel frame structures; \( k_i \) is the inter-story elastic stiffness of layers of steel frames; \( k_{EDCB} \) is lateral stiffness of the top of EDCB under unit force (Eq. 25).

Under frequent earthquakes, a recommendation for \( \beta_k \) to be controlled within 0.2 (Li.2018) is proposed, as further increase in \( \beta_k \) produces insignificant decrease in structural seismic response and
plastic damage. Elasto-plastic analysis can be further conducted for EDCB subjected to rare earthquakes to satisfy code requirements in accordance to the structural seismic response under frequent earthquakes. Based on the abovementioned two-stage method, a detailed design process is depicted in Fig. 7.

Fig. 7 Design flow of EDCB-frame system

3. Project analysis on seismic performance of EDCB-frame system

3.1. Engineering background

A five-story office building of steel frame structure without basement was investigated with a total building area of 5572 m². The corresponding construction site is in class III and seismic design category II with a characteristic site period of 0.55s and earthquake acceleration of 0.15g. C30 concrete and Q345B steel were utilized for all floors, frame columns and frame beams. Nearly identical planar dimension was adopted for standard floors of 56m×19.9m, as layout of the first floor is depicted in Fig. 8 with a height of 4.8m. Except for that, height of other four floors was 3.3m accompanied by two groups of cross sections summarized in Table 1. Under frequent earthquakes, the story drift of the steel frame in x direction can satisfy the standard requirement of 1/250 [25] that cannot be met simultaneously by those of layers 1-3 in y direction, especially
considering the reduced stiffness of the right side due to the stairwell in the up-left corner. Therefore, EDCB was adopted herein to improve the seismic performance of this steel frame in $y$ direction.

![Fig. 8. First floor structural plan](image)

![Fig. 9. Story drift under frequent earthquakes](image)

| Floor | Sections of frame beams/mm | Sections of frame columns /mm |
|-------|-----------------------------|-----------------------------|
| 1     | H550×20×10×14;              |                             |
|       | H300×200×8×10               | 350×350×16                  |
|       | H400×200×8×10               |                             |
|       | H550×20×10×14;              |                             |
| 2-5   | H300×200×8×10               | 350×350×14                  |
|       | H400×200×8×10               |                             |

### Table 1. Main section sizes of steel frame

**3.2. Cross-section and addictive stiffness of EDCB**

Except for steel beams with relatively large sections hinged with two lattice columns at floors, the horizontal dense battens between floors utilized steel I-beams with small cross section of Q235B with lattice columns made of square steel tubes by Q345B. Cross sections of EDCB are summarized in Table 2. Under designed or rare earthquakes, input seismic energy can be consumed by dense battens yielded first that are replaceable to fulfill rapid functional recovery after the earthquake. Structural models of steel frames and EDCB are depicted in Figs. 10 and 11. The stiffness ratio of EDCB and steel frame structure subjected to frequent earthquakes is 0.0579 in $y$ direction under a
unit force at top, as their joint is shown in Fig. 12.

(a) Steel frame  
(b) EDCB-frame

Fig. 10. Models of steel frame and EDCB

Fig. 11. Elevation of single frame with additional EDCB

Fig. 12. Connection joint of batten plates and steel pipe columns

| Floor | Sections of lattice columns /mm | Sections of link bars/mm | Sections of battens/mm |
|-------|---------------------------------|--------------------------|------------------------|
| 1     | 350×350×16                      | HN450×200                 | H200×150×6×8           |
| 2-5   | 350×350×16                      | HN450×200                 | H200×150×6×8           |

4. Seismic performance of steel frame with EDCB

According to the proposed scheme above, Etabs was utilized to evaluate story drifts in both directions satisfying the standard requirement of 1/250 [24]. Further validation on seismic performance of the steel frame with EDCB was conducted with ETABS through the dynamic...
elasto-plastic time history analysis. In accordance with GB 50011-2010 [24], seismic waves selected in this paper can satisfy requirements of both structures with or without EDCB, of which the first three periods are summarized in Table 3 and Fig. 13.

| Table 3. First three periods of steel frames with or without EDCB |
|---------------------------------------------------------------|
|                                                             |
| First period        | Second period    | Third period    |
|---------------------|------------------|-----------------|
| Steel frame with    | T1=1.156         | T2=1.096        | T3=1.040        |
| EDCB                |                  |                 |                 |
| Steel frame         | T1=1.256         | T2=1.126        | T3=1.130        |
|                     |                  |                 |                 |

Fig. 13. Spectra acceleration of three seismic waves

4.1. Displacement time history of the structure

Displacements of both above mentioned structures were utilized for comparison of their seismic performances. Through comparing lateral displacements shown in Fig. 14, EDCB provided insignificant effects under three seismic waves in the first five seconds, while periods of initial steel frame extended with the prolonging of time indicating a serious degradation of steel-frame stiffness. A relatively slow stiffness degradation was presented by the steel frame with EDCB with lower stiffness, as dense battens consumed seismic energy by yielding.
Under the CHI-CHI earthquake

Under the Coalinga earthquake

Under an artificial earthquake

(a) Under the CHI-CHI earthquake

(b) Under the Coalinga earthquake

(c) Under an artificial earthquake
4.2. Inter-story drift

According to GB50011-2010, the elasto-plastic inter-story drift of frame structures should not exceed 1/50 (0.02) rad. That of the steel frame with EDCB exhibited an obvious decrease for all floors (Fig. 15), where the maximum story drifts of 1/53 (in y direction) and 1/61 (in x direction) were both reached without severe damage in accordance with Appendix M (GB50011-2010). Specific strengthening measures are required for structures with the height of the first floor 1.45 times that of other floors. The results of calculation agree fairly closely with the assumption that EDCB can coordinate deformation of layers and prevent premature yielding of the weak layer, as the coordination can increase the elasto-plastic inter-story drift of the first floor. In the following chapter, a more uniform variation in inter-story drifts will be illustrated for the steel frame with EDCB.

\[ DCF = \frac{\theta_{\text{max}} H}{u_{\text{roof}}} \]  

where \( \theta \) is the maximum inter-story drift; \( u_{\text{roof}} \) is the displacement of top of the structure; \( H \) is height of the structure.

The maximum inter-story drift was obtained of 1/61 for the steel frame with EDCB with
maximum displacement of top of 186.16mm under the Coalingo earthquake. The corresponding DCF was hence determined as 1.595 in y direction. In comparison, those of the original steel frame were 1/56, 189.40 and 1.672 respectively. An improvement in inter-story deformation concentration was thus presented by the steel frame with EDCB.

4.4. Seismic performance

Results of the elasto-plastic time history analysis show that the lowest ratio of plastic energy dissipation of EDCB (the sum of plastic energy dissipation of frame beams, frame columns and dense battens) was obtained as 70% of the total energy dissipation in y direction under the Coalinga earthquake (Fig. 16). That proves fully the energy-dissipative effect of EDCB subjected to rare earthquakes, when plastic hinges first occurred in dense battens and gradually increased and degraded with prolongation of earthquake time. As shown in Fig. 17, the deformation of plastic hinges in the steel frame with EDCB was more severe than that in steel frames, especially for frame column bases in the first floor, revealing protection of EDCB.

![Fig. 16. Plastic energy dissipation](image)

![Fig. 17 Plastic hinge deformation states](image)
4.5. **Numerical verification**

A finite element analysis software FEA was utilized for further verification of shear energy dissipation of EDCB, as a batten plate at the bottom was introduced to Midas Gen after enmeshment and then assembled to the original 3D structural model for the effective elastic analysis under rare earthquakes. As shown in Fig. 18, a reasonable result was illustrated as most of the maximum shear stresses presented by webs of dense battens exceeded the shear yield strength of 235N/mm$^2$.

![Fig. 18. Maximum shear stress under rare earthquakes](image)

5. **Conclusions**

In this paper, an economical and feasible seismic strengthening measure was proposed as steel frame structures incorporating energy dissipation lattice column with dense battens (EDCB). Theoretically, the mechanism between EDCB and steel frame structures was discussed with derivation of equations through the continuous link model under a horizontal concentrated force at the top surface. Then, the dynamic elasto-plastic time history analysis was conducted on a five-story steel frame with EDCB to enhance its seismic performance in $y$ direction under rare earthquakes. Major conclusions are summarized as:

1. Theoretical equations regarding addictive elastic stiffness of EDCB were proposed with simplicity, efficiency and feasibility, utilizing shear stiffness of floors obtained from structural design software. A feasible design process was thus developed, utilizing the two-stage method from GB 50011-2010.

2. Reasonable stiffness can be provided by EDCB under designed and rare earthquakes, as dense battens yielded first to consume majority of the input seismic energy.

3. Through comparison, the quantity and damage state of plastic hinges in the steel frame with EDCB were significantly decreased and improved in comparison to those in the pure steel frame at column bases.
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