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Comparative Analysis of Buckling-Restrained Braced Frames in Eccentric Configuration (BRBF-Es) and Eccentrically Braced Frames (EBFs)

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by

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Background:

General:

Seismic loads are a consideration in building design. Structural design typically starts with a gravity loaded design, then extends to a lateral force design. Seismic events cause ground accelerations which affect building design. Dependent on the location and the weight of the building, the lateral forces associated with seismic design can vary and can control the design of a structure.

Seismic design around the world has various interpretations. In South America, walls are over-designed to allow for continued use following a seismic event. This approach limits architectural elements in the design. Additionally, as the building exterior is rigid, the approach may impose large forces on people and objects within the structure. In Japan, the buildings are often designed through seismic isolation. The idea is to limit seismic forces applied to the structural elements through a shift in the building's natural period of vibration. Seismic isolation is achieved through large, mechanical systems called base isolators. The base isolators are effective at dissipating, dispersing, and absorbing seismic loads. The primary drawback is that the base isolators are expensive to design and maintain (Gilsanz et al.).

In the United States, buildings are designed through capacity based design approach. The goal of capacity based design is to have the building fail at chosen locations (structural fuses) while giving occupants the ability to exit the structure safely (Gilsanz et al.). The structural fuse concept is similar to that of an electronics system where a fuse blows to protect the main hardware from high current. The structural fuses in building systems yield and provide ductility to protect the rest of the structural system from damage. In capacity based design, the rest of the members in the system (not including the structural fuse) are designed to be stronger than the ultimate capacity expected from the structural fuse. Designing this way limits the yielding to within the controlled areas having known ductility.

Eccentrically braced frames (EBFs) and buckling restrained braced frames in eccentric configuration (BRBF-Es) are examples of capacity based design systems. In EBFs, the structural fuse is the link. The rest of the members: the columns, the brace, and the beam, are designed at the capacity of the link. In BRBF-Es, the structural fuse is the brace. The rest of the members: the columns, the stub, and the beam, are designed at the capacity of the brace. The findings of (Prinz, 2010) prove, from a performance standpoint, that BRBF-Es could be a viable alternative to EBFs.

In certain building designs, architectural considerations can control the structural seismic system used. Research into eccentrically braced frames (EBFs) and buckling-restrained braced frames in eccentric configuration (BRBF-Es) are an effort to allow more flexibility to architects while giving structural engineers the system ductility necessary to provide safe performance during earthquakes.

Eccentrically Braced Frames:

Eccentrically braced frames are a ductile, braced frame system that provides alternatives to conventional moment resistant frames (MRFs) and a concentrically braced frames (CBFs). EBFs are sometimes advantageous, as they have the ductility and architectural flexibility of an MRF and the lateral stiffness of a CBF. The ductility in the frame is due to the link which, by design, is where the yielding is isolated. The stiffness is given by the brace, and, by isolating the yielding in the link, the brace is protected from buckling. This design allows for the frame to withstand minor seismic events due to its stability and “bend but not break” from more major seismic events due to its ductility. Additionally, the eccentricity in the design gives space for architectural elements like doors and windows to be used in the exterior and interior aesthetic design.

Buckling-Restrained Braced Frames in Eccentric Configuration:

Buckling-restrained braced frames in eccentric configuration are also a ductile, braced frame system. This system employs a buckling-restrained brace as a structural fuse. Buckling-restrained braces are made of a steel core covered in unbonding material, which is surrounded by concrete to restrain the core from buckling. The core and concrete are enclosed by a hollow steel section to restrain the brace from buckling and to seal the brace. A key design principle in braced frames is for the brace not to buckle. Remember, in EBFs this was accounted for by isolating yielding to the link to protect the brace from buckling. In BRBF-Es, this principle is taken care of due to the use of the buckling-restrained brace. As a ductile braced frame system, BRBF-Es are able to withstand minor seismic events with its stability and yield due to more major seismic events due to its ductility.

EBF to BRBF-E Comparison:

Comparing BRBF-Es to EBFs qualitatively, each system has advantages. The following comparisons are for EBFs and BRBF-Es that are designed to be equal in performance. First, BRBF-Es have a higher steel weight than EBFs. Therefore, BRBF-Es have more material cost than EBFs. The detailing cost associated with BRBF-Es is less than in the EBFs. Due to the beam splices, BRBF-E stub-column connections can be shop welded, while the EBF link-column connection must be field welded (as the beam spans entire bay). For welds of the same size, shop welds are approximately half the price of field welds. Next, the repetition of member size (beams and columns) in a BRBF-E is better design economy than EBF. In EBF design, there is more variety in member sizes. Having better design economy saves money by making it easier for contractors to construct the design. Additionally, BRBF-Es are easier to repair. As previously noted, BRBF-Es utilize beam splices which protect the beams (stub included) and columns when the buckling-restrained brace yields. EBFs incur damage to the beams when the link yields. Thus, when the link yields, the entire beam must be replaced during repair. Additionally, the primary damage to EBFs is to the beam which is on grade with the slab. Repairing a beam on slab is more difficult and more expensive. Also, this repair requires field welds for the EBF system. Conversely, the primary damage to BRBF-Es is to the buckling-restrained brace. This member is not on slab, and this system allows for more shop welds to repair (cheaper than on slab and field welds). Due to these attributes, it seems as if a BRBF-E would be cheaper to repair than an EBF. The ease of construction and repairs advantages in BRBF-Es may offset the initial cost of the material advantage of EBFs.

Goals of Project:

The purpose of this research project is to extend on the findings of (Prinz, 2010). There are three primary goals of this project. First, to gain experience designing buildings with EBFs and BRBF-Es with link-column and mid-bay braced configuration. Second, to determine which system (EBF or BRBF-E) is easier to design. Third, to calculate initial cost estimates on the given multi-story frames (Prinz, 2010) to determine economic viability of BRBF-E compared to EBF.

Design:

Determine Seismic Forces:

Design spectral accelerations were given from (Prinz, 2010). The S_{DS} value was 1.12g, and the S_{D1} was 0.63g. These spectral accelerations gave the site a seismic design category of E (section 11.6; ASCE 7-10).

To determine the seismic force the equivalent lateral force procedure was used (section 12.8; ASCE 7-10). The seismic response coefficient, Cs, was calculated using section 12.8.1.1

(ASCE 7-10). The value of the seismic response coefficient was determined to be 0.0743. Next, the seismic weight of the building was determined. For nodes 1-11, the seismic weight per floor was 3165 kips (Prinz, 2010). For node 12, the seismic weight per floor was 3425 kips (Prinz, 2010). The building had four braces per north-south and east-west direction. Therefore, the seismic weight per brace was 791.25 kips (node 1-11) and 856.25 kips (node 12). Summing these seismic weight per brace values, gave a total seismic weight of 9560 kips. Using equation 12.8-1, the seismic base shear, V, value was 710.65 kips. It should be noted that the seismic base shear was compared against the minimum lateral force (section 1.4.3; ASCE 7-10) and the seismic base shear controlled the design. The vertical distribution of seismic force, F_x, was calculated using section 12.8.3 (ASCE 7-10). The calculated values were detailed in Table 1.

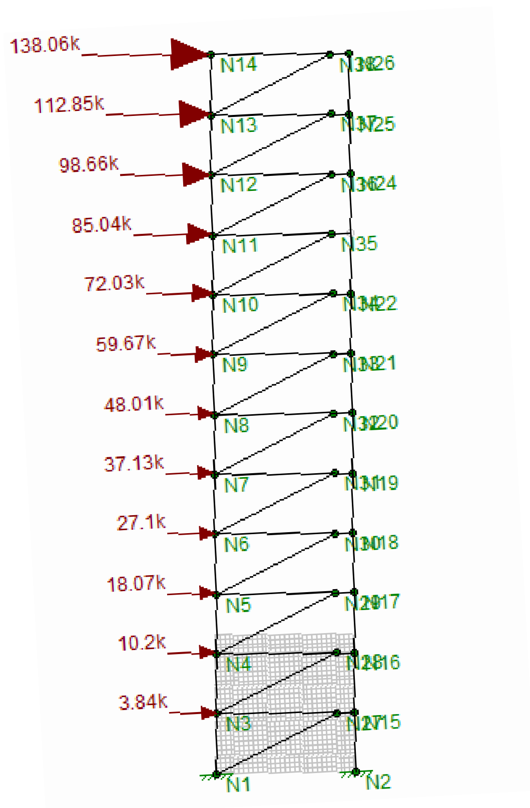
Table 1: Seismic Design Forces

Node	w (k)	h (ft)	$w_x h_x^k$	C_{vx}	F_x (k)	V_x (k)
1	791.25	13	29442	0.00540	3.84	710.65
2	791.25	26	78240	0.01435	10.20	706.81
3	791.25	39	138585	0.02542	18.07	696.61
4	791.25	52	207911	0.03814	27.10	678.54
5	791.25	65	284788	0.05224	37.13	651.44
6	791.25	78	368271	0.06756	48.01	614.31
7	791.25	91	457680	0.08396	59.67	566.30
8	791.25	104	552498	0.10135	72.03	506.64
9	791.25	117	652313	0.11966	85.04	434.61
10	791.25	130	756787	0.13883	98.66	349.57
11	791.25	143	865640	0.15880	112.85	250.91
12	856.25	156	1059025	0.19427	138.06	138.06
SUM=	9560		5451180	1	710.65	

The distributed seismic force was an applied force at that node. To determine the force within the members on each floor, the lateral forces were summed going down. This force was called the seismic design story shear, V_x, and the values per node were listed in Table 1. The structural fuses of the EBF and BRBF-E were sized using the seismic design story shear.

EBF Design:

The EBF design procedure used follows the procedure in AISC 341-05 (Seismic Design Manual, 2005). As an indeterminate system, RISA models were used to determine the forces in the link column and mid-bay EBF configurations (see Figure 1 and 2, respectively). Using the forces in the link, the link was sized using Table 3-1 of AISC 341-05.



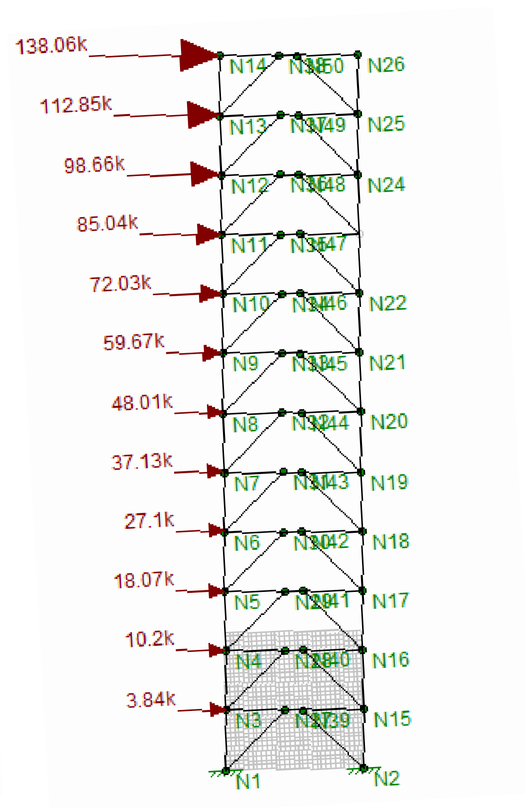


Figure 1: EBF Link-Col Model **Figure 2: EBF Mid-bay Model**

The remaining members were sized using an overstrength factor dependent on the link. This overstrength factor was calculated by dividing the shear capacity of the link by the actual shear in the link and multiplying by a member specific factor (1.21 for braces, 1.375 for beams, and 1.1 for columns). As the links change on each floor, the overstrength factor was floor specific. Additionally, the column loads on floors 1-6 were decreased by 30% to account for the findings in (Richards, 2009). (Richards, 2009) stated that column axial loads in the base were

55%-70% less than calculated by capacity based design for EBFs and BRBFs. The overstrength factor and base column load reduction were applied to the results of the RISA model. The braces, columns, and beams were then sized according to the capacity based design loads and AISC 341-10 (Steel Construction Manual, 2005). The EBF designs for the 12 story stub-column and mid-bay configurations (Appendix A: Table A1 and A2, respectively) were determined using the above procedure.

BRBF-E Design:

The BRBF-E design procedure used follows the procedure published in (Prinz, 2010). Using the design story shear, the forces in the buckling restrained braces were calculated using equations 3-1 and 3-2 (Prinz, 2010) for stub-column and mid-bay configurations, respectively. These forces were divided by the yield strength and the resistance factor to find the buckling restrained brace core area. Based on the ultimate brace strength, the remaining members were designed. The stub was sized using the maximum shear force and maximum moment in the member. The stub-column configuration maximum shear force values and maximum moment values were calculated using equations 3-3 and 3-4 (Prinz,2010). The mid-bay configuration maximum shear force values and maximum moment values were calculated using equations 3-5 and 3-6 (Prinz, 2010). Finally, the columns were sized as a beam column using the previously calculated moment values (stub-column only) and the axial force per equation 3-7 (Prinz, 2010) for stub-column and mid-bay configurations. The column loads on floors 1-6 were decreased by 30%, also, to account for the findings in (Richards, 2009). BRBF-E designs for the 12 story stub-column and mid-bay configurations (Appendix A: Table A3 and A4, respectively) were determined using the above procedure.

Design Comparison:

Comparing the BRBF-E and EBF design procedures, it was easier to design the BRBF-E system compared to the EBF system. The beam splices in the BRBF-Es allowed the axial forces, shear, and moment to be calculated using statics. EBFs were an indeterminate system requiring structural analysis software or intensive hand calculations. Therefore, the design cost was higher for the EBF system than the BRBF-E system. That being said, the member sizes in the EBF designs were considerably smaller than those used in the BRBF-E designs. With the construction cost being the primary cost of the buildings, the increase in design cost may be worth the investment depending on the material and detailing costs during the construction phase of the building.

Initial Cost Estimate:

Initial Cost Estimate Procedure:

Economic viability was determined by calculating the cost of a single lateral force resisting frame for the full building height. A complete building would have had multiple, and the buildings used in the initial cost estimate have eight lateral force resisting frames (four in each direction). Comparisons were made in 4 categories: bay length of 30 feet with I_e=1.0, bay length of 30 feet with I_e= 1.5, bay length of 20 feet with I_e = 1.0, and bay length of 20 feet with I_e = 1.5. The primary factors analyzed in these comparisons were material cost and detailing cost of link-column connection.

The buildings used in the initial cost estimate were the 3-story, 6-story, and 9-story, EBF and BRBF-E designs from (Prinz, 2010). Details were created of typical EBF link-column connection and BRBF-E stub-column connection. Figures 3 and 4 were sent to an anonymous steel fabricator for a cost estimate (steel fabricator was anonymous to protect their competitive advantage). Current steel price was researched to be $420 per US ton (SteelBenchmarker, Oct

2015). Typically, fabricator pricing was in the range of three to four times the material price. For this reason, a unit cost of $1500 per US ton was used for determining the material cost of frames. Using the steel weight of the frames, the unit cost, and the connection estimate from the steel fabricator, a spreadsheet was formulated to compute the total cost.

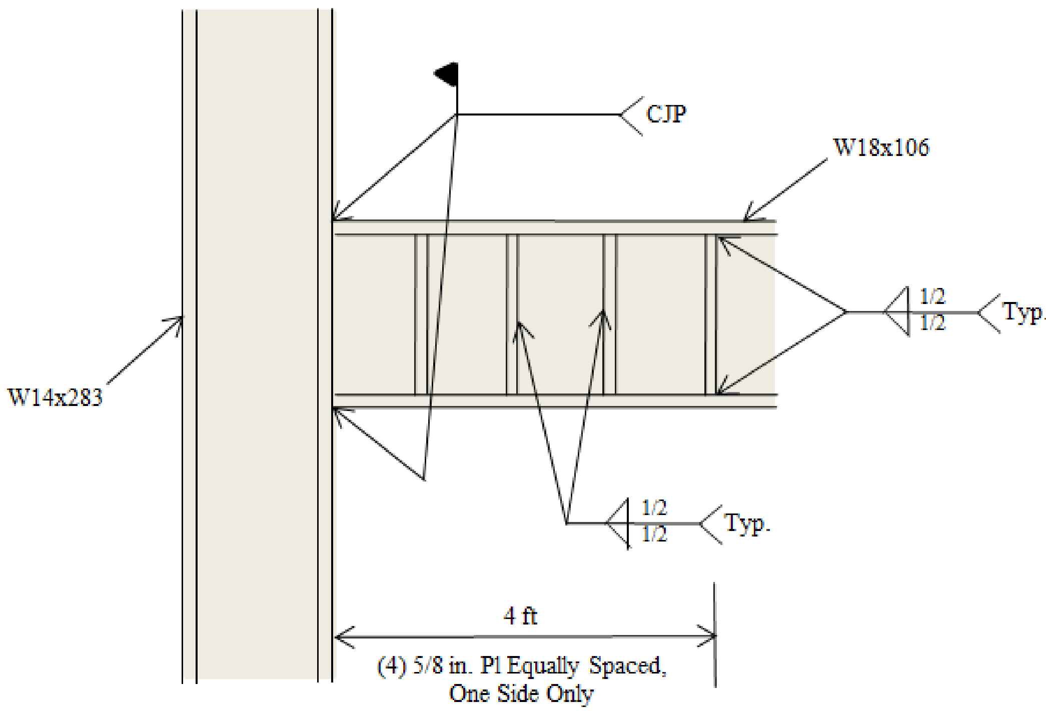


Figure 3: Typical EBF Link-Column Connection Detail

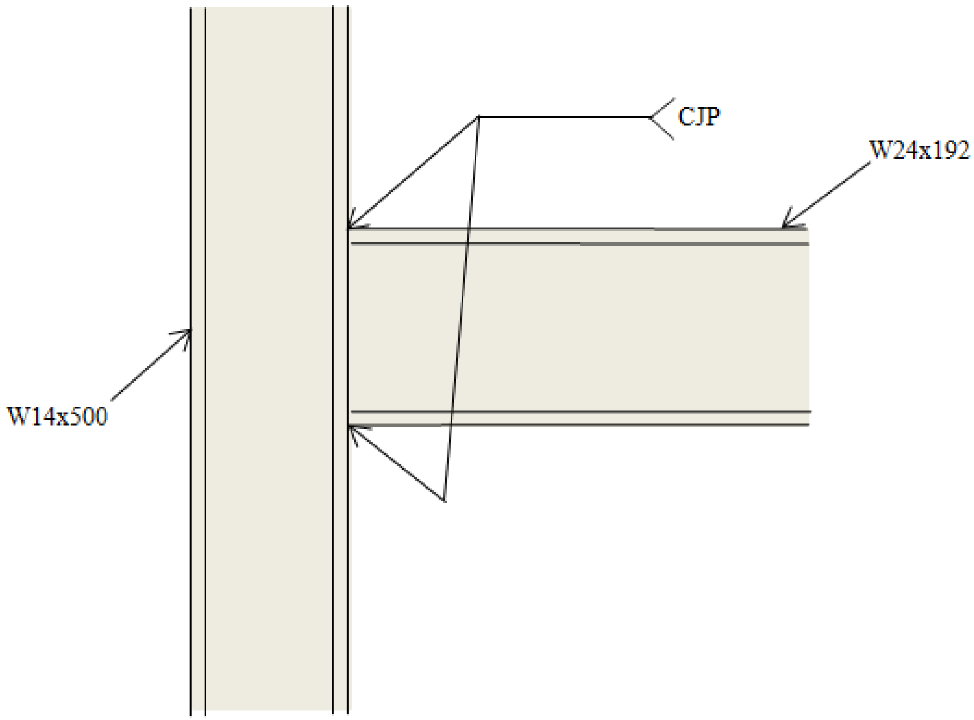


Figure 4: Typical BRBF-E Stub-Column Connection Detail

Material cost was computed by multiplying the unit weight of steel beams (lb/ft), the length of the member (ft), and steel unit price (US dollars/lb). The costs of all BRBF-E W-shape members were decreased by an aggressive 25% to account for potential discount for repetitive sizes from a steel fabricator. Additionally, buckling restrained brace costs were determined by an anonymous BRB manufacturer. Cost estimates received were for a brace length of 29 feet with core areas of 14.5 in^2 and 4.5 in^2 and a brace length of 20 feet with core areas of 14.5 in^2 and 4.5 in^2. BRBs for bay lengths of 30 feet and 20 feet had a brace length of approximately 29 feet and 20 feet, respectively. BRB prices were interpolated based on core area. All EBF and BRBF-E beams, braces, and columns were summed for material cost per frame.

Weld cost was calculated by using the steel fabricator estimate. The estimate was broken up into a flat cost per weld (weld prep and erection bolts), flat cost per stiffener, and cost of complete joint penetration welds. The cost of the complete joint penetration welds were adjusted based on the ratio of the flange areas of the specific member to flange areas of the member from the steel estimate (W18x106 and W24x192 for EBFs and BRBF-Es, respectively). The number of stiffeners used in the EBF cost was determined using minimum stiffener spacing for short links based on the beam size in Table 3-1 in AISC 341-05 (Seismic Design Manual, 2005). The link length of 48 inches was divided by the minimum stiffener spacing and rounded up to the next integer. The stiffener cost was calculated by multiplying the flat cost per stiffener by the number of stiffeners dictated by AISC 341-05. Total cost was computed by summing the material cost, weld cost, and stiffener cost (EBFs only).

Initial Cost Estimate Comparison:

Total costs were compared between the 24 buildings based on the previously stated four categories: bay length of 30 feet with I_e=1.0, bay length of 30 feet with I_e= 1.5, bay length of 20 feet with I_e = 1.0, and bay length of 20 feet with I_e = 1.5. Generally, the BRBF-Es were significantly more expensive. Table 2 summarizes the percent increase in initial cost to build a BRBF-E over an EBF. Figures 5 through 8 display the distribution of price dependent on height for each analysis category.

Table 2: Percent Increase in Initial Cost to Construct a BRBF-E over an EBF

Stories	% Increase in Total Cost			
	L= 30 ft, I_e= 1.0	L= 30 ft, I_e= 1.5	L= 20 ft, I_e= 1.0	L= 20 ft, I_e= 1.5
3	32%	76%	77%	93%
6	36%	49%	62%	70%
9	48%	45%	30%	28%

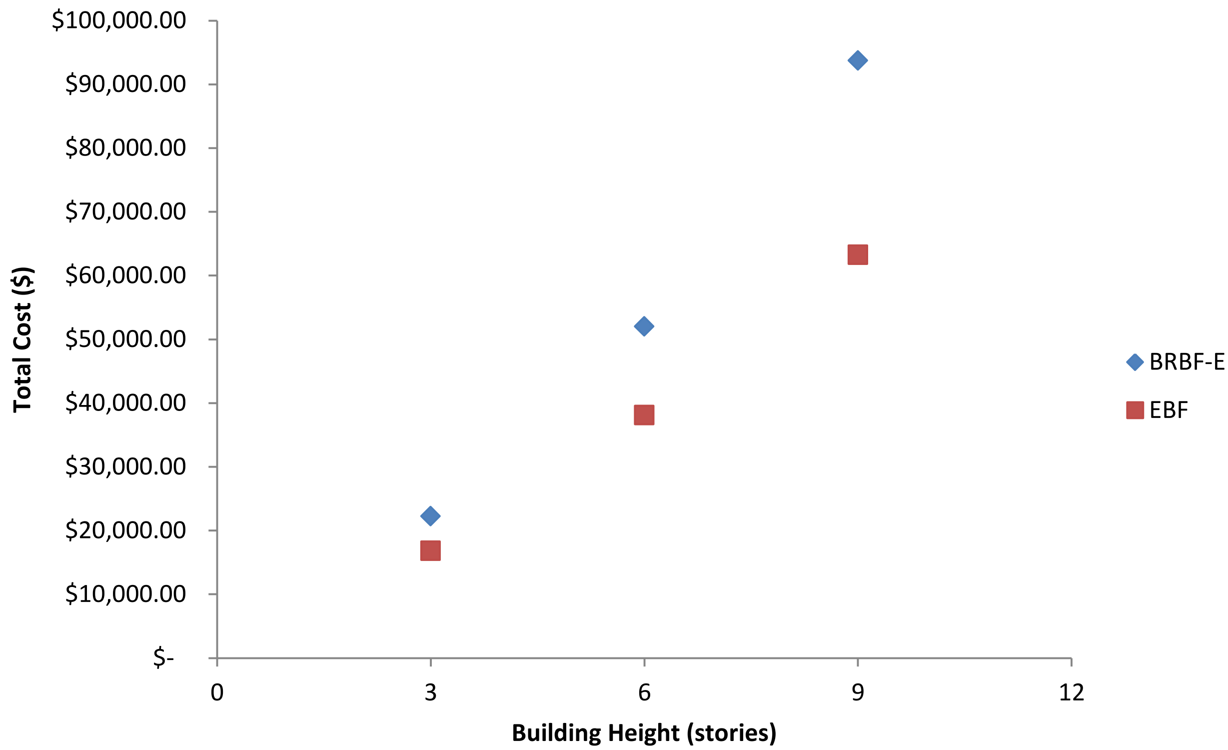


Figure 5: Total Cost Dependent on Building Height for Bay Length of 30 ft and I_e=1.0

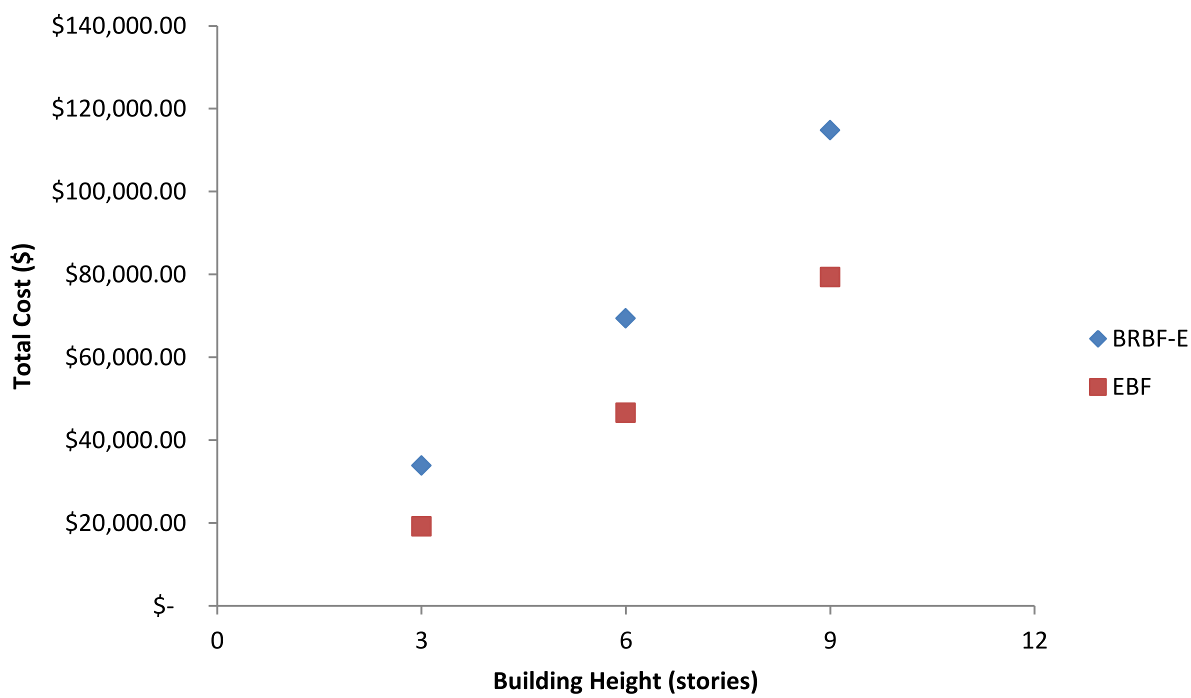


Figure 6: Total Cost Dependent on Building Height for Bay Length of 30 ft and I_e=1.5

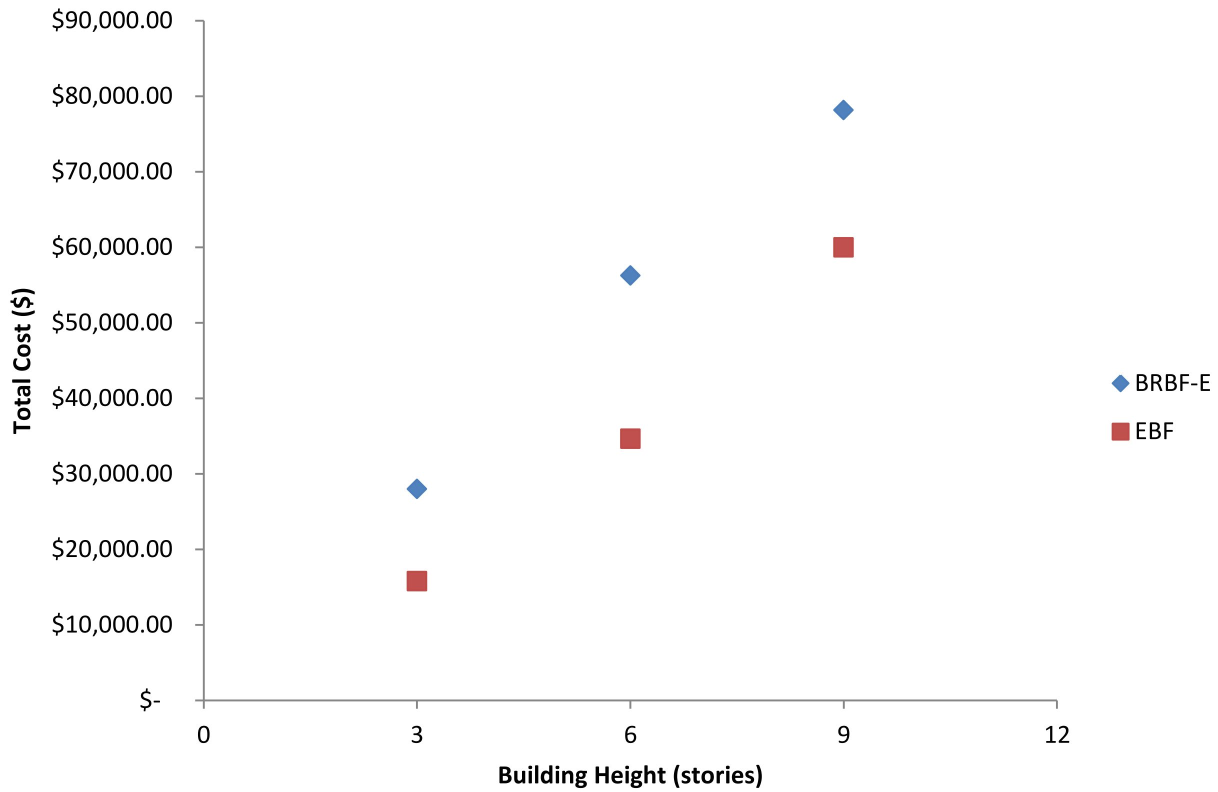


Figure 7: Total Cost Dependent on Building Height for Bay Length of 20 ft and I_e=1.0

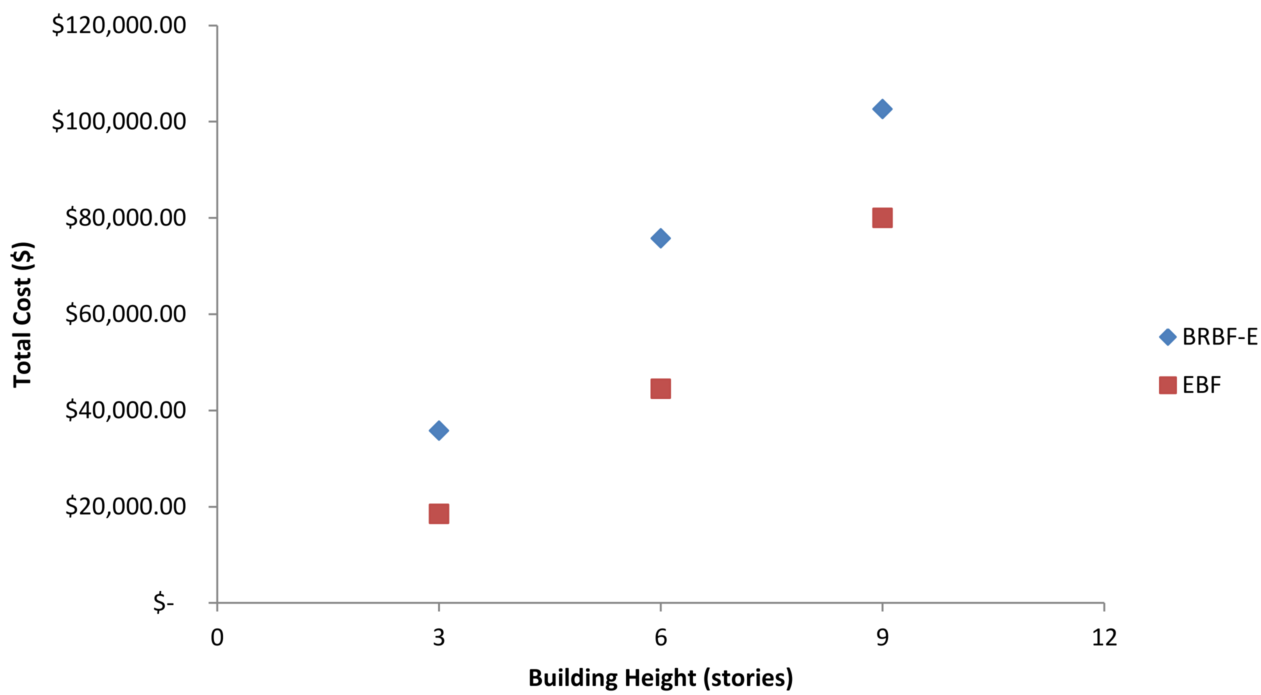


Figure 8: Total Cost Dependent on Building Height for Bay Length of 20 ft and I_e=1.5

With the exception of the bay length of 30 feet with I_e= 1.0 category, the design economy of the BRBF-Es made the systems cost more due to larger members being used when the strength was not required on the upper floors in the shorter buildings. With the taller BRBF-E buildings, typically, the increased variation of W-Shape member sizes allowed for more cost efficient designs. Tables 3 and 4 display the percent increase in material cost and detailing cost to build a BRBF-E over an EBF compared to the total cost of EBF system, respectfully.

Table 3: Percent Increase in Material Cost to Construct a BRBF-E over an EBF

Stories	% Increase in Material Cost			
	L= 30 ft, I_e= 1.0	L= 30 ft, I_e= 1.5	L= 20 ft, I_e= 1.0	L= 20 ft, I_e= 1.5
3	44%	83%	84%	98%
6	46%	55%	70%	75%
9	55%	50%	38%	33%

Table 4: Percent Increase in Detailing Cost to Construct a BRBF-E over an EBF

Stories	% Increase in Detailing Cost			
	L= 30 ft, I_e= 1.0	L= 30 ft, I_e= 1.5	L= 20 ft, I_e= 1.0	L= 20 ft, I_e= 1.5
3	-12%	-7%	-7%	-5%
6	-10%	-6%	-8%	-4%
9	-7%	-5%	-8%	-5%

Material cost of the frames was the primary constituent to the percent increase in total cost. This was because the W-Shapes of the BRBF-E were typically had 1.5 to 2 times the steel of the EBF frames. The BRBs had a similar increase in cost in relation to the W-Shape members used as braces in the EBF design. Steel fabricators did not give enough of a discount for repetitive sizes to cover the increase in material cost. In addition, the percent increase of BRBF-Es to EBFs in detailing cost was mostly constant throughout all designs with a range of a 5%-10% decrease to the total increase of cost of the system (dependent on bay length and seismic importance factor). A significant change dependent on height was not apparent. Therefore, the fluctuation in material cost depending on height of the building controlled the initial cost of the building.

The design portion of the comparative analysis determined that EBFs had an increased design cost due to the more intensive design calculations, but the investment in design may be worth it in the construction phase. Based on the significant increase in cost seen in the initial cost estimates, the increase in design cost for EBFs was worth the investment for the savings seen in the construction phase.

Conclusion:

This comparative analysis was composed of two parts: a design comparison and an initial cost estimate comparison. The design comparison involved designing four 12-story frames: BRBF-E with stub-column configuration, BRBF-E with mid-bay configuration, EBF with link-column configuration, and EBF with mid-bay configuration. Conclusions from the design comparison were as follows:

1. BRBF-Es were easier to design because to forces in the system can be calculated by statics. EBFs required a more intensive design process with structural analysis software to be efficient.
2. The increase in design cost may be worth the investment depending on the construction cost of the building.

Initial cost estimates were performed on 24 buildings representing four categories with respect to bay length and seismic importance factor (bay length of 30 feet with I_e=1.0, bay length of 30 feet with I_e= 1.5, bay length of 20 feet with I_e = 1.0, and bay length of 20 feet with I_e = 1.5). Each of these categories were analyzed for three building heights (3, 6, and 9 stories). The analysis compared total initial costs of BRBF-E and EBF lateral force resisting systems due to material cost and detailing cost. Conclusions from the initial cost estimate were as follows:

1. Material cost controlled the price of the BRBF-Es and EBFs. The detailing cost of BRBF-Es was cheaper than EBFs, but not nearly enough to cover the material cost increase.
2. Detailing cost of BRBF-Es remained constant about 5%-10% decrease in total cost compared to EBFs
3. Design economy of the BRBF-E system decreased the cost efficiency of the building (especially in shorter buildings). The increase in material cost did not keep up with a fabricator discount.
4. The increase in design cost for the EBF was justified to decrease the construction costs.

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Appendix A: Prototype EBF and BRBF-E Designs

Table Notes:

BM- Beam
BR- Brace
C- Column (CL- Left Column, CR- Right Column)

Table A2: 12-Story EBF Link-Column Configuration Design

Member	Shape (US Designation or BRB Area (in^2)
BM1	W24x84
BM2	W24x84
BM3	W24x84
BM4	W24x76
BM5	W24x62
BM6	W24x55
BM7	W24x55
BM8	W21x50
BM9	W21x44
BM10	W21x44
BM11	W21x44
BM12	W21x44
BR1	W14x120
BR2	W14x120
BR3	W14x120
BR4	W14x109
BR5	W14x109
BR6	W14x99
BR7	W14x99
BR8	W14x90
BR9	W14x90
BR10	W14x90
BR11	W14x90
BR12	W14x90
CL1-CL3	W14x176
CL4-CL6	W14x109
CL7-CL9	W14x90
CL10-CL12	W12x45
CR1-CR3	W14x233
CR4-CR6	W14x176
CR7-CR9	W14x159
CR10-CR12	W14x99

Table A3: 12-Story EBF Mid-Bay Configuration Design

Member	Shape (US Designation or BRB Area (in^2)
BM1	W24x84
BM2	W24x84
BM3	W24x84
BM4	W24x76
BM5	W24x62
BM6	W24x55
BM7	W24x55
BM8	W21x50
BM9	W21x44
BM10	W18x40
BM11	W14x38
BM12	W12x35
BR1	W14x99
BR2	W14x99
BR3	W14x99
BR4	W14x99
BR5	W14x90
BR6	W14x90
BR7	W14x90
BR8	W14x82
BR9	W14x74
BR10	W14x61
BR11	W12x53
BR12	W12x50
C1-C3	W14x176
C4-C6	W14x120
C7-C9	W14x90
C10-C12	W12x45

Table A3: 12-Story BRBF-E Stub-Column Configuration Design

Member	Shape (US Designation or BRB Area, in^2)
BM1-BM3	W24x229
BM4-BM6	W24x229
BM7-BM9	W24x192
BM10-BM12	W24x117
BR1	18.22
BR2	18.12
BR3	17.86
BR4	17.39
BR5	16.70
BR6	15.75
BR7	14.52
BR8	12.99
BR9	11.14
BR10	8.96
BR11	6.43
BR12	3.54
C1-C3	W14x605
C4-C6	W14x500
C7-C9	W14x398
C10-C12	W14x257

Table A4: 12-Story BRBF-E Mid-Bay Configuration Design

Member	Shape (US Designation or BRB Area, in^2)
BM1-BM3	W24x279
BM4-BM6	W24x279
BM7-BM9	W24x229
BM10-BM12	W24x146
BR1	21.30
BR2	21.19
BR3	20.88
BR4	20.34
BR5	19.53
BR6	18.41
BR7	16.98
BR8	15.19
BR9	13.03
BR10	10.48
BR11	7.52
BR12	4.14
C1-C3	W14x605
C4-C6	W14x398
C7-C9	W14x311
C10-C12	W14x109