Applications of hysteretic steel dampers for controlling the seismic damage in steel frames

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Abstract. The aim of seismic design is not intended to make earthquake-proof structures that will not experience any damage even during a strong earthquake; such structures will be too costly. However, controlling the damage to the desired level is a necessity for buildings with a particular purpose such as hospital, fire and police station, nuclear facilities, buildings that contain hazardous material, and other building that is critical for emergencies and defense. In this paper, a four-story steel frame, both with and without hysteretic steel damper are investigated in order to account the effect of stiffness ratio, and SR (ratio between steel damper and braces stiffness to the bare frame stiffness) in the global structural damage. For these purposes, three ground motions which are compatible to the design spectrum response in the Indonesian building code were selected to be applied to the structure with a stiffness ratio (SR) of 2, 3, 4 and 5 using a non-linear dynamic time history analysis. In this context, the damage index based on the works of Park and Ang is used as the criteria to define global structural damage, while story drift index is used as the criteria to measure the seismic performance level. The results demonstrated that the use of steel damper not only enhances the seismic performance, but it also reduces the damage index of the investigated structure. Furthermore, the damage index and story drift index are influenced by the stiffness ratio, in which the stiffness ratio of 4 (four) provides the smallest damage index and inter-story drift index. Moreover, the damage index and inter-story drift index were also affected by ground motions characteristics.

1. Introduction
The aim of passive dampers in a building system is to concentrate substantial portion of energy input induced by ground motions in a specific mechanical device to reduce the number of inelastic deformations in the primary structural components such as beams, column, or walls. Recently, hysteretic steel damper is well-known among passive dampers system since it is cheaper and more effective in reducing seismic responses by using the steel damper material as the source of energy dissipation when it forms inelastic deformation range under cyclic loading. Consequently, the steel damper system would be rendered defective and would need to be replaced. Minor damage that occurs in the structural component can be repaired without functional interruption. In addition, since the period of the system shortens due to the added stiffness, the base shear and floor acceleration might be increased. Several authors have reported the improvement of seismic performance in the new or retrofitted buildings with hysteretic steel damper [1-5]. Vargas and Bruneau [6] introduced the concept of structural fuse system in seismic design. In this concept, buildings are designed in such that a large portion of the input energy is absorbed by the structural fuse system, allowing the primary structural
member to remain elastic or incur minor inelastic deformation. To gain a realistic and efficient design, the ratio between the braced assembly stiffness to the lateral storey stiffness bare frame, $SR$, should be within the range of $2 \leq SR \leq 4$. The main objective of this paper is to evaluate the performance of four-story steel structures with and without steel damper that was placed under three selected areas with strong ground motion. The four-story steel structures were selected as the representative of the low-rise building that suffered more during earthquakes. The influence of stiffness ratio, $SR$, is investigated in controlling the global structural damage and performance using damage index and story drift index. In this paper, the damage index model is proposed by Bojorquez et al. [7] and applied to evaluate the damage level of the four-story steel frame with and without hysteretic steel damper under three selected areas with strong ground motions.

2. Concept of hysteretic steel damper

2.1. Traditional and damage controlled system
Damage-controlled structures [8] consist of two parallel structures, as shown in figure 1. The primary structures will behave elastically or incur minor inelastic deformation under the most severe earthquakes. While structures that are equipped with a damping system will respond better towards ground motion. The added structures possess a stiffness and a large energy dissipation. In this system, input energy induced by the earthquake is to concentrate the damping system in which the damaged elements might be replaced after the earthquake. Figure 2 illustrates a comparison of the behaviour of the traditional system and damage-controlled system.

2.2. Analytical modelling of hysteretic steel damper
They are several models to characterize the force and deformation relation of steel damper. A bilinear model was selected to represent the steel damper inelastic behavior due to its mathematical simplicity and its ability to trace the strain hardening and hysteretic behavior. Hereafter, unless otherwise mentioned, the term of hysteretic steel damper refers to the damper or the device. To give a better understanding behaviour of the damage-controlled system, a single frame that is equipped with steel damper with two braces support is shown in figure 3. In the model, the fuse system is represented as a lumped mass connected to the ground by non-linear springs, and a linear dashpot is used as the representation of the inherent viscous damping [9]. For the sake of simplicity, device-braced assembly or fuse system is a combination of a damper element (device) and two bracing elements. The lateral stiffness of the device-brace assembly, $k_{bd}$, is a function of the lateral stiffness of the braces, $k_{bs}$, and the device stiffness, $k_{dp}$. The stiffness of the device-braced assembly, $k_{bd}$, is given as,

$$
k_{bd} = \frac{1}{1/k_{bs} + 1/k_{dp}} = \frac{k_{bs}k_{dp}}{k_{bs} + k_{dp}}$$

(1)

The total lateral stiffness of the system leads to,

$$
k_{st} = k_{bd} + k_{fr}$$

(2)

where $k_{fr}$ is the lateral stiffness of the bare frame.

Another important parameter is called stiffness ratio, $SR$ which defined as a ratio between device system (damper and braces) stiffness, $k_{bd}$, to the bare frame stiffness, $k_{fr}$, then:

$$
SR = \frac{k_{bd}}{k_{fr}}$$

(3)
**Figure 1.** Damage-controlled structure.

\[ \Delta = \Delta_e + \Delta_p \]

\( \Delta_e = \) Elastic deformation of beam and column. The inter-story drift might be larger than 0.5%. Elastic deformation is too large

\( \Delta_p = \) Plastic deformation caused by plastic hinges

(a) Traditional system

\[ \Delta_f = \] Elastic deformation of beam and column. This structure deform elastically until the inter-story drift around 1%

\( \Delta_d = \) Elastic and plastic deformation of dampers

(b) Damage-controlled system

**Figure 2.** Comparison between the traditional system and damage controlled system [8].
Figure 3. Model of systems with steel damper [9]; (a) a single-story frame, (b) equivalent three spring system, (c) equivalent one-spring system.

3. Building description

Both four-story of steel building with and without steel damper were analyzed under three selected ground motion record. Steel building without steel damper is denoted as a bare frame (MRF), while the steel building with steel damper is denoted as DRF. The ground excitation records in this study are matched to the design spectrum response of the Indonesian Code (figure 4). The value of dead load and live load each story are designed equally to 32 kN/m. Figure 5 provided information with regard to the elevation view of the steel structure. Also, the dimension of the column and beam is listed in table 1. For a convenient discussion, the nomenclature of DRF frames is named DRF-2, DRF-3, DRF-4 and DRF-5, respectively. The braces for HSD-2 and HSD-3 frames are made from hollow structural section (HSS) 180x180xt, whereas the DRF-4 and DRF-5 are made from HSS 200x200xt. Moreover, table 2 shows the lateral story of damper stiffness and braced stiffness.

Figure 4. Design spectrum response of scaled ground motion.
Figure 5. Elevation view of the investigated buildings.

Table 1. Size of beam and column.

| Story | Column dimension | Beam dimension |
|-------|------------------|----------------|
|       | Interior          | Exterior       | Interior          | Exterior       |
| Story 1 | W360x110         | W360x101       | W460x74          | W460x74       |
| Story 2 | W360x101         | W360x91        | W460x74          | W460x74       |
| Story 3 | W360x91          | W360x79        | W460x68          | W460x68       |
| Story 4 | W360x79          | W360x79        | W460x60          | W460x60       |

Table 2. Stiffness parameters of the building.

| Structures | Story | 1     | 2     | 3     | 4     |
|------------|-------|-------|-------|-------|-------|
| MRF(10^3N/m) |       | 26752 | 15406 | 13101 | 10819 |
| DRF-2       | Damper (10^3N/m) | 80257 | 46218 | 39303 | 32457 |
| (SR=2)      | Brace (10^3N/m)  | 160514| 92364 | 78606 | 64914 |
| DRF-3       | Damper (10^3N/m) | 120384| 69327 | 58955 | 48686 |
| (SR=3)      | Brace (10^3N/m)  | 240768| 139654| 117901| 97372 |
| DRF-4       | Damper (10^3N/m) | 160515| 92436 | 78606 | 64914 |
| (SR=4)      | Brace (10^3N/m)  | 320130| 184872| 157212| 129828|
| DRF-5       | Damper (10^3N/m) | 200640| 115545| 98257 | 81142 |
| (SR=5)      | Brace (10^3N/m)  | 401280| 231090| 196514| 162284|

4. Damage indices

The evaluation of damage measure and seismic performance level based on hysteretic energy demand has been investigated by several authors such as Akiyama [10], and Choi and Kim [11]. However, the inter-story drift is accepted in many codes as the criteria for evaluation of seismic performance. Several researchers have developed a damage index model based on cumulative plastic deformation and maximum hysteretic energy demand [7, 12, 13]. The Bojorquez et al. damage index model is chosen in this paper since it is suitable to evaluate the damage level of the steel structure. This formula is based on energy concept and expressed as

\[ DI = \frac{E_{HSC}}{E_{HSD}} \]  

(4)
is the hysteretic energy capacity \( E_{HYSC} \) is the hysteretic energy demand, respectively. \( DI \) is an indicator of the damage of the structure, and in the range of \( 0 < DI < 1.0 \). If \( DI = 0 \) then there is no damage in the structure, while if \( DI = 0 \) indicates the condition as a failure. Bojorquez et al. [13] proposed hysteretic energy capacity with the assumption that the yield is concentrated at both ends of WF flanges in the beams, as given in equation (5).

\[
E_{HYSC} = \sum_{i=1}^{N_{st}} 2N_{bay}Z_{sf}F_y \theta_p F_{EHi}
\]

(5)

where \( N_{st} \) is the number of the story; \( N_{bay} \) is the number of bay; \( Z_{sf} \) is the plastic modulus of flanges; \( F_y \) is the yield stress of steel material; \( \theta_p \) is the cumulative plastic deformation at both ends of the beam and is taken 0.23; \( F_{EHi} \) is the energy contribution factor as expressed in equations (6) and (7).

\[
F_{EH} = \min \left\{ F_{EHi}, 1 \right\}
\]

(7)

where

\[
F_{EHi} = \frac{1}{2.33h/H} \exp \left\{ -\frac{1}{2} \left[ \frac{\ln(h/H) - \ln 0.52}{0.49} \right]^2 \right\}
\]

(8)

Arjomandi et al. [14] estimate the value of the damage indexes correlated to FEMA performance-level based on Bozorgnia and Bertero works, the estimation is listed in table 3.

| Performance level | Damage index | Damage state          |
|-------------------|--------------|-----------------------|
| IO(\( S_1 \))     | 0.11         | Elastic to minor      |
| DC (\( S_2 \))    | 0.25         | Minor to moderate     |
| LS (\( S_3 \))    | 0.44         | Moderate to heavy     |
| LSR (\( S_4 \))   | 0.67         | Heavy to severe       |
| CP (\( S_5 \))    | >1.0         | Collapse              |

5. Result and discussion

5.1. Time history of hysteretic energy demand

Figure 6 provides information on hysteretic energy for both MRF and DRF frame. It was found that the hysteretic energy depends on the ground motion characteristics and the lateral stiffness of the structure. The largest and the smallest hysteretic energy for MRF system were caused by Tabas, Imperial Valley, and Northridge earthquake, respectively. The presence of steel damper not only significantly reduced the hysteretic energy of structures (especially in Tabas earthquake), but it also changed the behavior of the structure in absorbing the energy that was induced by the earthquake. It can be seen that the most substantial hysteretic energy demand for DRF system is due to Imperial Valley. Furthermore, the use of steel damper with the value of \( SR = 3 \) and \( SR = 4 \) demonstrates good behavior in the reduction of hysteretic energy demand.
5.2. **Hysteretic energy demand distribution along the height of structure**

Figure 7 shows the distribution of hysteretic energy demand for both MRF and DRF system, respectively. It was clear that the distribution of hysteretic energy demand over the height of structure shows a similar pattern for both MRF and DRF system. However, the maximum hysteretic energy demand occurred on the first floor for the MRF system, while a similar occurrence happened on the second floor for DRF system. Also, the maximum hysteretic energy demand for MRF system is caused by Tabas ground motion, while the Imperial Valley earthquake causes the maximum hysteretic energy demand in the DRF system. Although the reduction percentage of hysteretic energy of DRF system is limited, most of the hysteretic energy demand occurred on steel damper, and it prevented several structural components from significant damages. In the case of Imperial Valley earthquake, the ratio of the hysteretic energy demand of DRF system to the MRF system that occurred in the structural components is 20%, 17%, 19%, and 46%, respectively.

5.3. **Inter-story drift**

Figure 8 display inter-story drift for MRF and DRF system. The value of the computed inter-story drift for MRF and DRF system were below the threshold of 2.5%, corresponding to the limit state of life safety (LS). The maximum inter-story drift for MRF system is 2.33% under Tabas earthquake, while for DRF system the maximum inter-story drift is 1.56%, it occurred in the DRF-5 frame under Imperial Valley. It was also observed that the maximum inter-story drift in the MRF system was due to the Northridge and Imperial Valley ground motions that occurred in the second floor, whereas Tabas ground motion occurred in the first floor. In general, there is a tendency that the higher hysteretic energy demands, the demand for the higher inter-story drift will also be higher. Furthermore, the presence of a steel damper and braces system reduced the inter-story drift significantly.
5.4. Global damage index

The global damage index, which was determined from equation (4), is plotted in table 4, the inter-story drift index was included for comparison. It was found that the MRF shows the largest inter-story drift index and damage index. Furthermore, the inter-story drift index of DRF frames is smaller than MRF frame due to the presence of braces and dampers. It was demonstrated that DRF-2, DRF-3 and DRF-4 showed the best structural performance level (IO level), while DRF-5 frame demonstrates damage control (DC level).

Table 4. Comparison between damage index and inter-story drift indexes.

| Frames   | $E_{HYSR}$ (kN-m) | $E_{HYSR}$ (kN-m) | Damage index | Inter-story drift (%) |
|----------|-------------------|-------------------|--------------|-----------------------|
| MRF      | 584.90            | 775.4             | 0.754        | 2.33                  |
| DRF-2    | 81.01             | 775.4             | 0.105        | 1.44                  |
| Frames   | $E_{HYSR}$        | $E_{HYSR}$        | Damage       | Inter-story           |

Figure 7. Distribution hysteretic energy demand over the height of structure.

Figure 8. Comparison of interstory-drift for MRF and DRF system.
6. Summary and conclusion
The behaviour and structural performance of a four-story of the moment-resisting frame both with and without hysteretic steel damper under three selected ground motion is presented in this paper. The ground motion characteristics (intensity, duration and frequency content) and the structural property (lateral stiffness) affected the responses of the investigated building. In general, the maximum inter-story drift index and maximum hysteretic energy demand occurred in the mid-story either for MRF system or DRF system. Furthermore, the MRF frame showed the worst performance level, either in terms of damage index or inter-story drift index. The use of steel damper and braces in the MRF frame reduced the inter-story drift index significantly and changed the damage state level of the structure. The selection of SR ratio is critical to exhibit an excellent structural performance level. Finally, the value of SR= 3 and 4 provide the best structural performance level and classified into elastic to minor damage state.

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