Numerical Behavior Study of Short Link, Intermediate Link and Long Link in Eccentrically Braced Frame Steel Structure

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Abstract

The paper discusses an analysis study on the design of Eccentrically Braced Frame (EBF) i.e. short link, intermediate link and long link by using diagonal web stiffener in the edge of the link. The analysis aimed at examining the influence of inelastic performance, particularly the effect of geometrical factors that occurred by its link and seismic hazard on the design performance of EBFs. The performance conditions are obtained from the link normalization to ratio capacity of plastic moment ($M_p$) and plastic shear ($V_p$). A numerical investigation was conducted on EBF portal system, Split K- Braces under three links condition. Subsequently, the analysis is performed by using SAP2000 and ABAQUS with the loading method is based on displacement control under the influence of cyclic parameter. In fact, to allocate the link parameters, spacing of web stiffener on each model is followed by using AISC -2010. A diagonal web stiffener is also added in each link scheme. The results indicate that the short link model considered have on a higher strength value and used as a proposed model when compared to intermediate link and long link model. It is stated by the failure mechanism as well, the failure is well-occurred in the short link condition. In addition, the added of diagonal web stiffener is possibly increase the link capacity. However, it could affect the performance behavior of link that typically proceed as a beam especially for a long link model.

Keywords: short link, intermediate link, long link, steel structure, eccentrically braced frame

INTRODUCTION

Eccentrically Braced Frame (EBF) structural system is a system that limits the inelastic behavior to only the link beam that lies between two eccentric braces, while the outer beam, column and diagonal braces remain elastic during the seismic loading. Therefore, Eccentrically Braced Frame (EBF) systems can meet high ductility levels such as Moment Resisting Frame (MRF), and can also provide high elastic stiffness levels such as Concentrically Braced Frame (CBF) [1]. Some possible placement of bracing for the EBF structure system is shown in Figure 1 [2].

The links in the EBF are formed from offsets at the braces connections on beam or braces adjacent to the columns so that during the seismic load the link becomes active and yielding [3]. Or in other words the link acts as a ductile fuse during an earthquake loading so that the link will undergo an inelastic rotation while the other components of EBF remain elastic [4]. The link behaves as a short beam with a shear force acting in opposite directions at both ends so that the moment produced at both ends of the beam has the same magnitude and direction. Figure 2 shows the force acting on the link where the deformation results S shape with the turning point in the middle of the span. The moment generated at both ends of the beam is equal to 0.5 times of the shear force multiplied by the length of the link.

There are three possible link beam criteria in the EBF structural system that are; short links, intermediate links and long links [5]. This criterias are determined from the normalization of link length with the ratio between plastic moment ($M_p$) and plastic shear capacity ($V_p$). The classification of these links is shown in Figure 3 [6] that are link with length ratios less than 1.6 is categorized are short links or shear links due to the more dominance of shear yielding. Links with a length ratio of more than 2.6 are categorized as long links or moment links due to the more dominance of shear yielding. Links with a length ratio of more than 2.6 are categorized as long links or moment links due to the more dominance of shear yielding. While links with long ratios ranging from 1.6 to 2.6 are categorized intermediate links or moment-shear links because the yielding occurred is a combination of shear and bending [5].

A study conducted by Musmar [7] showed that the EBF system with shear link was more stable and showed more ductility than the moment-shear link. This is due to the constant internal shear force along the links and the yielding on the web takes place along the web plane of the link. Numerical analysis carried out by Hashemi [8] to the EBF frame with long link criteria indicates that yielding on the link...
beam is because of the bending force. The energy absorption on the flange is less than the shear link condition due to the occurrence of premature buckling on the flange part of the link beam. To reduce this, it can be controlled by placing web stiffeners on the link beam although it is not very efficient because of the influence of torque. Yusisman et al. [9] and Budiono et al. [10] perform experimental testing and numerical analysis of short-link beam elements (shear links) and long links (bending links) using diagonal web diagonals (diagonal web stiffeners) shown in Figure 4. All of these models are then given cyclic loads according to AISC-2005 standards.

The use of long links is preferred in the architecture because it allows more use of the area under the link beam for opening area [11], while short links are always recommended in usage because it provides better ductility, stiffness and strength than other link types [1]. Hence many previous experimental and analytical studies are focused on studying the seismic behavior of short links.

Because of this reason, this study will analyze the three types of link beams applied to the model of the Split K-Braces EBF portal model to determine the behavior of each type of link beam. In addition, the role of other structural elements such as outer beams, columns and braces also affects the overall performance of link. So besides reviewing the three criterias of link beam, this research will also see the effect of variation of link length in one frame so that the behavior of the EBF structure system can be obtained completely. In addition to the effect on the length of the link beam, the variation of the stiffener configuration is also given to the link beam element that is the diagonal web stiffener refering to the research of Yusisman et al. [9] and Budiono et al. [10] in order to obtain also the effect of diagonal web stiffener use in each type of EBF system structure that has been determined.

**RESEARCH IMPORTANCE**

Experimental and numerical testing by previous researchers has shown that links that have shear yield (short links) provide great ductility and stability in resisting seismic loads. However, the possibility of giving an open area in the architecture makes shorter link selection sometimes insufficient. As a result, research on the length of the link is developed which is a link that experiences bending yielding. This research is to be a reference in the determination of link length on EBF structure planning and the use of diagonal link web stiffener.

**Figure 1:** Some Possible Placement of Bracing for EBF Structure System; (a) K-braces, (b) D-braces, (c) V-braces

**Figure 2:** Rotation Degree of Link for Each EBF System on Figure 1.
METHODOLOGY

In this study, numerical analysis of the EBF portal is divided into three portal models namely EBF-S, EBF-I and EBF-L. Each portal represents the category of short links, intermediate links and long links. The profile of EBF portal structure structure used is shown in Table 1. Each EBF portal has a width of 8 meters between columns and a height of 4 meters portal with the structure plan and frame line shown in Figure 5. The link length is determined from the capacity of plastic moment and plastic shear capacity as follows:

\[ M_p = Z_x f_y \]  
\[ V_p = 0.6 f_y (d - 2t_f) t_w \]

Using the link beam profile data and calculations with equations (1) and (2), as well as the link length classification in Figure 3, a 100 cm link length is chosen to represent a short link, a link length of 200 cm to represent a medium link and a link length of 300 cm to represent a long link as shown in Figure 5. Giving web stiffener on a link is required to prevent local buckling based on AISC [12] requirement shown in Table 2.
Table 1: Modelling of EBF Portal

<table>
<thead>
<tr>
<th>Floor</th>
<th>Column (KC)</th>
<th>Beam (WF)</th>
<th>Link Beam (WF)</th>
<th>Bracing (WF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-4</td>
<td>800x300x14x26</td>
<td>588x300x12x20</td>
<td>588x300x12x20</td>
<td>300x300x15x15</td>
</tr>
<tr>
<td>5-7</td>
<td>700x300x13x24</td>
<td>488x300x11x18</td>
<td>488x300x11x18</td>
<td>300x300x15x15</td>
</tr>
<tr>
<td>8-10</td>
<td>588x300x12x20</td>
<td>434x299x10x15</td>
<td>434x299x10x15</td>
<td>300x300x15x15</td>
</tr>
</tbody>
</table>

Tabel 2. Classification of intermediate stiffener distance and link rotation capacity (AISC, 2010)

<table>
<thead>
<tr>
<th>No.</th>
<th>Link Length</th>
<th>Link Type</th>
<th>Rotation</th>
<th>Maximum Stiffener Distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$1.6 \frac{M_p}{V_p}$</td>
<td>Full shear</td>
<td>0.08</td>
<td>$30 \frac{l_o}{d} - \frac{d}{5}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>&lt; 0.02</td>
<td>$52 \frac{l_o}{d} - \frac{d}{5}$</td>
</tr>
<tr>
<td>2</td>
<td>$1.6 \frac{M_p}{V_p} \leq e \leq 2.6 \frac{M_p}{V_p}$</td>
<td>Shear dominant</td>
<td>Can use number 1 and 3</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>$2.6 \frac{M_p}{V_p} \leq e \leq 5 \frac{M_p}{V_p}$</td>
<td>Bending dominant</td>
<td>0.02</td>
<td>$1.5 \frac{b_f}{d}$ from each link end</td>
</tr>
<tr>
<td>4</td>
<td>$e &gt; 5 \frac{M_p}{V_p}$</td>
<td>Full bending</td>
<td>Does not need intermediate stiffener</td>
<td></td>
</tr>
</tbody>
</table>

Figure 6. Cyclic Loading Protocol on EBF Portal

Figure 7: Modelling of EBF Portal
The model of each EBF structure design is made and analyzed using SAP2000 program version 14.2.5 to obtain the element forces and deformation results. Hinge properties is defined to elements in which plastic hinge is supposed to be occurred. The hinge properties on beam elements is defined to be caused by only strong-axis moment, column elements caused by axial-moment interaction and braces caused by axial only. Steel Design Check feature is then run for checking all elements fulfill the AISC [12] requirements.

After that, modeling is done using ABAQUS program version 6.14 towards the three models of EBF portal to obtain structural responses in single integrated system. Link, beams, columns and braces elements are modeled as Sholid 3D elements. The steel material data used are BJ41 steel (\(f_y = 250\) MPa, \(f_u = 410\) MPa) and elastic modulus \(E = 200000\) MPa. The material functions used in the analysis are same for all elements. The web stiffener is 10 mm thick and applied on both sides of the link beam. The connection between the elements are given in the Tie Constraints and Boundary Conditions applied in each portal model that is fixed joint at the column base. The loading assigned to the three EBF portal models is cyclic loading shown in Figure 6. A 50 mm of meshing is applied to obtain more accurate results. To prove the accuracy of result from ABAQUS, each portal is remodelled in SAP2000 and given a displacement control as pushover load. Plastic hinge location and portal deflection of the model is verificated from each program.

In the further modelling, the diagonal web stiffener is applied on the link by using configuration from Yurisman [9] and Budiono [10] experiment shown in Figure 8. The thickness of diagonal web stiffener is defined to be same with vertical web stiffener which is 10 mm.

**ANALYSIS AND DISCUSSION**

**Structural Analysis**

The result of the structural analysis shall be controlled by a certain limitations to determine the feasibility of the structure system which includes; mass participation control, vibration period control period, control of end-point spectrum response and drift control. Once the constraints are met, then it can be continued by controlling the cross-section of the structural elements used. For controlling the cross section is done by using the Steel Design Check feature in SAP2000. The result of the Steel Design Check and the color indicator in Figure 9 shows that the used cross section is still in safe condition, that is the maximum color indicator is green on the column element with the stress ratio value ranging from 0.5 to 0.7.

**Lateral Drift**

Figure 10 shows that the lateral displacement generated in the EBF-S building model is smaller than the other two models, and the EBF-L building model has the largest deck lateral displacement. With the EBF-S building model as a reference, the EBF-I and EBF-L building models increased by 8.36% and 16.35% respectively for x-direction and 8.20% and 16.13% for y-direction.

The same behavior applies to the floor drift except for decks which change to the opposite condition shown in Figure 11. The deck drift of the EBF-S building is larger than the other building models. EBF-I and EBF-L building models have a less 6.06% and 7.98% deck drift than EBF-S respectively for x-direction and 6.37% and 8.65% for y-direction.
Figure 9: Steel Design Check towards (a) EBF-S, (b) EBF-I, and (c) EBF-L structures

Figure 10. Lateral displacement (mm) of steel structure of each EBF model

Figure 11: Drift (mm) of steel structure of each EBF model

**Portal Behaviour Analysis using ABAQUS v6.14**

The behavior of the EBF-S, EBF-I and EBF-L models are discussed by taking each of the EBF portals on the bottom floor of the three model building models using ABAQUS software version 6.14 with a cyclic loading to obtain the behavior of each portal. The resulting output is a stress contour and element behavior on the EBF portal.

The behavior and stress occurring on the EBF-S portal due to cyclic loading are shown in Figure 12. Result in step-1 with the displacement of 15 mm indicates that the collapse mechanism on the link element has been seen, marked by the change of link beam form into elastic with the maximum stress of 266.25 N/mm² that occurs on the web. The initial signs of collapse in the links begin to be seen in step-3 which is marked by the significant gradation of the color contour on the web part with the maximum stress of 349.62 N/mm².
The entire web part finally reaches the ultimate stress ($f_u$) of 410 N/mm$^2$ at step-13 when displacement load is increased to 20 mm. In this condition, the link can be ascertained to have reached the plastic limit, so that the stress concentration that occurs begins to shift toward the outer beam of links, bracing and columns. This is indicated by the increasing color gradation, especially at the point of connection between beams and columns. In addition, the flange of the link beam in the link and beam joint area is deformed because of the local buckling.

Figure 13 shows the behavior and stress on the EBF-I portal. In step-1 with displacement of 15 mm, the maximum stress that occurs on the web is 250.46 N/mm$^2$. The sign of collapse on the link starts at the end of the link connected with the beam. This is seen with the change in the color contour gradation of the step-17 with the displacement of 20 mm where the maximum stress that occurs on the web of link beam is 367.96 N/mm$^2$. With the addition of the step especially the increase of displacement value to 30 mm at step-25, the stress on the web of the link beam has reached the ultimate stress ($f_u$) of 410 N/mm$^2$. The stress concentration that occurs begins to shift toward the outer beam of links, bracing and columns along with the increase of displacement at the given cyclic load. The flange of the link beam in the link and beam joint area is also deformed because of the local buckling.

The model portal EBF-L shown in Figure 14 shows the stress occurred on the link tends to be larger at the link end connected with the beam which is indicated by the color difference of the stress contour. When step-1 with initial displacement of 15 mm, the maximum stress value on the web of the link beam at the end is 250.25 N/mm$^2$. With the cyclic load increase at step 19 with 20 mm displacement, the stress result at the end also increased to 379.41 N/mm$^2$. When the displacement increases to 40 mm in step-37, the web of the link beam at the end has reached a ultimate stress ($f_u$) 410 N/mm$^2$. Local buckling also occurs in the flange of the link beam in the connection area with the beam. Similar to the previous two EBF models, the stress concentration also shifts toward the outer beam of links, bracing and columns but in this EBF-L model shows more significant behavior occurs in the beam-column connection area, as well as beam-link. The stress increase in this connection area causes the yielding to occur not only on links but also on beam and column elements.
Figure 15. Plastic hinge location of each portal model

Table 3. Deflection result comparison between ABAQUS and SAP2000

<table>
<thead>
<tr>
<th>Portal</th>
<th>Point 3</th>
<th>Point 4</th>
<th>Point 5</th>
<th>Point 6</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ABAQUS mm</td>
<td>SAP2000 mm</td>
<td>ABAQUS mm</td>
<td>SAP2000 mm</td>
</tr>
<tr>
<td>EBF-S</td>
<td>118.65</td>
<td>120.04</td>
<td>101.65</td>
<td>120.29</td>
</tr>
<tr>
<td>EBF-I</td>
<td>118.64</td>
<td>120.04</td>
<td>101.08</td>
<td>120.24</td>
</tr>
<tr>
<td>EBF-L</td>
<td>118.66</td>
<td>120.04</td>
<td>104.61</td>
<td>119.89</td>
</tr>
</tbody>
</table>

Result Verification

To verify whether the modeling created with ABAQUS has complied with the concept of the EBF system, the three portals is remodelled using SAP2000. In SAP2000 a pushover load by displacement control with the value equals to ABAQUS which is the step-53 cyclic load with displacement of 120.04 mm for the three EBF portal models. As shown in Figure 15, the review point on SAP2000 to be verified with ABAQUS is at points 3, 4, 5 and 6. Point 3 and 6 are joints between beam-columns, whereas points 4 and 5 are joints between links with external beams Links and bracing. Verification is done by comparing the mechanism of collapse that is the location of the plastic joints and the amount of deformation produced between the two softwares.

Figure 15 shows the starting and ending positions of plastic joint locations on the three models of the EBF portal. Overall, mechanism of collapse on the portal has been fulfilled that is occurred from the beginning. The plastic joint is occured on the link beam. The increase in displacement load causes other structural elements to start yielding which is marked by the occurrence of plastic joints on both columns and beams. The same mechanism is generated on each portal at the EBF portal modelling with ABAQUS.

From Table 3 it is found that the deflection result generated by ABAQUS and SAP2000 at each observation point shows that the value differences are slightly different so that the result of ABAQUS analysis has used an appropriate modeling and can be used for further analysis.

In addition, the ductility value of the three models of the EBF portal can be calculated by using pushover curve output of SAP2000. The ductility factor ($\mu$) is the ratio between the maximum drift ($\delta_m$) of the building structure upon reaching the conditions on the verge of collapse and the structure drift ($\delta_y$) at the time of the first yielding within the building structure. From the SAP2000 yield pushover curve shown in Figure 15, the ductility of the EBF structure can be calculated by using the formula:

$$\mu = \frac{\delta_m}{\delta_y}$$

Ductility of EBF-S structure:

$$\mu_{EBF-S} = \frac{\delta_m}{\delta_y} = \frac{119.25}{16.46} = 7.18$$

Ductility of EBF-I structure:

$$\mu_{EBF-I} = \frac{\delta_m}{\delta_y} = \frac{119.59}{24.52} = 4.88$$

Ductility of EBF-L structure:

$$\mu_{EBF-L} = \frac{\delta_m}{\delta_y} = \frac{119.92}{27.88} = 4.30$$

From the above calculation shows that the structure of EBF-S has the greatest ductility value among the three models that is equal to 7.18, EBF-L structure has the lowest ductility value that is equal to 4.30 and EBF-I structure has ductility value between the three models that is equal to 4.88. It can be concluded that the structure of EBF-S is more ductile than the structure of EBF-I and EBF-L.
Advanced Development using Diagonal Web Stiffener

The advanced development model is given on link elements that refer to the experimental research of Yurisman et al. [9] and Budiono et al. [10] by providing diagonal web stiffener. By modelling the three portals in the same step with the previous analysis without the diagonal of the web stiffener, the behavior and the stress result can be seen.

Figure 17 shows the behavior and stress on the EBF-S portal given the diagonal web stiffener. In step-1 with displacement of 15 mm, the maximum stress that occurs on the web is 257.53 N/mm² or decreased by 3.28%. The stress on the web increases to 341.60 N/mm² at step-3. Despite reaching the ultimate stress ($f_u$) 410 N/mm² at step-13 with displacement of 20 mm, there is a change in stress pattern that occurs in the web of the link. Not all parts of the web yields but the yielding more likely occurs at the end of the link. This suggests that with the addition of a diagonal web stiffener on the link simply affects the stress distribution along the web from the link beam. The flange of the link beam in the link and beam joint area is also deformed due to the local buckling effect.

The EBF-I portal model shown in Figure 18 gives result of varying stress contours on the link beam section. The stress at the end of the link beam is greater than the center, thus the effect of giving the diagonal web stiffener has been seen. The maximum stress on the web of the link beam end is 250.41 N/mm² or decreased by 0.02%. The same behavior also occurs in step-17 with the maximum stress at the end of the link beam of 403.53 N/mm². In step-25, the ultimate stress ($f_u$) 410 N/mm² on the web has been reached but only at the end of the border with the flange side. In other words the concentration of stress is more focused on this part rather than distributed evenly along the web plane as in the condition without the diagonal web stiffener. Local buckling also occurs in the flange of the link beam in the connection area with the beam.

Figure 19 shows the EBF-L portal stress contour with the addition of a diagonal web stiffener. In step-1 with initial displacement of 15 mm, the maximum stress value of 250.25 N/mm² on the web of the link beam at the end area occurs at the web end of the link especially in the intersection area of flange and the diagonal web stiffener. At step-19, the
maximum stress also occurs in the same area with the stress value that has reached the ultimate stress ($f_u$) 410 N/mm$^2$. In the next step, the same behavior also occurs. The web areas that experience an ultimate stress increasingly spread from the end of the intersection of flange and diagonal web stiffener to the middle of the web. In general, the addition of diagonal web stiffener on the link beam of the EBF-L portal causes the link behavior resembles a beam so that the initial collapse mechanism that should occur on the link becomes unfulfilled.

By looking at the comparison of Von Mises stress with displacements generated from the ABAQUS analysis as shown in Figure 20, the effect of changes in link length and the addition of a diagonal web stiffener to the performance of the EBF structure can be explained. The first yielding is firstly achieved on the EBF-S portal compared to other EBF portal types. With the addition of a diagonal web stiffener, the deformation result increases at the first yielding indicating the structure becomes more rigid due to the diagonal web stiffener. The same condition applies to EBF-I portals and EBF-L portals, but the EBF-L portal that adds diagonal web stiffener requires greater deformation so that the stress result reaches the ultimate stress. The use of a diagonal web stiffener on a long link causes the link to become more rigid and thus requires large deformations to yield the link, but on the other hand it can cause other elements of the EBF structure to also yield.

Thus it can be concluded that the portal model that uses short links is better and is recommended in its use on the structure compared with intermediate links and long links. The addition of a diagonal web stiffener to the link can increase the capacity of the link, but it can give more power to the link so that the collapse mechanism that should occur on the link becomes unattainable especially on long links.

**Stress-strain Diagram and Dissipation Energy**

The stress-strain diagram is taken on the link beam element of each EBF portal model with the initial conditions and the addition of a diagonal stiffener. The results given are shown in Figure 21(a).

From Figure 21(a), the EBF-S model that uses web stiffener based on AISC reaches maximum stress of 236.61 MPa at strain of 0.1415, while the model with diagonal web stiffener achieves maximum stress equal to 236.13 MPa with strain of 0.0677. The difference in stress values between the two types of stiffener placement is not much different, except in the strain value where the strain on the link beam with the diagonal web stiffener is much smaller than that without diagonal web stiffener.
In the EBF-I model, the difference in stress values between the link beam with and without the diagonal web stiffener begin to appear. For link without diagonal web stiffener, the maximum stress that occurs is 233.47 MPa with strain value of 0.0411. On the link with the addition of a diagonal web stiffener, the maximum stress that occurs is 210.79 MPa with the strain value of 0.0077.

For the EBF-L model, the stress value in the link beam element decreases. The maximum stress of 185.28 MPa with a strain value of 0.0071 is occured on the link without the diagonal web stiffener. The addition of diagonal web stiffener causes a significant stress reduction that becomes 120.60 MPa with the strain reaches 0.0035.

The stress-strain area of each of the EBF portal models as shown in Table 4 can explain the effect of link length and the addition of a diagonal web stiffener. The EBF-S portal has a larger stress-strain area from the three EBF portals, while the EBF-L portal has a smaller stress-strain area than the other EBF portals. The addition of the diagonal web stiffener causes a decrease in the area of stress-strain in each model of the EBF portal. The EBF-S portal decreases by 58.78%, the EBF-I portal decreases by 85.59%, while the EBF-L portal decreases by 76.45%.

The value of the energy dissipation is determined by the area of reaction force vs. displacement area produced by each EBF portal model in ABAQUS which is loaded with cyclic displacement control. Figure 21(b) shows a graph of the relationship between the forces, in this case is the reaction force, to the displacement result. EBF-S portals, EBF-I portals and EBF-L portals with stiffener based on AISC standards have relatively similar energy dissipation values but EBF-S portal has a tendency to be larger than other portals. The comparison of the energy dissipation value is shown in Table 4.

The table shows that the EBF-S portal has better energy dissipation than other EBF portal models. Due to the load given in the form of displacement control, the amount of energy dissipation of each EBF portal model with the diagonal web stiffener is decreased which indicates that the structure has increased strength and stiffness.
CONCLUSION

This research describes a result of a study done numerically with finite element approach to the behaviour of link on EBF portal by using variation of link length and the use of diagonal web stiffener on the link. Based on the discussion of the results of the analysis that has been done, the conclusions can be taken as follows:

1. The lateral displacement and drift generated in the EBF-S building model is smaller than the other two building models, and the EBF-L building model has the largest deck drift value. Thus the building structure using short link provides better response than intermediate link and long link.

2. The entire EBF portal model with the addition of web stiffener based on AISC has fulfilled the EBF system collapse mechanism that is yielding begins on link beam elements. The cause of collapse at short link is shear yielding on the web, while at intermediate link is the combination of shear and bending yielding, and for long link is bending yielding.

3. The addition of a diagonal web stiffener to the link can increase the capacity of the link but it can give more stiffness and power to the link so that the collapse mechanism that should occur on the link becomes unattainable especially on long links that cause links to behave like beam.

REFERENCES


