Damage-control evaluation of high-strength steel frames with energy dissipation bays

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This paper reports on a practical seismic-damage-control evaluation method of a high-strength steel (HSS) system comprising HSS frames with energy dissipation bays (EDBs). First, the design concept and seismic performance of the system are illustrated and verified by an experiment on a large-scale two-storey two-bay HSS frame with EDBs. Critical issues considering quantification of the seismic-damage-control threshold and the performance requirement of HSS members are clarified. A stepwise procedure that accounts for both the peak response of the HSS members and the cumulative effect of EDBs is then constructed by applying the energy balance concept and a micromechanics-based fracture model. Subsequently, the procedure is implemented on a prototype structure. The results indicate that the proposed procedure is effective for evaluating seismic performance of both HSS members and EDBs. Compared with direct application of non-linear history response analysis, which is time consuming, the proposed procedure simultaneously retains conceptual and computational advantages.

Notation

D_{ini}	damage index quantifying the limit state of energy dissipation bays (EDBs)	V'_{yEDB}	yield strength of the SDOF system
D_{prop}	crack propagation index	α	post-yielding stiffness ratio of the damage-control core (DCC)
dD_{ini}	incremental crack initiation index	Γ_1	participation factor of the fundamental mode
$d\epsilon_{eq}^p$	incremental equivalent plastic strain	γ	energy factor
E_a	nominal absorbed energy	δ	target roof displacement
E_e	nominal elastic energy	δ'	displacement of the equivalent single-degree-of-freedom (SDOF) system
E_h	hysteretic plastic energy	δ_e	maximum roof displacement of the corresponding elastic structural system
E_p	nominal plastic energy	δ'_e	maximum displacement of the equivalent elastic SDOF system
G	current absorbed energy of a unit area since the instant of fracture initiation	δ_{yEDB}	equivalent yield roof displacement of EDB
G_c	threshold value for absorbed energy of a unit area	δ'_{yEDB}	yield displacement of the equivalent SDOF system
M	moment of members	δ_{yHSS}	yield roof displacement of the HSS members
M_1	effective mass of fundamental mode	ζ	sequence factor
m	mass vector	ζ_T	threshold of the sequence factor
S	lateral load distribution vector	μ_a	accumulated ductility
S_v	pseudo-velocity	σ_e	effective stress
T	stress triaxiality	ϕ_1	modal vector of the fundamental vibration mode
T_1	fundamental period	ϕ_r	roof element of the modal vector corresponding to the fundamental mode
V_b	base shear corresponding to the target roof displacement	χ_{cr}	material parameter for the damage index
V_e	base shear corresponding to the elastic system	ψ	damage-control factor
V_{yEDB}	base shear corresponding to yield of EDBs		

1. Introduction

In order to expand the application of high-strength steel (HSS) (Bjorhovde, 2004; Miki *et al.*, 2002) in seismic regions, HSS structures implemented with energy dissipation members of mild carbon or low-yield steels have been proposed. Although the ductility of HSS members (Ellobody, 2012; Green *et al.*, 2002; Sause and Fahnstock, 2001) is generally not as satisfactory as those of mild carbon or low-yield steels, their excellent elastic performance and limited ductility are still usable for seismic-resistant structures. Furthermore, compared with some innovative seismic-resistant structures (Chey *et al.*, 2015; Fang *et al.*, 2014) that might be challenging due to the relatively complicated field construction efforts or more significant cost, HSS structures with energy dissipation members retain constructional and economic efficiency. Extensive research works considering HSS structures with energy dissipation members have thus been carried out. For instance, Wada *et al.* (1992) proposed damage-tolerant structures in which HSS members were expected to stay elastic under ground motions. Dual-steel structures involving HSS members have also been investigated (Dubina *et al.*, 2008; Ke and Chen, 2016; Tenchini *et al.*, 2014). These works indicated that when combining HSS members with energy dissipation members rationally, the system showed excellent seismic performance and inelastic deformation could be restricted to the energy dissipation members, thus achieving damage-control behaviour (Wada *et al.*, 1992). A fundamental issue for the application of these systems thus lies in the rational arrangement of HSS members to ensure that inelastic deformation is restricted to the energy dissipation members under expected earthquakes. However, as the combination of HSS members and energy dissipation members would generally lead to a variation of the hysteretic feature, applicability of current inelastic evaluation procedures (Kalkan and Kunnath, 2007; Lagaros and Fragiadakis, 2011) or empirical design equations (Chopra and Chintanapakdee, 2004; Hatzigeorgiou and Beskos, 2009) established on bilinear models with negligible post-yielding stiffness becomes questionable. In addition, since energy dissipation members are designated as the primary source of plastic energy dissipation and are expected to experience significant cumulative damage, they face the risk of fracture under a strong earthquake event. Therefore, cracking potential of energy dissipation members in these systems should also be critically assessed.

In this paper, a seismic evaluation procedure is established, with a focus on the damage-control behaviour of a proposed HSS structure (HSSs frame with energy dissipation bays (EDBs)). The design concept of the system is firstly illustrated. The rationality of the proposed system is then verified by means of a large-scale test. The test programme and the results are briefly described. Recognising that improper design might expose a structure to the risk of significant inelastic deformation development in HSS members or fracture due to excessive cumulative damage in EDBs, both the peak response and

the cumulative response of the structure should be involved in an evaluation procedure. To facilitate design and assessment, computational efficiency is also expected. Therefore, a stepwise evaluation procedure was established in this research. In this procedure, the performance of HSS members is assessed by a spectral method motivated by the energy balance concept, and the cracking behaviour of the EDBs is evaluated by a proposed fracture model. The procedure is applied to a preliminarily designed prototype structure for demonstration. To validate the accuracy of the procedure in identifying damage to HSS members, the predictions determined from the proposed procedure are compared with results extracted from non-linear response history analysis (NL-RHA) utilising the simplified structural model.

2. Design concept and test programme of HSS frames with EDBs

2.1 Design concept

Unlike a conventional steel moment-resisting frame (SMRF), the proposed system is realised by combining HSS members and energy dissipation beams of mild carbon steel, as illustrated in Figure 1(a). Hence, an EDB is constructed in a pre-selected bay. As the motive is to make the EDB dissipate energy, it is designed to possess a larger stiffness and smaller yield drift. Correspondingly, the remaining bays, composed of HSS members, are expected to be free of damage in a certain deformation range. Since the HSS has a larger yield stress, the elastic deformation capacity of a moment-resisting frame can be enhanced considerably. Accordingly, the early yielding of the EDB and the elastic behaviour of the HSS frame members lead to favourable damage-control behaviour, and a ‘damage-control stage’ is achieved (Figure 1(b)). In the ‘main structural damage stage’, in which the HSS members develop inelastic deformation, ductile behaviour can also be achieved owing to the energy dissipation capacity of the HSS members. Figure 1(b) shows a typical load–displacement curve of the proposed system. Compared with a conventional SMRF, which can be idealised as an elasto-plastic system with negligible post-yielding stiffness, the proposed system shows distinct post-yielding stiffness in the damage-control stage, and a bilinear kinematic hysteretic model can be used to describe the structural non-linearity, as shown in Figure 1(c). The hysteretic curve then forms a damage-control core (DCC).

In practice, even if the HSS frame is designed with the same sections as a conventional SMRF structure, the yield drift can be significant (around 1.5–2% for HSS with a yield stress of 460 MPa). In contrast, for a conventional SMRF, the yield drift is generally around 1%, and its increase inevitably involves heavy sections. To illustrate the attractiveness of the proposed system, two example structures are presented and compared in Figure 2. These systems meet the requirement of the Chinese seismic code (CMC, 2010). The design basic

Figure 1. Design concept of HSS frames with EDBs: (a) design concept; (b) non-linear feature; (c) DCC in the damage-control stage

acceleration is $0.4g$. A dead load of 4.8 kN/m^2 and a live load of 2 kN/m^2 are assumed. Frame A is representative of a conventional SMRF and all members are designed with a yield stress of 235 MPa . Frame B is a typical representative of the proposed system. Apart from the beams of the EDBs, which are designed with steel with a yield stress of 235 MPa , the remaining bays are designed with HSS of 460 MPa . Non-linear structural models were then established, considering a stress hardening ratio of 2% .

The pushover-unloading curves (push to a roof drift of 3%) of frames A and B considering the fundamental vibration mode are illustrated in Figure 2. As a conventional SMRF system, frame A develops entire yielding after the roof drift reaches about 1% , followed by negligible post-yielding stiffness. In contrast, the damage-control behaviour of frame B results in favourable significant post-yielding stiffness up to a roof drift of approximately 1.5% . More importantly, the clear enhancement of frame B does not involve heavier sections since the

Figure 2. Comparison of a conventional SMRF (frame A) and the proposed system (frame B). □, box section

weight and elastic stiffness of the structure are the same as frame A. In this context, the seismic demand of ground motions will not increase.

2.2 Test programme

To verify the seismic performance of the proposed structure, a large-scale welded frame with two storeys and two bays was designed and tested under cyclic loading. A preselected bay installed with energy dissipation beams of mild carbon steel is designated as the EDB and the remaining frames are constructed from HSS members with a yield stress larger than 460 MPa. Details of the experiment are given in Figure 3. During the experiment, a vertical load of 1500 kN, simulating the gravity load, was first applied and distributed to all the columns. The cyclic loads were then involved. To simulate the actual seismic action closely, lateral triangle loads (the ratio of the horizontal loads was 1:2) were applied. The loading protocol was modified based on AISC 341-10 (AISC, 2010a) by inserting the amplitudes of 0.2%, 1.25% and 1.75%.

The storey shear force versus storey drift curves considering the entire loading cycle and the DCC (maximum storey drift of 2%) are given in Figures 4(a) and 4(b), respectively. The model curve following the bilinear kinematic law is also presented. Before the HSS members developed significant inelastic deformation, there is satisfactory agreement between the bilinear model with significant post-yielding stiffness and the experimental curve. As a representative result, the moment distributions of structural members (members surrounding joint B in Figure 3) derived from strain data and damage phenomena at various load amplitudes are shown in Figure 5. In general, combination of HSS members and the EDB leads to a favourable damage evolution mode. After the EDB yielded at a drift of 0.5%, it began to dissipate plastic energy stably. With an increase in drift, slight local buckling was firstly detected in the beams of the EDB when the drift reached 1%. At this drift, the HSS members remained totally elastic. After the drift increased to 1.5%, the inelastic

deformation of the HSS members was insignificant. This is supported by the globally elastic behaviour illustrated in Figure 5. Therefore, the damage-control behaviour was achieved. After the drift reached 3%, all members developed plastic deformation. At this drift, visible fractures in the EDB were observed. However, the HSS members still exhibited ductile behaviour without failure.

3. A damage-control seismic evaluation procedure of HSS frames with EDBs

3.1 Quantification of structural non-linearity

To quantify the structural non-linearity of the damage-control stage, a sequence factor is defined as

$$1. \quad \zeta = \frac{\delta}{\delta_{yEDB}}$$

where δ is the target roof displacement and δ_{yEDB} is the roof displacement equivalent to yielding of the EDB, which can be solved by idealising the actual curve with a bilinear curve (Figure 1(b)). Note that the definition of this parameter can feature the yielding sequence of the entire system. In particular, a bilinear simplification is applied to the pushover curve in the damage-control stage, utilising two tangent lines going through the origin O and the point B in Figure 1(b) corresponding to the inception of yielding in the HSS frame. To ensure that the HSS frame is strictly damage-free, point B is determined by the first yield point in the HSS members. Thus, δ_{yEDB} is obtained from the intersection point of the two tangent lines. Correspondingly, the post-yielding stiffness ratio (α), defined as the ratio of the post-yielding stiffness to the initial stiffness (Figure 1(b)), is obtained. An equivalent target sequence factor denoting the HSS damage-control threshold is determined from

$$2. \quad \zeta_T = \frac{\delta_{yHSS}}{\delta_{yEDB}}$$

Figure 3. Specimen details and test setup of the HSS frame with EDB

Figure 4. Hysteretic curves: (a) considering entire loading cycle; (b) DCC

Figure 5. Damage-control behaviour reflected by members' internal moment distribution and the damage evolution at various loading drifts

where δ_{yHSS} is the yield roof displacement of the HSS members, corresponding to the HSS damage-control threshold. When the displacement is below this threshold, the HSS members are guaranteed to be damage-free.

3.2 Critical issues of damage-control behaviour and performance requirements

In general, the core of damage-control behaviour is the restriction of damage in EDBs under expected earthquakes. Thus, one method is to increase the gap between the yield drift of the HSS frames and the EDBs, which could facilitate restraining damage within the EDBs. However, even though this was considered during the specimen design, with the yield drift gap between the EDBs and the HSS frames intentionally enlarged, fracture of the EDBs occurred. This was due to the fact that significant damage was concentrated in the EDBs. In this regard, to achieve damage-control behaviour, both the HSS frames and the EDBs should be considered. This necessity can be further highlighted by the following examples. Single-degree-of-freedom (SDOF) systems following the bilinear hysteretic law with significant post-yielding stiffness verified by the experimental research are adopted here for clarification. In particular, a

series of SDOF systems with different DCC shapes (by varying α and ζ) is analysed under a ground motion (LA01, Imperial Valley, 19 May 1940, El Centro). The weight of the SDOF system is assumed to be 1000 kN. The results in terms of accumulated ductility (Nakashima *et al.*, 1996) given by Equation 3 and denoting the cumulative inelastic deformation demand of the structural members are presented in Figure 6.

$$3. \quad \mu_a = \frac{E_h}{(1 - \alpha)\delta'_{yEDB} V'_{yEDB}}$$

where E_h denotes the cumulative hysteretic energy dissipation and V'_{yEDB} and δ'_{yEDB} are, respectively, the yield strength and yield displacement of the SDOF system, corresponding to the yielding of EDBs in a structural system. For comparison, the result of perfectly elasto-plastic systems ($\alpha = 0$) representing conventional SMRF systems is also shown in the figure. It can be seen that an increase in the post-yielding stiffness ratio and the sequence factor both result in significant cumulative inelastic deformation demand, irrespective of the structural period (T_1). For the proposed system, since the accumulated ductility is expected to be experienced by the EDBs only in the

Figure 6. Accumulated ductility of SDOF systems of different DCC shapes

damage-control stage, a DCC of significant post-yielding stiffness ratio and sequence factor will likely expose the EDBs to the potential of failure.

In light of the above discussion, it can be seen that, although damage to HSS frames can be prevented by enlarging ζ_T and strengthening the HSS frame, the significant inelastic deformation introduced into the EDBs still needs to be considered. In this regard, the rational combination of HSS members and EDBs is a most critical issue for practical applications. Therefore, it is essential to explicitly account for the rational design of HSS members and their proper combination with EDBs. In particular, the performance requirements considering the damage-control stage are as follows.

- The HSS members should stay elastic under expected ground motions.
- When the HSS members stay elastic, the EDBs should provide a stable source of energy dissipation without the occurrence of fracture.

However, it is noted that these requirements are for the damage-control stage. For a system entering the main structural damage stage, the requirements will be dependent on specific design motives.

3.3 Quantification of the performance requirement

3.3.1 HSS member performance evaluation

According to the performance requirements, a critical issue to be solved is assessing the performance of HSS members under

expected ground motions. The modified energy balance concept is adopted here (Ke and Chen, 2014; Leelataviwat *et al.*, 2009; Sahoo and Chao, 2010; Wongpakdee *et al.*, 2014). The energy balance equation (Leelataviwat *et al.*, 2009) can be firstly applied, which is given by

$$4. \quad \gamma \left(\frac{1}{2} M_1^* S_v^2 \right) = E_e + E_p$$

where γ , M_1^* and S_v are the energy factor, the effective mass of the fundamental mode and the pseudo-velocity considering the fundamental vibration mode, respectively. The energy factor is defined as the ratio of the energy absorbed by an inelastic system to that of a corresponding elastic system under monotonic loading. E_e is the nominal elastic energy and E_p is the nominal plastic energy. Summation of E_e and E_p leads to nominal absorbed energy E_a , which is determined by the shaded area of the skeleton load–displacement curve with a target displacement (Figure 7(a)). Therefore, the result of the left-hand side of Equation 4 can be defined as the nominal energy demand and the right-hand side leads to the nominal energy capacity.

Based on the assumption that the system is dominated by the fundamental mode and the premise that the HSS members remain elastic, Equation 4 can be extended to structural systems using an equivalent SDOF system (Leelataviwat *et al.*, 2009). For a system satisfying damage-control behaviour reflected by the significant post-yielding stiffness provided by

Figure 7. Energy balance of HSS frame with EDBs in the damage-control stage (equivalent SDOF system): (a) the energy balance and the energy factor; (b) flow chart for computation of the energy factor

HSS members (Figure 7(a)), γ can be solved as

$$\gamma = \frac{E_e + E_p}{(1/2)M_1^* S_v^2}$$

$$= \frac{(1/2)V_{yEDB}\delta'_{yEDB} + (1/2)(V_{yEDB} + V_b)(\delta' - \delta'_{yEDB})}{(1/2)V_e\delta'_e}$$

where V_{yEDB} , V_b and V_e are, respectively, the base shear corresponding to yielding of the EDBs, the base shear corresponding to the target roof displacement and the base shear of the corresponding elastic system. It is noted that V_{yEDB} , V_b and V_e of the equivalent SDOF system are identical to the base shear of the structural system (Leelataviwat *et al.*, 2009), and δ'_{yEDB} , δ' and δ'_e of the equivalent SDOF system, which can be associated with structural features, are determined as follows.

$$\delta'_{yEDB} = \frac{\delta_{yEDB}}{\Gamma_1 \phi_r}$$

$$\delta' = \frac{\delta}{\Gamma_1 \phi_r}$$

$$\delta'_e = \frac{\delta_e}{\Gamma_1 \phi_r}$$

where Γ_1 , ϕ_r and δ_e are, respectively, the participation factor of the fundamental mode, the roof element of the modal vector corresponding to the fundamental mode and the maximum roof displacement of the corresponding elastic system. Therefore, with application of the equivalent SDOF system, the energy factor for HSS frames with EDBs can be determined from

$$\gamma = \psi[2\zeta - 1 + \alpha(\zeta - 1)^2]$$

$$\psi = \left(\frac{V_{yEDB}}{V_e} \right)^2$$

where ψ is defined as the damage-control factor. Computation of the energy factor γ can be solved by an iteration approach following the flow chart in Figure 7(b). Therefore, the performance of HSS members can be evaluated by performing a push-over analysis. Specifically, the lateral load corresponding to the fundamental vibration mode, determined by Equation 11, is applied to the structure.

$$S = m\phi_1$$

where S is the lateral load distribution vector, m is the mass vector and ϕ_1 is the modal vector of the fundamental vibration mode. Then, the non-linearity features (α , ζ and ζ_T) quantifying the shape of the DCC and HSS members' damage-control threshold can be determined based on the pushover curve utilising Equation 11. Therefore, the nominal energy demand diagram (Figure 8) can be constructed from Equations 4 and 9 by applying the sequence factor as the horizontal coordinate. Meanwhile, the energy capacity diagram in terms of the absorbed energy versus the sequence factor can also be obtained with the pushover curve, and the vertical coordinate can be determined by computing the work done by the lateral loads. Subsequently, a performance evaluation of the HSS members can be carried out with the constructed diagrams. When an intersection point of the demand curve and capacity curve is placed before the damage-control threshold, the nominal energy balance can be achieved before HSS members are damaged. Otherwise, occurrence of damage in HSS members is expected.

Figure 8. Performance evaluation of HSS members based on energy balance concept

3.3.2 Performance evaluation of EDBs based on micromechanics-based fracture model

In this research, to precisely account for the potential fracture of EDBs, a micromechanics-based ductile fracture model (Jia *et al.*, 2016) is adopted. Since the failure of beams in the EDB is typical ductile fracture under cyclic large-strain loading, the validated fracture model is applicable. Specifically, a damage index (Jia and Kuwamura, 2014, 2015), quantifying the crack initiation in a small increment, is given by

$$12. \quad dD_{ini} = \begin{cases} \frac{d\epsilon_{eq}^p}{\chi_{cr} \cdot e^{-(3/2)T}} & T \geq -\frac{1}{3} \\ 0 & T < -\frac{1}{3} \end{cases}$$

where χ_{cr} is a material parameter, $d\epsilon_{eq}^p$ is the incremental equivalent plastic strain and T is the stress triaxiality, defined by the ratio of the hydrostatic pressure to the equivalent Mises stress for metals. Ductile fracture initiation is postulated to occur when D_{ini} reaches 1. Thus, the damage index (D_{ini}) corresponding to the crack initiation is used to quantify the limit state of EDBs, given by

$$13. \quad D_{ini} = \sum (dD_{ini})$$

In the fracture model, a crack propagation rule is also employed using an energy approach (Jia *et al.*, 2016), where the corresponding crack propagation index D_{prop} is given by

$$14. \quad D_{prop} = \frac{G}{G_c}$$

where G is the current absorbed energy of a unit area since the instant of fracture initiation and G_c is the threshold value for absorbed energy of a unit area. G_c can be obtained based on Charpy impact tests (Jia *et al.*, 2016). Material deterioration is also considered by using the concept of effective stress (σ_e), given by

$$15. \quad \sigma_e = (1 - D_{prop})\sigma$$

To validate accuracy of the model for predicting ductile fracture of the EDB, a finite-element analysis (FEA) shell model was established using Abaqus. All members of the test frame were modelled with three-dimensional shell elements (S4R) based on the measured dimensions. Since out-of-plane braces were angle members that were relatively rigid in the axial direction, the 'MPC pin' simplification that idealises the braces with rigid truss elements was adopted. The procedure for the application of loads was the same as that in the experiments: the axial forces derived from the strain readings were first applied to the each column and kept constant during the whole simulation. Initial imperfections are considered in the model. Since this quantity was not precisely measured in the test programme, the magnitude of initial imperfections was determined by a trial-and-error procedure based on the buckling shape of the beams in the EDB. Eventually, a magnitude of $h/500$, considering buckling shapes identical to the specimen, was adopted for all the beams in the EDB. The fracture model was implemented in the material property and only assigned to beams in the EDB. To completely reflect the damage evolution, the crack initiation, crack propagation rules and deterioration of stress-carrying capacity were all considered in the EDB, which can be accounted for in the material module of Abaqus. The parameters of fracture properties were calibrated based on coupon test of the material following the

procedure proposed by Jia *et al.* (2014). During the modelling, sensitivity of mesh sizing was also studied and it was found that the mesh size had a minor influence on the results. Thus, a mesh size of 4 mm was adopted in the plastic zone of beams in the EDB and a mesh size of 1 mm was used in the welding area. As the welding wire and the welding process generally compromise the fracture behaviour of the material, elements in the weld region were considered by setting the fracture parameter as 30% of the base metal, based on a previous study on ductile fracture of materials in various subzones of a welded joint (Jia *et al.*, 2014), for the purpose of simplification. The crack initiation index (χ_{cr}) was equal to 2.2 and 0.7 for the base metal and the weld, respectively, and the crack propagation index (G_c) was 1.55 J/mm² for the steel Q235B considered in this study.

The numerical and experimental results of the tested HSS frame of the EDB are compared in Figure 9. The results show that the applied fracture model can be used to identify crack initiation and to account for crack propagation with satisfactory accuracy, strongly strengthening its potential in the proposed structure. To provide further understanding of the damage evolution in EDBs, the development of D_{ini} at the cracking location of spot A is plotted against the base shear force and the drift in Figure 10. It can be seen that when the drift is not significant, D_{ini} grows slowly with increasing drift, but the growth of D_{ini} becomes drastic at large drift amplitudes. Even though the specimen was loaded repeatedly with the same peak target drift, a substantial increase in D_{ini} was observed. This phenomenon exactly supports the findings regarding the SDOF system responses in Section 3.2.

Figure 9. Verification of the fracture model

Figure 10. Damage evolution at cracking spot A

More importantly, it can be seen that D_{ini} also increases during unloading; this is due to the fact that the restoring force of HSS members would lead to the yielding of EDBs in the unloading half cycle.

Considering that the motive of implementing EDBs in HSS frames is to provide stable energy dissipation under a ground motion or even a sequence of repeated ground motions, crack initiation of the EDBs is here taken as the limit state.

When applying the damage index for performance evaluation of a structure, the loading history of EDBs should be obtained first. In this regard, when performing an evaluation of EDBs, NL-RHA is needed first.

3.3.3 Damage-control seismic evaluation procedure

Based on the critical issues discussed above, it can be concluded that evaluating the damage-control behaviour of the system requires quantification of the performance of HSS members and EDBs simultaneously. In this regard, both the global response of a structure subjected to ground motions and the localised damage evolution in the EDBs should be explicitly assessed in practical design. However, the widely used design and evaluation procedures are still unable to comprehensively account for these issues, and running NL-RHA on detailed structural models in the evaluation procedure is time consuming. Therefore, a damage-control seismic evaluation procedure for HSS frames with EDBs was constructed, and the framework is given in Figure 11. In general, the entire procedure is composed of two modules. Specifically, module A can be used to quantify the performance of HSS members considering the potential inelastic behaviour, and module B is designated for the EDB performance evaluation with a focus on fracture identification. Essentially, the procedure starts with a non-linear static analysis to evaluate performance of HSS members first, corresponding to the first requirement discussed in Section 3.2. Since the second requirement is

subordinate to the first, the application of module B should be based on the evaluation results of the performance of the HSS members. In module A, ground motions that lead to intersections of demand curves and capacity curves (Figure 8) approaching the damage-control threshold can be regarded as the most dangerous, since these ground motions would lead to relatively significant peak inelastic deformation of the EDBs, contributing to a drastic increase in the damage index, as indicated in Figure 10. They can thus be used as excitations for module B. In cases where the energy dissipation of the EDBs can be ensured to be redundant (application of specialised material after validation) or the cumulative effect is insignificant, only module A is required. Therefore, the procedure provides sufficient flexibility in practice.

4. Application of the proposed procedure in damage-control evaluation

To illustrate the application of the proposed procedure, a six-storey HSS frame of EDBs was designed using the Chinese seismic code (CMC, 2010); the loading conditions used were the same as given in Section 2. Information about the design sections of the beams and columns is given in Figure 12(a). As a prototype structure, the EDBs were arranged in external bays. The HSS frame was designed with steel with a yield stress of 460 MPa. The beams in the EDB were designed with mild carbon steel with a yield stress of 235 MPa, and the reduced beam section (RBS) detail was considered to further enlarge the deformation range of the damage-control stage. The design of the RBS followed the Chinese seismic code (CMC, 2010) and AISC recommendations (AISC, 2010b). The plastic strength of the weakest section is also given in Figure 12(a).

Two models were developed (with Abaqus), considering evaluation module A or module B. Specifically, the model for module A was established based on beam elements (B31). For simplification, the connections were considered to be rigid in the model. To realise the RBS detail in the B31 elements

Figure 11. Flow chart of the proposed damage-control evaluation procedure

model, the yield stress of elements at the RBS location was reduced. The model for module B was established considering both beam elements and shell elements (S4R) (Figure 12(b)) to simulate fracture in the EDBs. The connection between the beam elements and shell elements was realised by means of kinematic coupling constraint. Thus, the loading history obtained from the NL-RHA based on the B31 model can be applied to the multi-scale model for module B. The vibration properties of the two models were compared to ensure that they would have almost identical non-linear dynamic responses when subjected to an earthquake (Table 1). In the model for module A, Rayleigh damping of 5% was considered for the first two vibration modes. For both models, the material property was assigned based on the coupon test described in Section 3.3.2. In particular, the combined hysteretic law was

considered for both the HSS members and the beams in the EBDs, and the fracture model was implemented in EDBs of shell elements. The mass was considered by the node mass distribution at floors. For the purposes of demonstration, a ground motion ensemble considering the stiff soil of Los Angeles (LA01–LA20) (Shome *et al.*, 1998) was assumed as potential earthquakes. The acceleration spectra of these ground motions with an elastic damping ratio of 5% are presented in Figure 12(c).

The procedure was as follows.

Step 1. The vibration properties considering the fundamental mode of the prototype structure were calculated and the results are given in Table 1. Good agreement

Figure 12. (a) Structural arrangement of the prototype structure. (b) Multi-scale model for module B in the procedure. (c) Ground motion acceleration spectra. □, box section

Table 1. Dynamic properties of the prototype structure (six-storey HSS steel frame with EDBs)

Property	Beam model: first mode	Multi-scale model: first mode
Period: s	1.35	1.40
Modal effective mass: t	471.3	467.2
Modal participation factor	1.29	1.29

between the beam model and the multi-scale model with beam and shell elements verifies the rationality of the modelling process.

- Step 2. After pushover analysis considering lateral load in proportion to the fundamental mode shape (applied to model with beam elements), the pushover curve can be obtained (Figure 13(a)). The shape of the DCC is confirmed.
- Step 3. The corresponding γ values considering the fundamental vibration mode associated with different sequence factors are plotted in Figure 14. Computation of this index is based on the self-developed algorithm BTESPEC (Ke *et al.*, 2015), following the flow chart given in Figure 7(b). In addition, to clarify the effect of the DCC, perfectly elasto-plastic systems for an identical elastic

Figure 13. Results of evaluation procedure regarding the prototype structure: (a) pushover curve and idealisation; (b) module A in the procedure, based on energy balance concept; (c) plastic energy dissipation of HSS members extracted from NL-RHA; (d) maximum damage index history (LA01)

Figure 14. Energy factor considering different hysteretic models

period were analysed. The clear difference induced by the different models validates the necessity of explicit consideration of the structural non-linear feature when combining HSS members with EDBs.

Step 4. A performance evaluation of the HSS members based on module A was performed; the results are shown in Figure 13(b). According to these results, it is predicted that there will be inelastic deformation in the HSS members in cases of LA09, LA10, LA11, LA13, LA14, LA15, LA16, LA17, LA18 and LA20 since the intersection points of the nominal capacity curve and the demand curves are above the damage-control threshold. To validate accuracy of the prediction, the plastic energy of the HSS members during ground motions computed by the NL-RHA is presented in Figure 13(c). The satisfactory accuracy of the prediction considering the damage in HSS members during ground motions demonstrates the rationality of module A.

Step 5. For cases in which the structure sustains damage in the EDBs without inelastic damage to the HSS members, module B regarding fracture identification can be conducted with information of the ground motions and the results extracted from the NL-RHA. The results of module A can provide an instructive reference for the most dangerous earthquakes that would induce fracture. Specifically, it was observed that the interaction points of ground motions LA01, LA02 and LA03 approach the damage-control threshold, indicating that the EDBs would also have to experience significant inelastic deformation. In this respect, they can be used as excitations for module B. As an example, LA01 is considered in this step. Specifically, after performing the NL-RHA based on the model with beam elements, the corresponding displacement history is then applied in the multi-scale model for module B by a static analysis. The maximum damage index (D_{ini}) response is then obtained (Figure 13(d)). Thus, there would be no fracture under this ground motion. However, it should be noted that ground motion LA01 corresponds to the design-based seismic hazard level. In this respect, the behaviour of the system subjected to more intense earthquakes should be critically assessed since damage to EDBs has already become significant under a design-based ground motion.

5. Conclusions

A seismic evaluation procedure regarding an innovative high-strength steel (HSS) structure (HSS frame with energy dissipation bays (EDBs)) has been proposed. With a focus on damage-control behaviour, rational requirements regarding the combination of HSS members and EDBs were clarified and an evaluation procedure was proposed in which a performance

evaluation of the HSS members and fracture identification in the EDBs are simultaneously employed. Compared with direct application of non-linear response history analysis (NL-RHA) considering the entire structure under a group of ground motions, which is computationally consuming, the proposed procedure is more efficient. The procedure was applied to a prototype structure and its rationality was validated. The main findings can be summarised as follows.

- Cyclic testing of a large-scale specimen indicated that a combination of HSS frames with EDBs is an effective way of realising damage-control behaviour. A damage-control core, quantified by a bilinear model with significant post-yielding stiffness, can be extracted.
- The seismic performance of the HSS can be reasonably evaluated based on an energy balance concept, and can be used to simplify or optimise the design of HSS members under provided ground motions. A comparison with conventional models indicated that the hysteretic feature variation induced by the combination of HSS members and EDBs would have a significant influence on the structural energy balance.
- The damage index extracted from the micromechanics-based fracture model can be used to identify the potential fracture of EDBs conservatively. However, the case study indicated that, for a code-designed structure subjected to a ground motion in the design-based hazard level, the damage index might be significant. The proposed procedure just puts forth a solution for identifying the potential fracture of EDBs under a provided ground motion.
- By following module A in the damage-control evaluation procedure, the performance of HSS frames under ground motions can be evaluated by performing non-linear static analyses. Potential earthquakes that would trigger fracture of the EDBs can also be targeted.
- The application of module B in the damage-control evaluation procedure can identify fracture occurrence associated with loading history of EDBs subjected to the targeted ground motions obtained in module A.
- The results of the case study showed that the combination of module A and module B in a stepwise procedure can effectively account for the performance of HSS frames and EDBs simultaneously. Compared with running NL-RHA on a detailed structural model subjected to a group of ground motions, computational effort is greatly reduced, and irrational design of HSS members and EDBs can be reasonably detected.

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