A Study on the Behavior of shear Link Beam made of Easy-Going steel in Eccentrically Braced Frames

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ABSTRACT :

The design of an eccentrically braced frame is based on creating a frame which will remain essentially elastic outside a well defined link. During extreme loading it is anticipated that the link beam will deform inelastically with significant ductility and energy dissipation. In this study, lower strength steel is used for the link beam according to the general concept of easy-going steel (EGS). The link beam made of easy-going steel yields in less displacement and so the energy dissipation capacity is significantly improved. Non-linear finite element models accounting for large displacements were developed according to the preceding experimental test specimens and the results are compared and good agreement was achieved between the measured and predicted behavior. After verifying the FE models, pushover and cyclic analysis was carried out for various link beam models made of ST37 steel and EGS. It was observed that the links made of easy-going steel had significantly better behavior. No local buckling was observed in the links and the hysteretic loops were stable.

KEYWORDS: Eccentrically Braced Frame, shear Link beam, Easy-Going steel, energy dissipation

1-Introduction

Seismic resistant eccentrically braced frames (EBFs) are a lateral load resisting system for steel buildings that are capable of combining high stiffness in the elastic range with good ductility and energy dissipation capacity in the inelastic range. In this system, the segment of beam placed between the braces absorbs the earthquake energy by large inelastic deformations and other members essentially remain elastic. In this paper, Easy-Going Steel is used to make the link beam and the behavior of link is studied and compared to those of a link made of constructional steel.

2- Easy-Going Steel (EGS)

Since the first use of steel in building construction, Human being has tried to increase the steel strength to decrease the size of structural members. In this way, the total weight of the structure decreases and it is assumed to be more economical. Increasing the steel strength and decreasing the cross section of structural members is not always efficient. In some cases and for some structural members, it is needed to decrease the steel strength as much as possible to improve the structural behavior. Examples for such situation are steel structures under earthquake or wind loading.

A useful method to increase the energy absorption of the frame is to use lower strength steel for the link beam. According to the general concept of easy-going steel, this lower strength steel is called Easy-going steel (EGS). The best EGS which is suggested for the link beam is pure iron with yield stress between 90 N/mm^2 to 120 N/mm^2. The percentage of the typical elements added to iron to make steel like Carbon, Manganese, Silicon and Chromium are much lower in easy-going steel (EGS) compared to other constructional steels.

The elasticity modulus of EGS is equal to that of other constructional steels. This significantly increases the EGS ductility, since the member made of EGS yields in smaller displacements and its energy absorption is increased. Stress-strain curves of constructional steel (ST37) and Iron (EGS) are shown in figure 4.

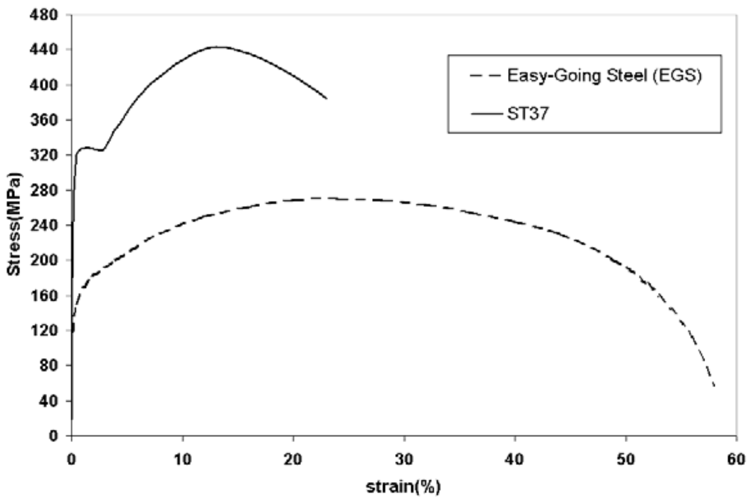

Figure 1: Stress-strain curve for Iron (EGS) and constructional steel

EGS is preferable to constructional steel or high-strength steel due to the following reasons:
- EGS has significantly higher ductility than other constructional steels.
- The elasticity modulus is equal for EGS and other constructional steels.
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Since the ultimate load carrying capacity should not change when EGS is used in the structural members, the thickness of these members should be increased due to the fewer yield stress of EGS. So the thickness of structural members made of EGS is greater compared to the thickness of the same members made of common constructional steel.

The use of EGS in lateral resisting systems such as the eccentrically braced frames (EBFs) or concentrically braced frames (CBFs), especially in the braces and link beams significantly increases the shear stiffness and decreases the shear displacements in different stories in a steel structure. By decreasing the lateral displacement, the moments in the vertical load-carrying members like columns are decreased. Moreover, by decreasing the lateral displacements, undesirable P-Δ effects are significantly decreased.

When the link beam in eccentrically braced frames is made of EGS, it yields in fewer displacements and the energy absorption of the link will increase, as it is shown in figure 2. Since the thickness of EGS link beam is increased compared to that of a link made of common constructional steel, local buckling in the flange and web do not occur and the hysteresis loops are more stable.

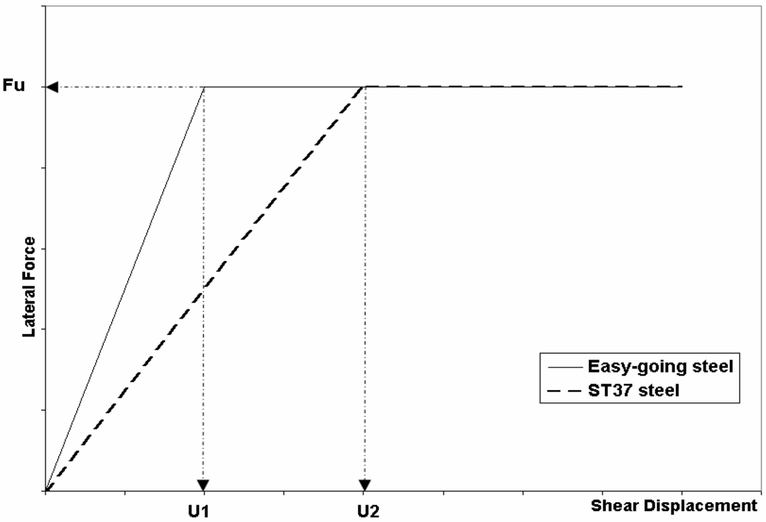

Figure 2: Comparison between shear force-displacement of members made of EGS and common constructional steel

3- Finite element modeling

To study the behavior of short links, ABAQUS finite element program is used. To verify the finite element modeling, models are developed according to the specimens tested by Popov et al and the results were compared. A classic method is also used to verify the force-displacement curves of links obtained by pushover analysis.

3-1 Specimens Geometrical and mechanical properties

The geometrical and mechanical properties of the tested specimen selected for finite element model verification is tabulated in table 1. This specimen is a short link and the stiffeners are provided on both sides of the beam. An arbitrary cyclic load was applied on the specimen and the result of the experimental test is given as force-displacement curve. These curves are used to verify the curves obtained by the finite element model.

Table1: Geometrical and mechanical properties of the specimen tested by Popov et al.

Section	t_f(mm)	b_f(mm)	t_w(mm)	D(mm)	A(mm^2)	ε_u(mm/mm)	σ_u(MPa)	σ_y(MPa)
W18*40	13.3	153	8.00	455	7613	0.26	441.41	331.06

3-2 Classic method

In this method, the web of the link is assumed as a shear steel wall and the stiffeners and the flanges are assumed to be the beams and the columns respectively in a steel frame around the shear steel wall.

The displacement of the link is divided in two parts: Shear displacement and bending displacement. The shear in the assumed frame is mainly supported by the shear wall (the link web) and the moment is resisted by the frame (stiffeners and the flanges of the link). The shear displacement is calculated for the frame and for the shear wall. The total shear displacement of a panel is equal to the summation of these two values. This value should be multiplied in the number of panels in the link beam.

The bending displacement of the frame is also calculated and added to the total shear displacement of the link to obtain the total displacement of the link beam.

3-2-1 Shear displacement

3-2-1-1 Shear displacement of the shear wall

Critical stress of a shear wall is calculated by equation 1:

$$\tau_{cr} = \frac{K\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{b}\right)^2 \tag{1}$$

Where t is the thickness of shear wall, b is the width of shear wall, E is the modulus of elasticity, μ is the poisson ration and k is calculated by equations 2 and 3:

$$K = 5.35 + 4\left(\frac{b}{d}\right)^2 \qquad \text{For} \qquad \frac{d}{b} \geq 1 \tag{2}$$

$$K = 5.35\left(\frac{b}{d}\right)^2 + 4 \qquad \text{For} \qquad \frac{d}{b} \leq 1 \tag{3}$$

Where d is the height of shear wall. The sear force and the shear displacement of a shear wall just before the buckling is calculated by equations 4 and 5 respectively.

$$F_{wcr} = \tau_{cr} bt \tag{4}$$

$$U_{wcr} = d\frac{\tau_{cr}}{\left(\frac{E}{2(1+\mu)}\right)} \tag{5}$$

For a thick shear wall, where the critical shear stress is more than the yielding stress of the wall, the buckling will not occur and the critical shear stress is calculated by equation 5:

$$\tau_{cr} = \tau_{wy} = \frac{\sigma_y}{\sqrt{3}} \tag{6}$$

σ_y is the yield stress which is determined by the coupon test. For the selected specimen, the geometrical and mechanical properties these are tabulated in table 2.

Table 2: geometrical and mechanical properties needed for calculation of τ_{cr}

d (mm)	b (mm)	t (mm)	σ_y (MPa)	μ	E (Mpa)
305	455	8	331	0.3	206000

By substituting the above values in the formulas 2 to 6, the critical shear stress τ_{cr} is equal to $191.14MPa$. F_{wcr} and U_{wcr} are equal to 695 KN and 0.73 mm respectively.

3-2-1-2 Shear displacement of the frame

The beam-to-column connections are assumed to be moment-resisting and the beam is assumed to be rigid and the behavior of frame is assumed to be elastic-perfect plastic, the shear displacement when the plastic hinges are formed in the columns is calculated by equation 7.

$$U_{fe} = \frac{M_{fp} d^2}{6EI_f} \tag{7}$$

where M_{fp} is the plastic moment of the columns and I_f is the moment of inertia of the columns. By substituting the corresponding values in the formula, the shear displacement of the frame is equal to 5.6 mm.
The total shear deformation of a panel is calculated by the summation of the shear displacements in the wall and the frame. It is obvious that most of the shear force is resisted by shear wall and the frame has a negligible proportion in shear resisting.
The value calculated for one of the link beam panels should be multiplied in the number of panels of the link beam. The link beam selected for verification has 3 panels and so the shear displacement of one panel should be multiplied in 3.

3-2-2 Bending displacement

It is assumed that the web of link beam cannot resist any moment. The total link beam is considered as a cantilever beam whose displacement can be determined by equation 8.

$$U = \frac{F}{\left(\frac{12 EI}{L^3}\right)} \tag{8}$$

Where E is the modulus of elasticity, I is the moment of inertia of the link and L is the length of the link. The bending displacements are calculated for the corresponding shear forces determined in previous part.
The total force-displacement curve of the link beam is obtained by summation of shear and bending displacements.

3-3 Finite Element model

Finite element program ABAQUS is used in this study. Shell elements are used for developing the models. The left side of the beam can move in horizontal direction and the left side can move vertically. Isotropic hardening rule was used for pushover analysis.
The pushover force-displacement curve obtained by experimental test, classic method and finite element method are compared in figure 3. As it can be seen, very good agreement is achieved.

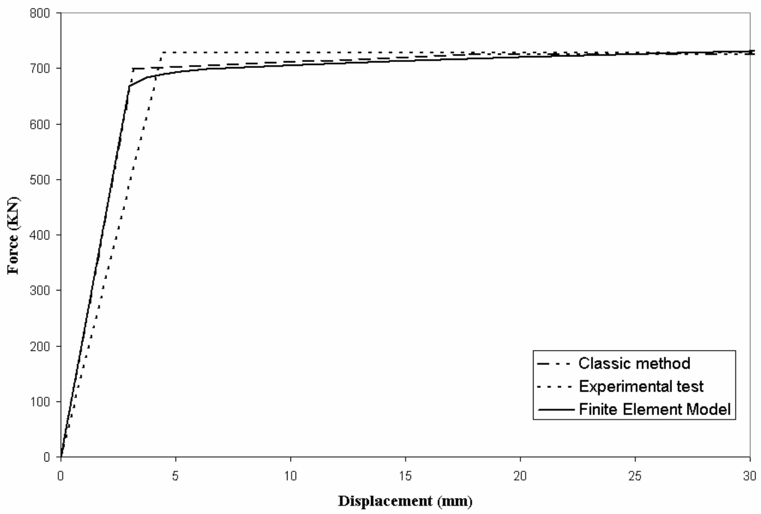

Figure 3: comparison between the force-displacement curves obtained by experimental test, classic method and finite element method

Moreover, the comparison is made for the specimens under cyclic loading. In this case kinematic hardening rule is used for the model. The curves obtained by the experimental test and by the finite element model are compared in figure 4. Again good agreement is achieved between the results. Since the Yield and ultimate stress of the specimens were not determined by coupon test and just the nominal values are reported, there are few differences between the curves.

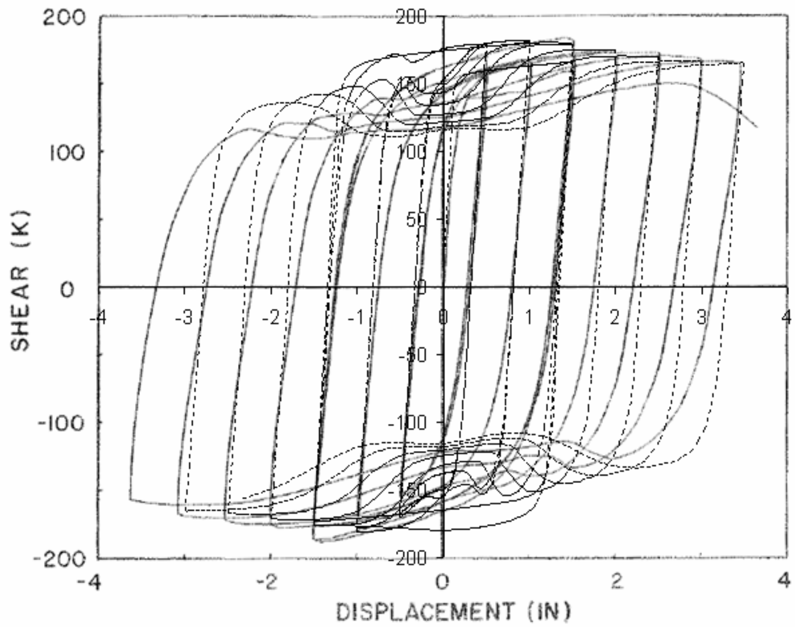

Figure 4: Comparison between the test results (solid lines) and model results (dashed lines)

4- Analysis of specimens under cyclic loading

After verifying the finite element model, the model made of constructional steel (ST37) which is called CSM were compared to the model made of EGS called EGSM. Applied loading protocols were the AISC provisions protocol

The yield stress, ultimate stress and the corresponding strains are tabulated in table 3 for constructional steel (ST37) and also EGS.

Table 3: yield stress, ultimate stress and the corresponding strains for constructional steel (ST37) and also EGS

	Yield point		Ultimate point	
	Stress (MPa)	Strain	Stress (MPa)	Strain
Constructional Steel	240	0.0012	360	0.26
EGS	120	0.0006	250	0.45

4-1 AISC loading protocol

AISC loading protocol is applied to both CSM and EGSM models. Short, intermediate and long links were designed and modeled using constructional steel and EGS. The energy absorption of each link is calculated and compared in the following figures for short, intermediate and long links respectively. As it can be seen, the hysteresis curves are improved and the absorbed energy is increased. Since the loading protocol in AISC provisions is not very severe, local buckling is very limited in the links made of constructional steel and so the benefits of using EGS cannot be fully observed here.

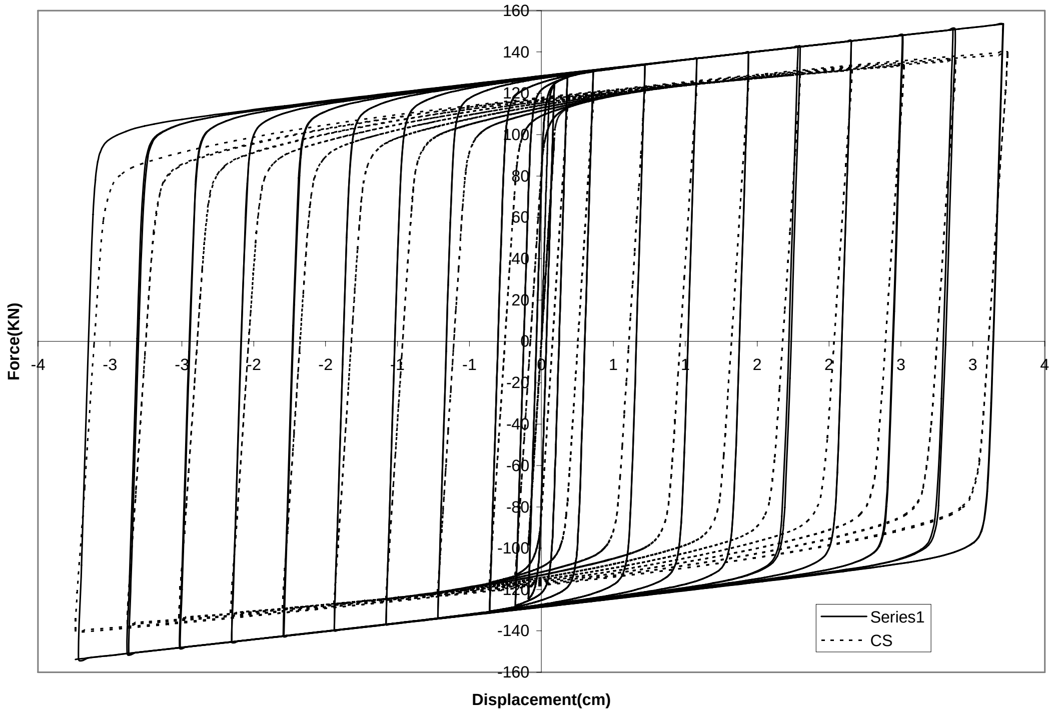

Figure 5: Comparison between EGS short link behavior (dashed lines) and constructional steel link results (solid lines)

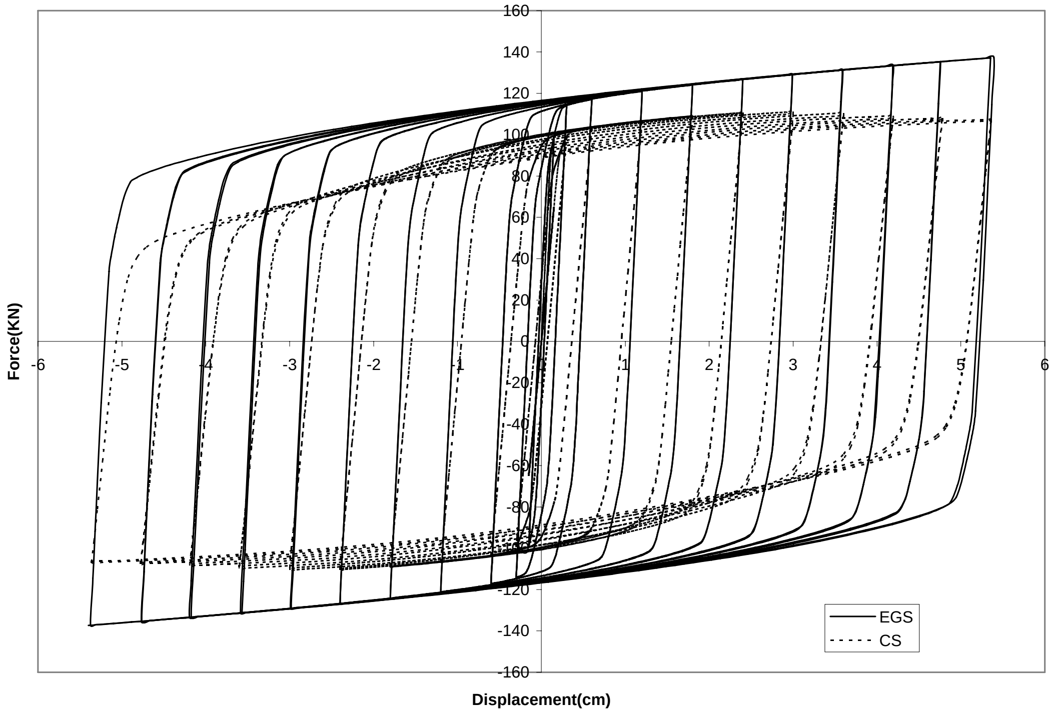

Figure 6: Comparison between EGS intermediate link behavior (dashed lines) and constructional steel link results (solid lines)

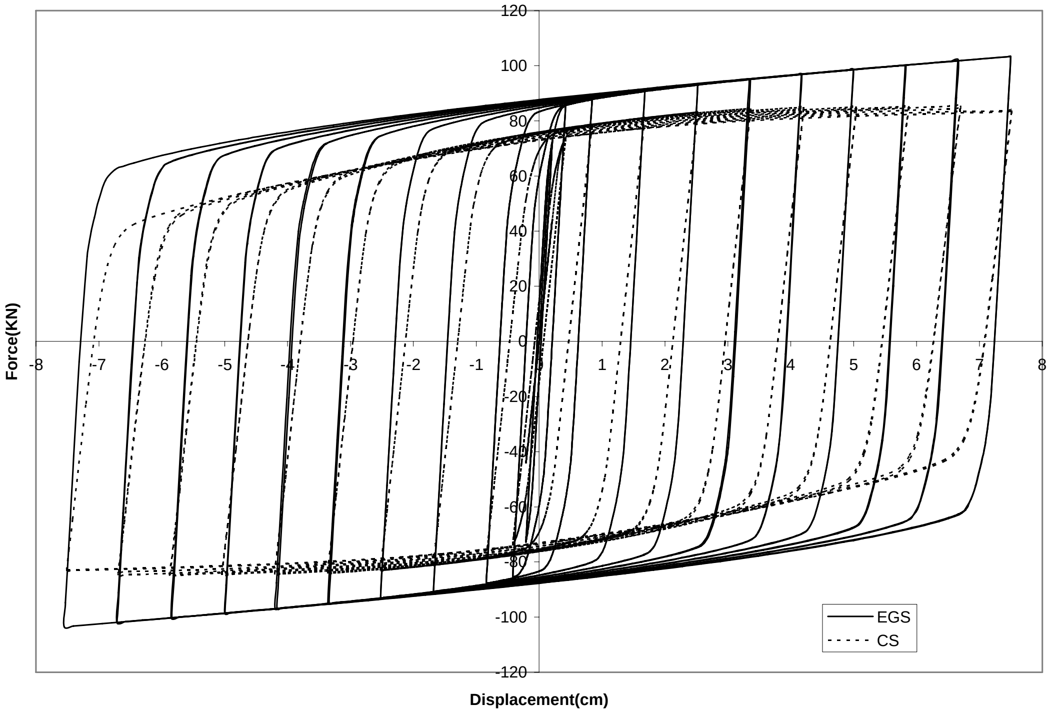

Figure 7: Comparison between EGS long link behavior (dashed lines) and constructional steel link results (solid lines)

5- Conclusion

In this paper, EGS is used instead of constructional steel in the link beam of eccentrically braced frames. The models were analyzed under cyclic loading protocol of AISC and the results were compared to the results of link beams made of constructional steel. The results showed that no local buckling occurred in the links made of EGS while the links made of constructional steel showed some local ducklings in the flange and web. Moreover, the hysteresis loops of EGS link are stable and the energy absorption is significantly increased compared to those of constructional steel links.

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ABAQUS finite element program manual