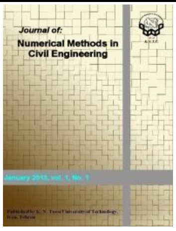

Numerical Methods in Civil Engineering


# The Effects of Using Easy-Going Steel Knee Element on Seismic Behavior of CKBF


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***Abstract:***

*A proper composition of stiffness and ductility parameters is required to obtain a resistant and economic structure. Accordingly, Chevron Knee Braced Frame (CKBF) seems appropriate due to its proper seismic performance. The advantage of this system comes to having the capability of rapid and cheap replacement of chevron knee elements after an earthquake occurs. In this research, response modification factor and over-strength factor are determined for CKBF in two statuses of chevron V bracing and chevron inverted V bracing. The effect of using Easy-Going Steel (EGS) on the knee is also studied in the 3,6,9 and 12 story frames. In addition, the methods used for this purpose are nonlinear static analysis, linear dynamic analysis and Incremental Dynamic Analysis (IDA). IDAs have been conducted on 17 records of important universal earthquakes using Opensees software.*


## 1. Introduction

Stiffness and ductility are the essential specifications of structural systems that play effective roles in the structural behaviour during earthquakes. While its stiffness is low, moment resistant frame shows good ductility by yielding beam flexural element. The concentrically Braced Frame (CBF) has higher stiffness; however, its ductility is reduced due to the diagonal brace buckling. In order to overcome this problem, Roder and Popov (1987) proposed Eccentrically Brace Frame system (EBF) [1]. At the same time, with proper performance, this system presents no disadvantages either. Therefore, Aristizabal-Ochoa (1986) suggested Knee Brace Frame (KBF) as the solution to improve braced system [2]. This system can provide ductility and energy dissipation through yielding knee element and likewise replace knee element rapidly with more convenience.

The frame with knee brace is seismic resistant steel systems with proper stiffness and ductility. Thus the CKBF is formed of two main knee and diagonal parts (Fig. 1).


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Basically, the former is connected to the middle of the beam from a different side. In such braces, the diagonal element provides stiffness of system, while the knee element provides the required ductility by yielding throughout strong earthquakes and preventing the diagonal element from buckling as well. Therefore, stiffness and ductility are provided for the structure simultaneously [3, 4]. In this system, the knee element is utilized as a ductile fuse to prevent the structure from collapsing by absorbing energy during shear or flexural yielding. Furthermore, it can be deduced that shear yielding mode has a more reasonable performance in comparison with flexural yielding mode [5, 6]. Recently Hsu et al. have introduced an alternative KBF system, in which the knee member remains elastic while energy dissipation occurs in the story beam [7]. Hence, the concept of this KBF system is different from what is shaped by Balendra et al. (1994) [4]. Knee braces can be utilized in retrofitting the structures where it improves the seismic performance of buildings [8]. Knee braced frames are recently being studied with regard to their nonlinear behaviour and dissipated energy based on cyclic analysis where the following results are presented [9].

Response modification factor is an essential parameter in seismic design which exhibits the capability of the system in absorbing and dissipating seismic energy. Nonlinear behaviour of a structure can be applied in a linear design using response modification factor. Response modification factor R=7 and over-strength factor $\Omega_0$ = 3.0 have been recently proposed by some researchers for CKB systems [10].

In the present paper, over-strength factor, ductility factor and seismic response modification factor are calculated and presented for chevron knee braced frames. For this purpose, IDA analyses using 17 earthquake records are conducted. The main objectives of this investigation are as follows:

- Study the effect of using Easy-Going Steel (EGS) on over-strength factor, ductility factor and seismic response modification factor
- Determination of response modification factor and over-strength factor CKBF
- Study CKBF in two statuses of chevron V bracing and chevron inverted V bracing
- Study IDA approach

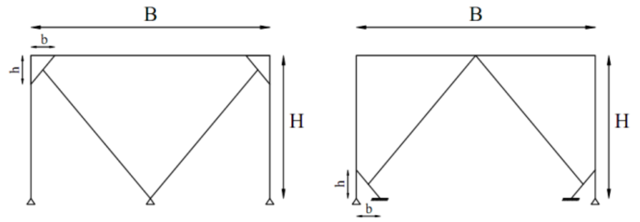


**Fig. 1:** Chevron knee braced frame (CKBF)

## 2. Using Easy Going Steel (EGS) in structures

Mankind has long endeavoured to enhance steel strength, as well as reduce the size of structural members, in order to reduce the total weight of structures and at the same time, making it economical. Nonetheless, it should be taken into account that increasing the steel strength and decreasing the cross section of structural members is not always efficient. In some cases, it is even required to reduce the steel strength as much as possible in order to improve the structural behavior [11, 12]. Examples for such situations are steel structures exposed to an earthquake or severe windy conditions. In order to augment the energy absorption of frames, low strength steel is utilized. Generally, this low strength steel is called Easy-Going Steel (EGS) where the best EGS is the pure iron with yield stress between 90 N/mm$^2$ and 120 N/mm$^2$.

Given that the ultimate load carrying capacity should not alter by applying EGS in a structure, the thickness of these members should be enhanced due to lower yield stress of the EGS. In other words, the thickness of structural members made of EGS should be greater than the ones made of common constructional steel [12]. Comparatively, since the thickness of the EGS knee element is increased, local buckling in the flange and webs do not occur and the hysteresis loops are more stable.

## 3. Geometric specifications of knee braces

The first step in designing knees is to determine their geometries and placements. Thus, the optimum placement of knee element in the CKBF is the condition in which the extension of brace passes the beam and column crossing. Moreover, the knee element on each side of the frame is parallel to the brace on the opposite side. In this state, the bending moment values are equal in the middle and end of knee and the shear values are the same on both sides of the brace, making the axial force reliable [13]. Based on the facts mentioned above, the h and b distances are assumed in the studied frames as follows:

$$H = 3m \, , B = 5m \, , \frac{h}{H} = \frac{b}{B/2} = 0.2 \rightarrow h = 0.6m \, ,$$

$$b = 0.5m$$

The assumed connection types of the members are given in Table 1.

**Table 1.** Members' connections

| Knee to beam and column | Diagonal to knee and beam | Beam to column | Column to ground |
|---|---|---|---|
| Fixed | Hinged | Hinged | Hinged |

## 4. The studied models

Response modification factor has been studied in regards to steel frames with chevron knee braces (V and inverted V chevron knee bracing) in order to evaluate their nonlinear behaviors. Two-dimensional frames with 3 spans of equal dimensions (5m) have been braced in their corner spans. Regarding the height, the selected frames consist of four types: 3, 6, 9 and 12 story, with 3m height. The studied 6 story sample has been shown in (Fig. 2). All structures have been designed based on ASCE 2010 and AISC 360, 2010 [14, 15]. In this research, 16 frames have been completely investigated: 1st group) the frames with chevron knee braces (V and inverted V), in the knee of which the building steel has been applied; 2nd group) the frames with chevron knee braces (V and inverted V), in the knee of which EGS has been applied.

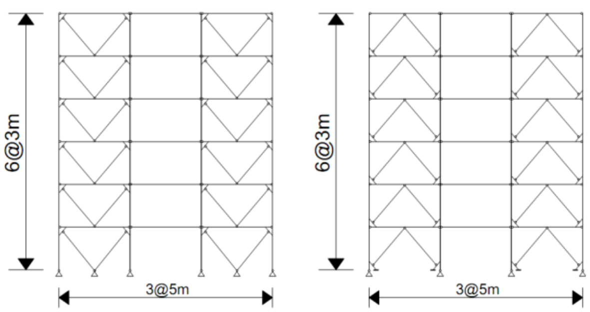


**Fig. 2:** Brace configuration of the studied frames

To predict linear buckling in an initial mid span, imperfection of 0.001L for all braces was assumed. In order to account for geometric nonlinearities, the simplified P-Δ stiffness matrix was applied [16].

Here, the frame located in D axis of the plan (Fig. 3), basically has been derived for each 3 story building and modelled in OpenSees software edition 2.2.2. Several assumptions have been considered for the sections and materials in order to adjust the behavior of two-dimensional frame with tri- dimensional model designed in OpenSees (Mazzoni, S 2007) software. These assumptions are indicated as follows:

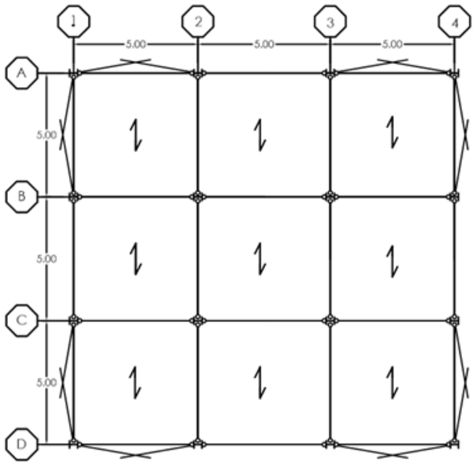


**Fig.3:** Plan of the structure

In this research, the Steel02 material model has been applied for modeling the frame's members (beams, columns and diagonals) and illustrated in (Fig. 4). Moreover, as the steel material used in this study is ST37 building steel, the yielding stress, ultimate stress and modulus of elasticity are $F_y$=2400 kg/cm$^2$, $F_u$=3700kg/cm$^2$ and E=2.1×10$^6$ kg/cm$^2$, respectively. The slope of strain hardening zone is considered as 2% of that of elastic one.

It should be noted that EGS with yielding stress of 1000 kg/cm$^2$ has been used for the knees in the second section of this research. The slope of strain hardening zone is considered as 2% of that of elastic one. Easy-Going Steel or EGS as briefly mentioned, is defined as a type of steel with very low percentage of carbon along with other types of alloys with very high ductility, having a nominal yield stress between 900 kg/cm$^2$ to 1200 kg/cm$^2$ [11].

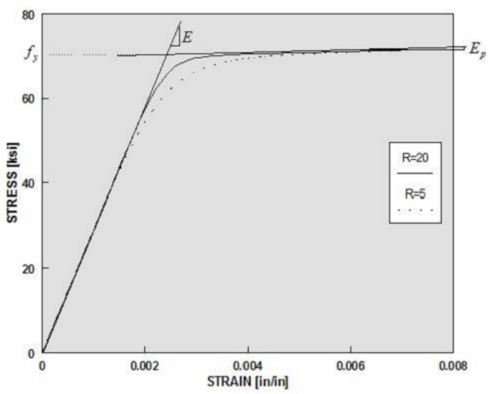


**Fig. 4:** The behavior of Steel02 material [17]

Fiber sections have been used for all structural members. In addition, the nonlinear beam-column element with Section Aggregator, as shown in (Fig. 5), has been utilized for knee member in order to provide the possibility of applying shear yielding properties. These fibers provide the capability of wide plasticity formation in the element with respect to the behavior of defined material (elastoplastic with considered strain hardening). The Zero-Length elements have been applied in modeling the hinge joints of beams along with braces, at the same time, only the transitional degrees of freedom have been rectified.

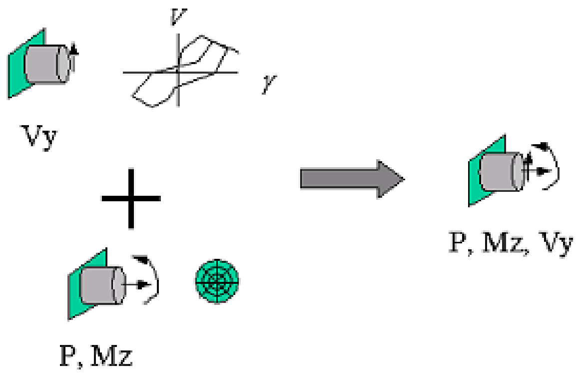


**Fig. 5:** The section used for modeling knee elements [17]

The eccentricity value of 0.001L has been considered for the element's length in the middle of each of the columns and braces in order to provide nonlinear geometric behavior in these members. Additionally, considering the construction error, this function can provide the buckling possibility in columns and braces due to the axial loads [16].

## 5. Response modification factor

Response modification factor is considered in almost all universal codes for reducing the calculated earthquake loads in order to consider inelastic behavior. This allows the designers to conduct elastic analysis under reduced loads, as well as design–structures based on the results obtained. The mentioned factor depends on different aspects, the most important of which are: ductility of structure, material properties, damping characteristics, cooperation of non- structural members, over-strength, etc.

In this study, response modification factor is calculated using Uang's ductility factor method in which real nonlinear behavior is usually idealized by a bilinear elastic perfectly plastic relation, (Fig. 6) [18]. In order to calculate response modification factor, some parameters are defined using the base shears shown in (Fig. 6). The first type is over-strength factor which is expressed in the following Eq. (2).

$$R_s = \frac{V_y}{V_s} \tag{2}$$

where, $R_s$ is overstrength factor; $V_s$ is base shear corresponding to the first yield in the structure; $V_y$ is base shear corresponding to the mechanism formation and collapse of structure. Over-strength factor considers the actual lateral strength of structure against its design lateral strength.

The reduction factor of force because of ductility ($R_\mu$): the linear shear force ($V_e$) can be reduced to yield shear force ($V_y$) due to the ductility and nonlinear behavior of structure. This factor depends on several aspects including the type of structural system, the quality of connections, number of stories, etc.

$$R_\mu = \frac{V_e}{V_y} \tag{3}$$

Allowable stress factor (Y): in the designing codes, $V_s$ is reduced to $V_w$ through a factor called allowable stress factor, the amount of which is considered as 1.44 in this research [18].

$$Y = \frac{V_s}{V_w} \tag{4}$$

In fact the origin of response modification factor is strength reduction factor due to ductility ($R_\mu$) and over-strength factor ($R_s$), which have both been defined in the previous sections. Response modification factor with ultimate strength method is defined as follows:

$$R_u = \frac{V_e}{V_y} \times \frac{V_y}{V_s} = R_\mu \times R_s \tag{5}$$

Response modification factor with allowable stress method is expressed as follows:

$$R_w = \frac{V_e}{V_y} \times \frac{V_y}{V_s} \times \frac{V_s}{V_w} = R_\mu \times R_s \times Y \tag{6}$$

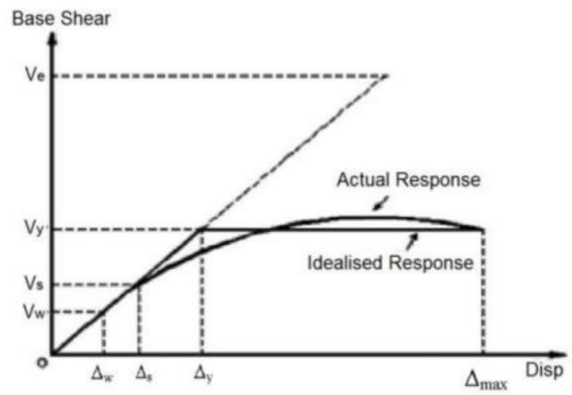


**Fig. 6:** Elastic and inelastic responses of structure [18]

The steps of calculating the response modification factor in this research is depicted in flowchart form in Fig. 7.

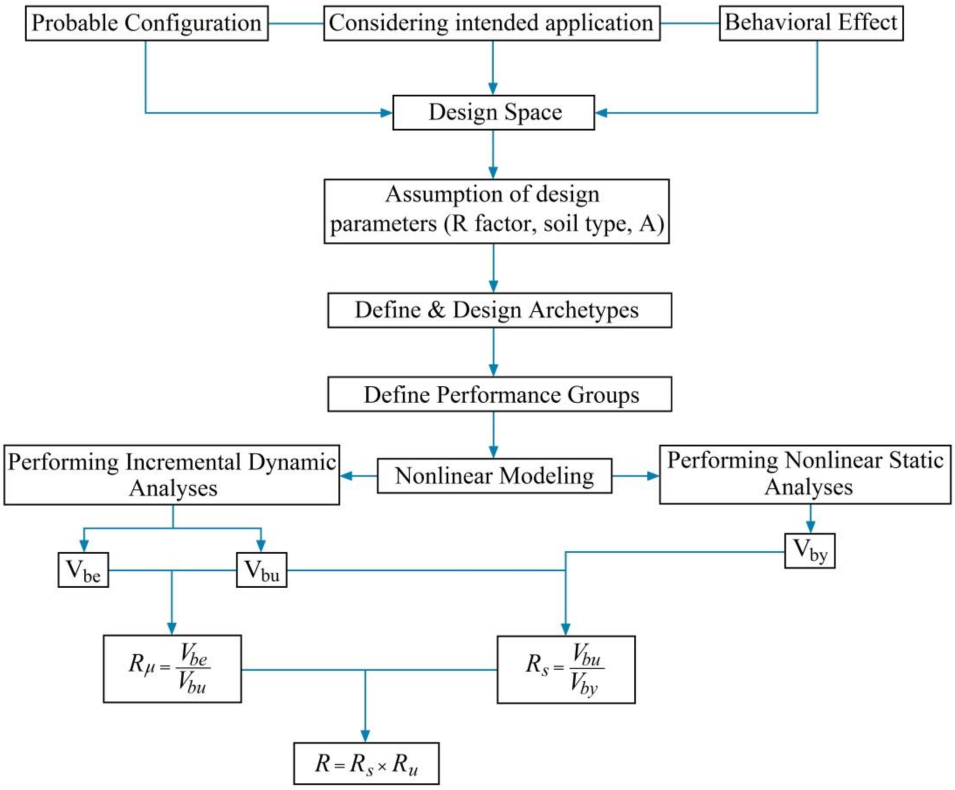


**Fig. 7:** Flowchart of calculating response modification factor

## 6. Nonlinear static analysis of studied models

This section presents the pushover curves and the results of nonlinear static analysis of the studied models. Finally, the base shear value corresponding to the first yielding occurrence $V_b$ (st,y) is derived from the pushover curves of the models (Figs. 8-11), and presented in Table 2.

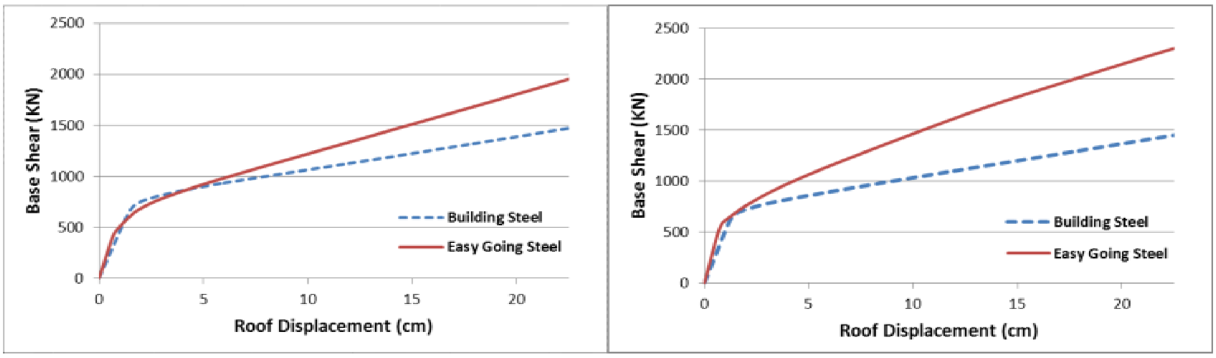


**Fig. 8:** Pushover curves of 3story structures: left) chevron V bracing; right) chevron inverted V bracing

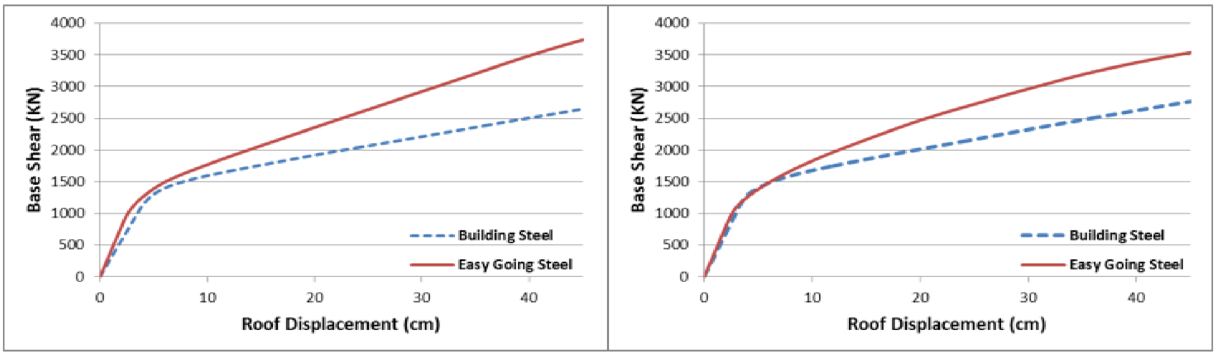


**Fig. 9:** Pushover curves of 6 story structures: left) chevron V bracing; right) chevron inverted V bracing

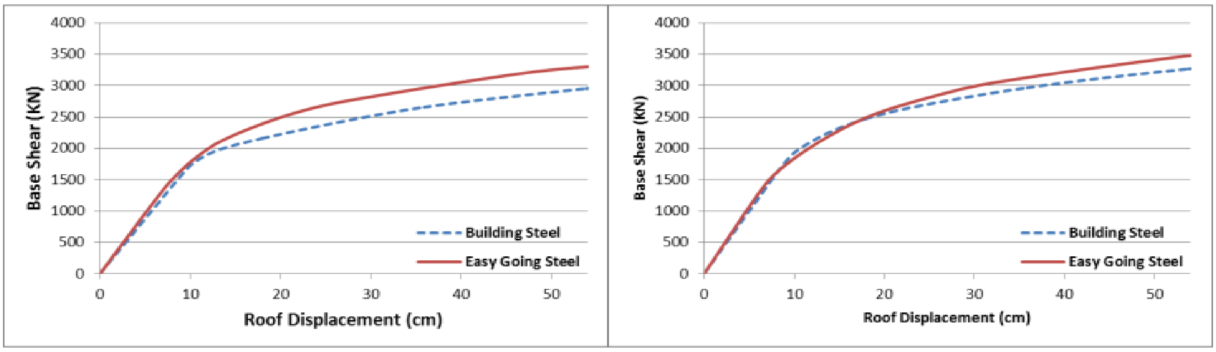


**Fig. 10:** Pushover curves of 9 story structures: left) chevron V bracing; right) chevron inverted V bracing

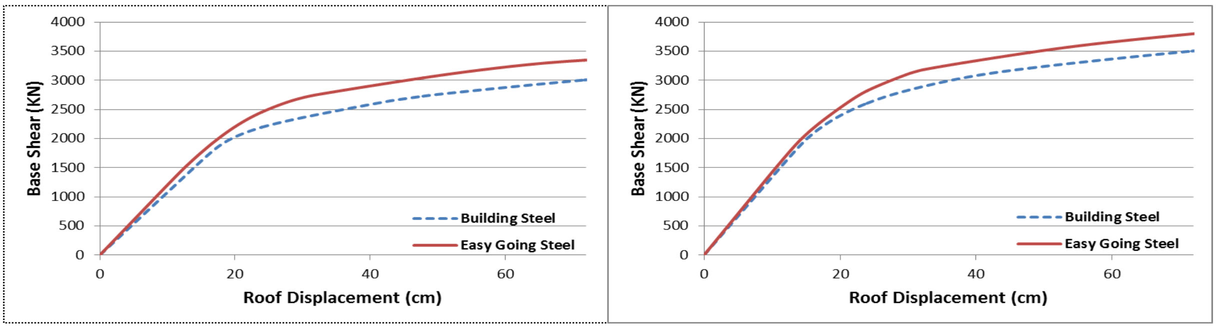


**Fig. 11:** Pushover curves of 12 story structures: left) chevron V bracing; right) chevron inverted V bracing

The area of the applied section increases, when yield stress decreases in EGS. At the same time, the stiffness of stories will increase and their drifts will decrease by increasing the thickness with respect to the constant modulus of steel elasticity.

**Table 2.** Base shear corresponded to the first yielding occurred in the structure

| Structure Type | No. of Stories | $V_{b(st,y)}$ (kN) | Structure Type | No. of Stories | $V_{b\,(st,y)}$ (kN) |
|---|---|---|---|---|---|
| Chevron V Bracing | 3 | 454.59 | Chevron Invented V Bracing | 3 | 422.43 |
| | 6 | 948.29 | | 6 | 987.49 |
| | 9 | 1449.13 | | 9 | 1412.41 |
| | 12 | 1492.26 | | 12 | 1167.24 |
| Chevron V Bracing (EGS) | 3 | 422.05 | Chevron Invented V Bracing (EGS) | 3 | 398.22 |
| | 6 | 990.18 | | 6 | 769.82 |
| | 9 | 1058.35 | | 9 | 933.32 |
| | 12 | 982.92 | | 12 | 824.4 |

## 7. Calculating response modification factor

There are different methods for calculating response modification factor; in this research the Uang method [18] has been used. In this method, nonlinear dynamic analysis and IDA have been applied for calculating over-strength factor. Moreover, IDA and linear dynamic analysis have been conducted on the two-dimensional frames for calculating ductility factor. Subsequently, the obtained results have been applied to calculate the final response modification factor of CKBF. Nonlinear static analysis, nonlinear dynamic analysis and linear dynamic analysis have been applied to calculate ductility and over-strength factors. Essentially, a total of 17 recorded world-renowned earthquakes have been selected to conduct IDA on the 3, 6, 9 and 12 story frames. The specifications of the applied records have been summarized in Table 3.

**Table 3.** The specifications of the records used for incremental dynamic analysis

| Earthquake | Station | Data | PGA(g) |
|---|---|---|---|
| Cape Mendocino | Rio Dell Overpass FF | 4/25/1992 | 0.549 |
| Chi-Chi, Taiwan | CHY080 | 9/20/1999 | 0.968 |
| Coyote Lake | Gilroy Array3 | 8/06/1979 | 0.434 |
| Kobe | KJMA | 1/16/1995 | 0.821 |
| Kocaeli, Turkey | Sakarya | 8/17/1999 | 0.376 |
| Landers | Coolwater | 6/28/1992 | 0.417 |
| Loma Prieta | Corralitos | 10/18/1989 | 0.644 |
| Morgan Hill | Anderson Dam | 4/24/1984 | 0.423 |
| N. Palm Springs | N. Palm Springs | 7/08/1986 | 0.694 |
| Northridge | Santa Monica | 1/17/1994 | 0.883 |
| Parkfield | Temblor Pre-1969 | 6/28/1966 | 0.357 |
| San Fernando | Lake Hughes #12 | 2/09/1971 | 0.366 |
| Superstition Hills | Usgs Station 5051 | 11/24/1987 | 0.455 |
| Victoria, Mexico | Unam/Ucsd Station 6604 | 6/09/1980 | 0.621 |
| Whittier Narrows | Obregon Park | 10/01/1987 | 0.45 |
| Tabas | Tabas, LN | 9/16/1978 | 0.836 |
| Bam | Bam | 26/12/2003 | 0.799 |

### 7.1 Calculating the over strength factor

There are limitations in calculating the over-strength factor through nonlinear static method, one of which is lateral loading pattern. Additionally, the over-strength phenomenon is important in earthquake occurrence and each frame presents different over-strength factors under different earthquakes. Over-strength factor is calculated through IDA in this research. Here, the method that is presented by Mwafy & Elnashai [19], is used for computing maximum base shear through IDA. Thus, it involves a structural model subjected to one (or more) ground motion record(s), each of which is scaled to multiple intensity levels [20]. The ratio of ultimate base shear to the base shear of the first yielding is presented as over-strength factor.

$$R_s = \frac{V_{b(Dyn,u)}}{V_{b(st,y)}} \tag{7}$$

It means that over-strength is the ratio of dynamic base shear obtained through mechanism formation in the structure to the static base shear corresponding to the first plastic hinge formation.

### 7.2 Calculating the ductility factor

In the method presented by Mwafy & Elnashai, the ductility factor is obtained directly, as well as by using the results of IDA and linear dynamic analysis as follows [19]:

$$R_\mu = \frac{V_{b(Dyn,el)}}{V_{b(Dyn,u)}} \tag{8}$$

In order to obtain $V_{b(Dyn,u)}$, the spectral acceleration of earthquake record (the intensity measure applied in this study) is increased to form mechanism in the structure or meet the considered damage. Basically, such spectral acceleration, which leads to the above mentioned mechanism or damage, is accepted as ultimate limit where the corresponding base shear is then obtained. Additionally, the maximum linear base shear of the structure is also calculated through dynamic analysis assuming elastic behavior of structure under the same spectral acceleration. The base shear corresponding to the first plastic hinge obtained through nonlinear static analysis, is used for calculating the over-strength factor. It means that the end of the linear zone, corresponded to the first plastic hinge, can be considered the same in both the static and dynamic analyses [19].

IDA curves have been plotted for 9 story frames in terms of maximum inter story drift-spectral acceleration corresponding to first mode shape and illustrated in Figs. 12-13, as examples. To consider a damage criterion in this research, after dynamic analysis, the deformations have been controlled according to standard No. 2800. For buildings with the main period less than 0.7 sec, maximum inter story drift is limited to 0.025h and for the buildings with the main period more than or equal to 0.7 sec, maximum inter story drift is limited to 0.02h where h is the height of each story.

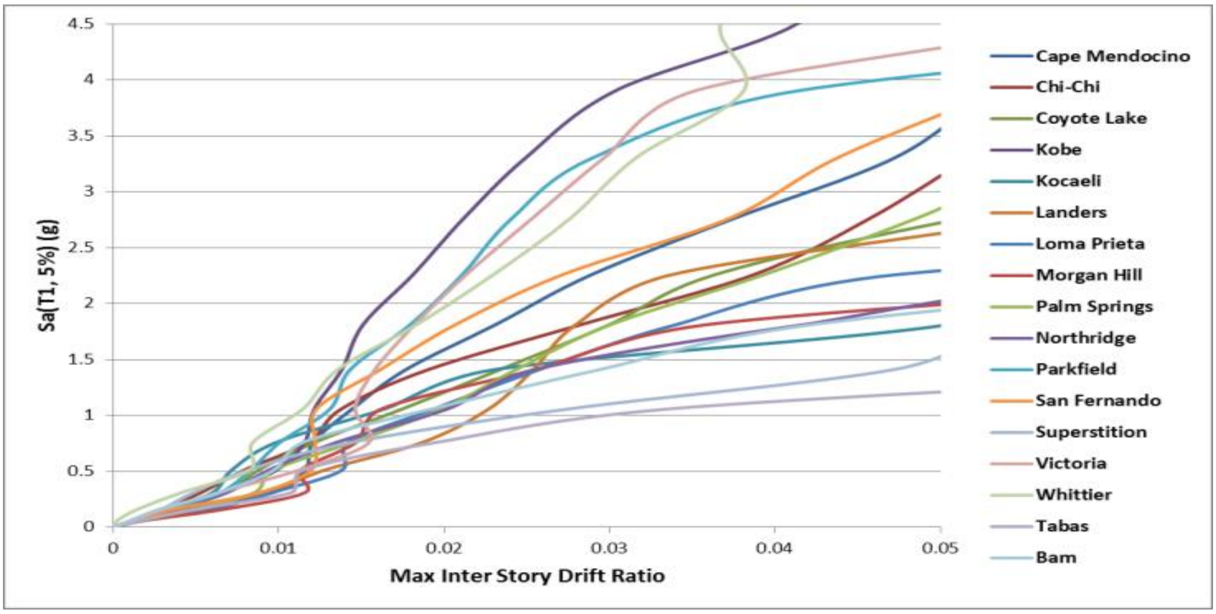


**Fig. 12:** IDA curves for 9 story frames, the first group, chevron V bracing

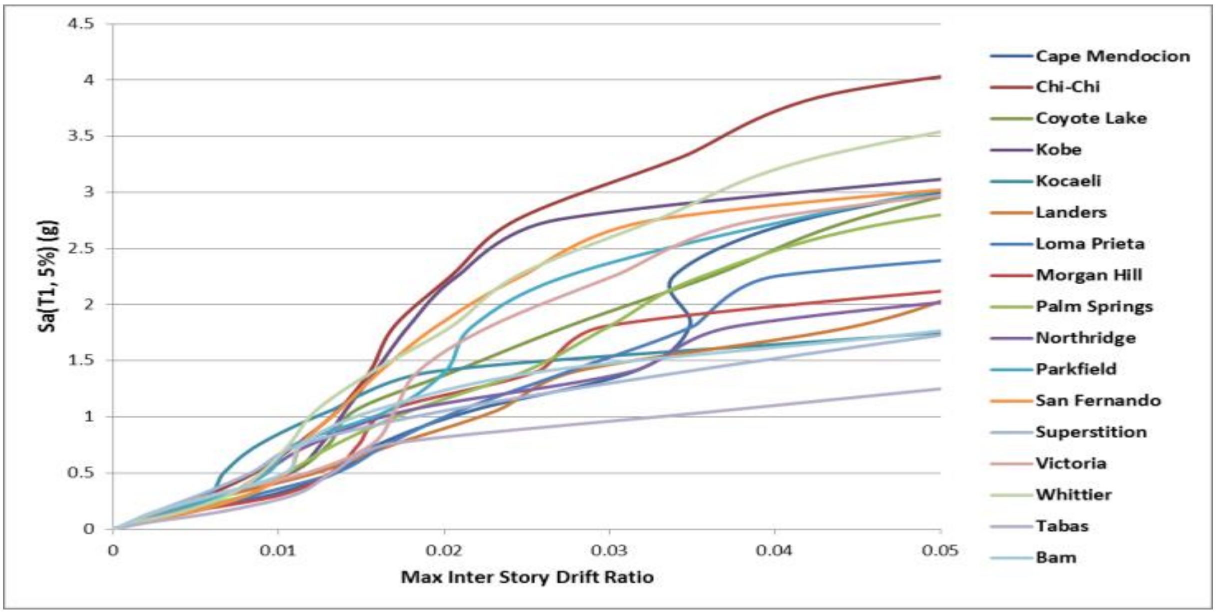


**Fig. 13:** IDA curves for 9 story frames, the second group, chevron V bracing

## 8. The results of incremental dynamic analysis (IDA)

Response modification factor has been calculated for all records pertaining to all 16 studied models. The final response modification factors have been obtained by averaging the results and presented as an example in Tables 4-5.

In this table, DM is Damage Measure; IM is Intensity Measure (here, spectral acceleration corresponded to the first mode of structure); σ is standard deviation and $\bar{x}$ is average; C.V. is Coefficient of Variation. If C.V. value is low in the statistical analysis, the results will be of higher accuracies.

$$\sigma = \sqrt{\frac{\sum (x - \bar{x})^2}{N}} \tag{9}$$

$$C.V. = \frac{\sigma}{\bar{x}} \tag{10}$$

**Table 4.** The values of over-strength and ductility factors for 9 story frame, the first group, chevron V bracing

| Records | DM Max Drift | IM Sa (T1, 5%) | $V_{b\,(Dyn,u)}$ (kN) | $V_{b\,(st,y)}$ (kN) | $V_{b\,(Dyn,e)}$ (kN) | $R_s$ | $R_\mu$ | $R_{ASD}$ | $R_{LRFD}$ |
|---|---|---|---|---|---|---|---|---|---|
| Cape Mendocino | 0.02 | 0.39 | 4092.15 | 1449.13 | 16179.32 | 2.82 | 3.95 | 16.08 | 11.17 |
| Chi-Chi, Taiwan | 0.02 | 1.93 | 3451.28 | | 11284.70 | 2.38 | 3.27 | 11.21 | 7.79 |
| Coyote Lake | 0.02 | 0.57 | 3006.84 | | 8949.23 | 2.08 | 2.98 | 8.89 | 6.18 |
| Kobe | 0.02 | 1.50 | 4011.29 | | 13991.49 | 2.77 | 3.49 | 13.90 | 9.66 |
| Kocaeli, Turkey | 0.02 | 0.38 | 3648.08 | | 8973.55 | 2.52 | 2.46 | 8.92 | 6.19 |
| Landers | 0.02 | 0.37 | 3136.46 | | 6016.09 | 2.16 | 1.92 | 5.98 | 4.15 |
| Loma Prieta | 0.02 | 0.4 | 3823.58 | | 13579.35 | 2.64 | 3.55 | 13.49 | 9.37 |
| Morgan Hill | 0.02 | 0.14 | 4592.47 | | 14874.02 | 3.17 | 3.24 | 14.78 | 10.26 |
| N. Palm Springs | 0.02 | 0.27 | 3471.69 | | 14496.45 | 2.4 | 4.18 | 14.41 | 10.00 |
| Northridge | 0.02 | 0.33 | 3270.31 | | 9387.01 | 2.26 | 2.87 | 9.33 | 6.48 |
| Parkfield | 0.02 | 0.21 | 4076.96 | | 19844.37 | 2.81 | 4.87 | 19.72 | 13.69 |
| San Fernando | 0.02 | 0.15 | 4579.99 | | 15710.86 | 3.16 | 3.43 | 15.61 | 10.84 |
| Superstition Hills | 0.02 | 0.97 | 3621.11 | | 6304.00 | 2.5 | 1.74 | 6.26 | 4.35 |
| Victoria, Mexico | 0.02 | 0.59 | 3155.06 | | 12861.07 | 2.18 | 4.08 | 12.78 | 8.88 |
| Whittier Narrows | 0.02 | 0.23 | 3823.17 | | 13278.50 | 2.64 | 3.47 | 13.20 | 9.16 |
| Tabas | 0.02 | 0.49 | 3512.73 | | 8329.78 | 2.42 | 2.37 | 8.28 | 5.75 |
| Bam | 0.02 | 1.06 | 3490.25 | | 9518.85 | 2.41 | 2.73 | 9.46 | 6.57 |
| Average | | | | | | 2.55 | 3.21 | 11.90 | 8.26 |
| $\sigma$ | | | | | | 0.31 | 0.80 | 3.65 | 2.54 |
| C.V. | | | | | | 0.12 | 0.25 | 0.31 | 0.31 |

**Table 5.** The values of over-strength and ductility factors for 9 story frame, the second group, chevron V bracing

| Records | DM Max Drift | IM Sa (T1, 5%) | $V_{b\,(Dyn,u)}$ (kN) | $V_{b\,(st,y)}$ (kN) | $V_{b\,(Dyn,e)}$ (kN) | $R_s$ | $R_\mu$ | $R_{ASD}$ | $R_{LRFD}$ |
|---|---|---|---|---|---|---|---|---|---|
| Cape Mendocino | 0.02 | 0.36 | 4449.10 | 1058.35 | 8936.49 | 4.2 | 2.01 | 12.16 | 8.44 |
| Chi-Chi, Taiwan | 0.02 | 2.29 | 3891.64 | | 12828.31 | 3.68 | 3.30 | 17.45 | 12.12 |
| Coyote Lake | 0.02 | 0.63 | 3732.48 | | 10037.18 | 3.53 | 2.69 | 13.66 | 9.48 |
| Kobe | 0.02 | 1.65 | 3899.91 | | 11545.85 | 3.68 | 2.96 | 15.71 | 10.91 |
| Kocaeli, Turkey | 0.02 | 0.38 | 4481.57 | | 9994.97 | 4.23 | 2.23 | 13.60 | 9.44 |
| Landers | 0.02 | 0.47 | 3350.48 | | 5544.32 | 3.17 | 1.65 | 7.54 | 5.24 |
| Loma Prieta | 0.02 | 0.45 | 4027.33 | | 11260.17 | 3.81 | 2.80 | 15.32 | 10.64 |
| Morgan Hill | 0.02 | 0.17 | 4886.31 | | 10550.71 | 4.62 | 2.16 | 14.36 | 9.97 |
| N. Palm Springs | 0.02 | 0.30 | 4207.83 | | 14137.93 | 3.98 | 3.36 | 19.24 | 13.36 |
| Northridge | 0.02 | 0.34 | 3582.51 | | 12905.91 | 3.38 | 3.60 | 17.56 | 12.19 |
| Parkfield | 0.02 | 0.18 | 4711.97 | | 15001.68 | 4.45 | 3.18 | 20.41 | 14.17 |
| San Fernando | 0.02 | 0.16 | 6050.90 | | 18873.69 | 5.72 | 3.12 | 25.68 | 17.83 |
| Superstition Hills | 0.02 | 1.01 | 4081.88 | | 7687.53 | 3.86 | 1.88 | 10.46 | 7.26 |
| Victoria, Mexico | 0.02 | 0.58 | 3547.13 | | 9902.74 | 3.35 | 2.79 | 13.47 | 9.36 |
| Whittier Narrows | 0.02 | 0.24 | 4833.02 | | 13725.16 | 4.57 | 2.84 | 18.67 | 12.97 |
| Tabas | 0.02 | 0.51 | 4403.43 | | 12820.40 | 4.16 | 2.91 | 17.44 | 12.11 |
| Bam | 0.02 | 1.07 | 3859.45 | | 9586.43 | 3.65 | 2.48 | 13.04 | 9.06 |
| Average | | | | | | 4.00 | 2.70 | 15.63 | 10.86 |
| $\sigma$ | | | | | | 0.60 | 0.54 | 4.07 | 2.83 |
| C.V. | | | | | | 0.15 | 0.20 | 0.26 | 0.26 |

The results obtained through analysis under different records have been averaged in order to find a unique value of response modification factor for application in the CKBF. However, the out of scope data should be omitted from the results before averaging process in order to reach more accurate results. For this purpose, the maximum and minimum results with C.V. over 0.4 have been omitted first and then the averaging process has been repeated. The values of final response modification factor, ductility factor and over-strength factor are presented in Tables 6-9.

**Table 6.** Average values of over-strength factor, ductility factor and response modification factor for the first group frames with chevron V bracing

| No. of Stories | $R_s$ | $R_\mu$ | $R_{ASD}$ | $\sigma$ | C.V. | $R_{LRFD}$ | $\sigma$ | C.V. |
|---|---|---|---|---|---|---|---|---|
| 3 | 3.66 | 3.52 | 18.65 | 7.1 | 0.38 | 12.95 | 4.9 | 0.38 |
| 6 | 3.06 | 3.38 | 14.86 | 3.9 | 0.26 | 10.34 | 2.7 | 0.26 |
| 9 | 2.55 | 3.21 | 11.90 | 3.6 | 0.31 | 8.26 | 2.5 | 0.31 |
| 12 | 2.61 | 2.72 | 10.31 | 3.5 | 0.34 | 7.16 | 2.5 | 0.34 |
| Average | 2.97 | 3.21 | 13.94 | 4.5 | 0.32 | 9.68 | 3.2 | 0.32 |

**Table 7.** Average values of over-strength factor, ductility factor and response modification factor for the first group frames with chevron inverted V bracing

| No. of Stories | $R_s$ | $R_\mu$ | $R_{ASD}$ | $\sigma$ | C.V. | $R_{LRFD}$ | $\sigma$ | C.V. |
|---|---|---|---|---|---|---|---|---|
| 3 | 4.24 | 3.27 | 19.94 | 6.2 | 0.31 | 13.85 | 4.3 | 0.31 |
| 6 | 3.08 | 3.75 | 16.58 | 4.1 | 0.24 | 11.51 | 2.8 | 0.24 |
| 9 | 2.81 | 3.19 | 12.99 | 3.3 | 0.25 | 9.02 | 2.3 | 0.25 |
| 12 | 3.07 | 2.44 | 10.94 | 3.2 | 0.29 | 7.60 | 2.2 | 0.29 |
| Average | 3.30 | 3.16 | 15.11 | 4.2 | 0.28 | 10.49 | 2.9 | 0.28 |

**Table 8.** Average values of over-strength factor, ductility factor and response modification factor for the second group frames with chevron V bracing

| No. of Stories | $R_s$ | $R_\mu$ | $R_{ASD}$ | $\sigma$ | C.V. | $R_{LRFD}$ | $\sigma$ | C.V. |
|---|---|---|---|---|---|---|---|---|
| 3 | 5.03 | 3.16 | 22.94 | 7.7 | 0.33 | 15.93 | 5.3 | 0.33 |
| 6 | 3.89 | 3.62 | 19.91 | 6.6 | 0.33 | 13.83 | 4.6 | 0.33 |
| 9 | 4.00 | 2.70 | 15.63 | 7.1 | 0.26 | 10.86 | 2.8 | 0.26 |
| 12 | 3.65 | 2.18 | 11.58 | 3.6 | 0.31 | 8.04 | 2.5 | 0.31 |
| Average | 4.14 | 2.91 | 17.52 | 5.5 | 0.31 | 12.16 | 3.8 | 0.31 |

**Table 9.** Average values of over-strength factor, ductility factor and response modification factor for the second group frames with chevron inverted V bracing

| No. of Stories | $R_s$ | $R_\mu$ | $R_{ASD}$ | $\sigma$ | C.V. | $R_{LRFD}$ | $\sigma$ | C.V. |
|---|---|---|---|---|---|---|---|---|
| 3 | 6.16 | 2.90 | 25.87 | 7.7 | 0.30 | 17.96 | 5.3 | 0.30 |
| 6 | 4.98 | 3.24 | 22.81 | 6.6 | 0.29 | 15.84 | 4.6 | 0.29 |
| 9 | 4.05 | 2.50 | 14.64 | 4.9 | 0.34 | 10.16 | 3.4 | 0.34 |
| 12 | 4.60 | 1.91 | 12.84 | 4.1 | 0.32 | 8.92 | 2.9 | 0.32 |
| Average | 4.95 | 2.64 | 19.04 | 5.8 | 0.31 | 13.22 | 4.1 | 0.31 |

The effect of Easy Going Steel on the response modification factor, over-strength factor and ductility factor have been calculated quantitatively for chevron V and inverted V bracing frames and presented in Tables 10-11, respectively.

**Table 10.** The effect of easy going steel on the response modification factor, over-strength factor and ductility factor (chevron V bracing)

| | No. of Stories | Percent of Increase | Average |
|---|---|---|---|
| $R_\mu$ | 3 | -10.39 | -8.01 |
| | 6 | 6.98 | |
| | 9 | -15.79 | |
| | 12 | -14.72 | |
| $R_s$ | 3 | 37.58 | 40.65 |
| | 6 | 26.99 | |
| | 9 | 57.07 | |
| | 12 | 45.00 | |
| R | 3 | 23.01 | 25.67 |
| | 6 | 33.67 | |
| | 9 | 31.38 | |
| | 12 | 12.32 | |

**Table 11.** The effect of easy going steel on the response modification factor, over-strength factor and ductility factor (chevron inverted V bracing)

| | No. of Stories | Percent of Increase | Average |
|---|---|---|---|
| $R_\mu$ | 3 | -11.28 | -16.78 |
| | 6 | -13.53 | |
| | 9 | -21.76 | |
| | 12 | -22.56 | |
| $R_s$ | 3 | 45.27 | 48.91 |
| | 6 | 61.65 | |
| | 9 | 44.03 | |
| | 12 | 45.70 | |
| R | 3 | 29.72 | 25.99 |
| | 6 | 37.60 | |
| | 9 | 12.72 | |
| | 12 | 17.33 | |

The values of response modification factor have been compared schematically in two conditions (using ST37 and EGS for knee element) in different stories and are presented in Figs. 14-15 for both chevron V bracing and inverted bracing, respectively.

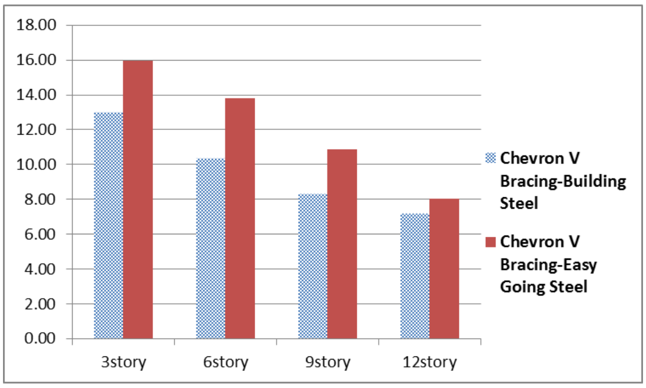


**Fig. 14:** Comparing the response modification factors of different stories in two conditions of chevron V bracing

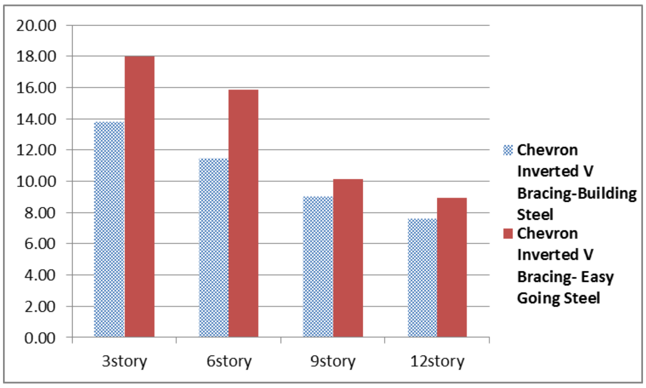


**Fig. 15:** Comparing the response modification factors of different stories in two conditions of chevron inverted V bracing

## 9. Conclusions

In this research the simple steel frame with chevron knee bracing has been investigated and several obtained results are summarized as follows:

1) The average values of response modification factor for CKBF (V bracing) are 9.68 and 12.16 using building steel and EGS in the knee elements respectively, in ultimate strength design method.

2) The average values of response modification factor for CKBF, (inverted V bracing) are 10.49 and 13.22 using building steel and EGS in the knee elements respectively, in ultimate strength design method.

3) The application of EGS will lead to an average increase of 25% in the response modification factor. This increase value is 25.67% in chevron V bracing status and 25.99% in chevron inverted V bracing status. Therefore, the application of EGS in CKBF will improve the overall seismic structural performance.

4) The application of EGS will lead to the average increase of 40.65% in the over-strength factor value in chevron V bracing status and 48.19% in chevron inverted V bracing status.

5) The application of EGS will lead to the average decrease of 8.01% in the ductility factor value in chevron V bracing status and 16.78% in chevron inverted V bracing status.

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