Linear and Nonlinear Seismic Analysis of a Tall Air Traffic Control (ATC) Tower

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SUMMARY:
Air Traffic Control (ATC) towers are one of the strategic and vital structures for the functionality of each airport. Due to the inadequate information about seismic design and the performance of ATC towers, structural engineers often refer to building codes. However, seismic performance and the demands of ATC towers significantly differ from common structures. In this paper, the seismic performance of Kuala Lumpur International Air traffic control tower is investigated through numerical simulations. Linear and nonlinear analyses are carried out and obtained results are compared. Results show that, in comparison to modal response spectrum analysis, equivalent static analysis overestimates overturning moments, drifts and lateral displacements. Moreover, linear analysis underestimate base shear, drifts and overturning moments in comparison to the results of nonlinear time history analysis. Furthermore, when the pile-foundation system is not considered in the nonlinear FE model, the damage severity at the mid-height of the tower is underestimated.

Keywords: airport traffic control tower, nonlinear time history analysis, seismic performance level.

1. INTRODUCTION

Air Traffic Control (ATC) towers are one of the most strategic and necessary buildings in each airport, as functionality of each airport directly depends on the operation of ATC towers (Roark et al., 2000). Seismic design and the performance of ATC towers are challenging matters for structure engineers. On the one hand, their significant role in the functionality of airports elevates their seismic performance level. On the other hand, lack of specific instructions and guidelines for seismic evaluation and design of ATC towers results in the misuse of existing building codes. It should be mentioned that, some building codes like ASCE 7-10 (American Society of Civil Engineering, 2010) considerer the seismic design of non-building structures, however due to the unique characteristics of ATC towers; their dynamic behavior does not completely comply with the characteristics of non-building structures presented in building codes.

Lateral resistance system of ATC towers often consists of only concrete shear walls. As a result, upper story drift can hardly comply with recommended safe values. In addition, having only one lateral resistance system is in contradiction to building codes recommendations in which tall and important structures should have a dual lateral resistance system.

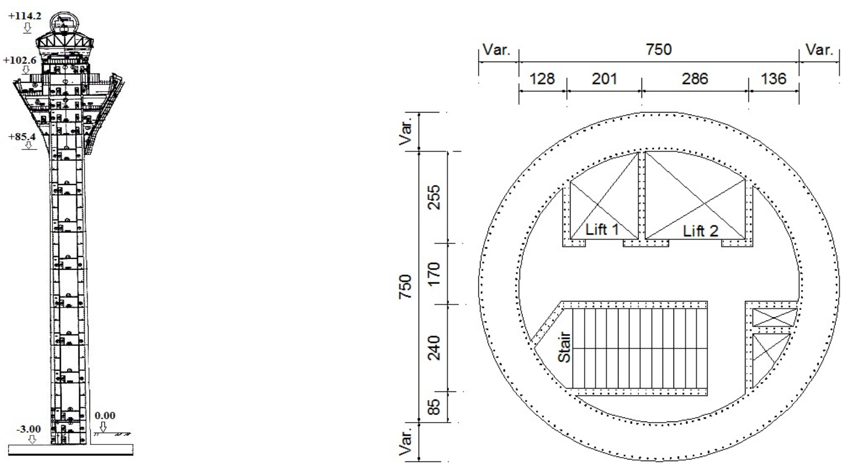
It should be noted that, in addition to structural damage, special attention must be paid to non-structural elements, because non-structural damage can easily halt the functionality of ATC towers.

This paper aims to investigate the seismic response of Kuala Lumpur International Airport (KLIA) traffic control tower using linear and non-linear analysis. At first, based on structural drawings Finite Element (FE) models were created. Then, the ability of created FE models in representing modal properties was confirmed through on-site measurement of natural frequencies. Then, linear and nonlinear analyses were carried out. By comparing the obtained results from linear and nonlinear analyses, the ability of conventional method for seismic evaluation of the ATC tower was investigated.

In addition, the effect of considering flexibility of pile-foundation system on the dynamic response of the tower was addressed.

2. ATC TOWER OF KLIA

KLIA tower is one of the tallest airport traffic control towers in the world. It rises about 120 meters above foundation level. Figure 1 depicts the longitudinal and cross-sections of this tower. The tower relies on a circular concrete core to carry gravity loads. In addition, the same system provides lateral stiffness and strength for the tower. The concrete core starts at foundation level and continues up to 106.2 m height above foundation level. Concrete core thickness varies along the height of the tower from 1 m to 0.6 m. The reinforcement ratio in concrete core varies from 0.9% to 2%. The tower settles on a 3 m thick circular mat foundation that is 24.8 m in diameter. Moreover, 57 cast-in-place concrete piles support the mat foundation. Piles are 30 m in length and are 1m in diameter. The observation room is located at the top of the tower. Utility and office rooms are located between 88.4 m and 105.6 m in height above foundation level. The gravity load of these areas is transferred to the concert core through inclined steel columns, radial and circumferential steel beams. Moreover, interior concrete walls with a constant thickness of 20cm and rebar ratio of 0.002, support lifts and staircase.



(a) (b)

Figure 1. (a) Longitudinal section of the tower. (b) Cross-section of the tower.

3. FINITE ELEMENT MODELS

In this study, two FE models are created. The first model is a linear FE model and is employed to run modal analysis. The second model is a nonlinear FE model and is used for running nonlinear time history analysis. Linear and nonlinear FE models are created using ETABS (Computers and Structure, Inc. 2007a) and Perform 3D (Computers and Structures, Inc., 2006) software, respectively. Figure 2 shows the created FE models. It should be mentioned that, obtained results from both softwares were in close agreement. All openings in the concrete core and slabs are considered in the FE models. Piles and foundation are included in both linear and non-linear FE models. In order to investigate the effect of pile-foundation flexibility on the dynamic characteristics of the tower, two more fixed-base linear and nonlinear FE models are created and shown in Figure 2. In the linear FE models, beams and columns are modeled by frame element while concrete shear walls are taken into account by shell elements. Table 1 represents considered material properties for linear FE models.

In this study, fiber elements are employed to consider the inelastic behavior of concrete shear walls. This approach has been used by other researchers to model the inelastic behavior of concrete structures (Chen et al., 2010). In this method, nonlinear materials are assigned to fiber elements. Table 2 shows nonlinear material properties for the employed concrete and rebars. It should be mentioned that, the nonlinear behavior of beams and columns are modeled using plastic hinges in accordance with Federal Emergency Management Agency (FEMA 356, 2000) provisions. Nonlinear moment-curvature and nonlinear axial force-axial deformation behavior of shell elements are considered in the FE models. In addition, since shear demand was lower than the capacity of concrete walls, elastic behavior is considered for shear deformation of the concrete walls. P-Δ effect is considered for all the FE models. Furthermore, it is assumed that, foundation and piles remain elastic when the tower is under seismic action. Moreover, Winkler's spring is employed in the FE models to account for interaction among soil, foundation and the tower. Stiffness of the springs is calculated based on the results obtained from geotechnical investigations.

Table 1. Modeling parameters of concrete and rebars for linear analysis.

Material	Modulus of Elasticity	Poisson's Ratio	Compressive Strength	Yield Strength
Concrete	3020 KN/cm^2	0.2	4 KN/cm^2	-
Rebars	19994 KN/cm^2	0.3	-	40 KN/cm^2

Table 2. Modeling parameters of concrete and rebars for nonlinear analysis.

Material	Ultimate Comparison Strain	Ultimate Tensile Strain
Concrete	0.005	-
Rebars	0.02	0.05

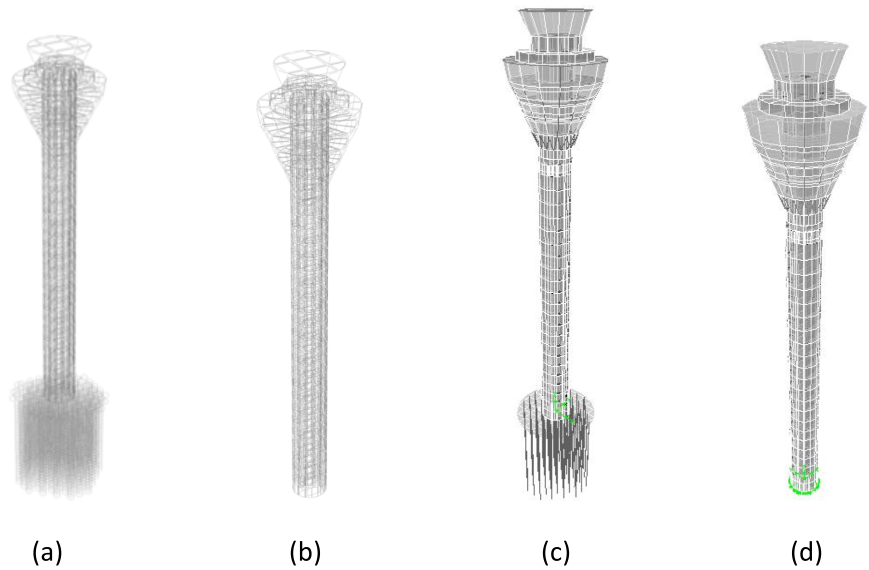


Figure 2. Finite element models. (a) Nonlinear FE model considering piles. (b) Nonlinear fixed base FE model. (c) Linear FE model considering piles. (d) Linear fixed base FE model.

4. MODAL ANALYSIS

In order to confirm the ability of created FE models in representing the modal properties of the tower, the first four natural frequencies in both principal directions were measured on-site by using recorded response accelerations of ambient vibrations. Table 3 compares measured natural frequencies with those obtained through FE models. As can be seen, there is close agreement between the measured natural frequencies and those obtained by FE models when the pile-foundation system is considered in the FE model. Moreover, it can be seen that when supports are assumed to be fixed, the obtained natural frequencies from FE model are bigger than those measured on-site.

The first three flexural mode shapes of the tower are presented in Figure 3. It should be mentioned that, obtained modal properties from both linear and nonlinear FE models are in close agreement.

Table 3. Obtained natural frequencies of the tower.

	Natural frequencies(Hz) -X direction				Natural frequencies(Hz) -Y direction			
	1st mode	2nd mode	3rd mode	4th mode	1st mode	2nd mode	3rd mode	4th mode
On-site	0.35	0.99	2.27	2.78	0.36	1	2.22	2.78
FE- piles	0.34	1.01	2.33	2.78	0.35	1.02	2.22	2.85
FE-Fixed	0.39	1.64	2.5	3.13	0.39	1.72	2.56	5.88

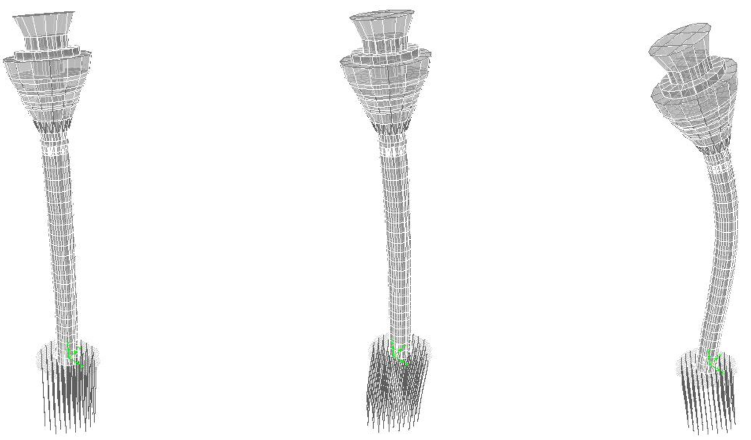

(a) (b) (c)

Figure 3. Mode shapes. (a) First flexural mode. (b) Second flexural mode. (c) Third flexural mode.

5. LINEAR ANALYSES

5.1. Equivalent lateral load

Equivalent static analysis is a simple and well-known method to calculate seismic base shear. However, according to building codes (Eurocode 8-1, 2004; ASCE 7-10, 2010) application of this method is restricted to short and regular structures where they respond in their fundamental mode in each principal direction. This philosophy is not consistent with the dynamic response of the ATC tower. In this study, the calculated seismic load based on this approach is compared with the results of other methods.

In equivalent static analysis, seismic base shear, V, is calculated using the following equation:

$$V=CW \quad (1)$$

Where, C, stands for seismic response coefficient and W is the effective seismic weight. Each building code has its own specific equation to calculate the C value. Here the equation suggested by ASCE 7-10 (ASCE 7-10, 2010) is employed as follows:

$$C_S=S_{DS}/(R/I_e) \quad (2)$$

Where; S_{DS} denotes the design spectral response acceleration in the short period range, R stands for response modification factor and I_e is the importance factor. Figure 4 shows the site-specific response spectrum of the tower in which S_{DS}=0.59g. Moreover, since the ATC towers must maintain their functionality after the occurrence of earthquakes, according to the building code, the importance factor is I_e=1.5.

Selecting an appropriate R value is a challenging matter for ATC towers. ASCE 7-10 (ASCE 7-10, 2010) suggests R=2 for chimneys and Inverted pendulum structures. On the other hand, this building code proposes R=4 for ordinary reinforced concrete shear walls. In this study, for the tower under consideration, it is assumed that, R=4. Authors believe that, due to complex seismic behavior of ATC towers, comprehensive studies need to be carried out to determine appropriate R values for different types of ATC towers. This issue is beyond the scope of this research and the considered R value in this study is only for comparing results of different approaches.

The vertical distribution of seismic load, $F_{i,}$ is determined from following equation:

$$F_i= (w_i\, h^k{}_i)/ (\Sigma\ w_i\, h_i{}^k) \qquad (3)$$

Where; w_i is the portion of total effective seismic weight at level i , h is the height from the base to level i, and k is an exponent related to the structure period. When the fundamental period of a structure is more than 2.5 sec, the value of k equals to 2. The obtained results from equivalent static analysis are presented in the next section in order to be compared with the results of modal response spectrum analysis.

5.2. Modal response spectrum analysis

Since this method considers the effect of higher modes, it is allowed to apply this method for tall structures. Figure 4 depicts applied 5% damped site-specific design spectrum with an exceedance probability of 10% in 50 years. In this study, complete quadratic combination (CQC) method is employed to combine results of mode shapes. Furthermore, results of modal response spectrum analysis are scaled by multiplying them to (I_e/R). In addition, the modal response spectrum analysis is implemented by using the first eight vibration modes that leads to more than 90% of mass participating ratio in both principal directions.

Figures 5 to 7 compare the obtained results from equivalent static analysis with those obtained from modal response spectrum analysis. It can be seen from Figure 5 that, the seismic load distribution along the height of the tower differs significantly for both cases and only for the last 20 m height of the tower, the lateral loads follow a similar pattern. Furthermore, for the first 20 m height of the tower, modal response spectrum analysis predicts more shear force than equivalent static approach whereas form this level up to 90 m height, shear force demand for equivalent static approach is considerably more than those obtained by modal response spectrum analysis. Moreover, in response spectrum analysis, after linear decreases in shear force from foundation level up to 60 m height, a sudden increase in shear force occurs which it continues up to 100 m in height. As can be seen in Figure 6, the calculated overturning moment via equivalent static approach is significantly greater than values obtained by modal response spectrum analysis. Figure 7 depicts the lateral displacements of the tower due to horizontal loads shown in Figure 5. It can be seen that similar to overturning moments, equivalent static procedure overestimates the displacements when it is compared with the results of modal response spectrum analysis.

Drift values are calculated and presented in Figure 8 for the head of the tower where utility and office rooms are located. According to SEAOC vision 2000 Committee (SEAOC, 1995), for fully functional performance level, the transient drift values should be less than 0.2%. In addition, for operational and life safety performance levels the transient drift values should be less than 0.5 and 1.5%, respectively. From Figure 8 it can be seen that, modal response spectrum analysis estimates a fully functional performance level for the tower while equivalent static approach predicts a life safety level. It can be observed that, for the tower under consideration, the equivalent static procedure generally overestimates the seismic demand when they are compared with the results of modal response spectrum analysis. The only exception is the calculated base shear at the foundation level.

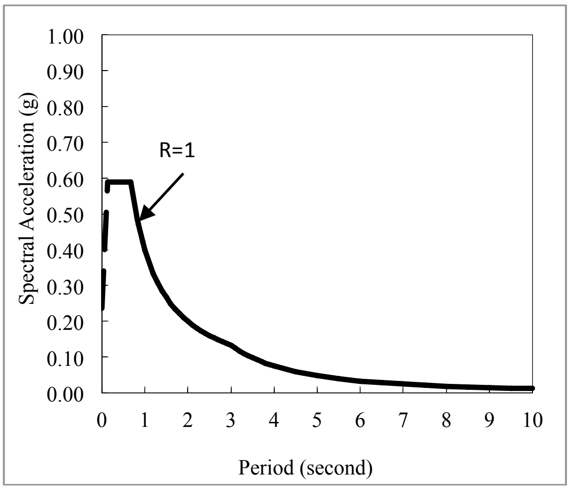


Figure 4. 5% damped site specific response spectrum with exceedance probability of 10% in 50 years.

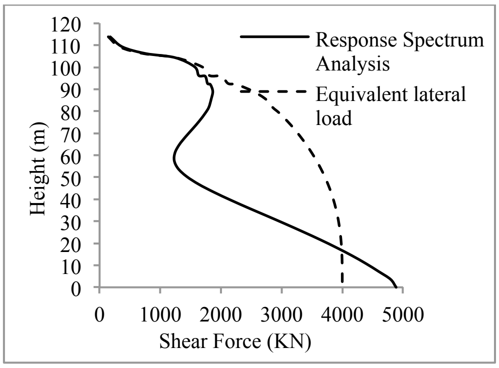


Figure 5. Shear force distribution along the height of the tower.

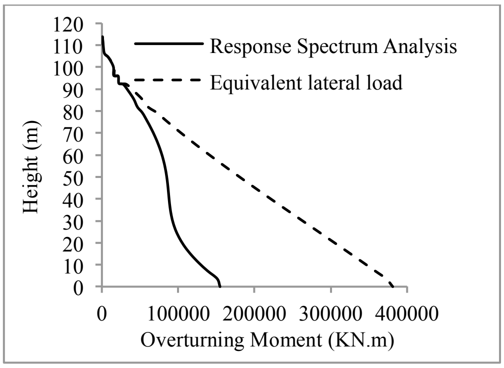


Figure 6. Overturning moment along the height of the tower.

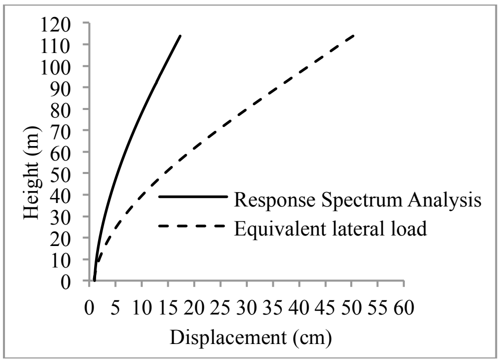


Figure 7. Displacement along the height of the tower.

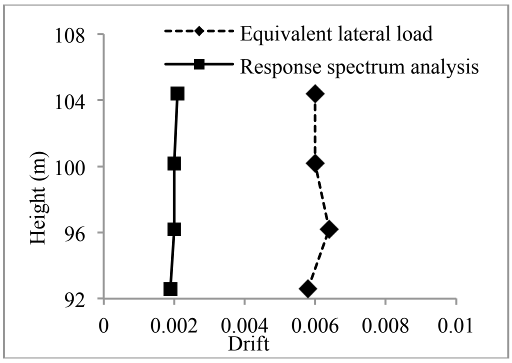


Figure 8. Drift values for utility stories located at the top of the tower.

6. NONLINEAR ANALYSIS

6.1. Nonlinear Time History Analysis

Nonlinear time history analysis is known as the most accurate procedure for representing inelastic seismic behavior of structures. In this study, several nonlinear time history analyses are carried out to investigate the seismic response of the KLIA traffic control tower more accurately. Due to lack of available real earthquake records for Malaysia, seven natural accelerograms were selected from Pacific Earthquake Engineering Research Centre (PEER, 2009) database. Table 4 shows the selected records. Response spectra of the selected records are shown in Figure 9 along with the mean spectrum and site-specific hazard spectrum for 10% probability of exceedance in 50 years. Earthquake records need to be scaled before use in nonlinear time history analysis. In this study, the proposed method by Eurocode (Eurocode 8-1, 2004) is adopted for scaling the selected records. The scaled response spectra based on this approach can be seen in Figure 10.

Figures 11 to 13 depict envelop of shear force, overturning moments and displacements along the height of the tower, respectively. From Figure 11, it can be seen that the mean value of base shear demand, at foundation level is around 25000 KN that is approximately 5 times more than those obtained by linear analyses. This clearly represents a remarkable difference between results of nonlinear time history analysis and those obtained by other approaches. Figure 12 shows an increase in overturning moments from 40 m to 70 m height of the tower. For this range, the value of overturning moment in equivalent static analysis decreases linearly. On the other hand, results of response spectrum analysis follow the same pattern that nonlinear time history analysis represents. However, the value of overturning moments along the height of the tower is significantly smaller than those obtained by nonlinear time history analysis.

Considering the deflection amplification factor proposed by ASCE 7-10 (2010), Cd=4, the maximum displacements of the tower are obtained 65 cm and 186 cm for modal response spectrum analysis and equivalent static analysis, respectively. On the other hand, as Figure 13 shows, the mean maximum displacement obtained from nonlinear time history analysis is 66 cm. So, in comparison to the nonlinear time history analysis, modal response spectrum analysis can estimate the maximum displacement of the tower accurately, while equivalent static approach overestimates it.

Figure 14 depicts obtained mean and maximum drift values for utility stories of the tower. Considering SEAOC Vision 2000 committee performance levels, the tower does not satisfy operational performance level and falls into life safety level. Furthermore, obtained drift values are significantly more than those obtained by linear analyses.

Figures 15 and 16 show obtained tensile and compressive strain values along the height of the tower. As these figures show, a considerable decrease in the strain values occur in the second segment while a remarkable increase occurs in the third and fourth segments. This means that, the damage concentrates at the foundation level as well as 40 m to 70 m height of the tower.

In order to investigate to what extent the support conditions effect the seismic behavior of the tower, tensile and compressive strains along the height of the tower are compared for the fixed supports against pile-foundation system. Results are presented in Figures 17 and 18. These figures show that both cases provide a similar damage pattern. However, when supports are assumed fixed, a significant decrease in tensile and compressive strains occurs at the third and forth segments when compared with results of a pile-foundation system.

Table 4. Selected earthquake records.

No.	Earthquake	Year	Duration(sec.)	PGA (g)	PGV(cm/s)	PGD(cm)
1	Kobe	1995	41	0.345	27.6	9.6
2	Loma Prieta	1989	30	0.268	22	5.15
3	Landers	1992	56	0.097	5.7	2.27

4	Morgan Hill	1984	36	0.068	3.9	0.63
5	Whittier Narrows	1987	37	0.051	2.4	0.48
6	Kocaeli	1999	49	0.249	40	30.08
7	Borrego	1942	33	0.068	3.9	1.37

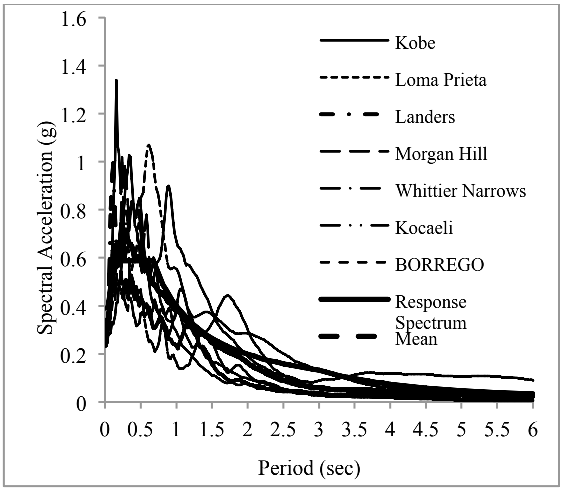


Figure 9. Response spectra of selected earthquake records along with the mean spectrum and site specific hazard spectrum for 10% probability of exceedance in 50 years.

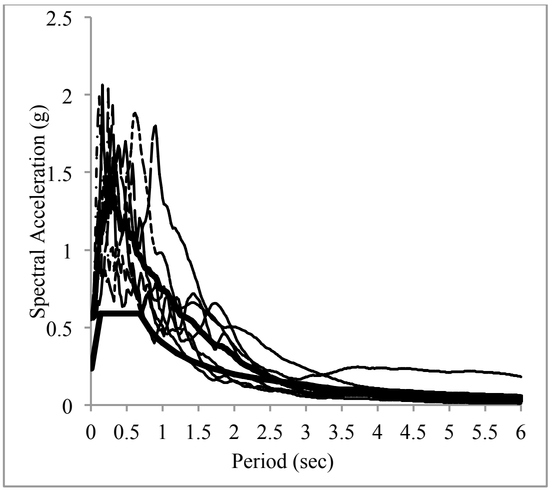


Figure 10. Scaled response spectra of selected earthquake records along with the mean spectrum and site specific hazard spectrum for 10% probability of exceedance in 50 years.

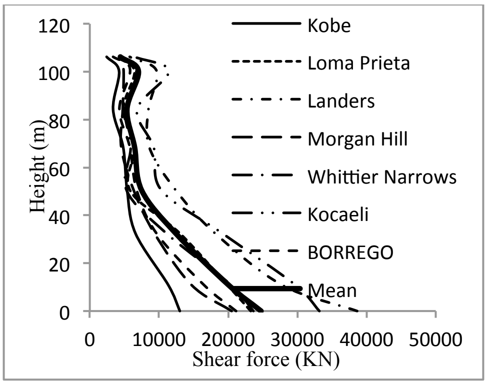


Figure 11. Envelope of shear force distribution along the height of the tower considering different earthquake records.

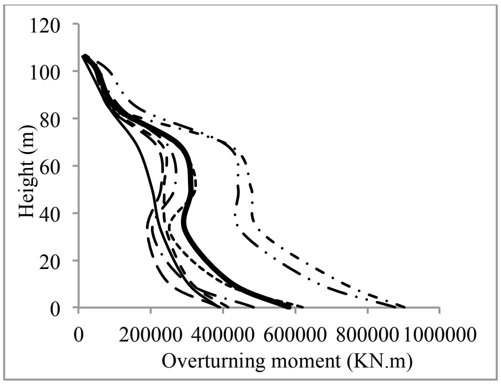


Figure 12. Envelop of overturning moment along the height of the tower considering different earthquake records.

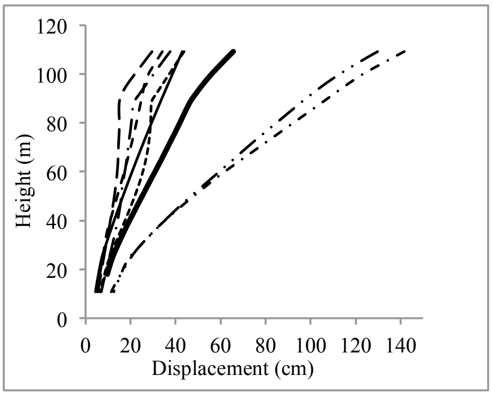


Figure 13. Envelop of displacement along the height of the tower considering different earthquake records.

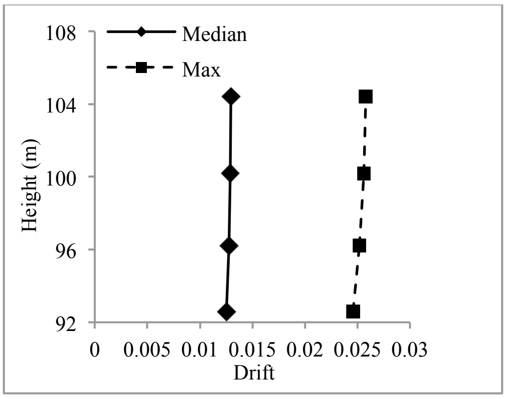


Figure 14. Mean and maximum drift values for utility stories located at the top of the tower.

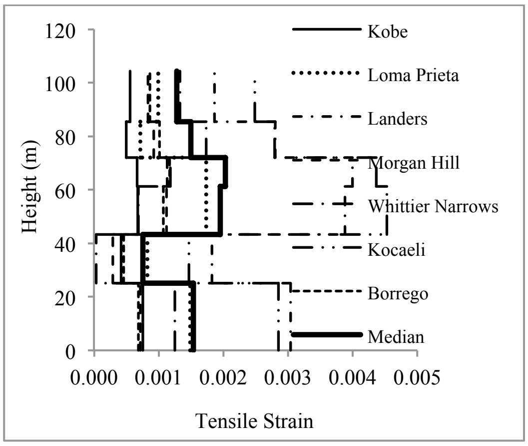


Figure 15. Envelop of tensile strain along the height of the tower considering pile-foundation system.

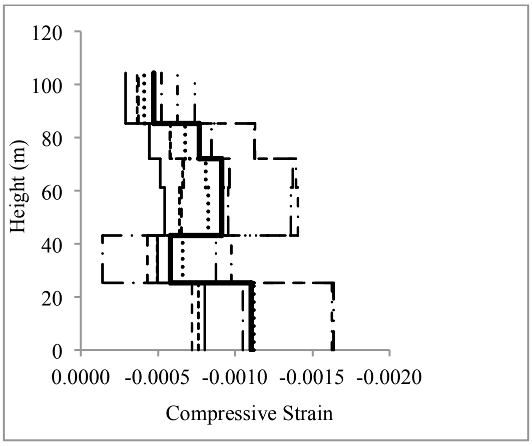


Figure 16. Envelop of compressive strain along the height of the tower considering pile-foundation system.

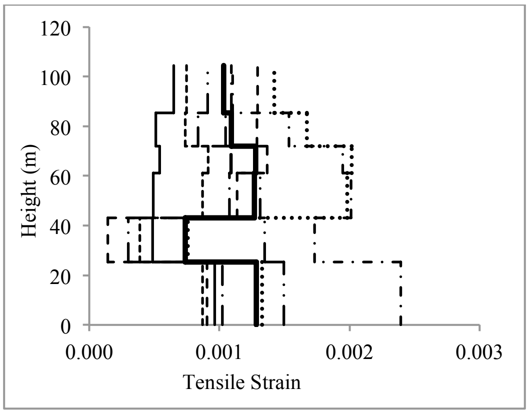


Figure 17. Envelop of tensile strain along the height of the tower considering fixed supports.

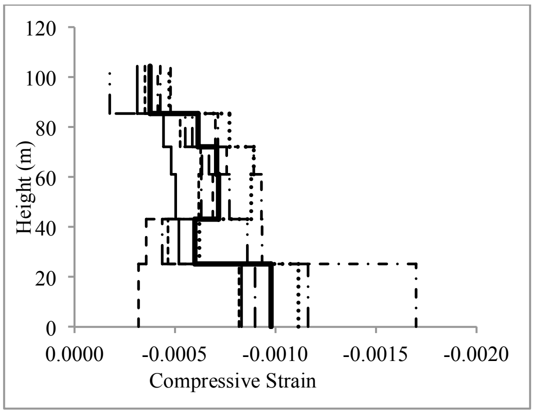


Figure 18. Envelop of compressive strain along the height of the tower considering fixed supports.

7. CONCLUSIONS

Seismic performance of Kuala Lumpur international air traffic control tower is evaluated using equivalent static analysis, modal response spectrum analysis and nonlinear time history analysis. Obtained results including shear force, overturning moment, displacement, drift ratio, tensile and compressive strains are compared with each other.

In comparison to the results of nonlinear time history analysis, linear analyses underestimate shear force and overturning moments. In addition, they do not provide a similar pattern for shear force and overturning moments along the height of the tower. Obtained drift values from modal response spectrum analysis shows that the tower can maintain fully functional seismic performance level. On the other hand, equivalent static method predicts life safety performance level for the tower. The same performance level is obtained through nonlinear time history analysis. However, obtained drift values are considerably more than equivalent static analysis.

Tensile and compressive strain values obtained from nonlinear time history analysis indicate that damage tends to concentrate at the mid-height of the tower as well as the foundation level. In comparison with nonlinear time history analysis, modal response spectrum analysis accurately predicts the maximum displacement of the tower. However, equivalent static analysis remarkably overestimates it.

Fixed-base FE model provides similar tensile and compressive strain distribution pattern to that of FE model considering pile-foundation system. However, when supports are assumed fixed the damage severity of mid-height of the tower is underestimated.

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