Recent Trends in Civil Engineering and Built Environment Vol. 3 No. 1 (2022) 1095-1107

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 Structural Configurations

Muhammad Badrun Amin Masri[1], **Ahmad Fahmy Kamarudin**[1]*

[1]Department of Civil Engineering, Faculty of Civil Engineering and Built Environment,
Universiti Tun Hussien Onn Malaysia, Batu Pahat, Johor, 86400, MALAYSIA

*Corresponding Author Designation

DOI: https://doi.org/10.30880/rtcebe.2022.03.01.125
Received 4 July 2021; Accepted 13 December 2021; Available online 15 July 2022

Abstract: This paper was determined the effect of dynamic characteristics on 5-storey steel tower with different structural configurations by using modal analysis through STAAD Pro software. There are two types of structural configurations found in the structure which is cross-section and connection. Dynamic characteristics of steel tower was investigated in terms of predominant frequency, maximum deflection amplitude and relative floor displacement amplitude. In this study was focused to verify the comparison result from bare frame with the previous laboratory measurement. There are three predominant frequency was produced from modal analysis which according to their modes. The result of bare frame is $f1$ and $f2$ = 5.55Hz, and $f3$ = 7.23Hz with percentage different less than 10% from previous study. Besides, there are three type of modes shapes for steel tower which can be explained in translational mode and torsional mode. The translational mode was produced for mode 1 and mode 2 while torsional mode for mode 3. The relationship of predominant frequency, maximum deflection amplitude and relative floor displacement were developed between the dynamic characteristics of steel tower. Result has been proved the maximum deflection for all cases in two types of structural configurations were located at top level of steel tower. While the maximum displacement amplitude for weak storey were located on the level 2 and level 3 which need an improvement to be done for maintaining the structure.

Keywords: Structural configurations, Modal analysis, STAAD.Pro, Dynamic characteristics

1. Introduction

Safety of the structure plays an important role to keep maintaining the characteristics of the dynamic behaviour of the structure from damage failure or collapse. The behaviour and vulnerability of buildings are highly dependent on the dynamic characteristics of buildings [1]. Dynamic characteristics of

*Corresponding author: fahmy@uthm.edu.my

building can be evaluated through the estimation of modal analysis such as natural frequencies and mode shape. Every structure has its own natural frequency which the structure tends to vibrate if it is subjected to a disturbance [2]. Therefore, the addition of vibration will lead to a change in natural frequency and mode shape that can cause structural damage and failure [3]. The variety of section properties allow the structural engineers to design excellent compression efficiency, high torsional resistance and aesthetic appearance [4]. Other than that, strength of the connection between members in the steel structures are concern one of the most important part in structural design and steel structure will support the load perfectly when the members and connection are in good design [5].

Building collapse and heavily damaged structure that causes by the earthquake were reported over nearly 10,000 people cases losing their lives around the world every year because of these natural hazards due to decreasing of the cross-section in columns [6]. Increasing of the beam and column dimension, it will increase in mass and stiffness of total structures [7]. The cross-section is an important component for every single aspect of analysis and structural design. Changing the cross-section of columns from square to circular increased the column's load-carrying power performance [8]. The change of density, length, young's modulus and area of cross sectional will affect the change of natural frequency and mode shape of the structures [9]. If the position of the connection is changed, the structural of the natural frequency and mode shape also was changed [10]. The connection will lead to fracture and structure damage or collapse when the connection did not receive sufficient resistance or ductility to support the large rotations simultaneously and normal force [11]. In this study, the effect of cross-section and connection has been investigated.

The objectives were to validate the modal analysis of STAAD Pro against the result of modal parameters obtained from previous laboratory measurement of ambient vibration testing and to investigate the trend of dynamic characteristics and maximum deflection amplitudes with different variables of structural configurations used for vulnerability assessment. The significances were to provide an economic design of the existing steel tower for maximum serviceability and equivalent dynamic behaviour with effective structural configurations. Besides, it also could assess the vulnerability of the existing steel tower with different cross-sections and connection conditions for safety and economic design purpose.

2. Literature Review

Every type of building has its own dynamic characteristics which are natural frequency and mode shape. The natural frequency and mode shape are the most important dynamic characteristics for tall buildings [12]. The most important aspects of structural health monitoring are determining the dynamic characteristics of structures from micro seismic and ambient vibration [13]. Next, the dynamic behaviour of structural building depends on dynamic characteristics of structures which are can be determined on the material properties, boundary condition, structural properties and the number of the structure damage [14]. The natural frequency unit was measure in Hertz (Hz), which can be determined as the period of time for the building to sways from side to side in seconds [15]. The modes of vibration and natural frequency can be determined based on free vibration analysis which can be done numerically or experimentally [7]. The study more focused on the findings from previous study about the dynamic characteristics on cross-section and connection that has been tabulated in Table 1 and Table 2.

Table 1: Investigation of dynamic characteristics on cross-section

Author	Methodology	Results
[16]	This study was investigated the vibration on the isotropic beam with the variable cross-section and three different type of boundary conditions were applied.	The result has been showing when the cross-section was changed, it will affect the natural frequency and vibration form. Besides, there were produced fifth mode of natural frequencies with increasing for each mode which is 4.72

		Hz, 24.20 Hz, 63.86 Hz, 123.10 Hz and 202.07 Hz.
[17]	The study has been conducted with different material for same I and T-cross section beam to determine the vibration characteristics which are natural frequency and mode shape.	The minimum natural frequency for cast iron in T-section is 14.3Hz while the maximum natural frequency for structural steel in I-section is 415.08 Hz

Table 2: Investigation of dynamic characteristics on connection

Author	Methodology	Results
[10]	Three tower tall building were connected to each other with a change of connection location and the effect on dynamic response were investigated by using finite element software.	The result has been shown the minimum of maximum displacement in x-direction is about 97.99% while in y-direction is about 98.39%.
[18]	The study was conducted based on the experimental study to evaluated of damage effect on the dynamic vibration characteristics of concrete encased composite column with different column and beam connection by using ambient vibration analysis	The damage and cracks happen in the column and beam connection will strongly affected the natural frequency due to decreasing in frequency because of decrement in flexural rigidity in the cracked parts. The maximum difference for natural frequency were calculated between 21.72% and 39.96%
[19]	The dynamic characteristics was performed by using Operational Modal Analyses by applied boundary conditions such as material properties to determine the effect on the dynamic behaviour. The semi-rigid was considered as the connection for the structural elements.	The first four natural frequencies were obtained between 7-22 Hz with maximum difference frequencies between analytical and experimental were reduced from 47% to 2.6%.

Mode shapes define as the deformation shape of the building when it oscillated and is shaken at the natural period [20]. The building structure has its numbers of natural frequency. There are three fundamental mode shapes of oscillation. The first two-mode shape is pure translational, and the third mode was in rotational mode shape. There was same regular translational in the mode shape for 1st and 2nd mode, while mode 3rd mode was produced in torsion [10]. From the Figure 1 has been showed the pattern of mode shape based on the previous research.

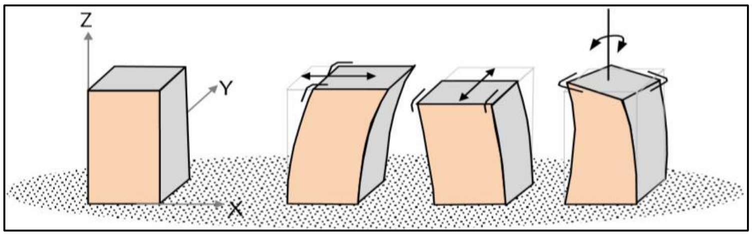


Figure 1: Fundamental mode of oscillation in translational and rotational mode shape [20]

3. Methodology

3.1 Stage 1: Observation and Data Collection

The steel frame structure of 5 storey steel tower was designed under the Research Grant TIER 1 Phase1/2017 and placed at Jamilus Research Centre (JRC) laboratory, Universiti Tun Hussein Onn Malaysia (UTHM). The structure was made up of mild steel with grade S275 and the dimension for beams and columns for square hollow section were 77 mm × 77 mm ×1000 mm with thickness 3mm for both elements. The height of steel frame was 5 m with the height for each level was 1 m. There are three different types of connectors of the steel frame which are base connector, intermediate connector and top connector. All elements (beams, columns and connections) were assembled into a 5-storey steel tower with four base connectors were placed fixed to the floor using four high tension bolts and nuts with diameter of 25mm. The spacing between connectors were 1000mm. The columns and beams were secured with 10mm diameter high tension bolts and nuts to grip the steel tower in position. According to the Figure 2 and Figure 3, the figure below has been showed the steel tower elements that designed under Research Grant TIER 1 Phase1/2017.

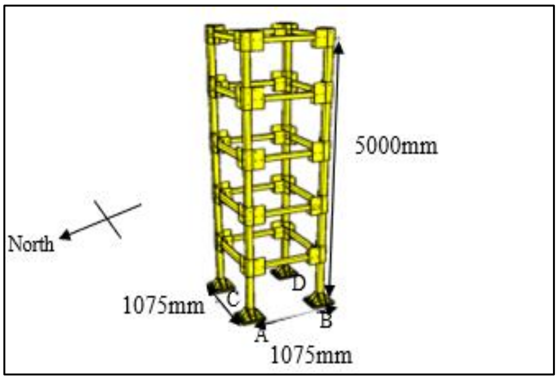


Figure 2: Moment resisting frame of a 5-storey steel tower [21]

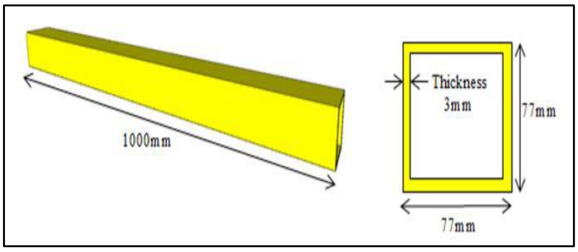


Figure 3: Dimension for beam and columns [21]

3.2 Stage 2: Modelling Work and Modal Analysis

By using STAAD Pro software, the modelling of the bare frame was conduct by referred to the dimension and parameters of the steel tower located at Jamilus Research Center (JRC) laboratory. STAAD Pro can be used for various structures for modelling, designing, and analysing. In this study, the structures are assigned as space which allowed the structure framed in 3D modelling. The unit needs to set up in meter and kilonewton to ensure the parameter of the model are accurate. To analyse the dynamic characteristics of steel tower model, the frequency results of bare steel tower model that was constructed in STAAD.Pro software was compared with frequency results from previous laboratory results that were conducted using accelerometer sensors and EFDD analysis. In order to model the steel tower for modal analysis, all dimensions of the steel tower were identified to be used in STAAD.Pro software. The detailing needed for STAAD.Pro modelling work are as in Table 3.

Table 3: Detailing of steel tower for modelling work [22]

No	Element	
1	Standard	BS EN 1993-1-8:200 Steel grade S275 fy: 275 × 10-3 kN/mm2 λ: 300 × 10-3 E: 205kN/mm2
2	Beam and column	Square hollow sections (SHS) Length = 1000 mm Width, B = 77mm Height, H = 77mm Thickness, t = 3 mm
3	Connection	High tension bolt grade 8.8 with diameter 10mm and 25mm
4	Support	Fixed at base floor
5	Loading	Self-weight

The output frequency results that was used for verification was from bare frame with regular design. Partial release moments can be release by using STAAD Pro for the frame members. By using the specification option in STAAD Pro, the moments were release at the start or end of the members. The moments were applied on the start-end of the beam and column with percentage of partial release moments by using try and error method. Besides, the moment was release 28% with highest reduction in column steel while beam take 80% moment release [23]. The percentage of partial release moments for beam are higher than the percentage applied for the column as shown in Table 4.

Table 4: The percentage of partial release moments [23]

Elements	Percentage (%)
Beam	80%
Column	28%

The study was conducted to investigate the effect of cross-section to dynamic characteristics. The steel frame needed to assign with two types of the cross-section which is square hollow section (SHS) and circular hollow section (CHS). There are three different types of size and thickness that have been provided for this study as shown in Table 5 and Table 6. The selection of size for both section properties are defined by the manual calculation for the total area which is needed to approximate between SHS and CHS.

Table 5: Dimension size for Square Hollow Section (SHS)

(SHS)	Outside Dimension Size of Steel (mm)	Inside Dimension Size of Steel (mm)	Thickness (mm)	Area for Outside of Steel (mm)	Area for Inside of Steel (mm)	Total Area of Steel (mm)
1	77 x 77	74 x 74	3	5929	5476	453
2	77 x 77	72 x 72	5	5929	5184	745
3	77 x 77	69 x 69	8	5929	4761	1168

Table 6: Dimension size for Circular Hollow Section (SHS)

(CHS)	Outside Diameter Size of Steel (mm)	Inside Diameter Size of Steel (mm)	Thickness (mm)	Area for Outside of Steel (mm)	Area for Inside of Steel (mm)	Total Area of Steel (mm)
1	88.9	85.9	3	6207.17	5795.30	411.87
2	88.9	83.9	5	6207.17	5528.58	678.59
3	88.9	80.9	8	6207.17	5140.28	1066.89

3.3 Stage 3: Result and Data Analysis

There are eight cases for the square hollow section (SHS) with three different sizes and thickness as tabulated in Table 7. Meanwhile, the circular hollow section (CHS) has been modelling with nine cases that refer to the three different types of diameters and thickness as given in Table 8.

Table 7: Size, thickness, and partial release moment of SHS

Type	Size	Thickness	Partial Release Moment	
			Column	Beam
Bare Frame	77mm x 77 mm	3mm	28%	80%
Case 1	77mm x 77mm	5mm	28%	80%
Case 2	77mm x 77mm	8mm	28%	80%
Case 3	77mm x 77mm	3mm	33%	75%
Case 4	77mm x 77mm	5mm	33%	75%
Case 5	77mm x 77mm	8mm	33%	75%
Case 6	77mm x 77mm	3mm	38%	70%
Case 7	77mm x 77mm	5mm	38%	70%
Case 8	77mm x 77mm	8mm	38%	70%

Table 8: Size, thickness, and partial release moment of CHS

Type	Size	Thickness	Partial Release Moment	
			Column	Beam
Bare Frame	77mm x 77 mm	3mm	28%,	80%
Case 1	88.9mm x 85.9mm	3mm	28%	80%
Case 2	88.9mm x 83.9mm	5mm	28%	80%
Case 3	88.9mm x 80.9mm	8mm	28%	80%
Case 4	88.9mm x 85.9mm	3mm	33%	75%
Case 5	88.9mm x 83.9mm	5mm	33%	75%
Case 6	88.9mm x 80.9mm	8mm	33%	75%
Case 7	88.9mm x 85.9mm	3mm	38%	70%
Case 8	88.9mm x 83.9mm	5mm	38%	70%
Case 9	88.9mm x 80.9mm	8mm	38%	70%

The analysis on the mode shape and natural frequency with the different variables such as cross-section and connection were analysed by conducted with two type analysis which are maximum deflection amplitude and relative floor displacement amplitude. In order to find the specific trend or pattern between the variation of the steel tower and the building configuration, it can be formed by three-dimensional (3D). The maximum deflection amplitude and relative floor displacement amplitude was calculated from their inter-storey deviation by using Equation 1 below.

Relative floor displacement = *AT – AB*

Where, AT = *Amplitude at top of storey*

AB = *Amplitude at bottom of storey* $Eq. 1.$

3.4 Stage 4: Development of relationship

The result of modal analysis for the bare frame has been verified with the previous laboratory outcomes. Next, the analysis of natural frequency and mode shape were beginning with two types of analysis which are relative floor displacement and maximum deflection amplitude. In order to find the vulnerability assessment of this study, a few relationships between natural frequency, maximum deflection amplitude, relative floor displacement amplitude with respective structural configurations were developed.

4. Results and Discussion

The output results from the modal analysis of a 5-storey steel tower in JRC laboratory were obtained. The dynamic characteristics were analysed using STAAD Pro software in order to determine the predominant frequency, maximum deflection amplitudes, and relative floor displacement amplitude with a different type of cross-section and connection variables.

4.1 Predominant frequencies of bare frame and result verification

Modal analysis has been conducted on a 5-story steel tower of the bare frame by using STAAD Pro software. The verification result of predominant frequency from the modal analysis method was carried out to ensure the accuracy of the model. The results of predominant frequency were obtained from the computational simulation. The output from the software has been produced three dominant modes where each mode has their own predominant frequency.

The first three fundamental frequencies of steel bare frame are 5.55 Hz, 5.55 Hz, and 7.23 Hz. The comparison results have been made between the modal analysis and previous ambient vibration testing that conducted by [24]. In this study, the frequency value for mode 1 and mode 2 shown clearly higher than previous laboratory tests. By referring to Table 9, the percentage difference for the first, second, and third modes were slightly decrease to lower values which is 5.6%, 1.1%, and 0%. The percentage difference for all modes has considered acceptable due to comparison made by [25] where from the computational analyses and laboratory testing as acceptable limit when percentage is less than 10%.

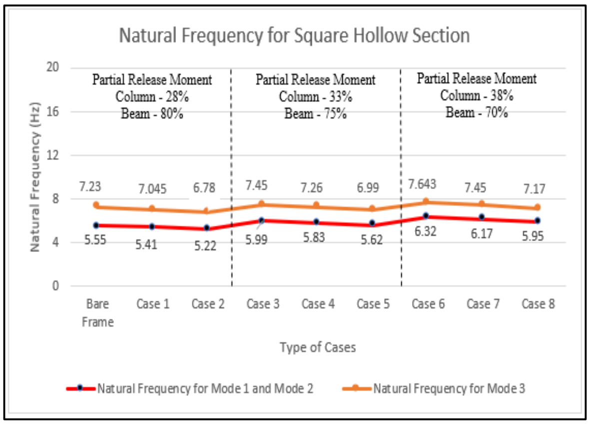
Table 9: Percentage difference for frequency data from modal analysis and previous laboratory measurements

Mode	Modal analysis for natural frequency from STAAD Pro (Hz)	Previous laboratory measurement for natural frequency (Hz) [23]	Percentage difference (%)
1	5.55	5.24	5.6
2	5.55	5.49	1.1
3	7.23	7.23	0

4.2 Predominant frequencies of multiple cases of structural configurations

The modelling of steel tower was modelled with multiple cases of structural configurations. According to the Figure 4 and Figure 5 below, the output of natural frequency for mode 1 and mode 2 were compliment to each other. Meanwhiles, the output for mode 3 results were higher than mode 1 and mode 2. The natural frequency for all cases were decrease when the structural properties were changes. The structural properties, material properties, boundary conditions were influenced the natural frequencies of the structure [14].

The bare frame has the lowest frequency between all cases of circular hollow sections. Meanwhile, highest frequency for square hollow section (SHS) and circular hollow section (CHS) were obtained in case 6 and case 7. The study conducted by [20], the structure which has short natural frequency in the range 0.5-25.0 Hz was performed well in vibration. Other than that, the natural frequency that produced outside the range were placed the structure in zone of danger which can caused collapsed or heavy damaged [26]. The trendline of natural frequency for all cases can be proved that mechanical properties are the main factor to the output of the results. The natural frequency and mode shape were influenced by non-uniformity of cross-section [16].



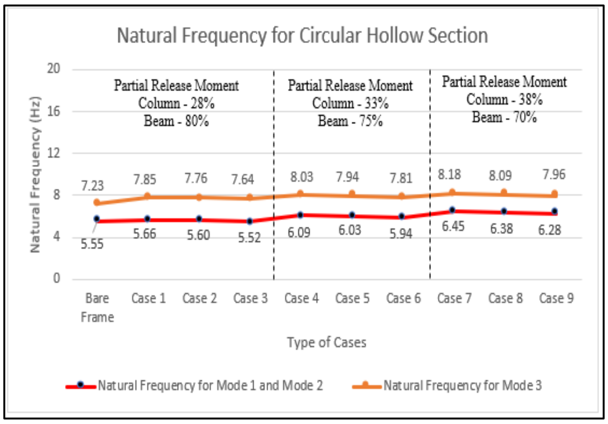
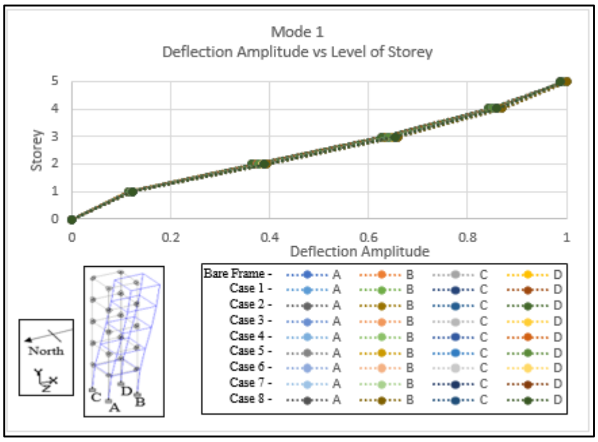


Figure 4: Natural frequency (SHS) **Figure 5: Natural frequency (CHS)**

4.3 Maximum deflection amplitude

4.3.1 First and second mode: Translational

The stiffness of the building was influenced to the deformation of translational and torsional mode [14]. There were almost cases had complemented each other for both type of cross-section. By referring to Figure 6 and Figure 7, the deflection amplitude of the steel tower was increased from the first floor until the fifth floor. The deflection amplitude starting to decrease in level 5 which starting in case 1 to case 8 for SHS. Meanwhile, the same pattern of decreasing value was obtained in the same level which starting to reduce from case 1 to case 9 in CHS. Besides, there was a maintaining value at level 5 for all cases in both cross section which can be translated in column A and column B. The deflection amplitude pattern for all column in mode 1 mostly in translation mode toward in the same direction which is parallel to x-axis. Furthermore, the maximum deflections amplitude for all nodes were located at the level 5. According to the [27], the maximum deflection amplitude has occurred on the top of the building. It was clearly showed the deflection pattern were presents relative displacements of all parts of the structure.



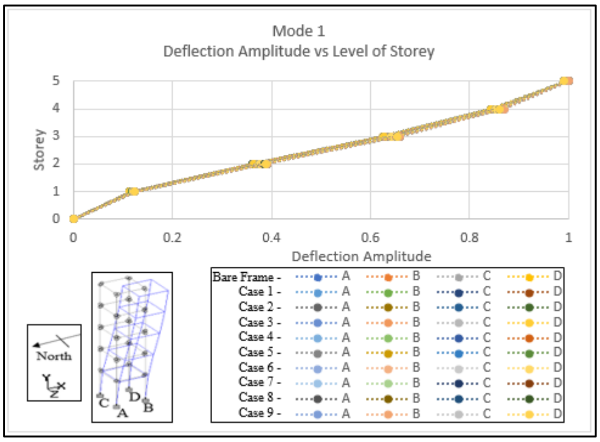


Figure 6: Deflection amplitude for mode 1 (SHS) Figure 7: Deflection amplitude for mode 1 (CHS)

Based on Figure 8 and Figure 9, the deformation for the second mode was translated into both directions of NS and EW directions. The translation of this mode was dominant in the direction of x-axis. There was an increasing pattern of deflection amplitude for both cross-sectional. The similar curve pattern in mode 2 were produced for all cases. From the observation, the lowest value for deflection amplitude were received at level 1 storey. The lateral deflection for second mode were relatively higher than first mode [28]. Besides, the deflection amplitude for column A and column B more deflect than column C and column D. The deflection amplitude value difference for each column was $\pm$ 0.0001 difference. The maximum deflection amplitude was located at the level 5. The results were similar as study conducted by [29] where the maximum deflection amplitude was determined on the top storey and reduce when reach to the ground level. The significant value for all cases in maximum deflection amplitude was closed to 1.0. Basically, there were not much difference between mode 1 and mode 2 due to structural configurations effect of steel tower.

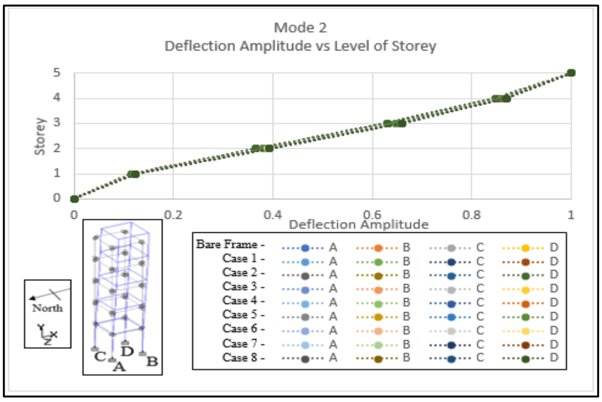


Figure 8: Deflection amplitude for mode 2 (SHS)

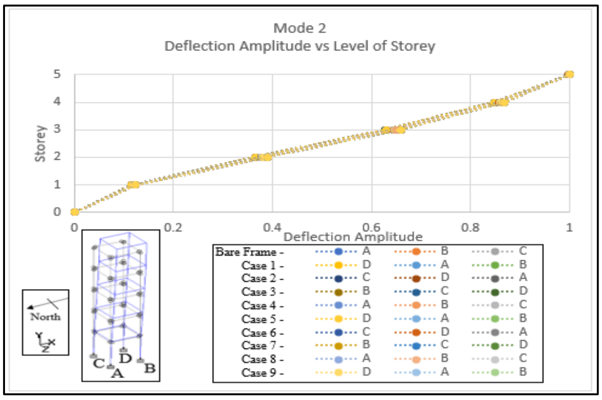


Figure 9: Deflection amplitude for mode 2 (CHS)

4.3.2 Third mode: Torsional

The steel tower was rotated in both direction of NS and EW for deflection amplitude. According to Figure 10 and Figure 11, the deflection amplitude for mode 3 were in torsional mode which split into two group of trendline. The curve pattern of deflection amplitude was overlapped each other. The direction for column A and column C was in negative EW direction, while the column C and column D in positive NS direction. Then, the negative NS direction was occurred at column A and column B. Meanwhile, position for column B and column D was in positive EW direction. There was a decreasing pattern of deflection amplitude for both cross-sectional. The maximum deflection amplitude for mode 3 was located at the highest level of a steel tower. Then, the value for maximum deflection amplitude was close to 1.00. By referring to study of [7], the maximum deflection was found at the peak of the model. The lateral deflection was related to many variables such as mass of structural, structure system and mechanical properties of material structures [30].

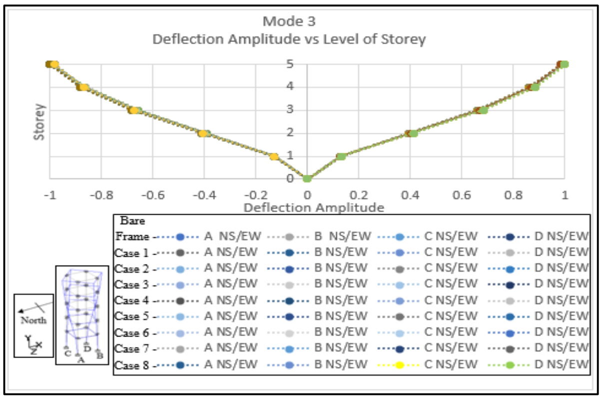


Figure 10: Deflection amplitude for mode 3 (SHS)

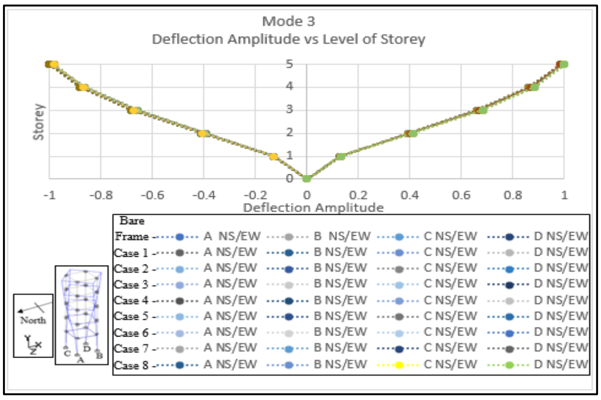


Figure 11: Deflection amplitude for mode 3 (CHS)

4.4 Relative floor displacement amplitude (RFDA)

4.4.1 First mode and second mode: RFDA

Relative floor displacement amplitude was determined from the displacement amplitude of top storey with subtracting from bottom storey. It has been calculated to determine the weak level of steel tower. The centre of the structure can be decided as the weak level. By referring to the Figure 12 and 13, the red circle was mentioned the maximum displacements amplitude for all cases was occurred at level 3. The trendline for all column had overlapped each other. The value of displacement amplitude was higher in column A and column B compared to column C and column D. The maximum displacement amplitude for all columns were concentrated at level 3 and determined as weak floor. The value of displacement amplitude was increase starting from the first floor until reach to the third floor. Then, the value was starting to reduce on the fourth floor and above. The highest value of maximum displacement amplitude that located at level 3 is case 8 and case 9 which refer to the square and circular hollow sections. The result study by [10], the response of connection between the displacement has a great significance.

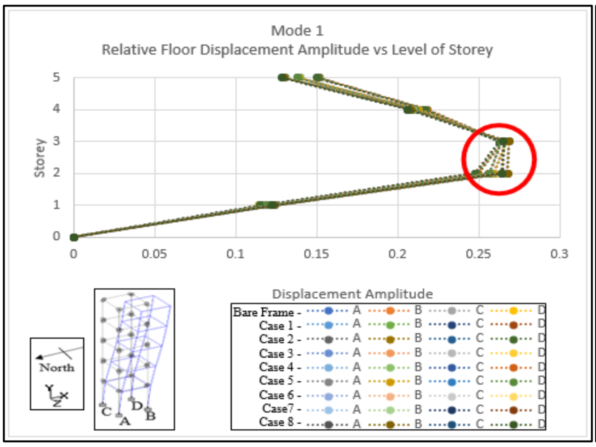


Figure 12: RFDA for mode 1 (SHS)

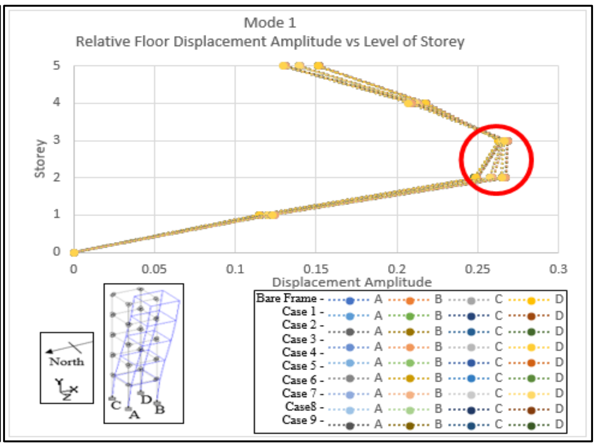


Figure 13: RFDA for mode 1 (CHS)

According to the Figure 14 and Figure 15, the figure showed the RFDA was plotted against level storey for both cross-sectional. The dominant value for cross-section on displacement amplitude were highest at level 3 as recognise on the red circle. The displacement amplitude at level 3 was maintain when performed in case 4 and case 8 which are 0.2669 and 0.2687 for SHS. At the same time, displacement amplitude in circular hollow section for case 3 and case 5 were similar which are 0.2646 and 0.2668 at level 3. Meanwhile, the maximum displacement amplitude was recorded at level 2 in case 9 for circular hollow section. Thus, it can be explained as the weak storey of steel tower. The larger of displacement amplitude, the wave become more intense. Most of the cases were showed the decreasing pattern in mode 2 at level 3 for both cross-sectional respectively.

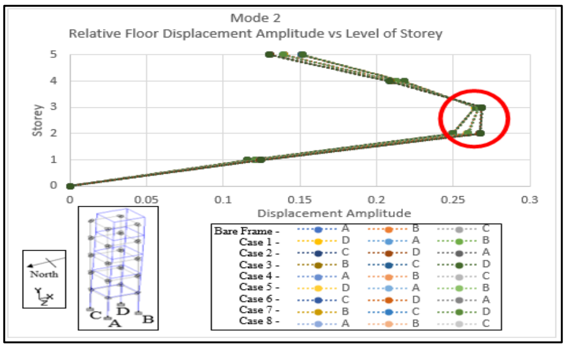


Figure 14: RFDA for mode 2 (SHS)

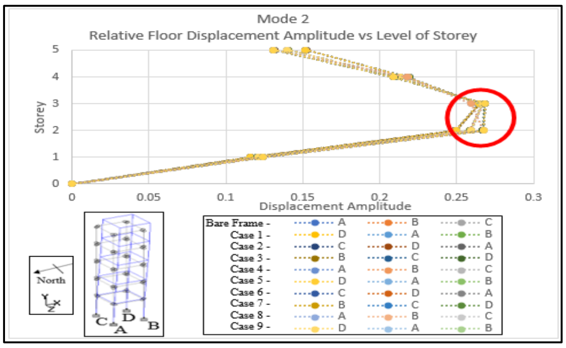
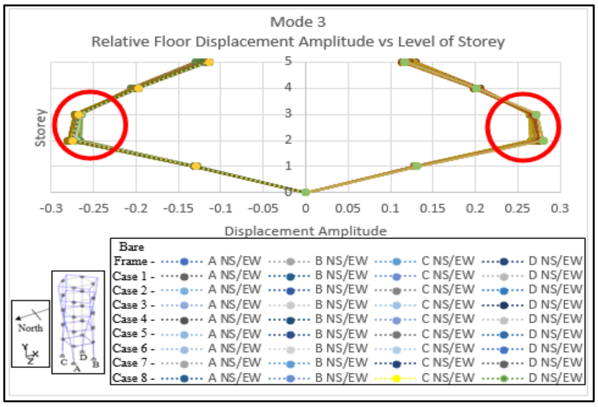


Figure 15: RFDA for mode 2 (CHS)

4.4.2 Third mode: RFDA

Based on the Figure 16 and Figure 17, the curve pattern of the RFDA of bare frame and structural configurations cases were present. The curve pattern of RFDA for cross-sectional were split into two group of trendlines. There are four different types of line has been produced for each case. The trendline can be translated in column A, column B, column C and column D. Besides, the highest displacement amplitude for bare frame was occurred at level 2. The direction column for highest displacement amplitude of bare frame can be translated in column B and C in EW direction and column C and D in NS direction. There was a similar pattern of displacement amplitude at level 2 of bare frame that provided in case 1 of square hollow section. Meanwhile, there are three kind of cases which has similar value of displacement amplitude for circular hollow section that obtained in case 4, case 5 and case 6.

By referring to the red circle on the Figure 16 and 4.17, there the two dominant displacement amplitude that located at level 2 and level 3. However, level 2 has the more dominant value of displacement amplitude. The maximum displacement amplitude was located at level 2 for all cases can be defined as the weak storey. It was similar to the previous study that explained the relative floor displacement amplitude was occurred at the mid of the structure [30]. There was a significance difference of maximum displacement amplitude between mode 1 and mode 2 compared to mode 3. This is because the maximum displacement amplitude for mode 1 and mode 2 occurred at level 3. While maximum displacement amplitude for mode 3 was located at level 2.



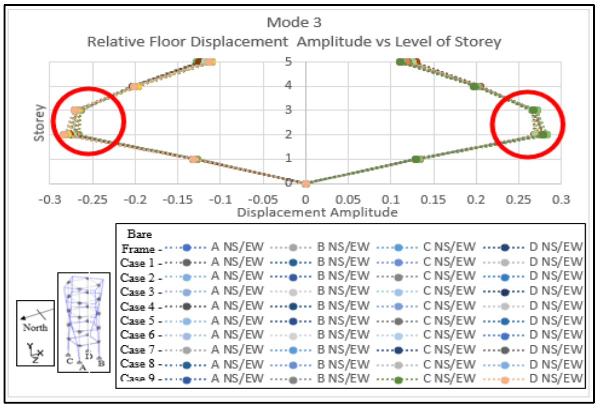


Figure 16: RFDA for mode 3 (SHS) **Figure 17: RFDA for mode 3 (CHS)**

5. Conclusion

The modal analysis of steel tower bare frame has been effectively verified with previous ambient vibration testing. The percentage different between modal analysis of steel tower and previous ambient vibration testing were less than 10%. Hence, the resulted can be determined as the acceptable analysis for modelling work. The steel tower has been produced three predominant mode shapes which can be explained in translation for modes 1 and 2, and torsional for mode 3. The maximum deflection amplitude for modes 1, 2, and 3 was always found at the top level of the steel tower. In the meantime, the maximum displacement amplitude for all modes was occurred at the centre of the steel tower. It was found as the weak storey for the steel tower. Most of the cases in both cross-sectional have a similar curve pattern for displacement amplitude and relative floor displacement amplitude.

Acknowledgement

The author would like to give appreciation to Faculty of Civil Engineering and Built Environment, Universiti Tun Hussein Onn Malaysia for the endless support and complete facilities for being able to conduct the study.

References

[1] Mucciarelli et al., “Analysis of RC building dynamic response and soil-building resonance based on data recorded during a damaging earthquake (Molise, Italy, 2002),” *Bulletin of the Seismological Society of America*, 94(5), 1943–1953, 2004

[2] Kishore et al., “Effect of Material Behavior on Dynamic Characteristics Determination of Marine Propeller Blade Using Finite Element Analysis,” *Procedia Engineering*, 144, pp. 767–774, 2016

[3] Tsai et al., “Vibration behaviours of single/multi-debonded curved composite sandwich structures,” *Composite Structures*, July 2019

[4] Meng et al., “Cross-sectional behaviour of cold-formed high strength steel circular hollow sections,” *Thin-Walled Structures*, January 2020

[5] Raghu, K., & Ramakrishnaiah, A, “Analysis and Study of Semi-rigid Joints in Peb- Steel Structure by Using STAAD Pro Fig: Pre-Engineered Building,” 8(3), pp. 924–931, 2019.

[6] Yön et al., “Earthquakes and Structural Damages,” *Earthquakes - Tectonics, Hazard and Risk Mitigation*, 2017

[7] Siddika et al., “Study on Natural Frequency of Frame Structures,” *Journal of Computational Engineering and Physical Modeling*, 2(2), pp. 38–49, 2019

[8] Khamees et al., “Effect of hollow ratio and cross-section shape on the behavior of hollow SIFCON columns,” *Journal of King Saud University - Engineering Sciences*, 2020

[9] Gao et al., “Natural frequency and mode shape analysis of structures with uncertainty,” *Mechanical Systems and Signal Processing*, pp. 24–39, 2007

[10] Yuan et al., “Influence of Connection Location on Dynamic Characteristics of Three-Towers-Connected High-Rise Building,” *Advanced Materials Research*, pp. 243–249, pp. 419–425, 2011

[11] Burgess and I, “Connection behaviour and the robustness of steel-framed structures in fire,” in *MATEC Web of Conferences*, 2018

[12] Edskär, I, “Modal Analysis, Dynamic Properties and Horizontal Stabilisation of Timber Buildings,” 2018.

[13] Dobre et al., “Dynamic Characteristics of Buildings from Signal Processing of Ambient Vibration,” *IOP Conference Series: Materials Science and Engineering*, 2017

[14] Bayraktar, A., Türker, T., ALTUNIşIK, A. C., Sevim, B., Şahin, A., & Özcan, D. M, “Determination of dynamic parameters of buildings by operational modal analysis,” *Teknik Dergi/Technical Journal of Turkish Chamber of Civil Engineers*, *21*(4), 2010.

[15] Eriksson, J, "Horizontal natural frequency in a 10-story building: A comparison between CLT and concrete using estimate calculations,". Building Technologies, September 2018. [Online]. Available: http://www.diva-portal.org/smash/get/diva2:1251690 [Accessed Dec. 2, 2020].

[16] Ece et al., “Vibration of a variable cross-section beam,” *Mechanics Research Communications*, pp. 78–84, 2007

[17] Kumar, V., Kumar Singh, K., & Gaurav, S, “Analysis of Natural Frequencies for Cantilever Beam with I-and T-Section Using Ansys,” *International Research Journal of Engineering and Technology*, pp. 1013–1020, 2015.

[18] Hüsem, M., Nasery, M. M., Okur, F. Y., & Altunişik, A. C, "Experimental evaluation of damage effect on dynamic characteristics of concrete encased composite column-beam connections," *Engineering Failure Analysis*, pp. 129–150, 2018

[19] Kartal, Murat, "Boundary Condition Effects on The Dynamic Characteristics of a Steel Bridge Model," American Society of Civil Engineers 6th International Engineering and Construction Conference (IECC'6), Cairo, Egypt, June 28-30, 2010.

[20] Murty, C. V. R., Rupen, Goswami., A. R. Vijayanarayanan., V. V. M, "Earthquake Behaviour of Buildings," Gujarat: Gujarat State Disaster Management Authority, 2012.

[21] 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.

[22] Kamarudin et al., "Dynamic Characteristics Evaluation on Portable Steel Frame due to Mass Irregularities," *International Journal of Integrated Engineering*, 12(9), pp. 27–35, 2020

[23] Gandhi, B., Gharat, P., & Murudi, D, "A Study on the Effect of Partial Moment Releases on the Behaviour of Steel Structure," *International Journal of Innovative Research in Science, Engineering and Technology*, 06(07), pp. 13223-13229, 2017.

[24] Khairul, M. F, "Investigation of Concentric Mass Action on Moment Resisting Steel Tower Against Dynamic Characteristic Modification.," Universiti Tun Hussein Onn Malaysia, 2019.

[25] Baldassarre et al., "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

[26] Aspects, E, "Engineering Aspects of the September 19, 1985 Mexico Earthquake," *National Bureau of Standards Building Science Series 165*, 1987.

[27] Patil Jaya, P., & Alandkar, D. P. M, "International journal of engineering sciences & research technology drift analysis in multistoried building," *International Journal of Engineering Sciences & Research Technology*, 5(12), pp. 490–505, 2016.

[28] Scheel et al., "A phase resonance approach for modal testing of structures with nonlinear dissipation," *Journal of Sound and Vibration*, 435, pp. 56–73, 2018

[29] Soni, A. G., Agrawal, D. G., & Pande, A. M, "Effect of Irregularity on buildings and their Consequences," *International Journal of Modern Trends in Engineering and Research*, 2(4), pp. 14–21, 2015.

[30] Shah et al., "Effect of Vertical Geometric and Mass Irregularities In Structure," vol 1, pp. 87–80, 2018