Response modification factors for dual moment-resisting frames with vertical links: Multilevel approach

Vahid Mohsenian1, Nima Gharaei-Moghaddam2 and Iman Hajirasouliha3

Abstract
Despite the growing applications of the performance-based design concepts for seismic design of structures, the response modification factors for structural systems proposed by the current design codes and standards do not generally consider different hazard and performance levels. Therefore, these factors are not directly applicable for performance-based design purposes. As a step to address this shortcoming, the present study aims to propose multilevel response modification factors for multistory dual moment-resisting frames equipped with eccentric braces and vertical links corresponding to different seismicity levels and performance targets. The concept of demand and capacity response modification factors is introduced, and these parameters are calculated for moment-resisting frame structures with 3-, 5-, and 7-stories before and after the addition of vertical steel shear links. It is shown that the calculated capacity response modification factors for the dual frames equipped with vertical links are generally higher than the demand response modification factors proposed by the design code for such systems under the design basis earthquake hazard level. This indicates the efficiency of the eccentric braces with the vertical links in improving the seismic reliability and performance of the moment-resisting steel frames. Based on the results of this study, the demand response modification factor for the studied dual lateral load-resisting system is calculated to be in the range of 7–10.

Keywords
demand response modification factor, capacity response modification factor, incremental dynamic analysis, eccentric braces, vertical link, steel structures

Highlights
- A novel multilevel approach is adopted to obtain response modification factors for dual moment-resisting frames with vertical links.
- The method takes into account the effects of using different seismicity levels and performance targets.
- Multilevel response modification factors are obtained for 3-, 5-, and 7-story systems before and after adding vertical links.
- Demand response modification factors of dual frames are up to 1.5 times higher than their moment-resisting frame counterparts.
- The response modification factor for the initial design of dual systems is calculated to be in the range of 7–10.

Introduction
As one of the efficient methods to improve the seismic performance of structures, shear link elements can be used to act as a seismic fuse and dissipate a part of the earthquake input energy through plastic deformations under shear and bending (Schmidt et al., 2004). Different studies have been performed on the performance assessment and design of structures with vertical and horizontal shear links. Shayanfar et al. (2014) studied the seismic performance of eccentrically braced frames equipped with vertical shear links using the performance-based plastic design concept. Subsequently, Bouwkamp et al. (2016)
developed an analytical model to simulate the nonlinear inelastic response of eccentric braces with vertical shear links. Lian and Su (2017) conducted an experimental program on eccentrically braced frame structures equipped with vertical shear links. Similarly, Caprili et al. (2018) performed a full-scale experimental program on a one-story eccentrically braced frame structure with horizontal and vertical shear links. Vetr et al. (2017) and Vetr and Ghamari (2019) also performed a wide range of experimental and analytical studies on the seismic behavior of eccentrically braced frames with vertical links. In more recent studies, Mohsenian et al. (2020a, 2021) proposed the application of the eccentric bracing system with vertical links for seismic retrofitting of moment-resisting steel frames. Based on the computed capacity curves of the retrofitted frames, the authors proposed the dual moment-resisting frame equipped with vertical links as a new lateral load-carrying structural system. Mohsenian et al. (2021) also evaluated the performance of the eccentric bracing system with vertical shear links subjected to consecutive ground motion excitations (mainshock–aftershock sequence) using pushover and incremental dynamic analysis. In general, the results of the abovementioned studies have demonstrated the ability of shear links to control lateral displacements and provide high energy dissipation capacity under earthquake excitations.

Accurate estimation of response modification factors (R-factors) is one of the most challenging issues in the seismic design of structures. These factors reflect a combination of nonlinear behavior and economical aspects considerations in the structural design process. It is stated that the response modification factors proposed by design codes ($R_{\text{Code}}$) are generally based on the experiences regarding the performance of various structural systems in the past severe earthquakes (Whittaker et al., 1999). In the current seismic design codes (as well as the majority of the previous research studies), the response modification factors are proposed without considering the seismic demand and/or the expected performance levels of structural systems (ASCE/SEI 7-16, 2016). It means that the response modification factors suggested by the seismic codes for different structural systems ($R_{\text{Code}}$) are generally derived only for the design basis earthquake and the life safety performance level. Therefore, these factors cannot be directly used for other earthquake intensity levels and performance targets. This is especially important for the new structural systems due to the lack of information regarding their performance in severe earthquakes. More accurate estimations of the response modification factors will lead to more reliable seismic design solutions (Bertero, 1989). Moreover, there are several issues regarding the application of response modification factors suggested by the design codes in the process of performance-based design of structures. For example, these factors are generally obtained for a specific performance target and therefore cannot be used to satisfy multi-performance targets. On the other hand, due to the lack of sufficient information and experiences from the past earthquakes about the structural performance of new structural systems (compared to the standard systems covered in the design codes), using the response modification factors proposed by the seismic design codes for the design of new systems may result in uncertainty in the seismic performance of these structures in future earthquake events.

The concept of the response modification factor has been widely investigated by different researchers in the past. For example, Malley and Cheever (2007) discussed the history of the development of response modification factors for different structural systems. By introducing the performance-based design approaches in the modern seismic design codes and utilizing various performance targets and hazard levels, the need for multilevel assessment of structures has been identified. To address the limitations with the conventional code-based modification factors, which are developed only based on a single hazard level and performance target, the response modification factors can be derived based on the seismicity of the site ($R_{\text{Demand}}$) or on the basis of the predefined performance levels which demonstrate the allowable damage levels in the structure ($R_{\text{Supply}}$). This approach results in multilevel response modification factors which are useful for the performance-based design methodologies. This concept has been adopted by Mohsenian and his colleagues to obtain response modification factors for different structural systems.

In one of the most relevant studies, Mohsenian and Mortezaei (2018) derived the multilevel response modification factor for the eccentric bracing frames equipped with vertical links. In this study, due to utilizing hinge connections, the frame elements only carried the gravity loads, and the lateral load-resisting structural system was provided only by the bracing elements. The response modification factors were calculated for the situation that the shear strains in vertical links reached the ultimate value and there was no remaining load-carrying capacity in the system. Based on the findings of this study, it was shown that using a response modification factor equal to 8 in the design process of this system results in acceptable reliability for the structures under high levels of seismic intensity. A similar study has been performed on eccentric bracing systems with dual vertical links by Mohsenian and Nikkhoo (2019). In their investigation, simple frames were used, and the response modification factors were calculated for different shear strain states in the vertical links. The results were then used to derive a matrix of response modification factors for the studied structure, considering different earthquake intensity levels and different damage levels in the vertical links. According to this study, a
response modification factor equal to 8 is desirable for the initial design of this structural system. The concept of the multilevel response modification factor was also utilized for other structural systems. For example, Mohsenian et al. (2020d) performed a reliability analysis on the steel diagrid structural system and determined multilevel response modification factors for this type of structure. In another study, Mohsenian et al. (2019a) evaluated the effects of geometrical irregularities in the plan on the seismic performance of tunnel-form structures and calculated multilevel response modification factors for the irregular tunnel-form systems.

In one of the first attempts, Mohsenian and Mortezaei (2019) utilized the concept of multilevel estimation of the response modification factor to evaluate the effects of the steel coupling beam on the seismic performance of tunnel-form concrete structures. However, the multilevel approach and calculation of demand and supply can be used for any kind of structural response. For example, Mohsenian et al. (2019b) utilized this approach for estimating the maximum acceleration demand of acceleration-sensitive nonstructural components. In a follow-up study, in order to reduce the computational time and effort during the estimation of multilevel response modification factors, Mohsenian et al. (2020c) proposed a new scenario based on the novel endurance time analysis method.

As a step to bridge the knowledge gaps discussed above, this article aims to (i) evaluate the efficiency of the eccentric bracing system with vertical shear links as a means for improving the seismic performance of moment-resisting frames; (ii) evaluate the seismic performance and reliability of a dual structural system, which consists of a moment-resisting frame structure equipped with eccentric bracing and vertical shear links, using a comprehensive parametric study; (iii) propose multilevel response modification factors for seismic design of the proposed dual lateral load-resisting system corresponding to different seismicity levels and performance targets; and (iv) demonstrate the efficiency of the suggested response modification factors for moment-resisting frame structures with 3-, 5-, and 7-stories before and after addition of vertical shear links.

Properties of the studied models

In this study, three multistory intermediate moment-resisting frame structures with 3-, 5-, and 7-stories are designed. The geometrical properties of these structures are shown in Figure 1. As it is evident, these structures are geometrically symmetric. The dead (QD) and live (QL) loads are also applied symmetrically with respect to the z-axis (QD = 2400 kgf/m and QL = 800 kgf/m). All the spans are 5 m wide, and the story height is equal to 3.2 m.

The site of interest is assumed to have very high seismicity with site soil type D (stiff soil) according to the classification of ASCE7-16, with the velocity of the shear wave varying between 183 and 366 m/s. Specification of the beam and column sections of the designed structures for each story level (Bi, Ci) are presented in Table 1 as identified in Figure 1. In the design and modeling of the

![Figure 1. Geometrical properties and gravitational loads of the moment-resisting frames.](image_url)
frames, the diaphragms are assumed to be rigid. Besides, the material used in all the models is mild steel (A36) with a yield stress of 250 MPa, a Poisson’s ratio of 0.26, and a modulus of elasticity of 210 GPa (ASTM A36/A36M-19, 2019).

As shown in Figure 2, the eccentric bracing system is mounted in the middle span of the dual frames. The equivalent static method is used for the design of the braces. In the dual structural system, the vertical links are designed to yield before any other structural element, so they can play the role of seismic fuses under the intensity corresponding to the design hazard earthquake (with the return period of 475 years) (Montuori et al., 2016). To achieve this, only 50% of the story shear values are used for the design of the eccentric bracing system. This means that the links at each story are designed to have a shear capacity equal to the 50% of the design story shear force at that level. After the selection of the link cross sections, assuming constant axial force in the braces when the link yields, the maximum axial force in the braces is determined using equilibrium equations. Subsequently, the cross-section area of the braces is calculated, and their slenderness is checked to satisfy the code requirements.

In this study, to ensure shear yielding of the vertical links and increase their energy absorption capacity, the length of these elements (e) is taken equal to 200 mm (Sabouri-Ghomi and Saadati, 2014; Duan and Su, 2017). The product of the plastic shear capacity of the link and the sum of the length of the link and half of the beam depth \( V_p \times \left( e + \frac{h}{2} \right) \) gives the concentrated moment applied to the story beam. This moment should be lower than the plastic moment capacity \( M_p \) of the story beam. Controlling the interaction of bending moments and axial force in the columns of the braced span is the last step of the design of the dual system. The utilized sections for the vertical links and braces \( (L_i, K_i) \), as shown in Figure 2, are presented in Table 2. The dimensions and static parameters of these sections can be found in DIN 1025 (1995).

### Table 1. Properties of the beam and column sections (dimensions are in mm).

| Columns | Beams |
|---------|-------|
| ID      | Section | ID      | Section           |
| C₀      | Box(200 × 15) | B₀      | Web(220 × 8)-flanges(150 × 20) |
| C₁      | Box(250 × 20) | B₁      | Web(200 × 8)-flanges(150 × 20) |
| C₂      | Box(200 × 20) | B₂      | Web(300 × 8)-flanges(180 × 20) |
| C₃      | Box(270 × 20) | B₃      | Web(270 × 8)-flanges(150 × 20) |
| C₄      | Box(240 × 20) | B₄      | Web(300 × 8)-flanges(180 × 25) |
| —       | —       | B₅      | Web(300 × 8)-flanges(150 × 25) |

**Modeling nonlinear behavior**

PERFORM-3D software is used for nonlinear modeling and analysis (Computer and Structures Inc., 2017). The beam and column elements are modeled using the generalized load–deformation diagram depicted in Figure 3. The parameters \( a, b, \) and \( c \) in this figure are taken from the table of acceptance criteria in nonlinear methods for steel components in accordance with the yield mode and cross-section conditions as suggested by ASCE/SEI 41-17 (2017). In Figure 5, \( Q, Q_y, \) and \( \theta_y \) represent the ultimate
strength, yield strength, and yield rotation of the members, respectively.

The beam and column elements are assumed to have linear behavior with concentrated plastic hinges at both ends of the members (points susceptible to the formation of plastic hinges). It should be noted that the utilized hinges for the beam elements are only flexural hinges (M-hinges), while for the columns, axial-flexural plastic hinges (P-M hinges) are utilized. P-delta effects are also included in the analysis using PERFORM-3D software (Computer and Structures Inc., 2017).

Although the eccentric bracing system is designed by considering linear behavior for the braces, in this study for the performance evaluation and verification of this design assumption, nonlinear elements are used to simulate the behavior of the braces. Due to the assumption of hinge connections at both ends of braces and their energy absorption through the formation of axial joints (yielding or buckling of the element due to tension and compression, respectively), axial deformations at the expected buckling load, $\Delta_b$, and the yield tensile load, $\Delta_t$, have been selected as the criteria to calculate their ductility and nonlinear behavior (ASCE/SEI 41-17, 2017). Equation (1) can be used to calculate the axial deformation of the braces

$$\Delta = \frac{FL}{EA}$$  \hspace{1cm} (1)

where $E$ and $A$ are the material modulus of elasticity and the cross-section area of the braces, respectively. Taking the free length for the brace as $L$, the parameters $\Delta_b$ and $\Delta_t$ can be derived by replacing $F$ in this equation with the expected tensile strength of the braces, $T_{CE}$, and their lower bound of the strength under compressive forces, $P_{CL}$, respectively.

In this study, the response of the brace elements in the nonlinear range of behavior is based on the generalized load–deformation relationship depicted in Figure 4. The parameters $a$, $b$, and $c$ in this diagram are taken from the corresponding table of the modeling and acceptance criteria in nonlinear methods for steel components (in this study, double channels) according to ASCE/SEI 41-17 (2017). To model the braces in the PERFORM-3D software (Computer and Structures Inc., 2017), a “Simple Bar” element is used that resists only against axial forces.

To model the vertical links, first, these structural elements are modeled in OpenSees software (Mazzoni et al., 2006) by taking into account their support and loading conditions. Subsequently, their capacity curves are derived using cyclic loading. Finally, the resulting curves are idealized as bilinear capacity curves for use in PERFORM-3D software (Computer and Structures Inc., 2017) (see Figure 5). The accuracy of this model compared to the experimental tests results on link elements is demonstrated in the previous study by Mohsenian et al. (2020b).

It should be noted that the shear capacity of the sections, $V_p$, in this case, is in good agreement with the capacity obtained from equation (2). In this equation, $A_w$ and $F_{ye}$ are the cross-section area of the vertical link and the expected yield stress of the material, respectively. The shear deformation angle of the link, $\gamma$, is another important parameter in the design. As shown in Figure 6, after modeling the link in ABAQUS (2014), four limit states are considered for this parameter based on the maximum stresses in the member. It should be noted that the effect of axial forces in the vertical links is negligible and therefore is ignored in the analyses

$$V_p = 0.6A_wF_{ye}$$  \hspace{1cm} (2)

In the previous experimental studies, the deformation angle of links before failure has been reported to vary from 0.128 to 0.156 radian (Zahrai and Mahroozadeh, 2010). However, to provide conservative results in the present study, the maximum acceptable shear deformation angle of the vertical links for the seismic design is limited to 0.1 radian (strain at which the ultimate stress of the material, $F_{u}$, develops in the link for the first time). As mentioned earlier, the vertical links are designed in a way to ensure they yield under the design basis earthquake intensity level. Thus, the deformation angle at the moment of yielding, $\gamma_{yr}$, is an important limit state that is considered as the lower

Table 2. Properties of the utilized sections for vertical links and braces (dimensions are in mm).

| ID | Section | ID | Section | ID | Section |
|----|---------|----|---------|----|---------|
| K₀ | 2UNP100 | L₀ | IPE 270 | L₄ | IPE 300 |
| K₁ | 2UNP120 | L₁ | IPE 240 | L₅ | IPE 200 |
| K₂ | 2UNP140 | L₂ | IPE 160 | L₆ | IPE 400 |
| K₃ | 2UNP160 | L₃ | IPE 330 | L₇ | IPE 360 |

Figure 3. Load–deformation diagrams used for beams and columns and the quantitative values for rotations of joints corresponding with different performance levels (adopted from ASCE/SEI 41-17 (2017)).

Figure 5. The accuracy of this model compared to the experimental tests results on link elements is demonstrated in the previous study by Mohsenian et al. (2020b).
bound for the shear deformation angle of the links. The parameter $\gamma_y$ can be calculated using equation (3)

$$\gamma_y = \frac{V_p}{k_e e}$$

In this equation, $e$ and $k_e$ are the length and stiffness of the link element, respectively. As is evident in Figure 6, the level of stress in the wings of the links is always lower than that of the web. This observation confirms that the links yield in shear. As shown in Figure 6, the considered boundary conditions for the link elements in ABAQUS (2014) are to represent the real applications (one end of the link is fixed and the other end is free to rotate).

To model the vertical links in the PERFORM-3D software (Computer and Structures Inc., 2017), a column element with linear behavior and a concentrated shear hinge (with hysteresis curve depicted in Figure 5) is used. It should be stated that all the section nodes at the free end are coupled for the lateral deformation.

The applied gravitational loads to the frames are identical in both linear and nonlinear phases. It should be noted that in the combination of gravitational and lateral loading, according to equation (4), the upper limit of gravitational loads is taken into account to provide the most critical conditions in terms of maximum stress in the beams and columns (ASCE/SEI 41-17, 2017)

$$Q_g = (Q_D + 0.25Q_L)$$

Calculation of multilevel response modification factors

In this section, the three different definitions for the response modification factor are briefly described (Mohsenian et al., 2019a, 2019b) and are subsequently derived separately for the modeled frames before and after the addition of eccentric braces with vertical links.

Design response modification factor ($R_{Code}$)

Since the ductility requirement is not involved in the determination of this response modification factor, the term “force method” is also used in seismic design codes to introduce this factor. In the current seismic design guidelines, the values proposed for the response modification factor, $R_{Code}$, are usually independent of the vibration period of structures and are mainly based on the structural system and the material types. The ambiguities about this response modification factor are described earlier in the Introduction section. In the present study, the moment-resisting frame structures are designed based on the proposed design response modification factor for “intermediate moment-resisting frames” equal to 7 in accordance with the Iranian seismic design standards (Permanent Committee for Revising the Standard 2800, 2014).
Demand (ductility/deformation) response modification factor \( (R_{\text{Demand}}) \)

The demand response modification factor, \( R_{\text{Demand}} \), is a function of the site seismicity. Moreover, the effects of the physical and geometrical properties of the structure are included in this factor. Based on the results of the past studies on demand response modification factors, the seismic parameters such as the magnitude of ground motion and its focal depth have no considerable influence on this factor. On the other hand, the value of this factor greatly depends on dynamic and geometrical properties of the structural system such as height, ductility, energy dissipation capacity, vibration period, indeterminacy, overstrength, and also the type of soil beneath the structure (ATC-19, 1995; Lia and Biggs, 1980; Miranda, 1991). In the present study, the demand response modification factor is derived using the relation below

\[
R_{\text{Demand}} = R_{\text{SDOF}}^\mu R_m \Omega_S R_d
\]  

(5)

In this equation, \( R_{\text{SDOF}}^\mu \) is the R-factor corresponding to the equivalent single-degree-of-freedom (SDOF) system. To account for the multiplicity of degrees of freedom, the R-factor of the equivalent SDOF is multiplied by the correction factor \( R_m \) (Moghaddam and Karami Mohammadi, 2001; Santa-Ana, 2004). However, since the R-factors in the present study are derived directly from the results of the analyses on multi-degrees-of-freedom systems (i.e., the original frame structures, not the equivalent SDOF systems), the correction factor \( R_m \) is not required. Accordingly, equation (5) can be rewritten in the following simpler form

\[
R_{\text{Demand}} = R_{\text{MDOF}}^\mu \Omega_S R_d
\]  

(6)

In equation (6), \( \Omega_S \) is the overstrength coefficient that includes the effects of indeterminacy of the structural system, and \( R_d \) is the allowable stress factor. This coefficient is used to reduce forces to the design strength level. \( R_{\text{MDOF}}^\mu \), \( \Omega_S \), and \( R_d \) can be calculated using the following relations (Elnashai and Mwafy, 2002; Mohsenian et al., 2020e)

\[
R_{\text{MDOF}}^\mu = \frac{V_e}{V_y}
\]  

(7)

\[
\Omega_S = \frac{V_y}{V_S}
\]  

(8)

\[
R_d = \frac{V_S}{V_d}
\]  

(9)

In the above equations, \( V_e \) and \( V_y \) are the elastic base shear and the yield base shear of the systems, respectively. In this study, first, the accelerograms are scaled to a given hazard level to derive the so-called demand earthquakes. Then these demand earthquakes are applied to the linear models of the structure, and the average of the derived base shear values \( (V_a) \) is considered as the elastic base shear \( (V_e) \) of the system (see Figure 7). In the present study, artificial accelerograms compatible with the design basis earthquake are used to achieve better consistency with the site seismicity and the selected design spectrum. In such circumstances, both the earthquake response spectrum and its peak acceleration are close to the values corresponding to the design hazard level. In Figure 8, the spectra of the produced artificial accelerograms are compared with the demand spectrum of the site.

The earthquake records introduced in Table 3 and the wavelet transform method have been used to produce the artificial accelerograms used in this study (Hancock et al., 2006). It should be noted that the records listed in Table 3 are extracted from the peer site based on the characteristics of the site soil (shear wave velocity between 175 and 375 m/s) (PEER, 2019).

To derive \( V_y \), according to schematic Figure 9, the demand earthquakes are applied to the nonlinear model of the structure and the maximum lateral displacement of the roof, \( \Delta_r \), and consequently, the average roof drift \( \theta_{\text{Avg}} \) is recorded. Then, the average of the derived values which is
called the maximum average roof drift corresponding to the design earthquake or the average target drift is calculated (see Table 4). As depicted in schematic Figure 10, pushover analysis is performed on the structures assuming both linear and nonlinear behaviors, and the corresponding capacity curves are extracted. Then, the nonlinear capacity curve is bi-linearized based on ASCE/SEI 41-17 (2017) guidelines. In the next step, the shear corresponding with the average target drift is extracted as $V_y$ from the derived bilinear capacity curve as suggested by Mwafy and Elnashai (2002) (see Figure 10).

As depicted in Figure 10, the base shear corresponding to the start of nonlinear behavior, $V_s$, is identified. The design base shear, $V_d$, is then derived by dividing the product of the total weight of the structure and the spectral acceleration obtained from the linear spectrum to the code R-factor $(i.e., S_h W / R_{\text{Code}})$. The demand R-factor of the studied structures, before and after the addition of the links, is calculated using equations (6)–(9). The obtained results are presented in Tables (5)–(7).

### Table 3. Earthquakes utilized for generating artificial accelerograms and incremental dynamic analysis.

| Record no | Earthquake and year | Station | $R_i$ (km) | Component | $M_s$ | $PGA(g)$ |
|-----------|---------------------|---------|------------|-----------|-------|----------|
| R1        | Chi-Chi (Taiwan), 1999 | TAP095  | 109.01     | 90        | 7.6   | 0.15     |
| R2        | Duzce (Turkey), 1999  | Bolu     | 41.3       | 90        | 7.1   | 0.82     |
| R3        | Imperial Valley (US), 1979 | El Centro Array#1 | 29.4 | 230 | 6.5 | 0.38 |
| R4        | Imperial Valley (US), 1979 | Delta      | 33.7       | 352       | 6.5   | 0.35     |
| R5        | Kobe (Japan), 1995    | HIK      | 95.72      | 0         | 6.9   | 0.14     |
| R6        | Loma Prieta (US), 1989 | CDMG58223 | 58.65     | 0         | 6.9   | 0.23     |
| R7        | Loma Prieta (US), 1989 | CDMG58224 | 72.2      | 290       | 6.9   | 0.24     |
| R8        | Loma Prieta (US), 1989 | CDMG58472 | 74.26     | 270       | 6.9   | 0.26     |
| R9        | Loma Prieta (US), 1989 | Capitola   | 27.0       | 0         | 7.1   | 0.53     |
| R10       | Landers (US), 1992    | Yermo Fire Station | 86.0 | 270 | 7.3 | 0.24 |
| R11       | Manjil (Iran), 1990    | Qazvin     | 49.97      | 336       | 7.4   | 0.13     |
| R12       | Northridge (US), 1994 | Canyon Country-WLC | 26.5 | 270 | 6.7 | 0.48 |

* Closest distance to fault rupture.

### Table 4. Maximum roof drift corresponding with the demand earthquake (target drift %).

| Frames $\rightarrow$ | 3-story | 5-story | 7-story |
|----------------------|---------|---------|---------|
| Original frame       | 1.69    | 1.55    | 1.53    |
| Dual system with vertical links | 0.331   | 0.378   | 0.455   |

### Supply (capacity) response modification factor ($R_{\text{Supply}}$)

The supply R-factor ($R_{\text{Supply}}$) is determined based on the allowable levels of damage in the structure and its capacity to withstand nonlinear deformation. The design of structures can be based on the same force method and the choice of strength reduction factors assuming a specified damage level under the predefined design basis earthquake. This is directly aligned with the performance-based idea that is currently used to evaluate the seismic performance of structures in seismic vulnerability requirements (Grecea et al., 2018). The supply response modification factors are derived based on the ATC suggested method considering the lateral strength of the systems (ATC, 1996; Mwafy and Elnashai, 2002). Incremental dynamic analysis (IDA) is performed on the structures in the nonlinear range of behavior. In the present study, the 12 ground motion records introduced in Table 3 are used for IDA. The maximum peak ground acceleration (PGA) and drift are selected as the intensity and response parameters in the IDA analysis,
respectively. It should be noted that the capacity response modification factor ($R_{\text{Supply}}$) is independent of the selected intensity measure for IDA. Accordingly, any intensity measure can be used for this purpose. Considering the comparative nature of the present study and also the significant effect of link elements on the results of eigenvalue analysis, PGA is selected since it is independent of the modal characteristics of the frames. Subsequently, for each ground motion record, the PGA corresponding to different damage levels is derived from the IDA curves. In this study, the maximum drift of the structure corresponding to the immediate occupancy (IO) (0.7%), life safety (LS) (2.5%), and collapse prevention (CP) (5%) is selected as the target performance levels (see Figure 10).

In the following step, as demonstrated in Figure 11, linear dynamic analyses are performed on the structures using the PGA values corresponding to the different damage levels derived in the previous step, and the average base shear, $V_e$, is then calculated. Using the target drift corresponding to the damage levels introduced in Step 1, the capacity curves obtained by the modal pushover analysis of the frames are bi-linearized based on ASCE/SEI 41-17 (2017) suggested method, and finally, the equivalent yield base shear, $V_y$, is obtained (see Figure 10). From this step on, the process and utilized relationships to compute the supply $R$-factor are the same as those used to calculate the demand response modification factor (equations (6)–(9)). The calculated supply response modification factors for the three performance levels are listed in Tables 5–7. In these tables, $R_{\text{Supply}}$, $R_{\text{Supply}}$, and $R_{\text{Supply}}$ represent the supply $R$ for the IO, LS, and CP performance levels, respectively.

**Results and discussion**

For each of the designed structures, in both moment-resisting frame and dual structural systems, the values of

![Figure 10. Schematic process of derivation of capacity curve for structure and its idealization (adopted from Mwafy and Elnashai (2002)).](image)

---

**Table 5.** The code, demand, and capacity response modification factors for the 3-story frame.

| Parameters                          | Moment-resisting frame | Dual system          |
|-------------------------------------|------------------------|----------------------|
|                                     | $R_{\text{Code}}$ | $R_{\text{Demand}}$ | $R_{\text{Supply}}$ | $R_{\text{Demand}}$ | $R_{\text{Supply}}$ | $R_{\text{Supply}}$ | $R_{\text{Supply}}$ |
| PGA (g)                             | 0.35          | 0.35                | 0.154               | 0.35          | 0.588                | 1.13                | 1.45                |
| $V_e$ (tonf)                        | 85.98         | 30.78               | 82.2                | 140.7         | 93.96                | 100.5               | 158.8               | 224.7               |
| $V_y$ (tonf)                        | —             | 44.51               | 44.1                | 45.12         | 30.54                | 30.93               | 48.44               | 60.2                |
| $V_y$ (tonf)                        | —             | 39.94               | 39.94               | 39.94         | 27.15                | 27.15               | 27.15               | 27.15               |
| $V_d$ (tonf)                        | —             | 15.06               | 15.06               | 15.06         | 15.06                | 15.06               | 15.06               | 15.06               |
| RMF due to ductility ($R_\mu$)      | —             | 1.93                | 1                   | 1.86          | 3.118                | 3.07                | 3.25                | 3.73                |
| RMF due to overstrength ($\Omega_s$) | —             | 1.11                | 1                   | 1.104         | 1.13                 | 1.13                | 1.14                | 1.78                | 2.22                |
| RMF due to allowable stress ($R_d$) | —             | 2.65                | 2.65                | 2.65          | 2.65                 | 1.8                 | 1.8                 | 1.8                 |
| RMF ($R = R_\mu \Omega_s R_d$)      | 7             | 5.71                | 2.65                | 9.34          | 6.24                 | 6.67                | 10.54               | 14.92               |

Mohsenian et al. 3307
the code, demand, and supply response modification factors are presented in Figure 12. Figure 13 also compares the average of the calculated values for the designed frames as the base of comparison for the selected lateral load-resisting systems.

As shown in Figure 12(b), for the dual systems the capacity behavior coefficients for all performance levels and their corresponding risk levels were higher than the demand response modification factors at the design risk levels ($R_{\text{Supply}, \mu}$, $R_{\text{Supply}, \Omega}$, $R_{\text{Supply}, \alpha} > R_{\text{Demand}}$). This demonstrates the
high strength and sufficient safety of the dual system for high seismic regions such as the one considered in this study. In the dual frames, for any pair of regular $A_0$ blue ranges, there will certainly be no limiting situations. Thus, when the design basis earthquake (and risk levels below that) and the average behavior coefficient of less than 6 are considered by the designer, the displacement demands will be certainly less than the values corresponding to the desired performance levels. The choice of the response modification factors between the demand and the supply R-factors corresponding to a given performance level will guarantee that the frame performs at the considered performance level for the intensity corresponding to the given performance level or lower intensities.

For instance, for each point in the red area in Figure 12, which is indicated by $A_2$, the story drift will be less than the value corresponding to the LS performance level (2.5%), or for each pair in the green area $A_3$, the relative story displacements are less than the value corresponding to the CP performance level (5%).

It is evident from the results that the selection of the R-factor equal to 7 for the initial design can ensure the safety and reliability of the frame. It is shown that in this case, the maximum story drift will not exceed the predefined target values for LS and the CP levels under the design basis earthquake. The results also indicate that the code R-factor of 7 is always within the safe range. As shown in Figure 13

Figure 12. Response modification factors for the studied structures (a) moment-resisting frame and (b) dual system.
(b), for the dual moment-resisting frame equipped with vertical links, the choice of any value of the response modification factor within the specified gray range will guarantee LS performance level under the design hazard level and lower intensities. Accordingly, the choice of values between 7 and 10 under the design hazard level will guarantee high reliability for the LS performance level. However, due to the wide range of inherent uncertainties associated with future earthquakes and possible aftershocks, choosing values above 10 for the response modification factor is not recommended.

Based on Figure 13 and the values presented in Tables 5–7, for the design earthquake and lower intensities, the maximum response modification factor to remain at the immediate occupancy performance level would be approximately 6.5. According to the small difference between the demand response modification factor ($R_{\text{Demand}} = 6$), the supply factor corresponding to the immediate occupancy level ($R_{\text{Supply}}^1 = 6.5$), and the code response modification factor ($R_{\text{Code}} = 7$), it is expected that the dual systems exhibit immediate occupancy level under design hazard level (and lower intensities). It can be concluded that at the abovementioned hazard level, the dual systems have high reliability for the LS and the CP performance levels.

In the original frames, the calculated supply response modification factors corresponding to the immediate occupancy level are less than the demand response modification factors ($R_{\text{Demand}} > R_{\text{Supply}}^1$). This finding proves the unreliability of the bare frames for the mentioned performance level under the design hazard level. In other words, the original frames under the design hazard level will most probably go beyond the immediate occupancy performance level.

For the studied frames, the supply response modification factor for the CP performance level is higher than the demand response modification factor ($R_{\text{Demand}} < R_{\text{Supply}}^2$). This indicated that under the design earthquakes and lower seismic intensities, the frames will have sufficient reliability for the CP performance level.

A quantitative comparison shows that the addition of eccentric braces with vertical shear links to the intermediate moment-resisting frames makes the capacity response modification factors for the IO, LS, and CP performance levels 6.5, 2, and 1.5 times greater, respectively. Moreover, the results show that the demand response modification factors for the 3-, 5-, and 7-story structures become 1.1, 1.3, and 1.5 times greater, respectively. As depicted in Figure 13(a), the utilized response modification factor equal to 7 for the initial design of frames leads to safe design solutions only for the CP performance level. When the LS performance level is considered, the response modification factor equal to 5 seems appropriate for the intermediate moment-resisting frame systems.

The conducted analysis showed that the dual system, in addition to increasing the demand and supply response modification factors of the frames, results in a significant increase in the response modification factor due to ductility, $R_\mu$. Moreover, it is observed that using the vertical links increases the overstrength response modification factor, while reduces the allowable stress response modification factor. This can be attributed to the early yielding of seismic

![Figure 13. Average response modification factors for the studied structures: (a) moment-resisting frame and (b) dual system.](image-url)
fuses (i.e., vertical links) and their high energy absorption capacity. It is shown that this factor decreases with increasing the height of the frames.

According to the given explanations, since each performance level corresponds to a level of intensity, it is possible to present the response modification factors for each frame in the form of a matrix. In this approach, which is demonstrated in Figure 14, each row of the proposed matrix represents a level of intensity, and each column indicates a performance level. For example, in the proposed matrix notation, when the hazard level PGA2 is considered, the frame will experience LS performance level. It should be noted that the response modification factor is proposed only for this performance level (R2). The reason is that at this level of intensity, the frames go beyond the IO performance level and also do not experience CP performance level. Therefore, no value is proposed for the first and second columns in the second row.

At each level of intensity, for the proposed response modification factor or lower values, the frames will experience the corresponding or higher performance levels. For example, at the PGA2 hazard level, for response modification factors lower than R2 (R ≤ R2), the frame will be at LS or higher performance levels. Similarly, at each row, the proposed response modification factor and the corresponding or lower intensities will guarantee the corresponding or higher performance levels. For instance, by using the response modification factor R2 at the hazard level PGA2 and lower hazard levels (PGA ≤ PGA2), the frame will exhibit LS or higher performance levels.

According to the provided explanation, matrices of the response modification factors for the studied frames, in both moment-resisting and dual structural systems, are presented in Tables 8–10. It should be noted that the presented idea can be utilized for different structural systems considering at least three hazard levels (e.g., serviceability, design, and maximum considered earthquakes) and three performance targets (e.g., IO, LS, and CP performance levels) in the new generation of performance-based design standards. This will lead to a considerable improvement in the safety and economical aspects of the design process.

Based on the results in Tables 8–10, for all original frames at the first hazard level (PGAА), the story drifts will be higher than the limit value corresponding with the IO performance level, which means at this level of intensity, the frames will not satisfy the target performance level. For these frames, on average, a response modification factor equal to 5 under the mentioned hazard level, which is corresponding with the selected design hazard level, will guarantee the LS performance level. The addition of eccentric braces with vertical links increases this value to 6.5. As it is evident, the dual frames even under higher levels of intensity (PGAБ and PGAС) remain at the performance levels higher than the CP.

As can be seen, the vertical links had a very positive effect on the seismic performance of the moment-resisting frames and could considerably improve their seismic performance to an acceptable level. This implies that the combination of the moment-resisting frame system with the

![Figure 14. Matrix presentation for the response modification factor.](image)

**Table 8.** The matrix of response modification factors for the 3-story frame.

| Frame Performance level | Moment-resisting frame | Dual system |
|-------------------------|------------------------|-------------|
| PGAА = 0.35 g           | IO         | 5.46       | 6.67       |
| PGAБ = 0.55 g           | --         | 9.34       | 6.67       |
| PGAС = 0.75 g           | --         | 9.34       | 10.54      |

IO: immediate occupancy; LS: life safety; CP: collapse prevention

**Table 9.** The matrix of response modification factors for the 5-story frame.

| Frame Performance level | Moment-resisting frame | Dual system |
|-------------------------|------------------------|-------------|
| PGAА = 0.35 g           | IO         | 5.23       | 6.53       |
| PGAБ = 0.55 g           | --         | 9.23       | 10.5       |
| PGAС = 0.75 g           | --         | 9.05       | 10.5       |

IO: immediate occupancy; LS: life safety; CP: collapse prevention

**Table 10.** The matrix of response modification factors for the 7-story frame.

| Frame Performance level | Moment-resisting frame | Dual system |
|-------------------------|------------------------|-------------|
| PGAА = 0.35 g           | IO         | 4.86       | 6.47       |
| PGAБ = 0.55 g           | --         | 4.86       | 11.5       |
| PGAС = 0.75 g           | --         | 8.21       | 11.5       |

IO: immediate occupancy; LS: life safety; CP: collapse prevention
eccentric bracing system equipped with vertical links provides a dual system with high energy absorption and dissipation capability that is capable of providing high performance levels under medium to high intensity levels. The dual system can be also designed as a two-phase system using different performance targets. For example, the design can be performed such that the eccentric bracings dissipate the applied earthquake energy up to the design hazard level and after this stage, the moment-resisting frame provides the main lateral strength and stiffness. In this case, the moment-resisting frames can be designed for higher performance levels. While the presented results are based on the frames and designed assumptions used in this study, the outcomes of this research should prove useful in practical design of dual moment-resisting frames with vertical links.

**Conclusion**

In the present study, for the first time, a multilevel approach was used to derive the response modification factors of a dual structural system consisted of intermediate moment-resisting frames and eccentric bracing systems equipped with vertical links. By introducing the concept of demand and supply response modification factors, these two factors are extracted based on the seismicity of the site and the desired performance level for 3-, 5-, and 7-story moment-resisting frames before and after the addition of the eccentric braces with vertical links. The matrix representation of the multilevel modification factors developed for the dual systems in the present study should prove useful for the performance-based seismic design of these systems under different seismic hazard levels and performance targets. Based on the results presented in this article, the following conclusions can be made:

1. In the studied dual structural systems, the intensity corresponding to the IO level is estimated to be close to the intensity of the maximum considered earthquake (return period of 2475 years). Therefore, the moment-resisting frames in the dual systems are expected to remain elastic even under strong earthquake events.
2. The addition of the eccentric braces with vertical links has increased the response modification factors of moment-resisting frames corresponding to IO, LS, and CP performance levels at least 6.5, 2, and 1.5 times, respectively.
3. In general, using vertical link increases the modification factors due to ductility and overstrength, while reduces the modification factors related to allowable stress. By choosing a response modification factor between 7 and 10 in the design process of the studied dual structural systems, it is expected that the LS performance level under the design basis hazard level will be provided.
4. Using a response modification factor of 7 for the initial design of moment-resisting frame structures does not guarantee the LS performance level under the design basis earthquake. Therefore, it is proposed to utilize a response modification factor equal to 5 for this system.

**Declaration of conflicting interests**

The author(s) declared no potential conflicts of interest with respect to the research, authorship, and/or publication of this article.

**Funding**

The author(s) received no financial support for the research, authorship, and/or publication of this article.

**ORCID iD**

Iman Hajirasouliha @ https://orcid.org/0000-0003-2597-8200

**References**

ABAQUS (2014) Version 6.14, Users-Manual, SIMULIA World Headquarters. Rising Sun Mills 166 Valley Street. Providence USA. RI 02909-2499.

ASCE/SEI 41-17 (2017). Seismic rehabilitation of existing buildings, Reston, VA: American Society of Civil Engineers.

ASCE/SEI 7-16 2016. Minimum design loads and associated criteria for buildings and other structures. Reston, VA: American Society of Civil Engineers.

ASTM A36/A36M-19 (2019). Standard specification for carbon structural steel. West Conshohocken, USA: ASTM International.

ATC (1995) Structural response modification factors. ATC-19 Report. Redwood City, CA: Applied Technology Council.

ATC (1996) ATC-40. In: Seismic evaluation of concrete buildings. Red wood, CA: Applied Technology Council, Vol 1.

Bertero VV (1989) Evaluation of response reduction factors recommended by ATC and SEAOC. In: Proceedings of the 3rd US national conference on earthquake engineering, South Carolina, USA, 24–28 August 1986. Oakland, CA: Earthquake Engineering Research Institute

Bouwkamp J, Vetr MG and Ghamari A (2016) An analytical model for inelastic cyclic response of eccentrically braced frame with vertical shear link (V-EBF). *Case Studies in Structural Engineering* 6: 31–44.

Caprili S, Morelli F, Mussini N, et al. (2018) Experimental tests on real-scale EBF structures with horizontal and vertical links. *Data in brief* 21: 1246–1257.

Computers and Structures Inc. (CSI) (2017) Structural and earthquake engineering software, PERFORM-3D nonlinear analysis and performance assessment for 3D structures. Version 7.0.0. CA, USA.
DIN 1025 (1995) hot rolled i and h sections: dimensions, mass and static parameters. Berlin, Germany: DIN Deutsches Institut Fur Normung EV.

Duan L and Su M (2017) Seismic testing of high-strength steel eccentrically braced frames with a vertical link. *Proceedings of the Institution of Civil Engineers - Structures and Buildings* 170(11): 874–882.

Elnashai AS and Mwafy AM (2002) Overstrength and force reduction factors of multistory reinforced-concrete buildings. *The Structural Design of Tall Buildings* 11: 329–351.

Greece D, Dubina D and Dinu F. (2018) Partial q-factor values for performance based design of MR frames. In: *STESSA 2003: Behaviour of Steel Structures in Seismic Areas*. Routledge, 23–28.

Hancock J, Watson-Lamprey J, Abrahamson NA, et al. (2006) An improved method of matching response spectra of recorded earthquake ground motion using wavelets. *Journal of Earthquake Engineering* 10: 67–89.

Lia SP and Biggs JM (1980) Inelastic response spectra for seismic building design. *Journal of the Structural Division (ASCE)* 106(6): 1295–1310.

Lian M and Su M (2017) Seismic performance of high-strength steel fabricated eccentrically braced frame with vertical shear link. *Journal of Construtional Steel Research* 137: 262–285.

Malley J and Cheever P (2007) History of structural response modification factors. In: New horizons and better practices, structures congress 2007, California, USA, 16–19 May, 2007. Long Beach, CA: American Society of Civil Engineers (ASCE).

Mazzoni S, McKenna F, Scott MH, et al. (2006) OpenSees command language manual pacific earthquake engineering research. Pacific Earthquake Engineering Research (PEER) Center, 264.

Miranda E (1991) *Seismic Evaluation and Upgrading of Existing Buildings*. A Ph.D. Thesis. University of California @ Berkeley.

Mohgaddam H and Mohammadi RK (2001) Ductility reduction factor of MDOF shear-building structures. *Journal of Earthquake Engineering* 5(3): 425–440.

Mohsenian V and Mortezaei A (2018) Evaluation of seismic reliability and multilevel response reduction factor (R factor) for eccentric braced frames with vertical links. *Earthquakes and Structures* 14(6): 537–549.

Mohsenian V and Mortezaei A (2019) Effect of steel coupling beam on the seismic reliability and R-factor of box-type buildings. *Proceedings of the Institution of Civil Engineers - Structures and Buildings* 172(10): 721–738.

Mohsenian V and Nikkhoo A (2019) Evaluation of performance and seismic parameters of eccentrically braced frames equipped with dual vertical links. *Structural Engineering and Mechanics* 69(6): 591–605.

Mohsenian V, Nikkhoo A and Hajirasouliha I (2019a) Estimation of seismic response parameters and capacity of irregular tunnel-form buildings. *Bulletin of Earthquake Engineering* 17(9): 5217–5239.

Mohsenian V, Ghaerei-Moghadam N and Hajirasouliha I (2019b) Multilevel seismic demand prediction for acceleration-sensitive non-structural components. *Engineering Structures* 200: 109713.

Mohsenian V, Filizadeh R, Ozdemir Z, et al. (2020a) Seismic performance evaluation of deficient steel moment-resisting frames retrofitted by vertical link elements. *Structures* 26: 724–736.

Mohsenian V, Hajirasouliha I and Filizadeh R (2020b) Seismic reliability analysis of steel moment-resisting frames retrofitted by vertical link elements using combined series-parallel system approach. *Bulletin of Earthquake Engineering* 19: 831–862.

Mohsenian V, Ghaerei-Moghadam N and Hajirasouliha I (2020c) Multi-level response modification factor estimation for steel moment-resisting frames using endurance-time method. *Journal of Earthquake Engineering*: 1–21. DOI: 10.1080/13632469.2020.1845875.

Mohsenian V, Ghaerei-Moghadam N and Hajirasouliha I (2020e) Reliability analysis and multi-level response modification factors for buckling restrained braced frames. *Journal of Constructional Steel Research* 171: 106137.

Mohsenian V, Padashpour S and Hajirasouliha I. (2020d) Seismic reliability analysis and estimation of multilevel response modification factor for steel diagrid structural systems. *Journal of Building Engineering* 29: 101168.

Mohsenian V, Filizadeh R, Hajirasouliha I, et al. (2021) Seismic performance assessment of eccentrically braced steel frames with energy-absorbing links under sequential earthquakes. *Journal of Building Engineering* 33: 101576.

Montuori R, Nastri E and Piluso V (2016) Theory of plastic mechanism control for MRF-EBF dual systems: closed form solution. *Engineering Structures* 118: 287–306.

Mwafy AM and Elnashai AS (2002) Calibration of force reduction factors of RC buildings. *Journal of Earthquake Engineering* 6(2): 239–273.

PEER Ground Motion Database (2019) Pacific Earthquake Engineering Research Center. Available at: http://peer.berkeley.edu/peer_ground_motion_data_base (Accessed November 2019).

Permanent Committee for Revising the Standard 2800 (2014) *Iranian Code of Practice for Seismic Resistant Design of Buildings*. 4th edition. Tehran, Iran: Building and Housing Research Center.

Sabouri-Ghomi S and Saadati B (2014) Numerical modeling of links behavior in eccentric bracings with dual vertical links. *Numerical Methods in Civil Engineering* 1(1): 14–20.

Santa-Ana PA (2004) Estimation of strength reduction factors for elastoplastic structures: modification factors. In: The 13th World conference on earthquake engineering, Vancouver, Canada, 1–6 August 2004. Vancouver, BC, Canada: Canadian Association for Earthquake Engineering, University of British Columbia, paper no. 126.
Schmidt K, Dorka UE, Taucer F, et al. (2004) Seismic retrofit of a steel frame and a RC frame with HYDE systems. Report No. EUR 21180 EN. European Laboratory for Structural Assessment (ELSA).

Shayanfar MA, Rezaeian AR and Zanganeh A (2014) Seismic performance of eccentrically braced frame with vertical link using PBPD method. The Structural Design of Tall and Special Buildings 23(1): 1–21.

Vetr MG and Ghamari A (2019) Experimentally and analytically study on eccentrically braced frame with vertical shear links. The Structural Design of Tall and Special Buildings 28(5): 1587.

Vetr MG, Ghamari A and Bouwkamp J (2017) Investigating the nonlinear behavior of eccentrically braced frame with vertical shear links (V-EBF). Journal of Building Engineering 10: 47–59.

Whittaker A, Hart G and Rojahn C (1999) Seismic response modification factors. Journal of Structural Engineering 125(4): 438–444.

Zahrai SM and Mahroozadeh Y (2010) Experimental study of using vertical link beam to improve seismic performance of steel buildings. Journal of Civil and Surveying Engineering 44(3): 379–393.