Controlling the Torsional Response of Asymmetric Low-rise Special Truss Moment Frame Structures through Truss Element Modification

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ABSTRACT

Special truss moment frame (STMF) is a seismic force-resisting system which dissipates the energy of an earthquake in specific predefined regions of the truss member called special segments. Past studies have shown that the STMF system is suitable for seismic regions. However, there is a need to evaluate and improve the seismic performance of asymmetric STMF buildings. This paper presents the arrangement of mass, stiffness and strength centers for improving the overall seismic behavior of asymmetric STMF buildings and controlling the undesired torsional responses of such structures without using any supplemental devices. The sensitivity of the seismic performance of the models to modifying chord, column and diagonal members is studied and the effect of a proper stiffness and strength distribution is presented. According to the results, modifying chord elements has the most effect on the overall strength capacity of the frames. Nonlinear pushover and time-history analyses were adopted to evaluate the seismic performance of low-rise asymmetric and modified asymmetric STMF buildings. The results showed that the performance of the modified model is considerably improved in terms of the deformations of the special segments, drifts of both stiff and flexible edges and torsional indices.

KEYWORDS: Special truss moment frame, Asymmetric buildings, Non-linear time-history analysis, Low-rise buildings.

INTRODUCTION

Special truss moment frames (STMFs) can be used for large spans by utilizing truss girders to provide high lateral stiffness and strength. The primary versions of truss moment frames cannot satisfy the adequate ductility required for seismic regions. As a result, they typically fail under inelastic deformations, which is also due to buckling of their members. On the other hand, special truss moment frames perform well in terms of ductility and can be an economical alternative for the conventional steel moment frames in seismic regions. One distinctive feature of STMFs is that there is a special segment (SS) in the middle of the truss girder that dissipates seismic force through flexural yielding of its members, while the other members outside the SS are designed to remain elastic. As a result of the high degree of redundancy, this system is a very rational choice for seismic regions. Besides, utilities and mechanical ducts can go through the open web of the trusses, which is so pleasant from an architectural point of view.

Goel and Itani (1994) introduced a novel truss moment frame system termed STMF in which a designed special segment dissipates seismic forces. They also proposed a validated design concept for the system. Then, Basha and Goel (1995) experimentally studied STMFs and proposed seismic design principles

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for this system. In addition, they presented an alternative configuration of the special truss segment in special truss moment frames. Chao and Goel (2008) proposed a performance-based plastic design (PBPD) procedure and a new seismic design lateral force distribution for special truss moment frames. Pekcan et al. (2009) proposed a damage-avoidance approach with energy-dissipating devices and evaluated an STMF with a buckling-restrained brace in the special segment. Ölmez and Topkaya (2011) studied special truss moment frames with vierendeel openings to address some of their design issues. They revealed that the expected shear-strength formulation presented in the AISC seismic provisions for structural steel buildings is overly conservative and developed the formula. Pekcan et al. (2014) proposed an innovative configuration and design methodology for STMFs to achieve predictable seismic response and mitigate damage to structural elements with efficient utilization of supplemental devices. Kim and Park (2014) studied the progressive collapse-resisting capacity of special truss moment frames with various span lengths, numbers of stories and lengths of special segments based on arbitrary column removal scenario. They finally developed a design procedure based on energy-balance concept to prevent the progressive collapse of such structures. Heidari and Gharehbaghi (2015) introduced a new configuration of special truss moment frame systems including energy-dissipation devices in order to improve their seismic performance. They used buckling-resistant braces located at the side of beam-column connections as the top and bottom members of truss-girders, which resulted in decreasing lateral displacement, inter-story drift ratio, overturning moment and shearing forces of stories. More recently, Kim et al. (2016) compared the seismic behavior of special truss moment frames with conventional moment frames and proposed a seismic rehabilitation approach for existing special truss moment frames using viscous dampers. At the same time, in 2016, Gade and Sahoo assessed the collapse behavior of two 9-story structures which consisted of special truss moment frames which were designed in accordance with ACSE7-10 and the performance-based plastic design method. Ntina et al. (2017) developed a simplified structural model of an STMF, with shape memory alloy bars as dissipation devices in the special segment. In 2019, Jiansinlapadamrong et al. assessed the seismic behavior of an STMF with a bay length longer than 20 meters, which is not allowed in current design codes. Besides, they proposed a design procedure for long-span STMFs using nonlinear pushover analysis and introduced some methods for enhancing the seismic behavior of STMFs under such conditions. Kumar and Sahoo (2021) studied the fragility curves of four 3-,6-,9- and 15-story STMFs having ductile special segments and explored the influence of vertical members in the Vierendeel panels of the special segments. Furthermore, they proposed design drift limits for different performance levels of STMFs.

An asymmetric building (AB) has an uneven distribution of mass and stiffness across the dimensions of the plan, which leads to both torsional and translational responses throughout a seismic excitation. Under such circumstances, the side of a building close to its center of mass is called flexible edge and the other side is called stiff edge. Uneven mass distribution is inevitable on most occasions and asymmetric buildings are more vulnerable to seismic forces. Past studies have shown that STMF systems are suitable for seismic regions. According to the literature, supplemental devices can be adopted as efficient means of controlling the response of buildings during an earthquake (Mansoori et al., 2011; Kim et al., 2016; Ntina et al., 2017; Bayat et al., 2018). Numerical tools have proved to be effective in evaluating the seismic performance of buildings, especially when three-dimensional large-scale models are to be studied (Bayat et al., 2018; Noori and Varaee, 2022; El-Naqeeb et al., 2022). Nevertheless, using such devices is not a reasonable option for some cases, such as low-rise residential buildings, as they cannot be economically justified in such construction projects. An alternative and reasonable approach that most structural designers adopt for such buildings with asymmetry is arranging the stiffness, strength and mass centers of a building to improve the seismic performance of an asymmetric building without the need for installing any supplemental devices. However, there is a lack of studies on strength and stiffness distribution is asymmetric STMF buildings in order to find a proper arrangement of centers for improving the seismic performance of these structures. Therefore, in this paper, the seismic performance of 4-story buildings that consisted of special truss moment frames with 20%

uneven mass distribution is compared to that of symmetric benchmark (BM) structures. Furthermore, in order to improve the overall seismic behavior of the asymmetric buildings and control undesired torsional responses of such structures without using any supplemental devices, different scenarios of modifying chord, column and diagonal members are studied and the sensitivity of the performance of the structures to proper stiffness and strength distribution is investigated. Proper center arrangements are introduced and non-linear pushover and time-history analyses were adopted to evaluate the seismic performance of the buildings. The results showed that the performance of the modified models was considerably improved in terms of the deformations of the special segments, drifts of both stiff and flexible edges, torsional indices and even the dissipated energy was slightly improved during the earthquakes in question.

Review of STMF Design

The studied buildings were designed according to AISC 360-10 (2010) using the LRFD method and subsequently, special truss moment frame requirements specified in AISC 341-10 (2010) were controlled. In these types of frames, span length and depth of trusses are not allowed to exceed 20 meters and 1.8 meters, respectively. The special segment of the truss should be designed in a way to provide inelastic-deformation capacity and consequently dissipate seismic forces, while columns and other parts of the truss (outside the special segment) should remain elastic. The length of the SS should be from 0.1 to 0.5 of the total length of the span and the length-to-depth ratio of the SS should be in the range between 0.67 and 1.5, according to AISC 360-10 (2010). Panels within a special segment shall either be all Vierendeel panels or all X-braced panels, where in this study, the former; a rectangular opening with fixed joints capable of transferring and resisting bending moments, is used. It should be noted that neither a combination of the mentioned types nor the use of other truss diagonal configurations is permitted. As per AISC 360-10 (2010), the non-yielding members outside the special segment should be designed based on the expected vertical shear strength (V_{ne}) of the special segments:

$$V_{ne} = \frac{3.75 R_y M_{nc}}{L_s} + 0.036EI\frac{L}{{L_S}^3} + R_y\,(P_{nt} + 0.3P_{nc})\sin\propto \qquad (1)$$

where E = modulus of elasticity of members in SS, I= moment of inertia of the members in SS, L= total length of the truss, L_s= total length of the SS, M_{nc}= nominal flexural strength of members of special segment, R_y = yield-strength modification factor, P_{nc}= nominal compressive strength of the diagonal members in the SS, P_{nt}= nominal tensile strength of the diagonal members in the SS and α = the angle between diagonal members and the horizontal axis.

Studied Buildings

Four-story code-designed buildings with three bays in both directions were considered as the benchmark study buildings in this study. The studied buildings are located in Tehran, Iran. The width of the bay in each direction is 9 m and the height of each story is 4.2 m. The height of the trusses is 1.8 m and the length of the special segments is 1.2 m in all of the models. Yield strength and ultimate strength were considered as those of ST37 and the designed sections are presented in Fig.1 and summarized in Table 1. From the south to the north of the plan, frames are numbered from 1 to 4 and from the west to the east of the plan, frames are named A, B, C and D, respectively. BOX and UNP sections in Table 1 refer to rectangular hollow section and standardized channel section, respectively. In Table 1, the first two numbers following BOX symbols represent their cross-sectional dimensions and the last number represents the thickness in millimeters. Also, in this table, the UNP sections adhered to DIN 1026-1. The structures were analyzed and designed under 6750 N/m^2 dead load, 5000 N/m^2 live load and seismic load stipulated in Iranian Standard 2800 seismic code (2014) and ASCE 7-10 (2010), where design basis acceleration (A), importance factor (I), site class and response modification factor (R) were considered 0.35, 1, soil type-2 and 7, respectively. In the models with uneven mass distribution, eccentricities in all floors were considered the same.

In this study, in order to consider the vertical component of all the earthquake records, instead of a lumped mass, the corresponding masses were distributed in each story. In the symmetric building, the center of mass and the center of stiffness are coincident with the center of the plan. When 20% uneven mass distribution was imposed on the AB model, its center of mass, CM (1), shifted away in X-direction, where it is 0.7L away from the left side of the plan (Fig. 2). So as to improve the overall seismic behavior of the asymmetric building, control undesired torsional responses of these models without using any supplemental devices and minimize the difference between the responses of the asymmetric building and the benchmark, different sections were used for the members of the frames in both stiff and flexible edges in a way that the overall stiffness and strength capacity of the models remained the same as those for the symmetric benchmark model. As a result, the center of strength (CS (2)) and the center of stiffness (CR (2)) of the MAB model moved closer to the center of mass, where CS (2) and CR (2) are located 0.65L and 0.7L away from the left side of the plan, respectively (see Fig. 2). The impact of the proposed center arrangements on the overall seismic performance of the buildings is presented in the following sections. It turned out that strengthening chord elements of the flexible edge had the most impact on improving the overall resistance capacity of the frames as well as reducing torsional responses of the asymmetric models. At the same time, using stronger sections for columns improved the capacity of the frames by as much as 10%, while the impact of altering the sections used for diagonal members was negligible. The pushover curves presented in Fig. 3 show the impact of altering each of the aforementioned elements on the overall capacity of the flexible edge. The finalized sections of the modified asymmetric building (MAB) models are summarized in Table 1.

Table 1. Section properties of the models

	Column			Truss		
Floor	**Frame**	**Interior**	**Exterior**	**Frame**	**Chord-vertical 2**	**Vertical 1-Diagonal**
BM and AB						
1-2	A-D	BOX 45×45×3	BOX 30×30×2	A-D-1-4	2 UNP 14	2 UNP 8
1-2	B-C	BOX 50×50×3	BOX 45×45×3	B-C-2-3	2 UNP 24	2 UNP 14
3-4	A-D	BOX 35×35×2	BOX 25×25×2	A-D-1-4	2 UNP 12	2 UNP 6.5
3-4	B-C	BOX 40×40×3	BOX 35×35×2	B-C-2-3	2 UNP 20	2 UNP12
	Column			**Truss**		
Floor	**Frame**	**Interior**	**Exterior**	**Frame**	**Chord-vertical 2**	**Vertical 1-Diagonal**
MAB						
1-2	C-D	BOX 50×50×3	BOX 45×45×3	C-D-2-3	2 UNP 24	2 UNP 14
1-2	A-B	BOX 45×45×3	BOX 30×30×2	A-B-1-4	2 UNP 14	2 UNP 8
3-4	C-D	BOX 40×40×3	BOX 35×35×2	C-D-2-3	2 UNP 20	2 UNP 12
3-4	A-B	BOX 35×35×2	BOX 25×25×2	A-B-1-4	2 UNP 12	2 UNP 6.5

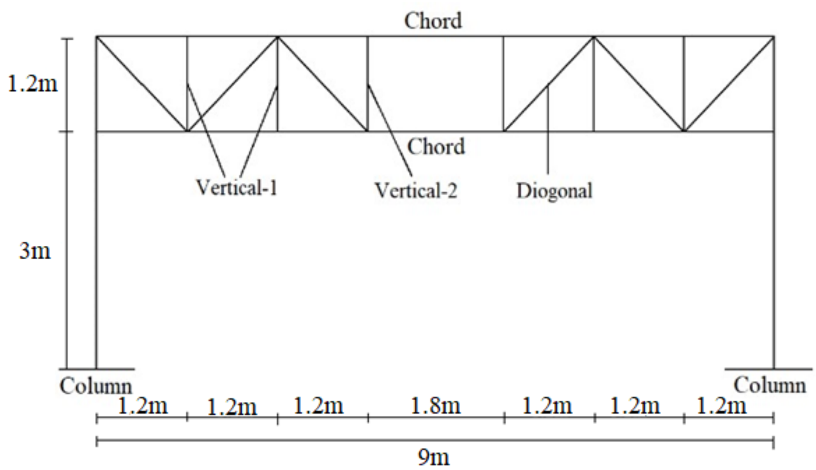


Figure (1): Name tags of the truss girders in question

Table 2. Ground motion characteristics of time histories

	Earthquake	Year	Magnitude	PGA x(g)	PGA y(g)	PGA z(g)	Site	Distance
1	Gazli, USSR	1976	6.8m	0.7	0.86	1.69	Karakyr	12.8km
2	Nahanni,canada	1985	6.8m	1.1	1.2	2.28	Site 1	6.8km
3	Loma Prieta	1989	6.9m	0.45	0.5	0.86	BRAN	9km
4	Loma Prieta	1989	6.9m	0.64	0.48	0.45	Corralitos	7.2km
5	Cape Mendocino	1992	7.0m	1.49	1.03	0.73	Cape Mendocino	10.4km
6	Northridge-01	1994	6.7m	0.75	0.93	0.31	LA - Sepulveda VA	8.5km
7	Chi-Chi, Taiwan	1999	7.6m	0.49	0.31	0.23	TCU067	28.8km
8	Landers	1992	7.3m	0.72	0.78	0.82	Lucerne	44km
9	Loma Prieta	1989	6.9m	0.51	0.32	0.39	Saratoga - Aloha	27.2km
10	Cape Mendocino	1992	7.0m	0.59	0.66	0.16	Petrolia	4.5km

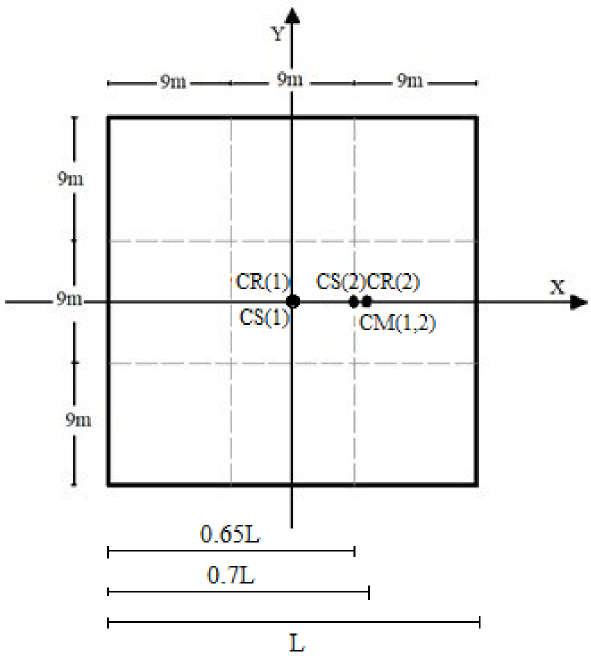


Figure (2): Centers' arrangement of the asymmetric building (AB) and the modified asymmetric building (MAB)

A set of ten near-field ground motions was selected in accordance with FEMA-P695 (2009) as per PEER-NGA database (see Table 2). These earthquake records were scaled according to the proposed procedure in ASCE 7-10 (2010) using the DBE response spectrum; the peak ground accelerations (PGAs) of Table 2 were multiplied by 0.84 (the computed scale factor for the model). It is noteworthy that the vertical components of all the earthquake records were considered in the analyses.

Analytical Study

In this study, the accuracy of the generated models was validated with the results of the research carried out by Kim et al. (2016). They studied a 3-story STMF structure with three 9 m bays. The first story height of the structure is 5.2 m, whereas a constant story height of 4.2 m is considered for the rest of the floor levels. The frame structure was modeled using the information about the member sizes of the STMF model structure presented in Table 1 of the paper. Non-linear static pushover analysis was performed using the lateral loads' distribution proportional to the fundamental mode shape of the model. The computed fundamental period of the model structure is 0.722 sec which is equal to the value reported in the paper in question. As shown in Fig. 4, the pushover curves of the paper and the generated model in this study are in great agreement in terms of maximum base shear and strength loss. Besides, the plastic hinges first formed at the first-and the second-story special segments and when the strength reached the maximum point, plastic hinges formed at the third story special segment which is the same as the scenario described in the aforementioned paper. Figure 5 shows plastic hinge formation of the paper, while Fig. 6 shows plastic hinge formation of the generated model of the present study.

Herein, three buildings including a 4-story symmetric building as the BM, a 4-story AB and a MAB were modeled and non-linear pushover and dynamic analyses were performed on each of them. For both static analysis and dynamic analysis, concentrated plastic hinges were assigned to the models and the simulation parameters (a, b and c) and acceptance criteria (immediate occupancy (IO), life safety (LS) and collapse prevention (CP)) were defined in accordance with ASCE 41-13 (2013). Beam, column and diagonal elements with elasto-plastic behavior were employed to define the non-linear behavior of the members. Table 3 shows the dynamic parameters of the model structures. It can be observed that the natural period of the modified STMF structure is significantly lower than that of the AB and indeed so close to that of the BM. The effective mass of the dominant mode (second mode) of MAB model in Y-direction is bigger than the effective mass of the dominant mode (first mode) of AB model and is so close to the effective mass of the dominant mode (third mode) of the BM model. This indicates that the overall seismic behavior of the MAB model is similar to that of the BM model.

Table 3. Dynamic parameters of the models

Mode Number	Type	Mode Period (s)	Effective Mass (ux)	Effective Mass (uy)
Benchmark (BM)				
1	torsional	0.8236	0	0
2	transitional	0.776	0.7913	0
3	transitional	0.776	0	0.776
Asymmetric (AB)				
1	torsional	0.9587	0	0.5169
2	transitional	0.776	0.7914	0
3	torsional	0.6037	0	0.2747
Modified (MAB)				
1	transitional	0.7916	0.7855	0
2	transitional	0.7798	0	0.746
3	torsional	0.6914	0	0.04698

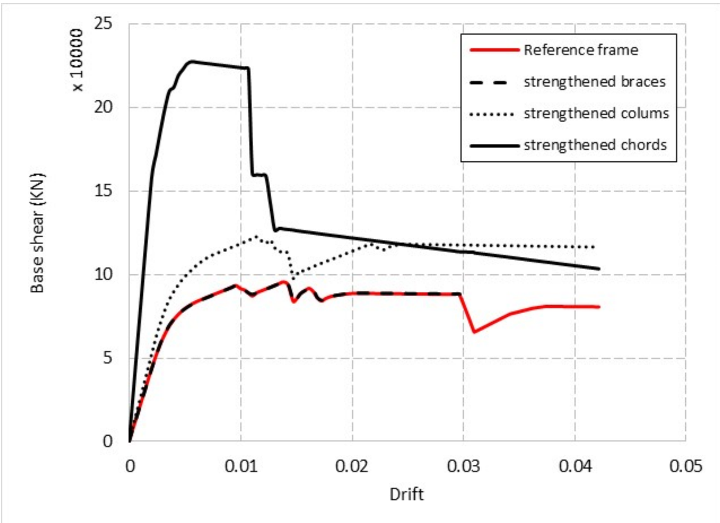


Figure (3): Pushover curves of the frames with strengthened braces, columns and chords

Non-linear Static Analysis

Since the sections used in the asymmetric model were the same as those for the benchmark, a non-linearstatic analysis using the first-mode load pattern as suggested in ASCE 41-13 (2013) was performed on the 4-story symmetric model (BM) and the results are depicted in Figs. 7 and 8. It can be observed from the pushover curve of the model (Fig. 7) and the visual results of the analyzed model (Fig. 8) that none of the members of the building exceeded the life safety acceptance level at the target displacement which was 11.02 cm. This can be interpreted from the color of the elements in Fig. 8, where none of the elements turned red during the analysis. As it was expected, only link elements in the special segments of the lower floors experienced non-linear deformations, the elements in blue or green.

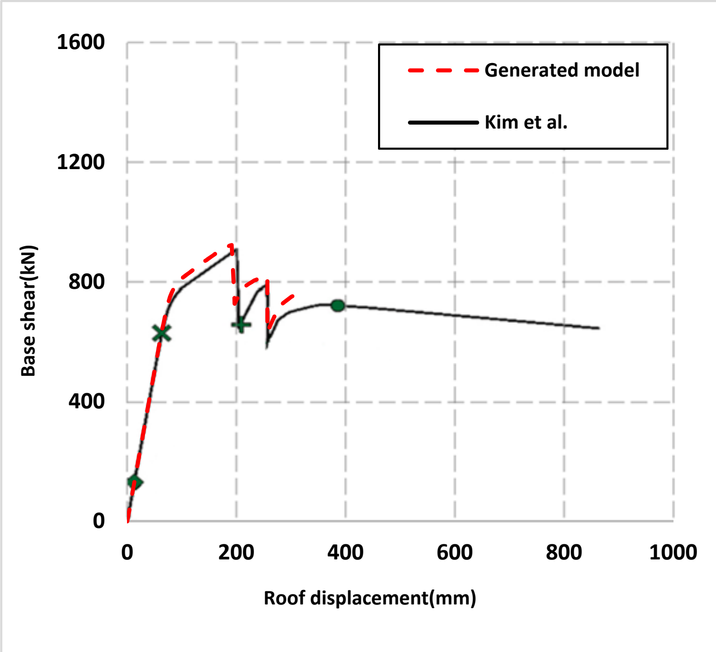


Figure (4): Pushover curve of the verified model *vs.* Kim et al.'s paper (2016)

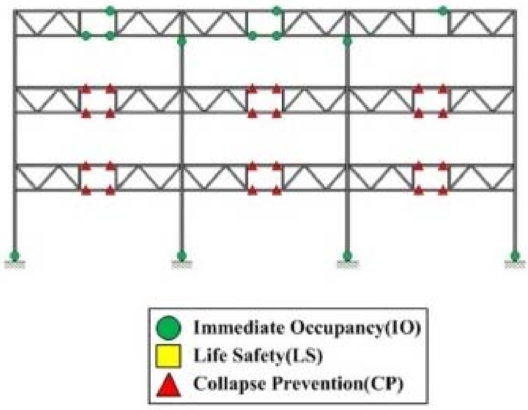


Figure (5): Plastic hinge formation in Kim et al.'s paper (2016)

Non-linear Time-history Dynamic Analysis

Non-linear dynamic analyses were performed on all models subjected to 10 different earthquake records considering the vertical components of the records. The results of acceptance criteria, displacements, torsional indices and dissipated energy of the models are presented in this section.

Acceptance Criteria

Evaluating the models in terms of acceptance criteria showed that under the earthquakes, some beams (link elements) of the modeled buildings exceeded life safety (LS) acceptance level. The BM model (symmetric) performed better under all the earthquakes and the number of the members exceeding this acceptance level was considerably fewer compared to the AB model. As an example, Fig. 9 shows the results of the analysis of the MAB model under capemend-pet earthquake in LS. In the BM model, none of the element exceeded LS, whereas in the AB model, many beams exceeded this acceptance level. It is noteworthy that although in the MAB model few beams exceeded LS, the performance of this model is notably improved, compared to the AB model. This can be attributed to the enhanced total seismic behavior of the MAB model.

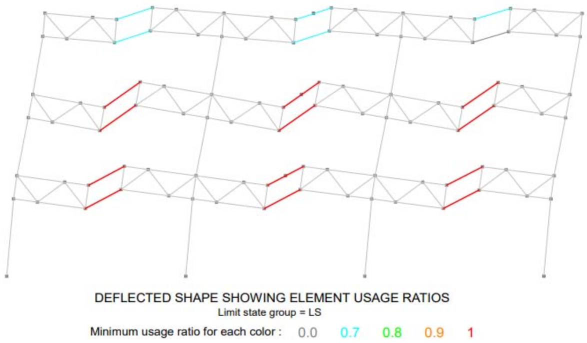


Figure (6): Plastic hinge formation of the generated model based on Kim et al.'s model

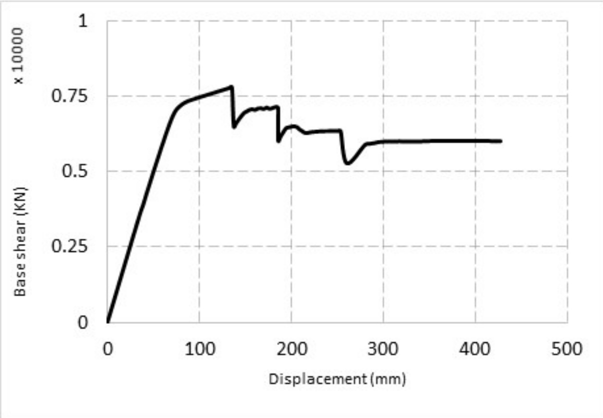


Figure (7): Pushover curve of the 4-story building

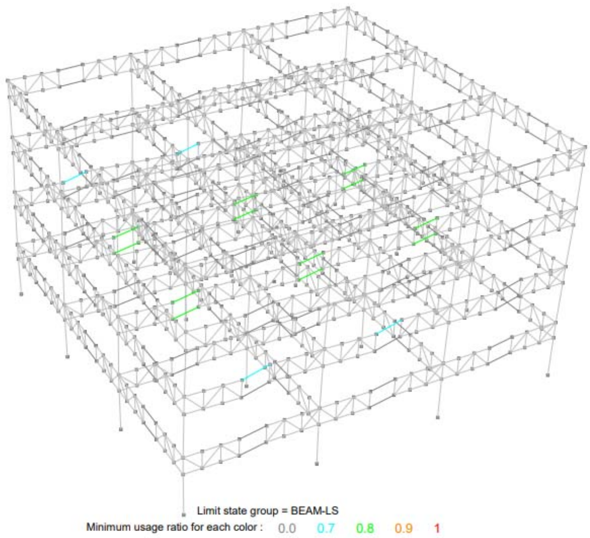


Figure (8): LS performance level of the generated model at target displacement in non-linear static analysis

The flexible edge of an asymmetric building is the most critical part as a result of higher earthquake-induced deformations. In other words, the members of flexible edge are more vulnerable. Therefore, a critical example of the flexible edge of the models under capemend-pet earthquake is depicted in Fig. 10. The link elements of the first floor and the second floor in the AB model exceeded LS, while there was no such an element in the BM model. The number of link elements of the MAB model passing LS was considerably less than in the AB model.

It can be derived that although the code designed building in this study showed a desirable behavior during non-linear static analysis and none of its members exceeded LS acceptance level before the target displacement (Fig. 8), in the non-linear dynamic analysis, which is more realistic, in some occasions the members in the special segments passed this acceptance level.

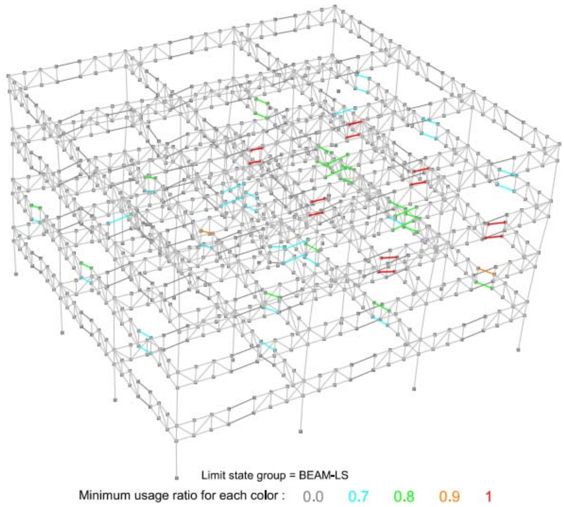


Figure (9): LS usage ratio of the modified asymmetric building (MAB) under capemend-pet earthquake

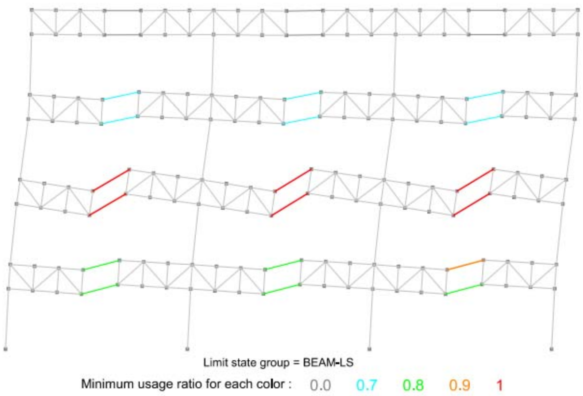
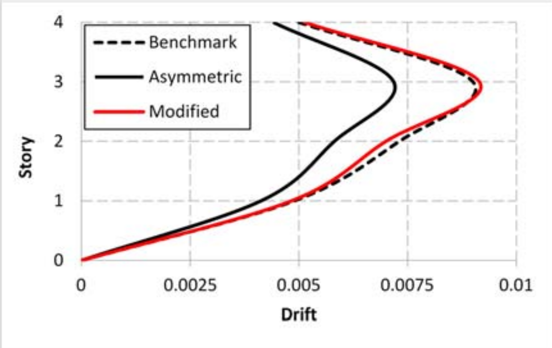


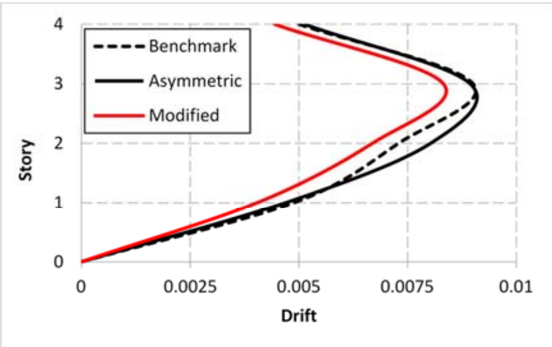
Figure (10): LS usage ratio of the flexible edge of the modified asymmetric building (MAB) under capemend-pet earthquake

Relative Displacement

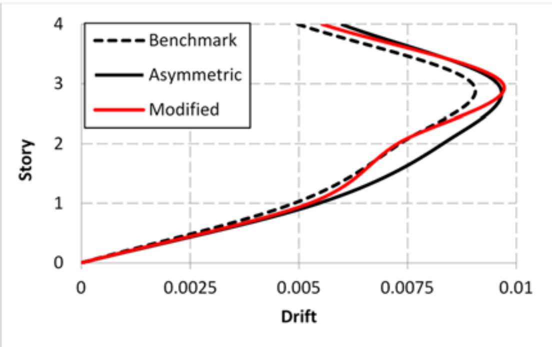
The averages of the maximum interstory drift of the models in each direction under the earthquakes in question are compared with each other. The drift of the MAB model is quite close to that of the BM model, where in Fig. 11 (a), these patterns almost overlap. Meanwhile, there are notable differences between the BM model and the AB model with a maximum difference of 27% in their third floors. Also, Figs. 11 (b) and (c) depict that the drift of the modified model in both flexible and stiff edges is adequately close to the benchmark model, compared to the asymmetric building. In sum, regarding the mentioned figures, drift which is of great importance for preserving the performance of the lateral load-carrying elements was considerably controlled.



(a)



(b)



(c)

Figure (11): Maximum average drift of the a) centroid, b) stiff edge and c) flexible edge of the AB, MAB and BM models

Torsional Index

Torsional index (TI) was computed for each story of the mass-asymmetric and modified mass-asymmetric buildings as follows:

$$TI_i = \frac{D_{FEi} - D_{SEi}}{D_{Ci}} \quad (2)$$

where D_{FEi}, D_{SEi} and D_{Ci} are the drifts of the flexible edge, stiff edge and centroid of the ith story of the models, respectively. As indicated in Fig. 12, the computed TIs for the modified asymmetric building are smaller than the values of the asymmetric buildings. Compared with the asymmetric models, the maximum TI of the MAB model reduced by 27.5%.

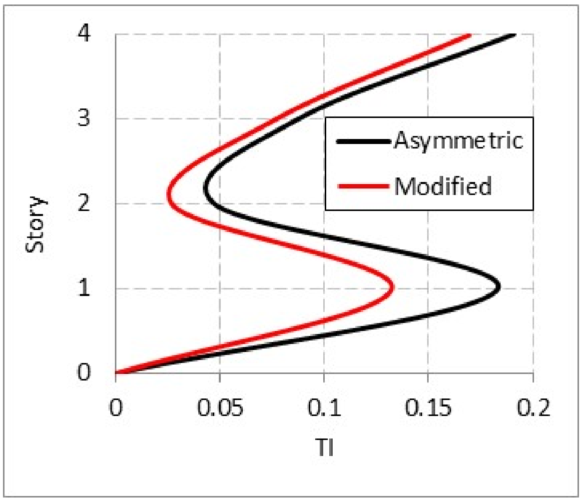


Figure (12): Computed torsional index for the 4-story AB and MAB models

Energy Dissipation

The amounts of the average dissipated energy during the analyses under the ten earthquakes are shown in Fig. 13. In all structures, roughly over 70% of the dissipated energy was through inelastic deformation of the members in the special segment of the trusses, while proportions of the modal damping, strain and kinetic energy were relatively smaller. The results show that there are minimal differences between the amounts of total dissipated energy in the BM model, the AB model and the MAB model. Nevertheless, the results of the MAB and BM models are almost equal. This could be attributed to the fact that its overall seismic behavior is close to that of the BM model and consequently, the elements that enter the non-linear domain in this model are distributed more uniformly in the plan, compared to the AB model. It can be observed that the differences among the amounts of the total dissipated energy of the models arise from the differences among the proportions of the modal damping energy.

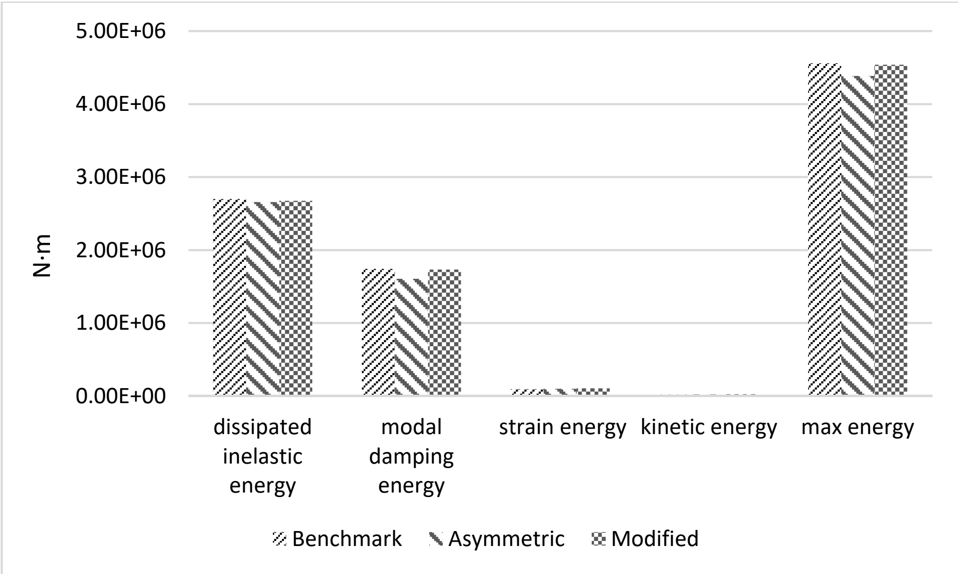


Figure (13): Average dissipated energy of the 4-story buildings

CONCLUSIONS

There is a lack of studies that cover the comprehensive assessment of strength and stiffness distributions in asymmetric STMF buildings. Therefore, in this paper, the seismic performance of low-rise mass asymmetric STMF buildings is compared to that of the symmetric benchmark building structures. Furthermore, in order to improve the overall seismic behavior of asymmetric buildings and control undesired torsional responses of such structures without using any supplemental devices, different scenarios of modifying chord, column and diagonal members are studied and the sensitivity of the performance of the structures to proper stiffness and strength distributions is investigated. For this purpose, non-linear pushover and time-history analyses were adopted.

The major conclusions of this study are as follows:

1- It turned out that strengthening the chord elements of the flexible edge has the most impact on improving the overall resistance capacity of the frames as well as reducing the torsional responses of the asymmetric models, whereas the impact of modifying columns is less pronounced and the impact of altering diagonal members is negligible.
2- The mass, stiffness and strength center arrangements of this study can be used by designers for improving the overall seismic behavior of asymmetric STMF buildings and controlling the undesired torsional responses of such structures without using any supplemental devices.
3- The natural period of the modified STMF structure is significantly lower than that of the AB model and so close to that of the BM model. Studying the effective mass of the models indicates that the overall seismic behavior of the MAB model is similar to that of the BM model.
4- Non-linear static analysis revealed that all of the buildings fit LS acceptance level, while the results of non-linear dynamic analysis indicated that under some of the earthquakes in question, some members in the special segments of the assessed buildings exceeded the LS acceptance level. This could be interpreted as that the current design procedure for STMFs is not adequately conservative.
5- Regarding the results, the proper distributions of stiffness and strength through the approach presented in this paper improved the performance of the asymmetric-building models in terms of LS acceptance level. The results of the modified asymmetric buildings are considerably close to the results of the benchmark models.
6- The average of the maximum drift of the models shows that the drift of the centroid, stiff and flexible

edge of the MAB model significantly decreased. The maximum drift of the centroid of the 4-story MAB model is 27% smaller than that of the 4-story AB model.

7- Torsional indices are computed for the MAB and AB models. It can be observed that the TI of the former is 27.5% smaller than that of the AB model.
8- Although the amounts of the dissipated energy during the analyses by the models are close, the results of the MAB and BM models are almost equal.

The results of this study demonstrate that suitable arrangement of stiffness, strength and mass centers for asymmetric low-rise STMF buildings through chord-element modification reduces the undesired torsional response of such structures. The impact of the same approach on mid-rise asymmetric STMF buildings is worth being evaluated.

Declaration

On behalf of all authors, the corresponding author states that there is no conflict of interest.

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