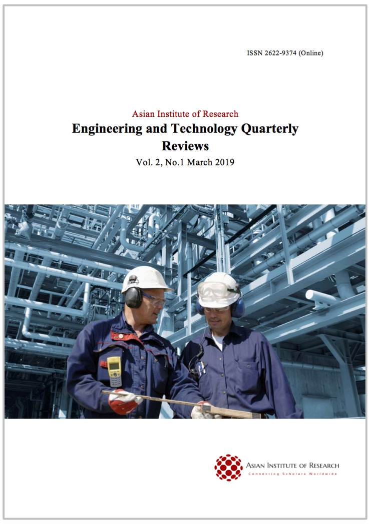

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Effect of Height on Retrofitting of Existing Steel Frames Using Buckling Restrained Brace Frames

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Abstract
Recent earthquakes indicate the importance of retrofitting existing structures to achieve an acceptable level of performance. Several different methods for retrofitting of existing structures were used by structural designers; Use of bracing systems is a cost-effective method for seismic retrofitting of existing steel frames. In particular, Buckling Restrained Braces (BRBs) are workable choices to be used because of their large energy dissipation capacity especially under moderate to severe earthquakes. Buckling restrained braces yield in tension and compression, exhibits stable and predictable hysteretic behavior. In this paper two existing structure with different heights are retrofitted with BRB. At next stage vulnerability assessment is done according to ASCE 41-06 with pushover analysis by SAP2000 software and compare the seismic parameters with each other to evaluate effect of height in retrofitted buildings with BRB. By having focus on results, it's observed that, Stiffness of four story building that retrofitted with BRB is larger than eight story but Earthquake energy that dissipated by eight story BRB is more also, four story BRB may undergo less lateral displacements than eight story BRBF and ductility of eight story is larger, so with increasing height, effective stiffness will decrease but ductility will increase. So it's better to use of BRB for retrofitting high raise building in moderate to high seismicity regions.

Key Words: Bracing System, BRB, Ductile, Retrofit, Steel Frame

1. Introduction

The hazard to life in case of earthquake is almost entirely associated with man-made structures such as buildings, dams, bridges etc. Prevention of disasters caused by earthquake has become increasingly important in recent years (Uang et al., 2001). Disaster prevention includes the reduction of seismic risk through retrofitting existing buildings in order to meet seismic safety requirements. The planning of alterations to existing buildings differs from new planning through an important condition; the existing construction must be taken as the basis for all planning and building actions (Prinz, 2007).

Many existing buildings do not meet the seismic strength requirement. The need for seismic retrofitting in existing building can arise due to any of the following reasons: (1) building not designed to code (2) subsequent updating of code and design practice (3) subsequent upgrading of seismic zone (4) deterioration of strength and aging (5) modification of existing structure (6) change in use of the building, etc. These buildings are more vulnerable, and in the event of a major earthquake, there is likely to be substantial loss of lives and property (Roeder et al., 2011).

Retrofit specifically aims to enhance the structural capacities (strength, stiffness, ductility, stability and integrity) of a building that is found to be deficient or vulnerable. In the specific context of enhancing the resistance of a vulnerable building to earthquakes, the term seismic retrofit is used. Sometimes, the terms 'seismic rehabilitation', 'seismic up gradation' and 'seismic strengthening' are used in lieu of 'seismic retrofit'. Seismic retrofit is primarily applied to achieve public safety, with various levels of structure and material survivability determined by economic considerations.

Several techniques have been used to retrofit buildings that have experienced structural damage as a consequence of moderate or severe earthquake shaking, or for the seismic upgrading of outdated existing buildings. Among these techniques, diagonal steel bracing has been considered attractive to enhance the lateral strength and stiffness of existing multi-story steel buildings. Nevertheless, it should be kept in mind that traditional braces (ductile or not) tend to exhibit global buckling when subjected to compressive strains, which in turn results in local buckling, fracture of the base material, and a highly unstable behavior under cyclic loading.

2. Buckling Restrained Braced Frames (BRBF)

The concept of buckling-restrained braces was introduced about thirty years ago in Japan by Uang and Nakashima in 2003. The idea behind a buckling-restrained brace is to fabricate a structural element that is able to work in a stable manner when it is subjected to compressive deformations (because braces are normally able to behave in a stable manner when subjected to tensile forces**.** The concept of eliminating the compression buckling failure mode in intermediate and slender compression elements has long been a subject of discussion. The theoretical solution for eliminating the buckling failure mode is very simple: laterally brace a compression element, at close regular intervals, so that the compression element's un-braced length effectively approaches zero (Bozorgnia and Bertero, 2004).

2.1 Components of BRBF:

BRBF composed of following components, as it is shown in figures 1 and 2.

- *Restrained yielding segment:* This steel segment can be rectangular or cruciform in cross section. Although it is common that a steel plate be surrounded in a casing, more than one plate can be used, if it is desired. Because this segment is designed to yield under cyclic loading, mild steel that exhibits high ductility is desirable. Also desirable are steel materials with predictable yield strength with small variations.

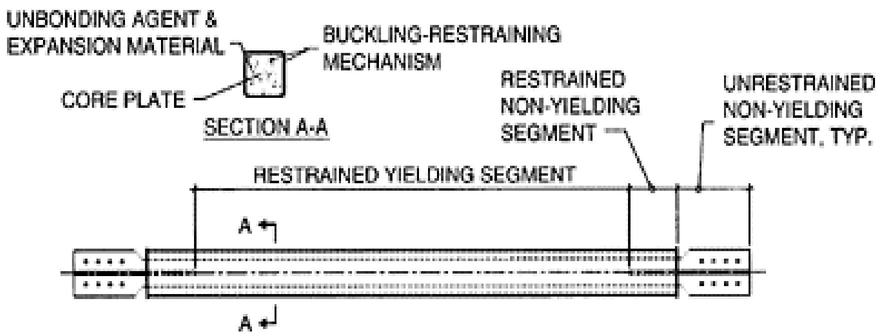

Figure 1. *Components of buckling restrained brace (Bozorgnia and Bertero, 2004)*

- *Restrained non yielding segment:* This segment, which is surrounded by the casing and mortar, is usually an extension of the restrained yielding segment but with an enlarged area to ensure elastic response.

- *Unrestrained non yielding segment*: This segment is usually an extension of the restrained non yielding segment, except that it projects from the casing and mortar for connection to the frame. This segment is also called the steel core projection.

- *Unhanding agent and expansion material:* Inert material that can effectively minimize or eliminate the transfer of shear force between the restrained steel segment and mortar can be used; materials like rubber, polyethylene, silicon grease and mastic tape have been reported.

- *Buckling-restraining mechanism:* This mechanism is typically composed of mortar and steel casing (Bozorgnia and Bertero, 2004).

The single-diagonal configuration is also an effective way to take advantage of the high strengths possible for BRBs. Note that neither X-bracing nor K-bracing is an option for BRBF. K-braced frames are not permitted for BRBF due to the possibility of inelastic flexural demands on columns. The chevron (V or Inverted-V) configuration is also popular for BRBF, as it maintains some openness for the frame. Because of the balance between brace tension and compression strength, the beam is required to resist only modest loads (Seismic Provisions, 2010)

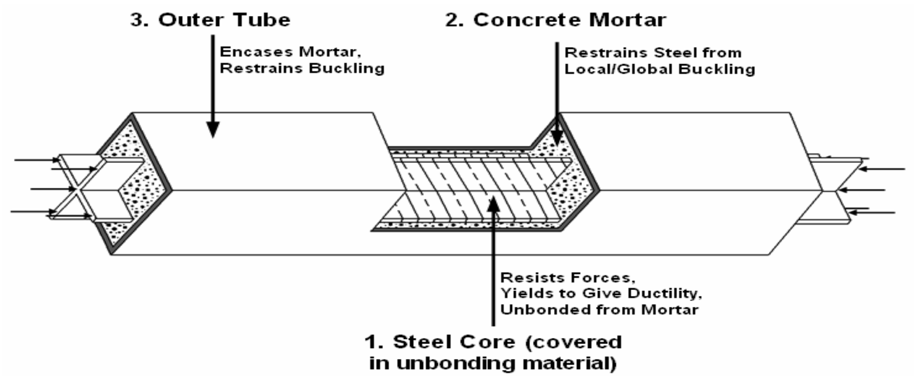

Figure 2. *Buckling-restrained brace cross section view (Prinz, 2007).*

3. Methodology

In this paper two existing structure with different height is retrofitted with BRB. At next stage vulnerability assessment is done according to ASCE 41-06 with pushover analysis by SAP2000 software and compare the seismic parameters with each other to evaluate effect of height in retrofitted buildings with BRB.

3.1 Design Methodology

According to AISC-2010, the steel core of BRBF is composed of a yielding segment and steel core projections; it may also contain transition segments between the projections and yielding segment. The length and area of the yielding segment, in conjunction with the lengths and areas of non-yielding segments, determine the stiffness of the brace. The steel core shall be designed to resist the entire axial force in the brace. The brace design axial strength, φP_{ysc} (LRFD), in tension and compression, in accordance with the limit state of yielding, shall be determined as follows:

$P_{ysc} = F_{ysc}A_{sc}$ (φ=0.90 (LRFD)) **(Eq. 1)**

Where; (A_{sc}): cross-sectional area of the yielding segment of the steel core, in^2 (mm^2) and (F_{ysc}): specified minimum yield stress of the steel core, or actual yield stress of the steel core as determined from a coupon test, (MPa).

3.2 Nonlinear Static Procedure (NSP)

The NSP is generally a more reliable approach to characterizing the performance of a structure than are linear procedures. The control node shall be located at the center of mass at the roof of a building. If the Nonlinear Static Procedure (NSP) is selected for seismic analysis of the building, a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components and elements of the building shall be subjected to monotonically increasing lateral loads representing inertia forces in an earthquake until a target displacement is exceeded.

Separate mathematical models representing the framing along two orthogonal axes of the building shall be developed for two-dimensional analysis. A mathematical model representing the framing along two orthogonal axes of the building shall be developed for three-dimensional analysis. Independent analysis along each of the two orthogonal principal axes of the building shall be permitted unless concurrent evaluation of multidirectional effects is required. The target displacement (δ_t) at each floor level shall be calculated in accordance with Equation 2:

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \qquad \textbf{(Eq. 2)}$$

Where, (C_0): modification factor to relate spectral displacement of an equivalent single-degree of freedom (SDOF) system to the roof displacement of the building multi-degree of freedom (MDOF) system. (C_1): modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response. (C_2): modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation, and strength deterioration on maximum displacement response. (T_e): effective fundamental period of the building in the direction under consideration, in seconds. (S_a): response spectrum acceleration at the effective fundamental period and damping ratio of the building in the direction under consideration ($S_a = A \times B$). g: acceleration of gravity. (C_0) coefficient is modification factor to relate spectral displacement of an equivalent single-degree of freedom (SDOF) system to the roof displacement of the building multi-degree of freedom is calculated using of the following table 1.

Table 1. Values for modification factor C_0 [1] (ASCE 41-06, Table 3-2)

	Shear building [2]		Other building
Number of stories	Triangular load pattern	Uniform load pattern	Any load pattern
1	1.0	1.0	1.0
2	1.2	1.15	1.2
3	1.2	1.2	1.3
5	1.3	1.2	1.4
10+	1.3	1.2	1.5

[1] Linear interpolation shall be used to calculate intermediates values
[2] Buildings in which, inter-story drift decreasing with increasing height.

According to ASCE 41-06, C_1 is calculated for linear elastic response shall be calculated in accordance with following equation:

$$C_1 = 1 + \frac{R-1}{aT_e^2} \qquad \textbf{(Eq. 3)}$$

Where; *a* = site class factor: =130 site Class A,
B; = 90 site Class
C; = 60 site Class D, E, and F;

C_2 for periods greater than 0.7 seconds, C_2=1.0. So C_2 shall be calculated in accordance with equation 4, as follow:

$$C_2 = 1 + \frac{1}{800}\left(\frac{R-1}{T_e}\right)^2 \qquad \textbf{(Eq. 4)}$$

4. Description and geometry of existing structure

A four-story residential building in Iran, Tehran was assumed as the sample building. This structure will be rehabilitated with some retrofitting techniques including BRBFs that are added as structural elements to the existing sample structure. For designing the sample building, a common place plan of a residential building in Tehran, which has been built before the Islamic revolution of Iran about years of 1975, is assumed. The configuration and plan of the sample building is shown in figure 3 and 4. The height of stories is assumed to be 3.2 meters.

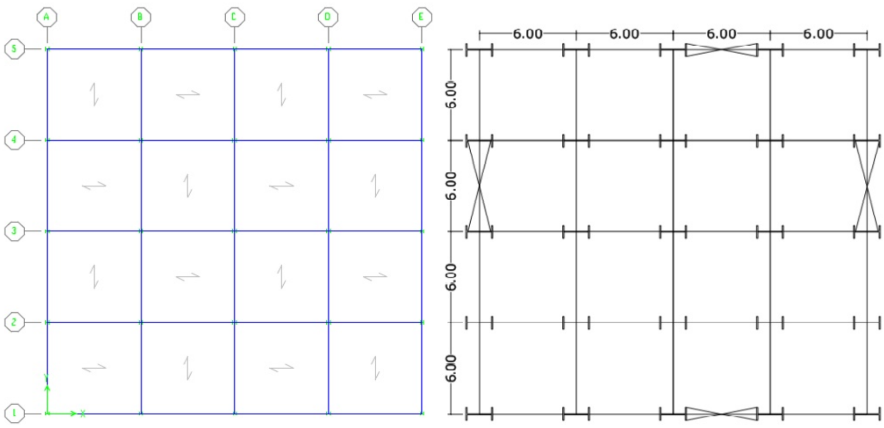

Figure 3. Plan of the sample building

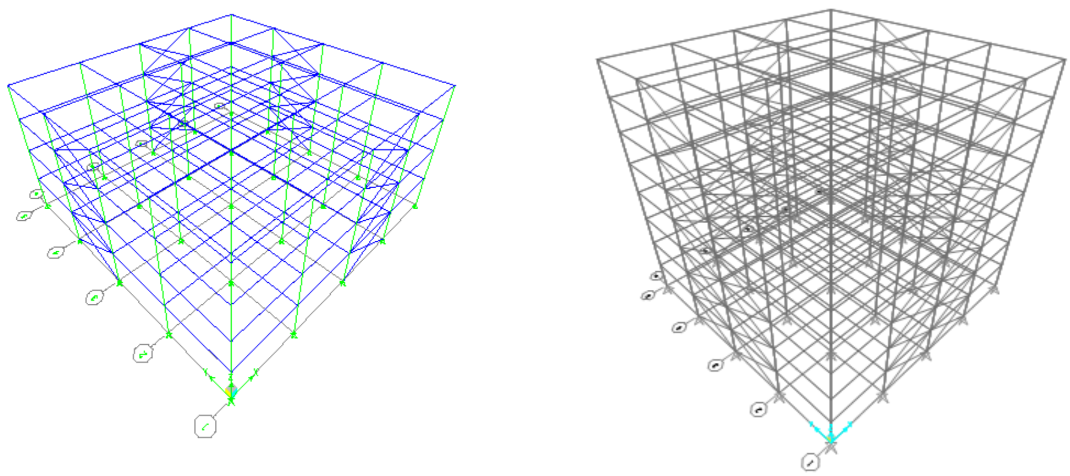

Figure 4. General view of the sample buildings

4.1 Material properties of structure

By Using St-37 steel, the yield strength of columns, beams and braces are assumed to be 240 MPa and the ultimate strength of columns, beams and braces are assumed to be 370 MPa. The details of material properties are shown in Table 2.

Table 2. Material properties data

F_y (MPa)	F_u (MPa)	E (GPa)
240	370	210

4.2 Computer model description

The building is analyzed and designed by means of commercial SAP 14.1 software and following procedures are used:

1. Floor diaphragms are assumed to be rigid.
2. Frame columns are modeled as pinned at their bases.
3. Beams to columns connections are pinned.
4. Brace connections to adjacent beams and columns are pinned.
5. HE-B (IPB) sections are used as columns.
6. IPE sections are used as beams.
7. Double angles (2L) sections are used as braces in sample buildings
8. Plate sections are used as core of braces in BRBFs system for retrofitted buildings.
9. Box sections are used as casing of braces in BRBFs system for retrofitted buildings.

4.3 Frame sections properties

Beams and columns of buildings with BRBFs are similar to the sample building with braces configuration, except those columns that adjacent to the braces that were strengthened with plates and are shown in tables. Plate section is used as brace's core of BRBF building, as shown in table 4. Table 3 and figure 5 show frame sections of sample building**.**

Table 3. Frame sections properties for four (left) and eight story (right) sample building

Column sections (mm)	**Beam sections (mm)**	**Brace sections (mm)**
IPB 160	IPE 360	2L120x120x12
IPB 180	IPE 330	2L150x150x12
IPB 200	IPE 220	
IPB 220	IPE 180	
IPB 240		
IPB 260		
IPB 280		
IPB 320		
IPB 340		
IPB 360		
IPB 400		

Table 4. Plate section for four (right) and eight story (left) building

	BRBF brace sections
Story	steel core section (mm)
1	Plate 110x40
2	Plate 110x40
3	Plate 100x40
4	Plate 100x40

	BRBF brace sections
Story	steel core section (mm)
1,2	Plate 160x40
3,4	Plate 160x40
5,6	Plate 140x40
7,8	Plate 140x40

Column-sections (mm)	Beam-sections (mm)	Brace-sections (mm)
IPB 160	IPE 360	2L120 x 120 x 12
IPB 180	IPE 330	2L100 x 100 x 10
IPB 200	IPE 220	-
IPB 220	IPE 180	-
IPB 240	-	-

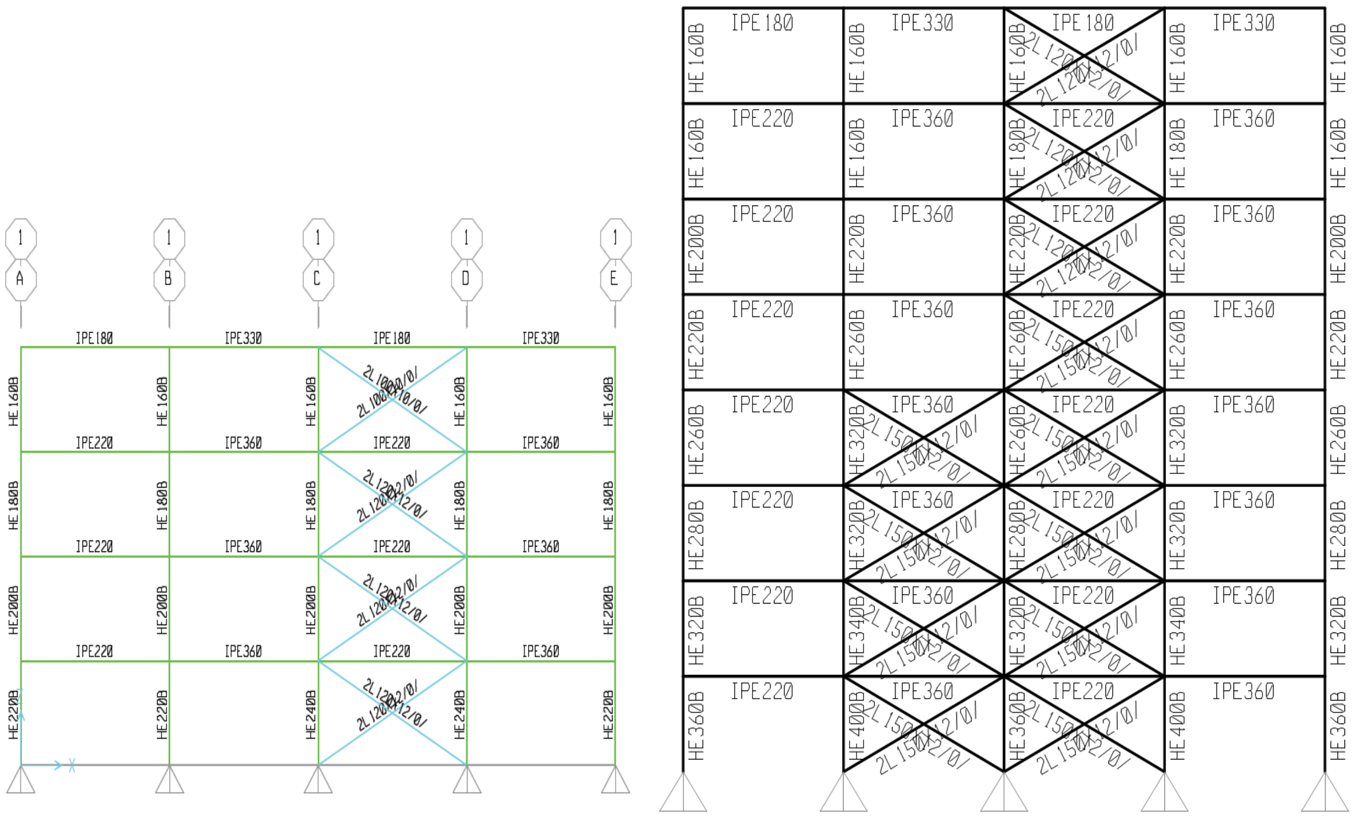

Figure 5. Illustration of frame sections at elevation

4.4 Calculating of equivalent static procedure for BRBF

Equivalent static procedure for four and eight story BRBFs are done according to STD NO.2800-3. Design base acceleration ratio, A, is equal to 0.35g. T_0 Coefficient is equal to 0.15. Fundamental period of vibration, T, is equal to 0.338 Second for four story and equals to 0.56 second for eight story structure. Building importance factor, I, is equal to 1.0. Building behavior factor, R, is equal to 8.0 for BRBF, according to ASCE/SEI7-10.

5. Design of four and eight story buildings with BRBFs

Design lateral loads that is applied to the structure by means of equivalent static procedure. BRBs Design control of steel core is according to AISC 2010, that shown in table 5 for four story and table 6 for eight story building as follow:

Table 5. Design control of steel core in four-story BRBFs building

Story	Steel core section, (mm)	A_{sc} (mm^2)	F_y (MPa)	ϕ	P_{ysc} (kN)	$P_{allowed} = \phi P_{ysc}$ (kN)	$P_{available}$ (kN)	$P_{allowed} \rangle P_{available}$
1	PL 110 x 40	4400	2400	0.9	1035.6	932.05	620.58	ok
2	PL 110 x 40	4400	2400	0.9	1035.6	932.05	570.31	ok
3	PL 100 x 40	4000	2400	0.9	941.47	847.32	450.27	ok
4	PL 100 x 40	4000	2400	0.9	941.47	847.32	244.50	ok

Table 6. Design control of steel core in eight-story BRBFs building

Story	Steel core section (mm)	A_{sc} (mm^2)	F_y (MPa)	ϕ	P_{ysc} (kN)	$P_{allowed} = \phi\ P_{ysc}$ (kN)	$P_{avalible}$ (kN)	$P_{allowed} \rangle P_{available}$
1	PLATE 160X40	6400	240	0.9	1506.56	1355.9	1193.04	ok
2	PLATE 160X40	6400	240	0.9	1506.56	1355.9	1168.76	ok
3	PLATE 160X40	6400	240	0.9	1506.56	1355.9	1120.98	ok
4	PLATE 160X40	6400	240	0.9	1506.56	1355.9	1056.95	ok
5	PLATE 140X40	5600	240	0.9	1318.24	1186.41	906.39	ok
6	PLATE 140X40	5600	240	0.9	1318.24	1186.41	764.84	ok
7	PLATE 140X40	5600	240	0.9	1318.24	1186.41	576.53	ok
8	PLATE 140X40	5600	240	0.9	1318.24	1186.41	309.53	ok

6. Calculation of story drifts for BRBF

According to ASCE/SEI 7-10, the design story drift (Δ) shall be computed as the difference of the deflections at the centers of mass at the top and bottom of the story under consideration. The deflection at level x (δ_x) (in. or mm) used to compute the design story drift, Δ, shall be determined in accordance with equation 5, as follow:

$$\delta_x = \frac{C_d \times \delta_{xe}}{I_e} \leq \delta_a \qquad \textbf{(Eq. 5)}$$

h_{sx}= story height, in mm.
C_d = the deflection amplification factor.
δ_{xe} = the deflection determined by an elastic analysis.
I_e = the importance factor.
δ_a= allowable story drift, according to table 12.12-1 from ASCE/SEI 7-10

As defined in earlier, BRBFs must satisfy check for designing story drifts as recommended in ASCE/SEI 7-10. Check for satisfying the allowable story drifts for four story BRBFs are shown in table 7 and for eight story in table 8, (the deflection amplification factor, C_d, for BRBs is equal to 5 according to ASCE/SEI 7-10.

Table 7. Control of BRBFs story drifts in four-story retrofitted building

Story	Story height (mm) h_s	Elastic story displacement $\delta_{xe}(mm)$	Story drift (mm) Δ_e	design Story Drift (mm) $\Delta = C_d \times \Delta_e$	Allowable Story Drift (mm)
4	3200	13.77	2.35	11.75	64
3	3200	11.42	3.52	17.6	64
2	3200	7.9	3.89	19.45	64
1	3200	4.01	4.01	20.05	64

Table 8. Control of story drifts in eight-story BRBFs building

Story	Story Height (mm) h_s	Elastic Story Displacement (mm) δ_{xe}	Story Drift (mm) Δ_e	Design Story Drift (mm) $\Delta = C_d \times \Delta_e$	Allowable Story Drift (mm)
8	3200	53.80	3.54	17.7	64
7	3200	50.26	5.08	25.4	64
6	3200	45.18	6.53	32.65	64
5	3200	38.65	7.44	37.2	64
4	3200	31.21	7.58	37.9	64
3	3200	23.63	7.81	39.05	64
2	3200	15.82	7.94	39.7	64
1	3200	7.88	7.88	39.4	64

7. Findings and results:

After calculation of story drifts for four and eight story BRB, it is observed that both systems have story drifts less than allowable story drift. Elastic story displacement in eight story BRB is larger than four story and obviously design story drift of eight story BRB is larger too. Comparison of design story drift in four and eight story BRBR buildings are shown in Figure 6 and it's observed that stiffness of four story BRB is more therefore, in low to relatively moderate earthquake four story BRB may undergo less damage specially in secondary (non-structural) elements, like in-fills or in general, elements like mechanical and electrical instruments which are sensitive to story drift.

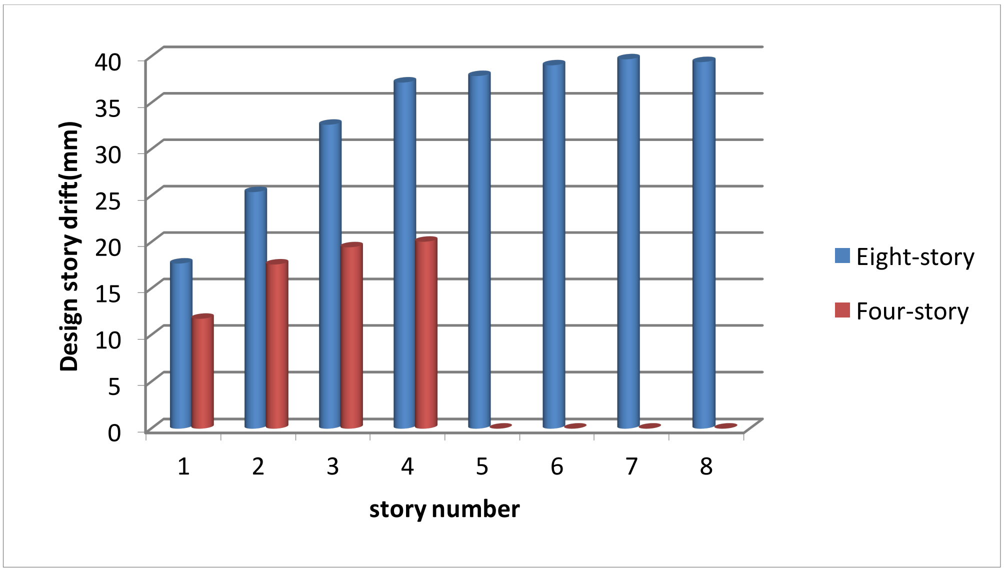

Figure 6. Comparison of four and eight story design story drift

8. Nonlinear static analysis of BRBF

8.1 Determination of target displacement of four and eight story BRBF

Target displacements are calculated manually and compare with results of software that extract from SAP2000; details of calculation are shown in tables 9 to 12.

Table 9. Target displacement parameters of four story BRBF in X-direction4

W (kN)	C_0	C_1	C_2	C_m	R
15548.3	1.25	1.0761	1.0107	0.9	2.8723
a	**V_y (kN)**	**S_a**	**T_e (Sec)**	$\delta_t\,(mm)$	$1.5\delta_t\,(mm)$
60	4694.28	0.9625	0.64	133.3308	199.9962

Table 10. Target displacement parameters of four story BRBF in Y-direction4

W (kN)	C_0	C_1	C_2	C_m	R
15548.3	1.25	1.0772	1.0109	0.9	2.8966
a	**V_y (kN)**	**S_a**	**T_e (Sec)**	$\delta_t\,(mm)$	$1.5\delta_t\,(mm)$
60	4656.14	0.9625	0.64	133.4898	200.2347

Table 11. Target displacement parameters of eight story BRBF in X-direction

W (kN)	C_0	C_1	C_2	C_m	R
31625.51	1.3	1.031438	1.0048	0.96	3.0480
a	**V_y (kN)**	**S_a**	**T_e (Sec)**	$\delta_t\,(mm)$	$1.5\delta_t\,(mm)$
60	6900.87	0.73828	1.042	268.6488	402.9731

Table 12. Target displacement parameters of eight story BRBF in Y-direction

W (kN)	C_0	C_1	C_2	C_m	R
31625.51	1.3	1.032549	1.005176	0.96	3.1190
a	**V_y (kN)**	**S_a**	**T_e (Sec)**	$\delta_t\,(mm)$	$1.5\delta_t\,(mm)$
60	6739.87	0.73828	1.042	269.0237	403.5356

8.2 Capacity curve of four and eight story BRBFs under pushover analysis

Capacity curves of the BRBF buildings under vertical distribution that are proportional to the building first mode of vibration are computed and they are shown in figures 7 and 8.

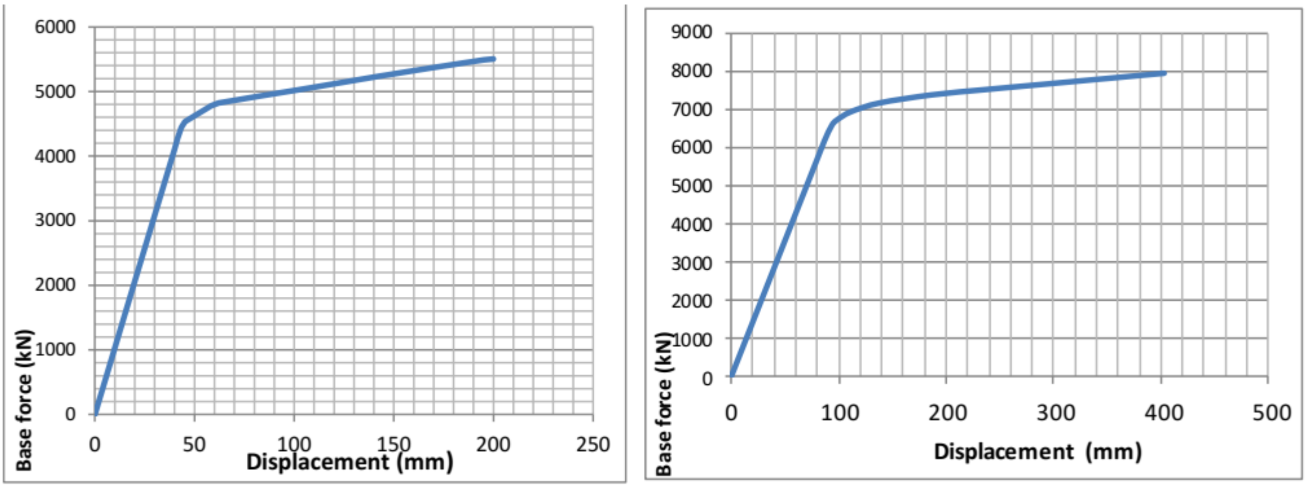

Figure 7. Capacity curve of four (left) and eight story (right) BRBFs building in X-direction (kN-mm)

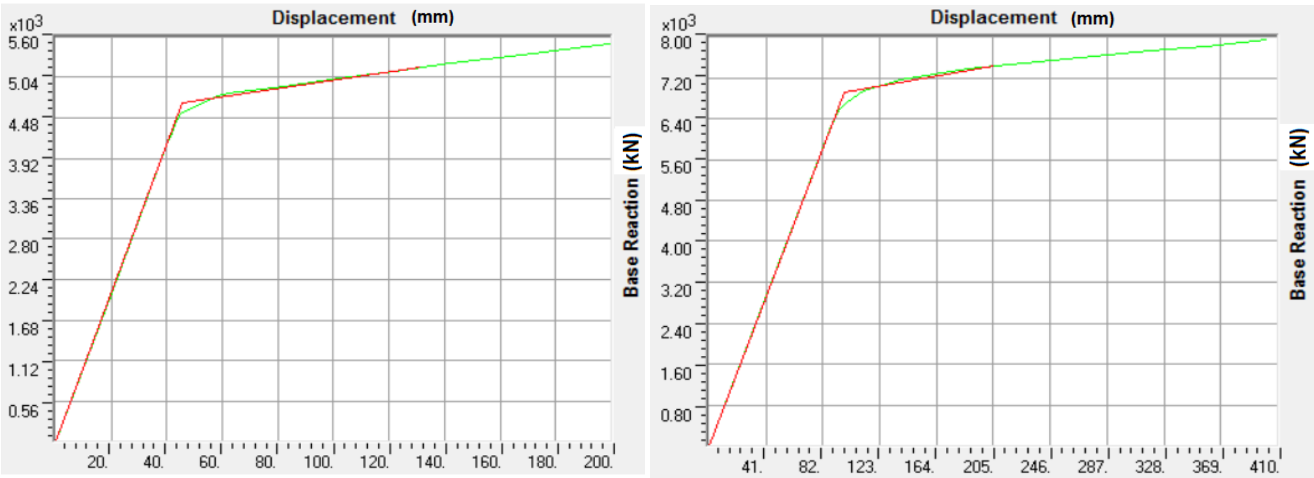

Figure 8. Adjustment of pushover with standard curve in X-direction for BRBFs four-story (left) and eight-story (right) buildings (kN-mm)

8.3 Hinge formation sequence in four story BRBFs

After performing nonlinear static analysis for four-story retrofitted building with BRBs, sequence of hinges that are formed in the structure can be observed in tables 13 and figure 9. As it is shown from following figures, plastic hinges are formed in braces only and there is not any hinges in columns of this system and it is because of very good manner of dissipation of seismic energy in BRBFs.

In X-direction (table 13), the first hinge is formed from (B to IO) limit at step 3 with displacement equals to 41.60 mm. At step 6, we have three hinges from (B to IO) limit and four hinges from (IO to LS) limit, increasing in number of hinges that are formed from (IO to LS) limit continue from step 6 with four hinges to step 12 with eight hinges. Around target displacement (133.33 mm) in step 11, eight hinges are formed from (IO to LS) limit and it shows that the performance of structure is acceptable according to our desire performance level (LRO) of ASCE 41-06. At 1.5 times of target displacement (199.99 mm) at step 12, we have two hinges from (B to IO) limit plus six hinges from (IO to LS) limit and it shows even in 1.5 times of target displacement performance of structure is good and dangerous plastic hinges are not formed yet.

Table 13. Hinges formation sequence of BRBFs retrofitted building in X-direction4

Step	Displacement	Base Force	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	mm	kN									
0	0	0	233	0	0	0	0	0	0	0	233
1	19.945087	2054.536	233	0	0	0	0	0	0	0	233
2	39.944087	4109.079	233	0	0	0	0	0	0	0	233
3	41.608413	4280.059	232	1	0	0	0	0	0	0	233
4	43.400615	4443.3	230	3	0	0	0	0	0	0	233
5	45.145349	4534.042	229	4	0	0	0	0	0	0	233
6	58.442411	4778.863	226	3	4	0	0	0	0	0	233
7	62.087373	4823.134	225	4	4	0	0	0	0	0	233
8	82.086373	4925.301	225	0	8	0	0	0	0	0	233
9	102.085373	5027.478	225	0	8	0	0	0	0	0	233
10	122.084373	5129.666	225	0	8	0	0	0	0	0	233
11	142.083373	5231.909	225	0	8	0	0	0	0	0	233
12	168.002752	5362.447	223	2	8	0	0	0	0	0	233
13	195.023777	5489.167	221	3	5	4	0	0	0	0	233
14	199.936087	5503.876	221	2	6	4	0	0	0	0	233

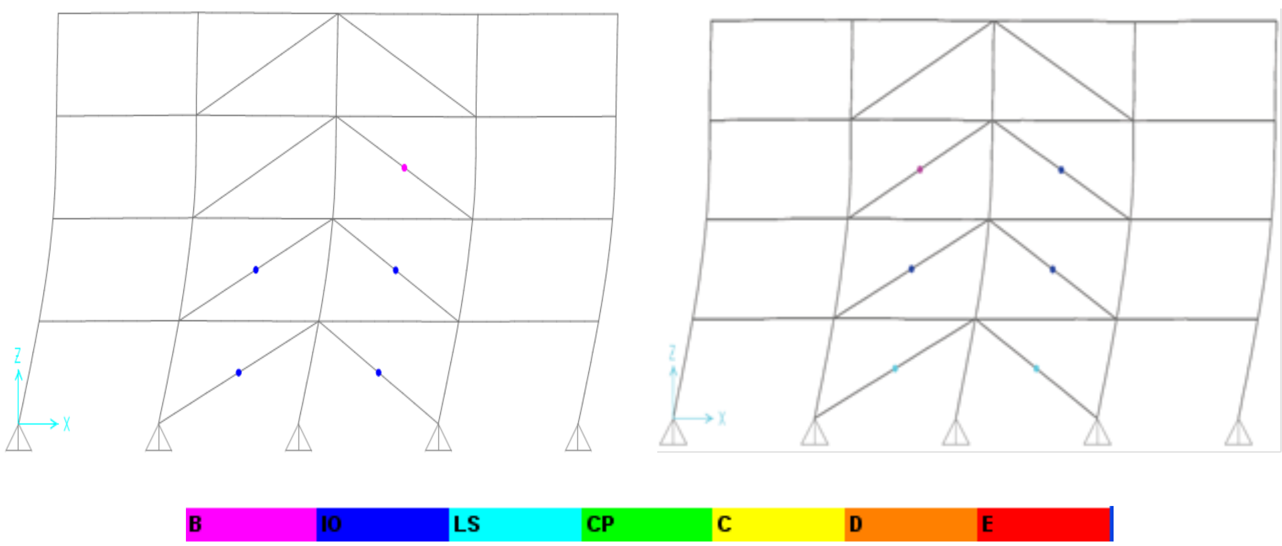

Figure 9. *Hinges formation of four story BRBFs building in X-direction at step 12 with displacement equals to 168 mm (left) and step 13 with displacement equals to 195.02 mm (right)*

8.4 Hinge formation sequence in eight story BRBFs

After performing nonlinear static analysis of eight-story retrofitted building with BRBFs, sequence of hinges that are formed in the structure can be observed in tables 14 plus figure 10.

In X-direction (see table 14), the first plastic hinge from (B to IO) limit is formed in step 3 at displacement equals to 86.88 mm. Around target displacement (268.64 mm) at step 10, we have twenty hinges from (IO to LS) limit and it shows that performance of retrofitted building is acceptable and in light to moderate earthquakes performance of structure is good according to ASCE 41-06. At 1.5 times of target displacement (402.94 mm) at step 13, We have one hinges from (B to IO) limit and eighteen hinges from (IO to LS) limit plus two hinges from (LS to CP) limit and it shows that performance of retrofitted building even in 1.5 times of target displacement is also acceptable and it is almost reliable even in sever earthquakes.

Table 14. Hinges formation sequence of eight story BRBFs retrofitted building in X-direction8story

Step	Displacement	Base Force	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	mm	kN									
0	0	0	464	0	0	0	0	0	0	0	464
1	40.248561	2864.759	464	0	0	0	0	0	0	0	464
2	80.545561	5729.516	464	0	0	0	0	0	0	0	464
3	86.886364	6180.291	463	1	0	0	0	0	0	0	464
4	93.513037	6569.012	458	6	0	0	0	0	0	0	464
5	98.274293	6711.741	455	7	2	0	0	0	0	0	464
6	111.459276	6929.503	451	5	8	0	0	0	0	0	464
7	137.898712	7158.634	448	3	13	0	0	0	0	0	464
8	186.314889	7370.419	445	3	16	0	0	0	0	0	464
9	237.212127	7511.729	444	0	20	0	0	0	0	0	464
10	277.509127	7615.391	444	0	20	0	0	0	0	0	464
11	317.806127	7719.064	444	0	20	0	0	0	0	0	464
12	358.103127	7822.782	444	0	20	0	0	0	0	0	464
13	402.921561	7937.283	443	1	18	2	0	0	0	0	464

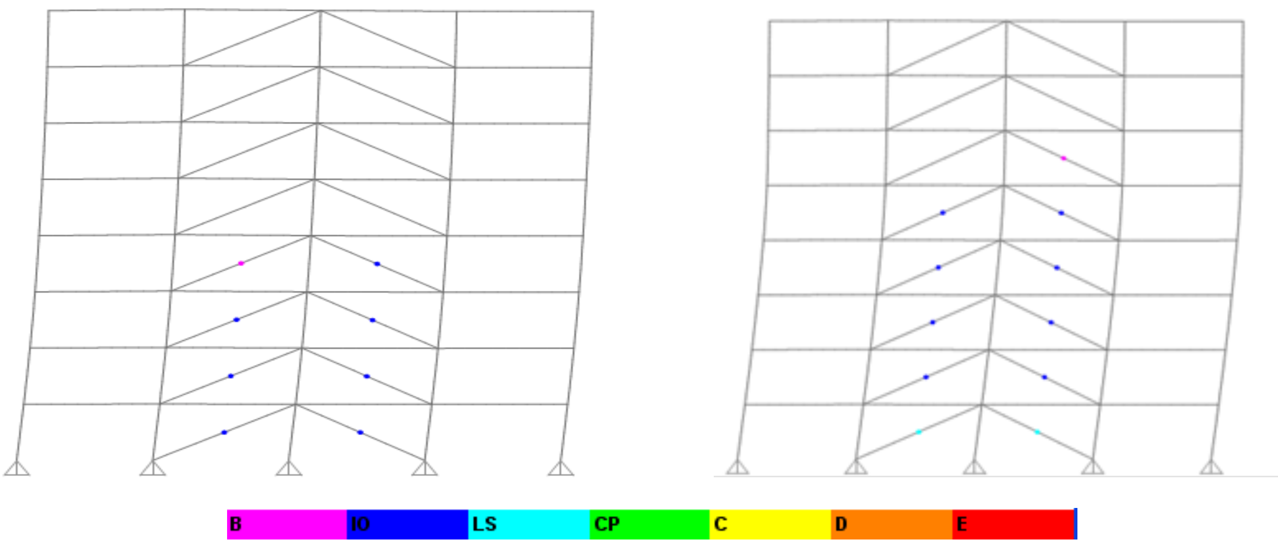

Figure 10. *Hinges formation of BRBFs building in X-direction at step 7 with displacement equals to 137.89 mm (left) and step 13 with displacement equals to 402.92 mm (right)*

9. Discussion

It is observed that in X-direction the first plastic hinge in eight-story BRBF system occurs at displacement equals to 86.60 mm where in four-story BRBF systems at almost same displacement we have eight hinges from (IO to LS) limit and the first plastic hinges occurs at displacement equals to 41.60 mm.

Target displacement in eight story building is equal to 268.64 mm with base force of 7615.40 (kN) that 20 hinges from (IO to LS) are formed but in four story structure with same displacement, it's passed the CP limit and structure is probable to collapse, target displacement in four story structure is equal to 133.33 mm with base force of 5150 (kN), so it shows that energy dissipation in eight story structure is more and displacement and base force that eight story structure is able to tolerate is almost 2 times of four story BRB. At 1.5 time of target displacement also response for eight story structure is better.

Figure 11 shows the stiffness of eight story BRBF is lesser than four-story. We can see this fact by compare the effective stiffness, K_e, of both systems in pushover curves too (figure 7 and 8). As it is seen the K_e for four-story BRBF systems is equals to 103 (kN/mm) and K_e for eight-story BRBF systems is equals to 71.21 (kN/mm) so for BRBF with increasing height, effective stiffness will decrease so we can see that ductility is increased too.

According to figure 11, the initial stiffness of four-story building retrofitted with BRBF is also more than eight-story and as the result four-story buildings may suffer less damage in low to moderate earthquakes if story drifts be a measurement of damage to the building.

So consequently structure's response for eight story structure is better than four story and it's recommended to use of BRBF for high raise building to get best of it. BRBF shows its best seismic performance in high raise building, (after target displacement) and confronting with moderate to severe earthquake (in moderate to high seismicity regions). It's mainly because of, with increasing height it gets more ductile and it will increase energy dissipation. Also from economical aspect, use of BRB is rather expensive in comparison with other braces so it's better to use of other braces system like SCBF for retrofitting of low raise building in low to moderate seismicity regions and use of BRB for retrofitting high raise building in high seismicity regions (Farizani et al., 2015).

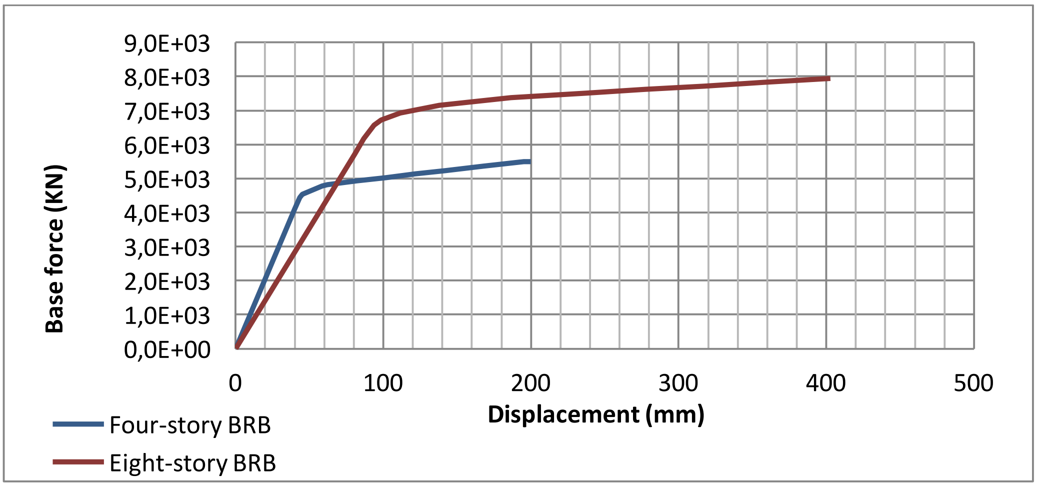

Figure 11. Compare of capacity curves for eight-story and four-story BRBF

10. Conclusions

I. Initial stiffness of four story building is more than eight story and as the result buildings with four story BRB may suffer less damage in low to moderate earthquakes if story drifts be a measurement of damage to the building.
II. Effective stiffness in four story building is larger than eight story and four story BRB may undergo less lateral displacements than eight story BRBF so Damage to nonstructural element is eight story is more.
III. Drift in eight story retrofitted building is more that four-story retrofitted buildings.
IV. The ductility and energy dissipation of eight story BRB is more than four story buildings.
V. With increasing height, effective stiffness will decrease so we can see that ductility is increased too.
VI. It's recommended to use of BRB for retrofitting of high raise buildings in relatively moderate to high seismicity regions.

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