Recent Trends in Civil Engineering and Built Environment Vol. 3 No. 1 (2022) 1320-1331

RTCEBE

Homepage: http://publisher.uthm.edu.my/periodicals/index.php/rtcebe
e-ISSN :2773-5184

Effects of Dynamic Characteristics on 5-storey Steel Tower with Different Orientation of Bracing Members

Jin Qian Yew[1], Ahmad Fahmy Kamarudin[2]*

[1]Faculty of Civil Engineering and Built Environment,
Universiti Tun Hussein Onn
Malaysia, 86400 Parit Raja, Johor, MALAYSIA

[2]Senior Lecturer at Faculty of Civil Engineering and Built Environment,
Universiti Tun Hussein Onn Malaysia, 86400 Parit Raja, Johor, Malaysia

*Corresponding Author Designation

DOI: https://doi.org/10.30880/rtcebe.2022.03.01.149
Received 4 July 2021; Accepted 13 December 2021; Available online 15 July 2022

Abstract: This paper was determined the effects of dynamic characteristics on the 5-storey steel tower with different orientations of bracing members by using the modal analysis through STAAD Pro software. There are two types of bracing systems used in the structure which is chevron bracing systems and single diagonal bracing systems. The dynamic characteristics of the 5-storey steel tower were investigated in terms of peak frequency, mode shape, maximum deflection amplitude and relative floor displacement amplitude. This study was focused on verified the comparison of the modal analysis result of the bare frame and the previous laboratory testing result. Three modes of peak frequency were produced from the modal analysis. The first three modes of result for the bare frame is 5.55Hz, 5.55Hz and 7.23Hz with the percentage difference less than 6% from the previous laboratory testing. The relationship of peak frequency, mode shape, maximum deflection amplitude and relative floor displacement were developed between the dynamic characteristics of the steel tower. The peak frequency of bracing frames occurred higher than the bare frame. Each bracing frame was presented with different types of mode shape, maximum deflection amplitude and relative floor displacement amplitude. As result, the bracing systems had improved in terms of the dynamic characteristics of the 5-storey steel tower. The chevron bracing systems were presented the more stable results in dynamic characteristics of the 5-storey steel tower.

Keywords: Dynamic Characteristic, 5-storey Steel Tower, Bracing Systems, STAAD Pro, Modal Analysis

**Corresponding author: fahmy@uthm.edu.my*

1. Introduction

Steel structures is widely used for residential building construction in the world [1]. Application of bracing systems in structure is effective to enhance the existing strength which is more practical and economic. Bracing systems commonly used to resist lateral load such as earthquake, wind and etc [2]. Bracing systems is considered as an effective system in enhancing the stiffness and strength of steel frames [3]. The bracing system effectively reduces the lateral displacement and drift of structures [2]. According to [1], bracing frames improve the seismic resistance of frames. The braced structure improves the strength and stiffness of the original (unbraced) structure, as well as the natural frequency and reduce the lateral drift [4]. Dynamic characteristic of a building can be evaluated through the estimation of modal analysis such as natural frequencies and mode shapes. Every structure has its own natural frequency which the structure tends to vibrate if it is subjected to a disturbance [5]. Besides that, additional vibration anomaly will lead to a change in mode shape and natural frequency that can cause structural failure and damage [6].

In this few years, the experimental and numerical studies on the effect of the installation bracing systems on the dynamic characteristic of steel structures has getting increase. According to [7], the effects of brace configurations on the dynamic properties of frames shows increasing of steel frame stiffness. Modal analysis can become a widespread means to investigate the modes of vibration of structure since every structure vibrates with high amplitude of vibration at its resonant frequency. For improving its strength and reliability at the design stage, the modal parameters such as resonant frequency, mode shape and damping characteristics of the structure must be known at its varying operating conditions [8]. Therefore, an analysis of modal analysis required to determine the dynamic characteristic of structural. In this study of 5-storey steel tower used different types of bracing orientation by using STAAD Pro software for investigation. This study involved 2 different braced steel frames with each 4 configurations as the parametric studies for the mode shapes, natural frequencies, maximum deflection amplitude and relative floor displacement amplitude of the braced steel structures. The output of dynamic characteristics and lateral displacement amplitude were produced, and relationship developed to observe the dynamic behaviour of the structures.

2. Methods

2.1 Phase 1: Pre-modelling

The steel tower was made up of mild steel with grade S275. The beams and columns were designed with 77mm×77mm×1000mm with 3mm thickness of square hollow section. Figure 2.2 shows the total height of the 5-storey steel tower which is 5m and each storey is 1m height.

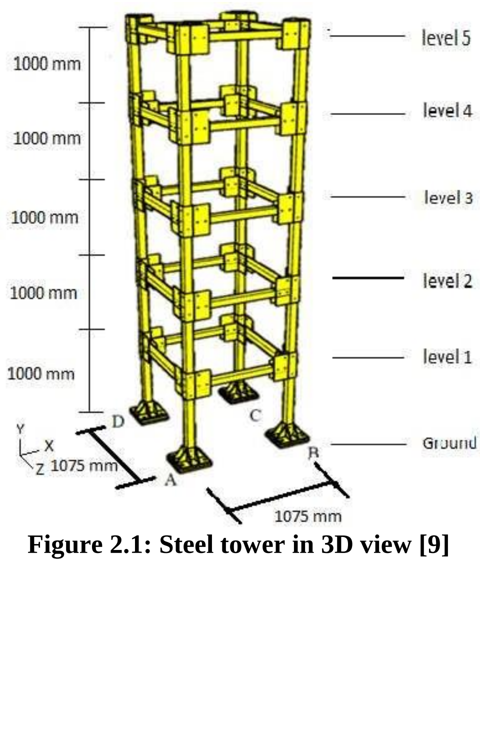


Figure 2.1: Steel tower in 3D view [9]

2.2 Phase 2: Modelling

Based on the dimensions of existing 5-storey steel tower from JRC lab in UTHM as Table 3.1, the steel tower modal was modelled by using STAAD Pro software. The partial moment release as Table 2.2 was applied on the modal steel tower in STAAD Pro editor to represent the partial fixity of all connections between elements. Figure 2.3 shows the partial moment at the start and the end of every member assigned to the beams and columns.

Table 2.1: Detailing needed for modelling [10]

No.	Element	
1	Standard	BS EN 1993-1-8:2005 fy: 275×10^{-3}kN/mm^2 λ: 300×10^{-3} E: 205kN/mm^2
2	Beam and column	Square hollow sections (SHS) Steel grade S275 Length = 1000 mm B x H = 77 mm × 77mm Thickness 3 mm
3	Connection	Steel grade S275 High tension bolt grade 8.8 with diameter 10mm and 25mm
4	Support	Fixed at base floor
5	Loading	Self-weight

Table 2.2: Percentage of partial moment release assigned at member

Member	Percentage of moment release
Beam	80%
Column	28%

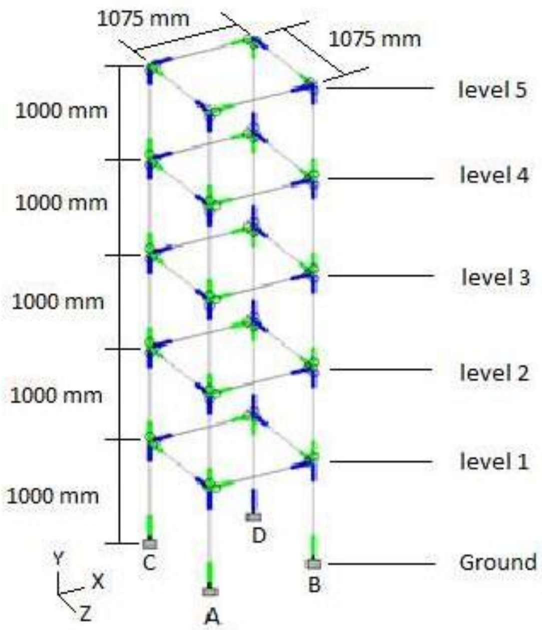
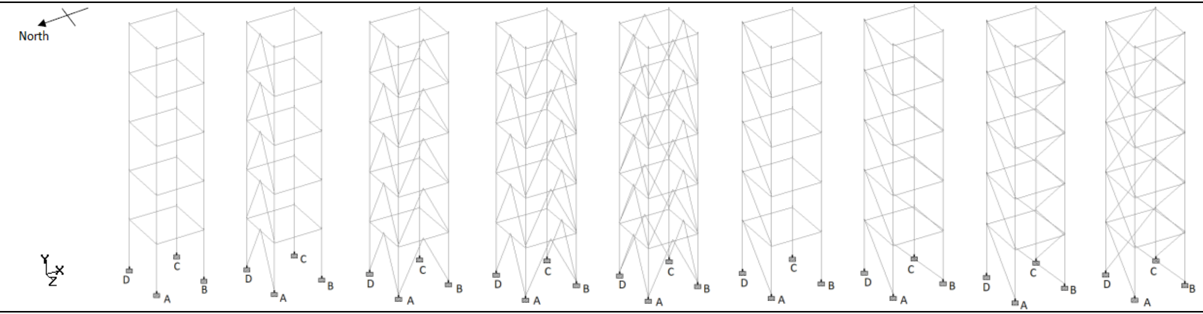


Figure 2.2: STAAD Pro steel tower model condition (green-start, blue-end) partial moment applied on beam and column

2.3 Phase 3: Result and analysis

Bare frame	Chevron bracing frames				Single diagonal bracing frames			
	Case 1	Case 2	Case 3	Case 4	Case 1	Case 2	Case 3	Case 4



The natural frequency of the steel tower was obtained from the modal analysis assigned in the STAAD Pro editor. The bare frame was analysed as first model to obtain the peak frequency value and mode shape pattern. After that, the 4 configurations of chevron bracing frame were analysed one by one and follow by 4 configurations of single diagonal bracing frame. Figure 2.4 shows the modelling of bare frame, chevron bracing frames and single diagonal bracing frames.

Figure 2.4: Model of bare frame, chevron bracing frames and single diagonal bracing frames

2.4 Phase 4: Development of relationship

A comparison of percentage differences was made between the natural frequency value of bare frame from modal analysis and previous laboratory ambient vibration testing to study the natural frequency differences between the two analysis methods. The peak frequency, mode shape, maximum deflection amplitude and relative floor displacement amplitude of bare frame, chevron bracing frames and single diagonal bracing frames were analysed through the output from STAAD Pro software.

3. Results and Discussion

3.1 Peak frequency

STAAD Pro software used for modal analysis to conduct the 5-storey steel tower of the bare frame. The result and peak frequencies of the bare frame that output from modal analysis had carried out for verification to ensure the accuracy of the model's data. the output of STAAD Pro software produced peak frequencies result in 3 dominant modes and each mode had its own peak frequency. From Table 3.1, the first three modes of peak frequencies for the bare frame were 5.55Hz, 5.55Hz and 7.23Hz. The comparison results were made between modal analysis method frequency and previous laboratory frequency conducted by [11] in Table 3.1. From Table 3.1, the frequency value of modal analysis for mode 1 and mode 2 had shown a higher value than the previous laboratory testing. According to Table 3.1, the percentage difference for mode 1, mode 2 and mode 3 was 5.92%, 1.09% and 0.00%, which shown a slight decrease in the value. The percentage difference for all modes was considered acceptable due to the comparison made by [12]. According to the study of [12], the acceptable limit of difference between computational analysis and laboratory testing was the percentage lesser than 10%.

Table 3.1: Percentage difference of previous laboratory frequency and modal analysis method frequency

Mode	1	2	3
Previous Laboratory Frequency [11] (Hz)	5.24	5.49	7.23
Modal Analysis Method Frequency (Bare frame) (Hz)	5.55	5.55	7.23
Percentage Difference (%)	5.92	1.09	0.00

The modelling of the steel tower was modelled with multiple cases of different bracing structural as shown in Table 3.2. From Table 3.2, the output of peak frequencies in mode 1 was the lowest value.

Meanwhile, the output of peak frequencies in mode 3 was the highest value. The peak frequency for all cases increased when the bracing members of the steel frame increasing. Compared to the bare frame, chevron bracing frames and single diagonal bracing frames presented a higher natural frequency with

Mode	Peak frequency (Hz)								
	Bare frame	Chevron bracing frames				Single diagonal bracing frames			
		Case 1	Case 2	Case 3	Case 4	Case 1	Case 2	Case 3	Case 4
1	5.55	4.97	5.87	5.60	6.41	5.24	7.07	11.99	24.48
2	5.55	6.41	6.11	6.86	6.41	7.08	16.00	25.60	24.48
3	7.23	8.05	8.37	8.59	8.68	18.56	22.60	28.27	41.97

the increase in bracing members. Meanwhile, single diagonal bracing frames presented the highest value of peak frequencies. According to the study of [13], chevron braced frames have the lower natural frequency than the diagonal braced frames. The highest frequency was presented in case 4 of single diagonal bracing frame. In the study of [14], the natural frequency of structure perform well in vibration was range between 0.5Hz and 25.0Hz.

Table 3.2: Frequency of bare frame, chevron bracing frames and single diagonal bracing frames

3.2 Mode shape

The mode shape of the steel tower was generated based on the first three frequencies of the structure as extracted in Figure 4.2 to Figure 4.7. From the observation from Figure 4.2 to Figure 4.7, the bare frame, chevron bracing frames and single diagonal bracing frames were shown in different mode shape for every case. From the study of [7], the different frequency of braced frame shown the different mode

1st Mode					
	Bare frame	Chevron bracing frames			
		Case 1	Case 2	Case 3	Case 4
Frequency (Hz)	5.55	4.97	5.87	5.60	6.41
Mode shape	Translation mode (East-West direction)	Translation mode (North-South direction)	Translation mode (South-West & North-East direction)	Translation mode (North-South direction)	Translation mode (South-West & North-East direction)

shape of brace frame. In Figure 4.2 to Figure 4.7, the mode shape of chevron bracing frames was more

1st Mode					
	Bare frame	Single diagonal bracing frames			
		Case 1	Case 2	Case 3	Case 4
Frequency (Hz)	5.55	5.24	7.07	11.99	24.48
Mode shape	Translation mode (East-West direction)	Translation mode (North-South direction)	Translation mode (North-South direction)	Translation mode (North-South direction)	Translation mode (North-South direction)

like the mode shape of bare frame compare to the mode shape of single diagonal bracing frames. Other than that, the mode shape of chevron bracing frames in all modes was more regularity repeat compare to the mode shape of single diagonal bracing frames in all modes. Form Figure 4.7, the 3rd mode of single diagonal bracing frames was presented the irregularity mode shapes compare to bare frame and chevron bracing frames. It was similar with the mode shape occurred in [7].

Table 3.3: Mode shape of bare frame and chevron bracing frames in 1st mode

Table 3.4: Mode shape of bare frame and single diagonal bracing frames in 1st mode

Table 3.5: Mode shape of bare frame and chevron bracing frames in 2nd mode

2nd Mode					
	Bare frame	Single diagonal bracing frames			
		Case 1	Case 2	Case 3	Case 4
Frequency (Hz)	5.55	7.08	16.00	25.60	24.48
Mode shape	Translation mode (North-South direction)	Torsional mode (Clockwise direction)	Translation mode (South-East & North-West direction)	Translation mode (East-West direction)	Translation mode (East-West direction)

Table 3.6: Mode shape of bare frame and single diagonal bracing frames in 2nd mode

3rd Mode					
	Bare frame	Chevron bracing frames			
		Case 1	Case 2	Case 3	Case 4
Frequency (Hz)	7.23	8.05	8.37	8.59	8.68
Mode shape	Torsional mode (Clockwise direction)	Torsional mode (Clockwise direction)	Torsional mode (Anti-clockwise direction)	Torsional mode (Clockwise direction)	Torsional mode (Anti-clockwise direction)

Table 3.7: Mode shape of bare frame and chevron bracing frames in 3rd mode

3rd Mode					
	Bare frame	Single diagonal bracing frames			
		Case 1	Case 2	Case 3	Case 4
Frequency (Hz)	7.23	18.56	22.60	28.27	41.97
Mode shape	Torsional mode (Clockwise direction)	Bending mode (North-South direction)	Bending mode (North-South direction)	Bending mode (North-South direction)	Bending in-plane mode

Table 3.8: Mode shape of bare frame and single diagonal bracing frames in 3rd mode

3.3 Deflection amplitude

Figure 3.1 show the deflection amplitude curves for mode 1, mode 2 and mode 3. From observation of mode 1 in Figure 3.1, the bare frame experienced in translation mode shape along the East-West direction. All columns were deflected in the same direction and resulted to an overlap deflection curve pattern for every column same as the chevron bracing frame for case 1 and case 4. But some columns of the chevron bracing frame in case 2 and case 3 were deflected with different amplitude values. As the result, some columns were split apart from other columns in the deflection curve.

The translation mode of the bare frame in mode 2 showed in the opposite direction against the translation mode for mode 1 (Table 3.3 & Table 3.5). From Figure 3.1, the deflection curve pattern of

2nd Mode					
	Bare frame	Chevron bracing frames			
		Case 1	Case 2	Case 3	Case 4
Frequency (Hz)	5.55	6.41	6.11	6.86	6.41
Mode shape	Translation mode (North-South direction)	Translation mode (East-West direction)	Translation mode (North-West & South-East direction)	Translation mode (East-West direction)	Translation mode (North-West & South-East direction)

the bare frame in mode 1 and mode 2 was deflected in the same direction and resulted in an overlap deflection curve pattern for every column. For case 2 and case 4 of the chevron bracing frame, the translation mode along the North-West and South-East direction was showed. Some of the columns in case 2 and case 4 of the chevron bracing frame presented another opposite deflected direction in the result. Case 1 and case 3 of chevron bracing frame translation along East-West direction, but some columns in case 1 was deflected in different amplitude values and split apart from other columns in deflection curve. All columns in case 3 of the chevron bracing frame were deflected in the same direction and resulted to an overlap deflection curve pattern for every column.

From Figure 3.1, the mode 3 of the bare frame and chevron bracing frames were presented in the torsional mode. But case 2 and case 4 of the chevron bracing frame were rotated in opposite directions with other cases. In Figure 3.1, the bare frame and the chevron bracing frames were deflected in the same deflection direction. Some of the columns in case 1 and case 3 of the chevron bracing frame deflected with different amplitude values and split apart from other columns in the deflection curve.

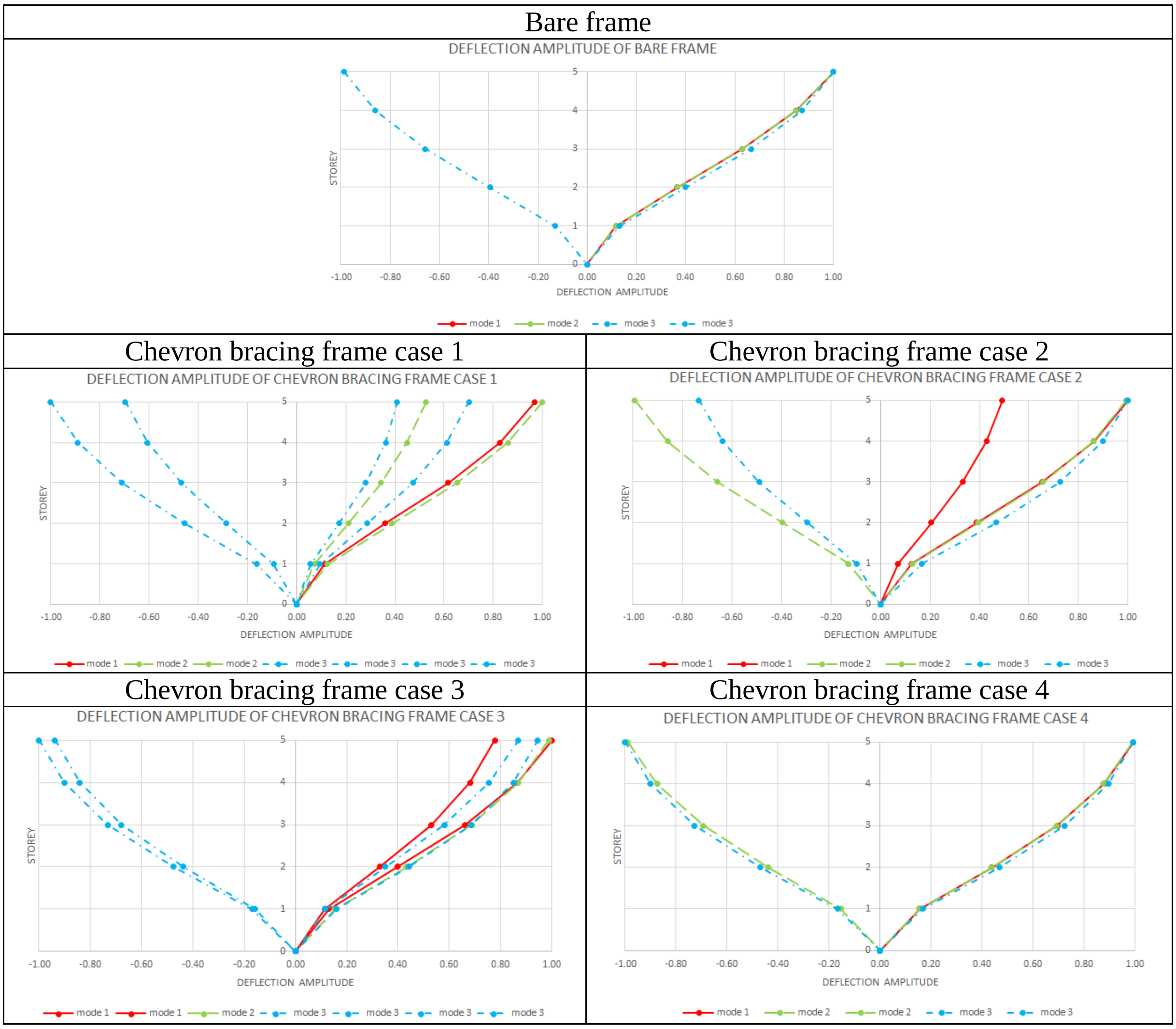


Figure 3.1: Deflection amplitude of bare frame and chevron bracing frames

From Figure 3.2, the mode 1 of the single diagonal bracing frames have the same translation mode along the North-South direction. For case 1 and case 4 of the single diagonal bracing frame, all columns were deflected in the same direction and resulted in an overlap deflection curve pattern for every column. That same deflection curve pattern was given by the bare frame. But some of the columns in case 2 and case 3 were deflected with different amplitude values. As the result, some columns were split apart from other columns in the deflection curve.

From observation of mode 2 in Figure 3.2, case 3 and case 4 of the single diagonal bracing frame were in translation mode along East-West direction, different translated direction with the bare frame. But the deflected direction of the bare frame and case 3 and case 4 of the single diagonal bracing frame were the same. All columns deflected in the same direction and resulted in an overlap deflection curve pattern for every column. Case 1 of the single diagonal bracing frame was in torsional mode, it showed some of the columns deflected in different deflection amplitude values and split apart from other columns in the deflection curve. For case 2 of the single diagonal bracing frame, it was in translation mode along the South-East and North-West direction. Some of the columns in case 2 was deflected in opposite direction.

From observation of mode 3 in Figure 3.2, the single diagonal bracing frames were presented in different deflection directions with the bare frame. All columns in case 1 of the single diagonal bracing frame were deflected in a direction and resulted in an overlap deflection curve pattern for every column while shown a bending curve. Other than that, case 2 and case 3 of the single diagonal bracing frame were showed some of the columns deflected with different amplitude values and split apart from other columns while shown a bending curve. Case 4 of the single diagonal bracing frame was showed slightly different deflection amplitude values with the bare frame.

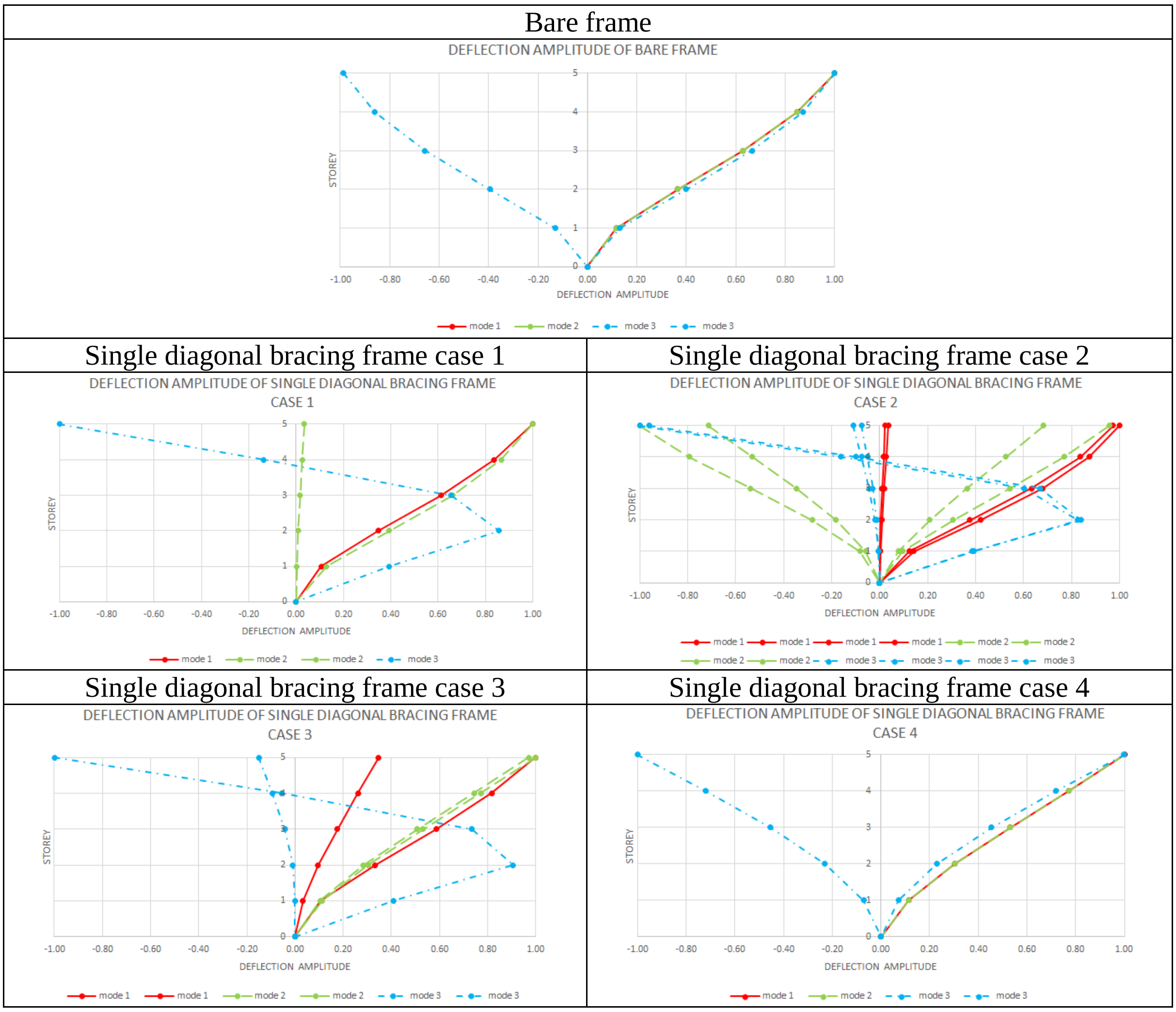


Figure 3.2: Deflection amplitude of bare frame and single diagonal bracing frames

From Figure 3.1 and Figure 3.2, the maximum deflection amplitude for the bare frame, chevron bracing frames and single diagonal bracing frames occurred at storey 5. According to [15], the maximum deflection of a building frequently located at the top level of the building.

3.4 Relative floor displacement amplitude

The maximum deflection amplitude at the top storey subtracted bottom storey of the steel tower was to determine the relative floor displacement amplitude. [16] demonstrated similar results in their study, where relative floor displacement occurs at the mid-high of all frame structures in terms of inter-storey drift. The relative floor displacement amplitude can present the weak storey of the steel tower. By referring to Figure 3.3 and Figure 3.4, the maximum displacement amplitude for the bare frame in all modes occurred among storey 2 and storey 3, which the middle storey of the steel tower. In Figure 3.3, the highest displacement amplitude value of the bare frame and chevron bracing frames occurred in mode 3. For the maximum displacement amplitude of chevron bracing frames, that from Figure3.3 was occurred among storey 2 and storey 3. The location occurred maximum displacement was determined as the weak floor. According to [17], the weak storey of the structures was existing at the middle storey.

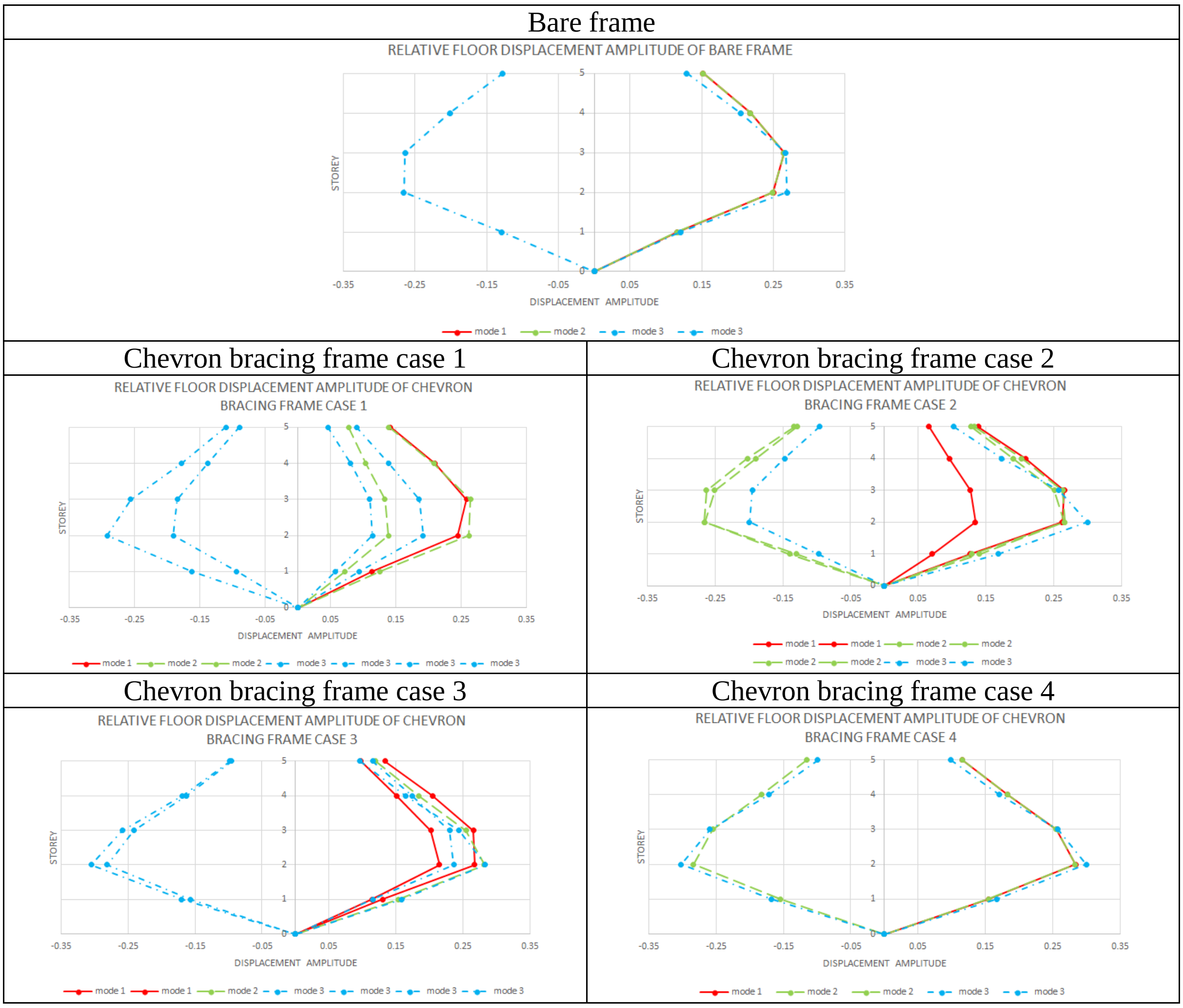


Figure 3.3: Relative floor displacement amplitude of bare frame and chevron bracing frames

Figure 3.4 were showed the relative floor displacement amplitude of bare frame and single diagonal bracing frames. From Figure 3.4, the maximum displacement amplitude of single diagonal bracing frames in mode 1 occurred at storey 3. Following with the maximum displacement amplitude of single diagonal bracing frames in mode 2 occurred at storey 4. The maximum displacement amplitude of single diagonal bracing frames in mode 3 occurred at storey 5. From mode 1 to mode 3, the location that occurred the maximum displacement amplitude for single diagonal bracing frames was getting high. The highest displacement amplitude value of single diagonal bracing frames was in mode 3 as showed in Figure 3.4. The location occurred maximum displacement amplitude was showed that the improvement of strength was needed to decrease the displacement amplitude.

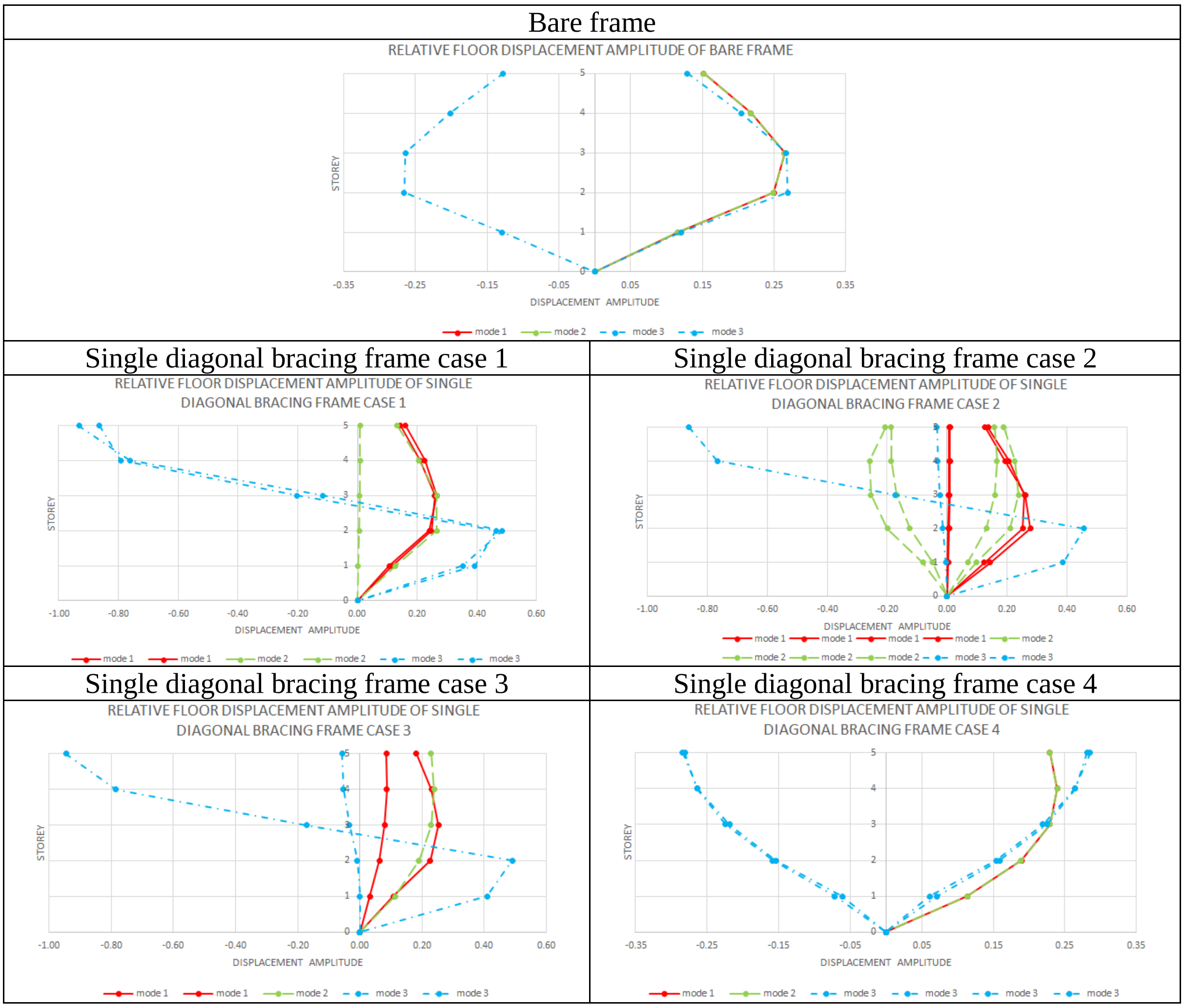


Figure 3.4: Relative floor displacement amplitude of bare frame and single diagonal bracing frames

4. Conclusion

This study was successful verification between the modal analysis of the 5-storey steel tower using the STAAD Pro software with the previous ambient vibration testing result. The frequencies in the first three modes for the bare frame was 5.55Hz, 5.55Hz and 7.23Hz with percentage differences of less than 6%. Hence the result can be determined as the acceptable analysis for modelling. The application of chevron bracing systems and single diagonal bracing systems were determined in terms of the dynamic characteristics applied on the 5-storey steel tower. Both bracing systems showed the increase of peak frequency when increasing the bracing members in the steel tower. From the result of applied the chevron bracing systems and single diagonal bracing systems in this study, the chevron bracing systems presented the more stable results in dynamic characteristics of the 5-storey steel tower. First, the peak frequency of chevron bracing frames was in the range of performing well in vibration. Besides, the mode shape of chevron bracing systems was more regularity repeat compare to the mode shape of single diagonal bracing systems. So, it can predict building mode shape and reduce the unexpected behavior of building mode shape. Other than that, the maximum deflection amplitude for chevron bracing frames was always on the top storey of the steel tower. Besides, the maximum displacement amplitude which found as the weak storey for all modes occurred at the middle of the chevron bracing frames. The chevron bracing systems have the most similarity of curve pattern for maximum deflection amplitude and relative floor displacement amplitude with the bare frame

Acknowledgement

The authors are grateful to the Universiti Tun Hussein Onn Malaysia and Faculty of Civil Engineering and Built Environment for providing the resource data for this study. Appreciation to everyone who involved directly or indirectly toward the completion of this study.

References

[1] D. E. Nassani, A. K. Hussein, and A. H. Mohammed, "Comparative response assessment of steel frames with different bracing systems under seismic effect," In Structures, 11, pp. 229-242, 2017.

[2] S. Metre, S. Ghule, and R. Kiran, "Comparative Study of Different Types of Bracing Systems by Placing at Different Locations," International Research Journal of Engineering and Technology, 4(8), pp. 2023-2027, 2017.

[3] R. Sabelli, C. W. Roeder, and J. F. Hajjar, "Seismic design of steel special concentrically braced frame systems," Gaithersburg, USA: NEHRP Seismic Design Technical Brief, 8, 2013.

[4] M. Omar, "Seismic response of braced steel frames with shape memory alloy and mega bracing systems," International Journal of Computer and Systems Engineering, 8(2), pp. 131-138. 2014.

[5] M. P. Kishore, R. K. Behera, S. K. Pradhan, and P. K. Parida, "Effect of Material Behavior on Dynamic Characteristics Determination of Marine Propeller Blade Using Finite Element Analysis," Procedia Engineering, 144, pp. 767-774, 2016.

[6] S. N. Tsai and A. C. Taylor, "Vibration behaviours of single/multi-debonded curved composite sandwich structures," Composite Structures, 226, pp. 1-13, 2019.

[7] T. Türker, and A. Bayraktar, "Experimental and numerical investigation of brace configuration effects on steel structures," Journal of Constructional Steel Research, 67(5), pp. 854-865, 2011.

[8] M. L. Chandravanshi, and A. K. Mukhopadhyay, "Experimental modal analysis of the vibratory feeder and its structural elements," International Journal of Applied Engineering Research, 10(13), pp. 33303-33310, 2015.

[9] TIER 1, "Dynamic Modal Analysis in Assessing Structural Integrity and Vulnerabilities on Portable Steel Frame using Ambient Vibration Method," Universiti Tun Hussein Onn Malaysia, 2017.

[10] A. F. Kamarudin, M. K. Musa, S. N. Mokhatar, T. T. Chik, S. M. Zuki, A. A. Bakar, and H. A. Johari, "Mechanical Properties of Single Shear Plane of Bolted Steel Connection," In IOP Conference Series: Materials Science and Engineering, 713(1), pp. 1-8, 2020.

[11] M. F. Khairul, "Investigation on the effects of alternating mass configuration on dynamic characteristics of moment resisting steel bare frame," Universiti Tun Hussein Onn Malaysia: Bachelor's Degree Project, 2019.

[12] A. Baldassarre, J. Ocampo, M. Martinez, and C. Rans, "Accuracy of strain measurement systems on a non-isotropic material and its uncertainty on finite element analysis," Journal of Strain Analysis for Engineering Design, 56(2), pp. 76–95,2021.

[13] S. M. Razak, T. C. Kong, N. Z. Zainol, A. Adnan, and M. Azimi, “A Review of Influence of Various Types of Structural Bracing to the Structural Performance of Buildings,” In E3s Web of Conferences, 34, pp. 01010, 2019.

[14] C. V. R. Murty, R. Goswami, A. R. Vijayanarayanan, and V. Mehta, “Earthquake behaviour of buildings,” Gujarat State Disaster Management Authority, Gandhinagar, pp.53-79, 2012.

[15] T. Jayakrishna, K. Murali, P. Satish, J. Seetunya, and M. L. Reddy, “Seismic Analysis of Regular and Irregular Multi-Storey Buildings by Using STAAD-pro,” International Journal of Civil Engineering and Technology (IJCIET), 9(1), pp. 431-439, 2018.

[16] K. Shah and P. Vyas, “Effects Of Vertical Geometric And Mass Irregularities In Structure,” International Conference on Research and Innovations in Science, Engineering & Technology, pp. 87-92, 2017.

[17] N. Kirac, M. Dogan, and H. Ozbasaran, “Failure of weak storey during earthquakes. Engineering Failure Analysis, 18(2), pp. 572–581, 2011.