© Universiti Tun Hussein Onn Malaysia Publisher's Office

IJIE

http://penerbit.uthm.edu.my/ojs/index.php/ijie
ISSN : 2229-838X e-ISSN : 2600-7916

The International Journal of Integrated Engineering

Effect of Span Length on the Seismic Design Modification Factors of Steel Frames with High Ductility

Nadhirah Borhan[1], Mohammadreza Vafaei[1*], Sophia C Alih[2]

[1]School of Civil Engineering, Faculty of Engineering,
Universiti Teknologi Malaysia, 81310, Skudai, Johor, MALAYSIA

[2]Institute of Noise and Vibration, School of Civil Engineering,
Universiti Teknologi Malaysia, 81310, Skudai, Johor, MALAYSIA

*Corresponding Author

DOI: https://doi.org/10.30880/ijie.2022.14.09.005
Received 25 April 2022; Accepted 15 July 2022; Available online 30 November 2022

Abstract: Seismic design modification factors (SDMFs) control the estimated force and displacement to structures during seismic events. Seismic design codes recommend similar SDMFs for short and long-span structures. Therefore, this study investigated the effect of span length on the SDMFs of steel frames with high ductility. For this purpose, nine steel frames with three different span lengths of 5, 10, and 15 m were selected and designed according to the specifications of ASCE/SEI 7-16. The designed structures were single, double, and three-story frames. The obtained results showed that the values of response modification factor (R), overstrength factor (Ω), and deflection amplification factor (C_d) were increased as the span length was increased. Besides, an increase in the number of stories decreased the value of R and Ω-factors. It was concluded that the inelastic displacement of long-span steel frames might be underestimated if the given C_d values in the seismic design code were used.

Keywords: Span length, behavior factor, overstrength factor, displacement amplification factor, steel frame

1. Introduction

Response modification factor (R-factor) which is also referred to as behavior factor in Eurocode 8, overstrength factor (Ω-factor), and deflection amplification factor (C_d-factor) are referred to as seismic design modification factors (SDMFs). Majority of existing seismic design codes use these parameters to estimate the intensity of seismic force and inelastic lateral displacement of structures when a linear analysis is conducted. The R-factor is an indication for the ability of a structural system to resist seismic forces, the Ω-factor is a measure for the reserve strength in the structural system and Cd-factor is a coefficient which is used to estimate the inelastic lateral displacements of a structural system through the calculated elastic lateral displacement. Seismic design codes often determine the values of these factors empirically based on engineering judgment and observed damage during past earthquakes [1].

Since seismic design codes have not proposed the value of these factors for all types of structural systems, many researchers have estimated them through numerical simulations. For example, Vafaei & Alih [2] assessed the SDMFs of air traffic control tower and showed that an increase in the height of towers decreased the R-factor significantly. Asgarian & Shokrgozar [3] investigated the R-factor of buckling restrained braced frames and proposed 8.35 and 12, respectively, as their R-factor for the ultimate and allowable stress design methods. In another study, Aliakbari & Shariatmadar [4] estimated these factors for steel slit panel-frames through pushover and nonlinear incremental dynamic analysis. They proposed an overstrength factor of 4.16 and a R-factor of 8.11 for the studied frames. Mahmoudi & Zaree [5] worked on the R-factor of concentrically braced steel frames. They reported that concentrically braced steel frames had a smaller R-factor than buckling restrained braced frames.

*Corresponding author: vafaei@utm.my

Mahmoudi & Ghasem Abdi [6] assessed the seismic R-factor of special moment resisting frames equipped with TADAS devices. Results of numerical simulation showed that the installation of TADAS device on the frames increased R-factors. Mohsenian et al. [7] worked on the R-factor of steel diagrid structural systems and showed that an R-factor equal to 4 could ensure a life safety seismic performance level. In another study, Alih et al. [8] showed that the R-factor of RC frames with high ductility was increased as the intensity of live load was increased. Shariati et al. [9] showed that an R-factor of 3 could provide an adequate safety margin against the collapse of tension-only braced frames.

Akbar et al. [10] evaluated the R-factor of haunch retrofitted reinforced concrete frames and suggested an R-factor of 7.5 for them. In another study, Siddiquee et al. [11] assessed the R-factor of concrete frames reinforced with SMA rebar and suggested an R-factor of 3.5 for them. The R-factor for steel structures equipped with friction dampers was investigated by Sadeghi et al. [12]. They reported that the number of dampers and their slip force affected the value of R-factors. Besides, the mean R-factor of frames equipped with friction dampers was 22.8 to 110.1% larger than bare frames. Khalili et al. [13] worked on the R-factor and Cd-factor of single-layer barrel vaults. They considered the effect of rise-to-span ratio and the length of structures in their calculations. The obtained results indicated that the R- factors were dependent on the rise-to-span ratio.

This study investigates the effect of the span length on the SDMFs of steel frames with high ductility. The span length in most structures is short to medium, ranging from 3 to 7 m. However, in some structures like terminals, cinemas, and libraries, architectural constrains require a larger span length. On the other hand, design engineers often employ similar SDMFs for the seismic design of structures regardless of their span length. The main reason relies on the fact that seismic design codes have no specific recommendation for the effects of span length on SDMFs. Therefore, it is of great interest to investigate to what extent span length can alter these parameters.

2. Investigated Structures

In this study, the SDMFs of 9 steel frames were investigated. As shown in Fig. 1, the selected frames are assumed to be part of a library building with three bays in both principal directions. In one direction, the span length of the building was fixed to 8 m, but in another direction, a variable span length was considered. The variable span lengths were 5, 10, and 15 m. The effect of the number of stories on the results was also investigated. For this purpose, the frames were assumed to be single-story, 2-story, and 3-story structures. The height of frames in the ground floor was assumed to be 5 m while in other stories was 4 m. In the design of all frames, it was assumed that a one-way slab covered the floors and the design dead and live loads are, respectively, 6 and 5 kN/m^2. It was assumed that the frames have fixed supports and are located at the seismic design category D and have the site class B as per specifications of ASCE/SEI 7-16 [14].

As shown in Fig. 1, the frame located at axis “C” was selected to calculate the SDMFs. The frames were designed as the special moment-resisting frames using ANST/AISC 360-16 [15] mainly because long-span frames are often used for important structures that require a higher level of ductility. In the design of steel frames, the yield stress and the modulus of elasticity of steel were assumed to be 240 MPa and 200 GPa, respectively. The considered response modification factor (R), overstrength factor (Ω), and deflection amplification factor (C_d) as per the recommendation of ASCE/SEI 7-16 [14] were, respectively, 8, 3, and 5.5 [14]. Besides, the approximate fundamental period (i.e., T_a) of frames with single-story, 2-storey, and 3-storey frames were calculated according to the specification of ASCE/SEI 7-16 [14] and equaled 0.26, 0.42, and 0.56 sec., respectively. Since the frames were assumed to serve as a structure for a library building, their occupancy risk category was II; therefore, their important factor, I_e equaled 1.0 [14]. The seismic base shear of frames was calculated using the equivalent static analysis [14]. ETABS [16] software was used for the seismic design and inelastic analysis of frames.

3. Calculation of The Seismic Design Modification Factors

Pushover analysis (also known as nonlinear static analysis) was conducted to calculate the designed frames' capacity curves. The lumped plasticity method, which has been widely employed for the simulation of inelastic response of structures [17]– [21] was used to determine beams and columns’ inelastic behavior. The moment-rotation relationships of plastic hinges were calculated based on the cross-section size of structural elements and the specifications of ASCE/SEI 41-17 [22]. Two plastic hinges were assigned to both ends of beams and columns. Besides, one plastic hinge was assigned to the mid-span of beams in order to account for the probable plastic deformation due to the positive bending moment. Two different lateral load patterns were used in the pushover analysis. The first lateral load pattern was proportion to the product of floors’ mass and referred to as Uniform. The second lateral load pattern was similar to the first mode shape of each frame and referred to as Mode. As shown in Fig. 2, the obtained capacity curves for frames were idealized using the recommended approach by FEMA 356 [23]. In this approach, the capacity curve can be replaced by a bilinear representation with similar energy to that of the actual curve. The bilinear representations of capacity curves determine the effective yield strength (F_y) and the yield displacement (Δ_y) of frames.

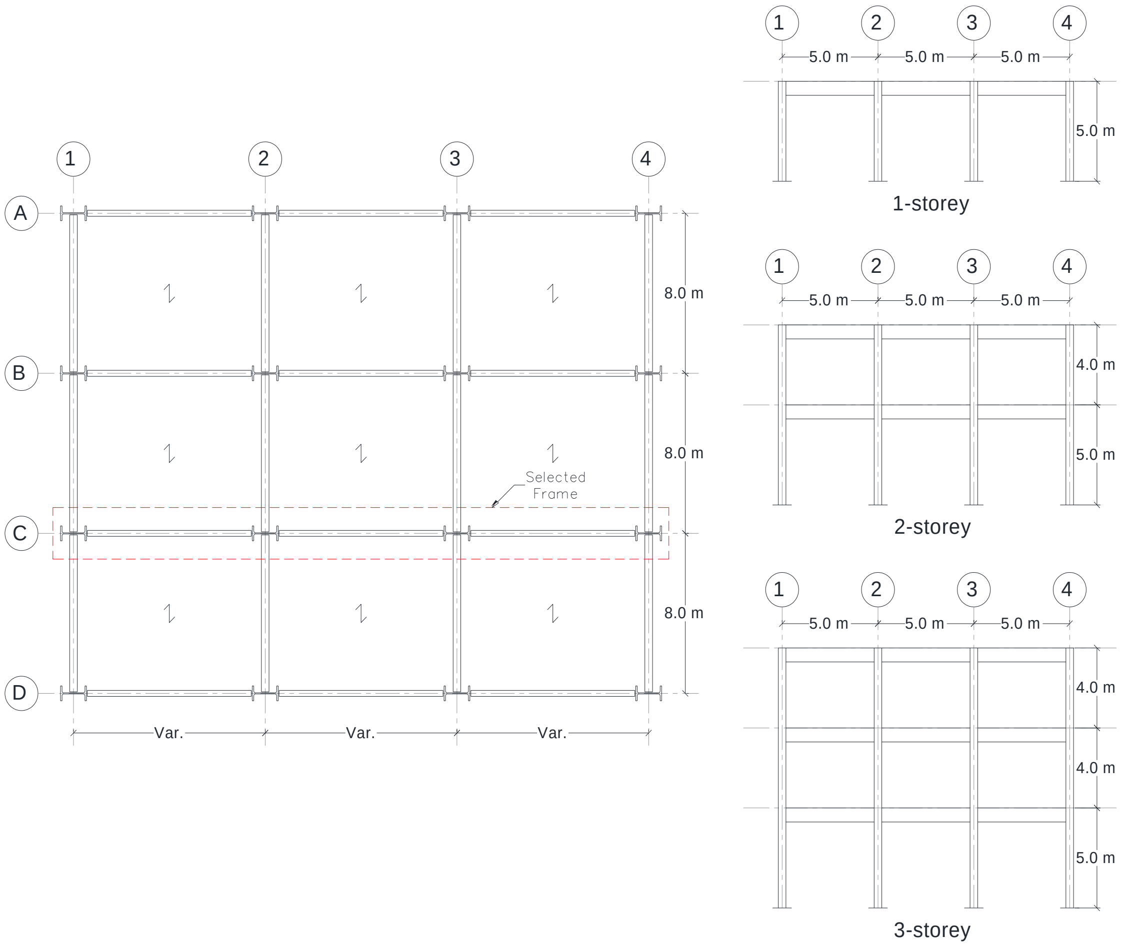

Fig. 1 - Schematic view of the plan and elevation of selected structures

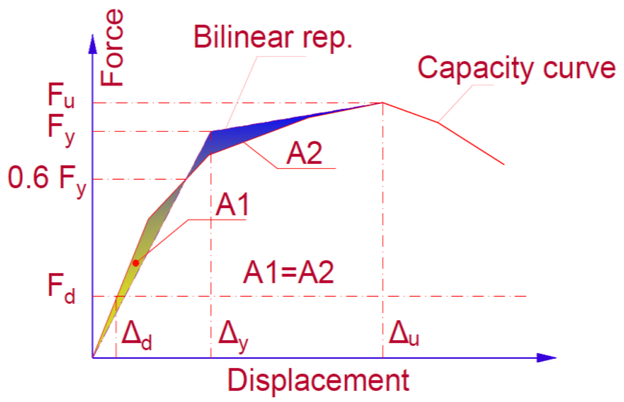

Fig. 2 - Bilinear representation of capacity curves

As shown below, the SDMFs of frames were calculated using the proposed equations by the SEAOC seismology committee [24]:

$$\Omega = F_y/F_d \tag{1}$$

$$C_d = \Delta_u/\Delta_d \tag{2}$$

$$R = R_\mu.\Omega \tag{3}$$

$$\mu=\Delta_u/\Delta_y \tag{4}$$

$$\text{If } T<0.03 \text{ sec.} \quad \text{then } R_\mu=1 \tag{5}$$

$$\text{If } 0.12<T<0.5 \text{ sec.} \quad \text{then } R_\mu=\sqrt{(2\mu-1)} \tag{6}$$

$$\text{If } T>1.0 \text{ sec.} \quad \text{then } R_\mu=\mu \tag{7}$$

In these equations, Δ_u is the ultimate displacement of frames measured at the roof level. F_d is the design base shear and Δ_d is the displacement at the roof level that corresponds to the design base shear.

4. Results and Discussions

4.1 Capacity Curves and Their Bilinear Representations

The obtained capacity curves and their bilinear representations for frames with a 5 m span length are shown in Fig. 3 and Fig. 4 for the Mode and Uniform lateral loading. Tables 1 and Table 2 summarize the obtained results from pushover analysis for all investigated frames. As can be seen, in frames with a 5 m span length, an increase in the number of stories has increased the ultimate load (i.e., F_u) of frames. In contrast, in frames with 10 and 15 m span lengths, an increase in the number of stories has decreased the ultimate load of frames. An almost similar pattern can be seen for the effective yield strength (i.e., F_y) of frames with short and long spans. Results also indicate that as the span length increases the ultimate load and the effective yield load of frames increase.

Besides, an increase in the span length has decreased the displacements corresponding to the yield and ultimate strength of frames. The decrease in the yield and ultimate displacements is more pronounced when the span length increases from 5 m to 10 m. It is also observed that taller frames have a larger yield and ultimate displacements. As can be seen from Table 1 and Table 2, there is no strong correlation between the increase in the span length or height of structures and the calculated displacement ductility ratios (i.e., μ). It is also noteworthy that the difference between the obtained results from the employed lateral load patterns is insignificant. The reason relies on the fact that the studied frames are short structures.

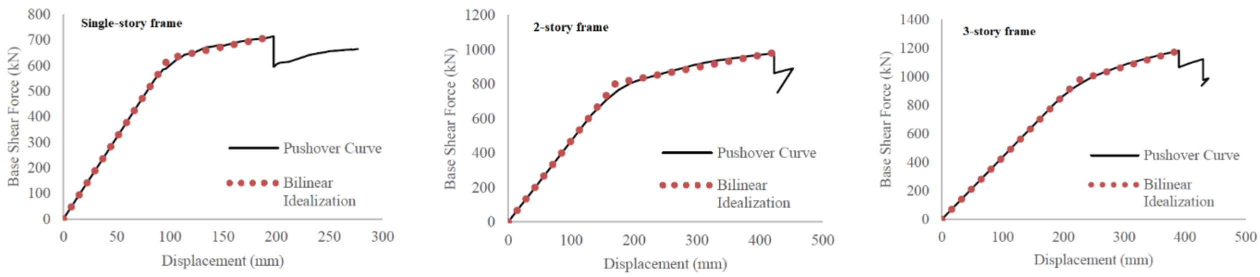

Fig. 3 - Capacity curves and their bilinear representation for frames with 5 m span length under the Mode lateral load pattern

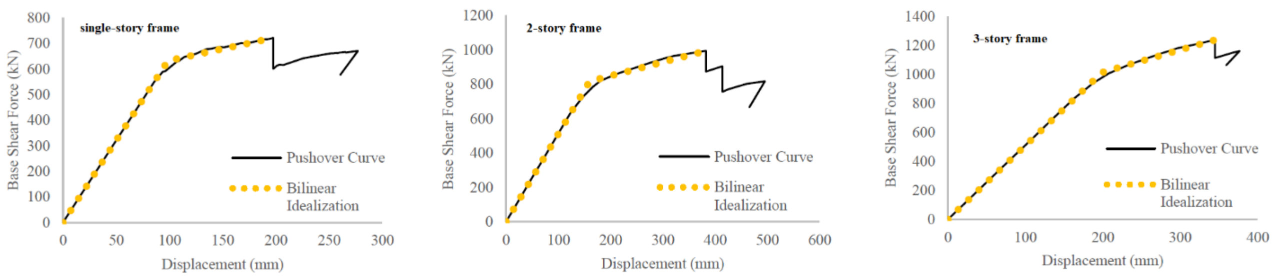

Fig. 4 - Capacity curves and their bilinear representation for frames with 5m span length under the Uniform lateral load pattern

Table 1 - Obtained results from pushover analysis using the Mode lateral load pattern

Frames	Span Length (m)	Δ_d (mm)	F_d (kN)	Δ_y (mm)	F_y (kN)	Δ_u (mm)	F_u (kN)	μ unitless	R_μ unitless
Single-story	5	41	111	109	579	198	714	1.8	1.6
	10	18	224	94	1999	117	2536	1.2	1.2
	15	16	305	95	3918	115	5115	1.2	1.2
2-story	5	76	213	211	746	423	953	2.0	1.7
	10	20	428	118	2253	165	2554	1.4	1.3
	15	16	593	105	3622	161	4593	1.5	1.4
3-story	5	79	240	285	897	381	1083	1.3	1.3
	10	32	146	146	1965	191	2257	1.3	1.3
	15	28	657	139	3269	181	3824	1.3	1.3

Table 2 - Obtained results from pushover analysis using the Uniform lateral load pattern

Frames	Span Length (m)	Δ_d (mm)	F_d (kN)	Δ_y (mm)	F_y (kN)	Δ_u (mm)	F_u (kN)	μ unitless	R_μ unitless
Single-story	5	41	111	109	582	198	721	1.8	1.6
	10	18	224	93	2002	116	2561	1.2	1.2
	15	16	305	96	3927	117	5231	1.2	1.2
2-story	5	76	213	200	758	363	995	1.8	1.6
	10	20	428	109	2306	149	2619	1.4	1.3
	15	16	593	100	3720	153	4637	1.5	1.4
3-story	5	79	240	269	935	335	1129	1.2	1.2
	10	32	481	133	2044	175	2321	1.3	1.3
	15	28	657	139	3269	169	3952	1.2	1.2

4.2 Seismic Design Modification Factors

The obtained results for the overstrength factor of frames have been shown in Fig. 5. It is observed that an increase in the number of stories has decreased the value of the calculated overstrength factors. Besides, frames with a larger span length have a larger overstrength factor. It is also noteworthy that the obtained overstrength factors are all larger than the proposed values by the seismic design code (i.e., Ω=3). As shown in Fig. 6, the R-factor increases as the span length increases. Besides, an increase in the number of stories has decreased the values of the R-factor. Although the R-factor of single- and 2-story frames are equal or larger than the proposed value by the seismic design code (i.e., R=8), the obtained R-factors for 3-story frames are smaller than that. As shown in Fig. 7, an increase in the span length has increased the value of calculated displacement amplification factors. It is also evident that an increase in the number of stories has no strong correlation with the value of calculated C_d. In addition, the obtained displacement amplification factors for frames with 10 and 15 m span lengths are larger than the proposed value by the seismic design code (i.e., C_d=5). However, the C_d values of short-span frames are equal to or smaller than the code proposed value. The obtained results also show that the employed lateral load patterns have resulted in close values for the R, Ω, and C_d.

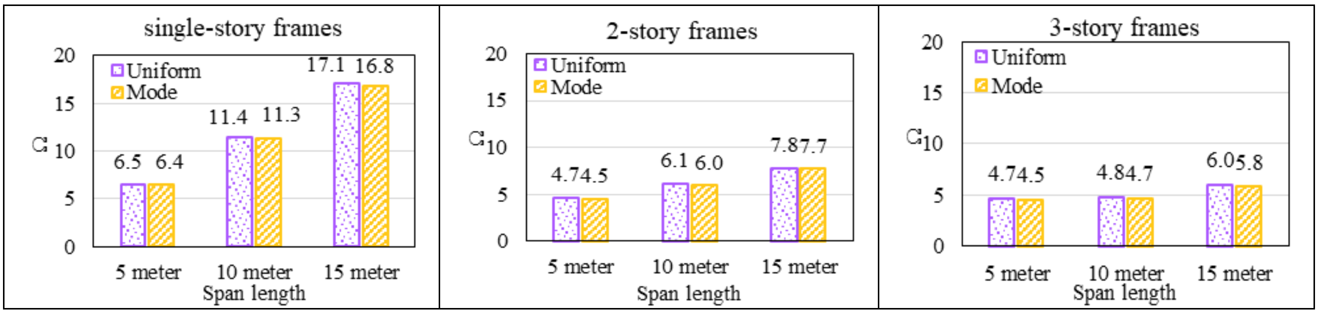

Fig. 5 - Obtained results for overstrength factors

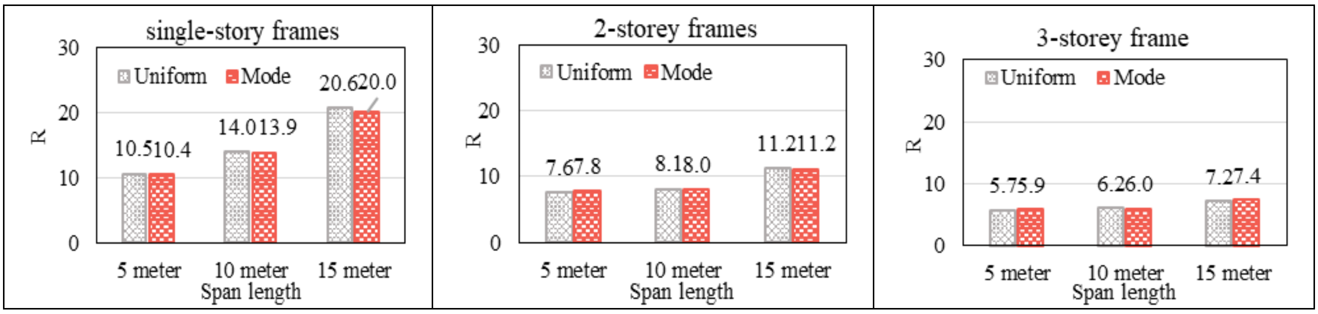

Fig. 6 - Obtained results for response modification factors

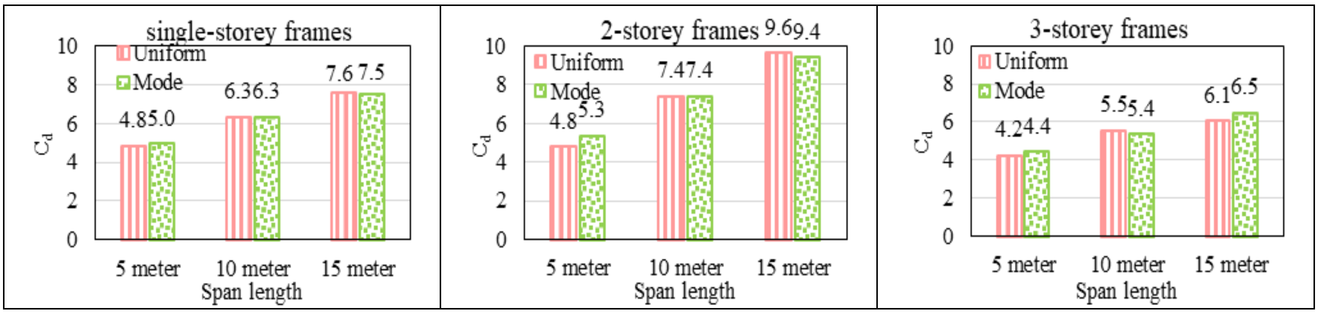

Fig. 7 - Obtained results for displacement amplification factors

5. Conclusions

This study investigated the effect of span length on high ductile steel frames' seismic design modification factors. Nine steel frames with different span lengths (i.e., 5, 10, and 15 m) and the different number of stories (i.e., single-, 2-, and 3-story) were designed according to the requirements of ASCE/SEI 7-16 [14] and ANST/AISC 360-16 [15]. The frames were subjected to two different lateral load patterns, and their capacity curves were obtained using the pushover analysis. The capacity curves were idealized and represented by bilinear curves to calculate their SDMFs. Results indicated that as the span length was increased, the ultimate load and the effective yield load of frames were increased. Besides, an increase in the span length decreased the displacements that corresponded to the yield and ultimate strength of frames. It was also found that an increase in the span length increased the values of SDMFs (i.e., R, C_d, and Ω) of all frames. Furthermore, taller frames exhibited smaller overstrength and response modification factors. It was concluded that the inelastic lateral displacement of long-span steel frames might be underestimated if the seismic design code's C_d value is used. On the other hand, the design base shear of the long-span steel frames can be conservatively estimated if the seismic design code's R-factor is used. It should be mentioned that the obtained results are valid only for the investigated structures, and more studies are needed to generalize the findings of this study.

Acknowledgement

The provided finical supports by Universiti Teknologi Malaysia and the Ministry of Higher Education of Malaysia through the research grant No. 5F365 are acknowledged.

References

[1] Whittaker A., Hart G. & Rojahn, C. (1999). Seismic response modification factors. Journal of Structural Engineering, 125, 438–444. https://doi.org/10.1061/(ASCE)0733-9445(1999)125:4(438)

[2] Vafaei M. & Alih S.C. (2016). Assessment of seismic design response factors of air traffic control towers. Bulletin of Earthquake Engineering, 14, 3441–3461. https://doi.org/10.1007/s10518-016-0008-3

[3] Asgarian B. & Shokrgozar H. R. (2009). BRBF response modification factor. Journal of Constructional of Steel Research, 65, 290–298. https://doi.org/10.1016/J.JCSR.2008.08.002

[4] Aliakbari F. & Shariatmadar H. (2019). Seismic response modification factor for steel slit panel-frames. Engineering Structures, 181, 427–436. https://doi.org/10.1016/J.ENGSTRUCT.2018.12.027

[5] Mahmoudi M. & Zaree M. (2010). Evaluating response modification factors of concentrically braced steel frames. Journal of Constructional of Steel Research, 66, 1196–1204. https://doi.org/10.1016/J.JCSR.2010.04.004

[6] Mahmoudi M. & Abdi M. G. (2012). Evaluating response modification factors of TADAS frames. Journal of Constructional of Steel Research, 71, 162–170. https://doi.org/10.1016/J.JCSR.2011.10.015

[7] Mohsenian V., Padashpour S. & Hajirasouliha I. (2020). Seismic reliability analysis and estimation of multilevel response modification factor for steel diagrid structural systems. Journal of Building Engineering, 29, 101168. https://doi.org/10.1016/J.JOBE.2019.101168

[8] Alih S. C., Vafaei M., Alhariri M. & Ghanim I. (2020). Effects of the intensity of design live load on the seismic design modification factors of reinforced concrete frames. IOP Conference Series: Materials Science and Engineering, 884, 012029. https://doi.org/10.1088/1757-899X/884/1/012029

[9] Shariati M., Majid L., Shervin M., Ali S. & Thoi T. N. (2020). Evaluation of seismic performance factors for tension-only braced frames. Steel and Composite Structure, 35, 599–609. https://doi.org/ 10.12989/SCS.2020.35.4.599

[10] Akbar J., Ahmad N. & Alam B. (2020). Response modification factor of haunch retrofitted reinforced concrete frames. Journal of Performance of Constructed Facilities, 34, 04020115. https://doi.org/10.1061/(ASCE)CF.1943-5509.0001525

[11] Siddiquee K. N., Billah A. M. & Issa A. (2021). Seismic collapse safety and response modification factor of concrete frame buildings reinforced with superelastic shape memory alloy (SMA) rebar. Journal of Building Engineering, 42, 102468. https://doi.org/10.1016/J.JOBE.2021.102468

[12] Sadeghi A., Abdollahzadeh G., Rajabnejad H. & Naseri S.A. (2020). Numerical analysis method for evaluating response modification factor for steel structures equipped with friction dampers (2020). Asian Journal of Civil Engineering, 222(22), 313–330. https://doi.org/10.1007/S42107-020-00315-2

[13] Khalili R., Poursha M. & Abedi K. (2020). Behavior factor and displacement amplification factor for the seismic design of single-layer barrel vaults. Journal of Constructional of Steel Research, 169, 105987. https://doi.org/10.1016/J.JCSR.2020.105987

[14] Loads M. D. O. (2017). Minimum design loads and associated criteria for buildings and other structures. American Society of Civil Engineers.

[15] ANSI/AISC 360-16 (2016). Specification for Structural Steel Buildings. American Institute of Steel Construction.

[16] CSI (2016). Integrated finite element analysis and design of structures basic analysis reference manual. Computers and Structures Inc.

[17] Vafaei M., Alih S. C. & Fallah A. (2020). The accuracy of the lumped plasticity model for estimating nonlinear behavior of reinforced concrete frames under gradually increasing vertical loads. Structural Concrete, 21, 65-80. https://doi.org/10.1002/suco.201800357

[18] Berry M. P., Lehman D. E. & Lowes L. N. (2008). Lumped-plasticity models for performance simulation of bridge columns. ACI Structural Journal, 105, 270–279. https://doi.org/10.14359/19786

[19] Brunesi E., Nascimbene R., Bolognini D. & Bellotti D. (2015). Experimental investigation of the cyclic response of reinforced precast concrete framed structures. PCI Journal, 60, 57–79. https://doi.org/ 10.15554/pcij.03012015.57.79

[20] Vafaei M., Alih S. C., Abdul Rahman Q. (2016). Drift demands of low-ductile moment resistance frames (MRF) under far field earthquake excitations. Journal Teknologi, 78, 82–92. https://doi.org/10.11113/jt.v78.5076

[21] Vafaei M., Bin Adnan A. & Rahman A. B. A. (2014). Seismic performance evaluation of an airport traffic control tower through linear and nonlinear analysis. Structure and Infrastructure Engineering, 10, 963–975. https://doi.org/10.1080/15732479.2013.774030

[22] ASCE/SEI 41-17 (2017). Seismic evaluation and retrofit of existing buildings. American Society of Civil Engineering.

[23] FEMA 356 (2000). Prestandard and commentary for the seismic rehabilitation of buildings. American Society of Civil Engineering.

[24] Structural Engineers Association of California (2008). A brief guide to seismic design factors. Structure Magazine, pp. 30–32.