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SEISMIC BEHAVIOUR OF 4-LEGGED SELF-SUPPORTING TELECOMMUNICATION TOWERS CONSIDERING EARTHQUAKE EFFECTS IN MALAYSIA

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Abstract: Malaysia's strategic location has spared it from major seismic activities. Nevertheless, plethora of far field tremors resulting from earthquakes in neighboring countries can be felt locally despite the fact that the epicenters of these earthquakes are mostly hundreds of kilometers away. With increasing frequency and intensity of late, this scenario has raised a major concern pertaining to the ability of the existing buildings and sensitive structures in the country to withstand earthquakes events in the future. Considering that no specific study has been contemplated with particular regards to the effect of earthquakes to telecommunications facilities in Malaysia, it is requisitely of prime significance to commence this study. The main objective of the study is to determine the seismic behavior and then evaluate the structural integrity of the existing telecommunication tower structures considering earthquake effects in Malaysia. The study covers the analysis of the four (4) legged self supporting steel telecommunication towers based on different types of seismic zones, peak ground accelerations (PGAs), and soils, using the SAP2000 finite element software, model development with reference to the International Building Code (IBC2000) and Euro Standards (EC8). Among the main components of the study are; data analysis, site visits, model development, analysis and structural evaluation. Linear static analysis involving joints displacement and base shear reactions with axial forces analysis are being carried out. In the end, the study has able to determine the seismic behavior experienced by the modeled towers on the differing type's conditions exposed on them. With the results it will enable the researcher to further proceed with the study including developing a complete maintenance assessment system which also includes the justification/determination of the types of damages inflicted to the towers as well as the severity of the damages.

Keywords: *Structural Integrity Telecommunication Towers, Earthquake Effects, Peak Ground Acceleration.*

1.0 Introduction

Telecommunication towers are categorised among the tallest man-made structures and can be found standing high on every parts of the globe with different heights and purposes. Communication needs during disaster are unique and critical. It becomes more crucial when disaster such as flood, typhoon, hurricane and earthquake events happened. In a major emergency caused by an earthquake it is likely that telephone lines may be down, other alarm and telecommunications facilities are adversely affected, and a vast increase in the work load imposed upon personnel and equipment in the control center. One distinguishing characteristics is the dramatic increase in the number of people who must make use and communicate among them. The malfunction of the communication facilities immediately after an earthquake that happens in other countries should be a lesson learned especially for telecommunication service providers.

Malaysia is situated on the southern edge of the Eurasian plate within the most two seismically active plate boundaries. Several possible active faults have been delineated and local earthquakes in the country appear to be related to some of them. Due to its strategic location, Malaysia is generally spared from any major active seismic activities. However, when earthquakes occur in neighboring countries, the effects can be felt locally. After the 2004 tsunami disaster that strikes Aceh, the government has taken early initiatives to look into the impact of earthquake events originating from our neighbours. Important buildings and sensitive structures such as telecommunication towers are among the most crucial to be looked upon. These specifically light and slender structures are particularly sensitive to the environmental loads to which they are subjected but also to ground movements. With the divergence of the high speed broad band projects initiated by the Malaysian government in mid 2008, more telecommunication towers are being and will be erected to cater for the country needs. Therefore the preservation of serviceable communication infrastructure as critical links of communication or post disaster networks is essential in the event of an earthquake disaster.

In this paper the effects of seismic behaviour acting on the 4-legged self-supporting steel towers have been studied based on detailed linear static analyses of six existing telecommunication towers.

2.0 Background

The safety issue of building structures including sensitive structures in Malaysia has always been of public concern and has been highlighted after the 2004 earthquake which triggered tsunami that caused a number of fatalities. The country too experienced direct impact by seismic waves emanating from earthquakes in Sumatra. As was mentioned in the Position Paper report (IEM, 2005) a seismic hazard assessment has been carried out in the country in 2004. The outcome indicates that a PGA of 50 gals (for 500 years return period) has been determined for ‘before Aceh’s event’ and PGA value of 100 gals has been ascertained for ‘after Aceh’s event’. It was also noted that the earthquake characteristic that has affected the structures here was of that with a long period of vibration. Preliminary study carried out by Adnan *et al*. (2006) on soil samples of five cities in the West coast of Peninsular Malaysia has shown that the average local soil amplification ranges from 1.4 to 3.6. Since the soil condition in many parts of the country is underlain of limestone bedrock the study implies that the local soil effect could not be neglected. Incident of sinkholes in ex-mining area with loose sand and tailings have occurred (Komoo, 2005). The geological predisposition was ‘ripe’ for the popping-up of sinkholes, and the earthquake tremor provided the ‘triggering’ factor.

Reports produced by the Japan Society of Civil Engineers (JSCE, 1995) stated that the main cause of damage to structures observed during the Great Hanshin earthquake on January 17, 1995 is due to their response to ground motions which are the loadings at the base. According to JSCE in order to evaluate the behavior of the structure under this type of loading condition, the principles of structural dynamics must be applied to determine the stresses and deflections, which are developed in the structure. Along with JSCE, the reports produced by the National Institute of Standards and Technology declared the same conclusion on damage caused by earthquake (NIST, 1995). From the past earthquakes events in year 2007 and 2009 that happen in neighboring countries, some buildings already experienced defects such as cracks. Though it is still considered not a major one for Malaysia, it is utmost important to perform a pre-earthquake assessment to evaluate their behavior under earthquake loads due to seismic ground motion. Could the existing structure in the country be able to withstand the tremor? This has been an unanswered question till today. According to Luin (2008), as for the ability of local structures in Malaysia to withstand such tremors, the effects would be minimal for low-rise building structures (up to four or five storeys), whereas for the inhabitants’ of high-rise buildings (up to 7 storeys and above), they may feel sideways movements of the structure in response to the tremors. Generally, these buildings would still be structurally sound.

McClure (1999) quoted a survey of the earthquake performance of communication structures that summarised documented reports of 16 instances of structural damage related to seven important earthquakes between 1949 and 1998, none of which were a direct threat to life safety. However, several towers may have been damaged or have become unserviceable without having collapsed or suffered damage visible from the ground during post earthquake inspections. Many strong earthquakes have happened since then and more damage has been reported as more telecommunication equipment is deployed worldwide. Indonesia, the country that lies in the Ring of Fire area has witnessed many of its telecommunication towers failed during earthquakes events. As mentioned by Smith (2007) the 1995 Kobe earthquake is a good example where communication facilities malfunction has given a big impact. This event was said to have prevented local governments from knowing the level and the scope of casualties caused by the disaster. According to Faridafshin *et al*. (2008) the preservation of serviceable communication infrastructure as critical links of communication or post disaster networks is essential in the event of an earthquake. According to Bai *et al*. (2010) numerical results indicate that seismic responses of transmission towers and power lines are amplified when considering the local site effect. This makes no difference for telecommunication towers to have experienced the same. In the 2011 Fukushima earthquake, communications were badly broken and knocked out where many residents are relying on the small number of surviving pay phones.

In the human society today the telecommunication structure are fundamental components of communication and post-disaster networks and their preservation in the case of not only a severe earthquake but also to locations experiencing the far fields' effect is essential. Telecommunication towers are typically tall structures whose function is to support elevated antennas for radio and television broadcasting, telecommunication and two-way radio systems. Therefore, immediate serviceability or even continuous function of first-aid-station infrastructure is of critically high priority in the case of a disaster. Because of their unique geometry, telecommunication towers are categorised as slender-tall multi-support structures. Due to that, they are intrinsically more sensitive to some physical characteristics of earthquakes which are ignored in seismic analysis of common self-supporting structures like buildings. In particular, the effect of the spatial variation of the excitation at ground support is one aspect which deserves attention. Amiri et.al (2004) confirmed that due to their vital role, the preservation of these telecommunication structures during natural disaster such as an earthquake is of utmost priority and hence their seismic performance should be properly evaluated.

Previous work performed by researches on transmission and telecommunication towers structures has proven that ground motion due to seismic waves do contribute some effects on these structures. The seismic behavior and response of telecommunication towers is very different from any building structures. In areas prone to earthquakes, the main issue for strategic telecommunication towers is their functionality during or immediately after an earthquake.

Early study by Konno et. al (1973) who performed on instrumented tower owned by Nippon Telegraph and Telecom (NTT) and mounted it on a building rooftop during the 1968 Off Tokaichi Earthquake, Japan. It was one of the first studies on the effects of earthquake loads on the lattice telecommunication towers. The objective of their studies was to obtain the mode shapes, the natural frequencies and the damping properties of such structures. It was observed that in some of the members the forces due to earthquake were greater than those of the wind. Later, their data were analysed and studied by Sato et. al (1984) on the input seismic force to be used for the design of appendages, particularly telecommunication towers and found that a maximum acceleration amplification of 4 at the rooftop was appropriate. Later Hiramatsu et. al (1989) reported the continuation of this investigation of the seismic response and in general their results also agreed with the earlier observations of Sato *et al*. A seismic response spectrum method for the analysis of secondary systems while considering the dynamic interactions between the primary and secondary structures was developed by Kanazawa et. al (2000). To evaluate their proposed method, they performed time-history analyses on a building-tower model consisting of a tower mounted on a single degree of freedom primary system.

Mikus (1994) performed the seismic response of six 3-legged self supporting telecommunication towers with heights ranges from 20 to 90 meters. The selected towers were numerically simulated as bare towers. In the analysis three accelerograms were considered as the real earthquake forces. He concluded that the lowest four modes of vibration would ascertain the sufficient precision. In addition also, he pointed out that the vertical component of earthquake-induced forces had no effects on the results. Galvez and McClure (1995) performed additional studies to introduce simplifying methods for the seismic analysis of telecommunication towers. They used 45 earthquake records and investigated on three different numerical models of 3 legged lattice steel towers with heights ranges from 90 to 121 meters. It was concluded that contribution of second and third transversal modes of vibration on the maximum acceleration at the top of the towers, depending on the tower type varies from 15% to 50%. The main disadvantage of Galvez and McClure method was the bilinear shape of the acceleration profile, which did not thoroughly include towers with different geometries. A modified method for the horizontal acceleration profile later was introduced by Khedr *et al* (1999) where for every specified tower a separate acceleration profile being obtained. Amiri (1997) performed work on seismic sensitivity on tall guyed telecommunication towers. The objective of his work is to propose some seismic sensitivity indicators for tall guyed masts which would help tower designers decide whether seismic effects are important and whether detailed dynamic analysis of the structure is required. The indicators proposed relate to the maximum base shear and the dynamic component of the axial force in the mast and guy cable tensions. Work done by Faridafshin et. al (2008) with computational modeling of the dynamic response of tall guyed masts under seismic loads has shown that different earthquakes records with diverse scenarios of motion may produce quite different responses in the towers. Also the towers showed sensitivity to asynchronous shaking of their ground supports. Besides that they also confirmed as the towers become taller, the sensitivity to asynchronous shaking is initiated on a stiffer soil.

Sackmann (1996) like most of other researchers have performed studies to obtain the fundamental frequencies of self supporting towers. In most of these studies especially in the field of seismic effects the researchers were focused on the 3 legged towers and few studies have been conducted on the 4 legged type towers. Due to the lacking of design provisions in codes for telecommunication tower structures, Amiri *et al*. (2007) has performed linear dynamic analysis for several towers in Iran to obtain the earthquake amplification factors for 4 legged self supporting towers. Assi *et al*. (2007) stated that in an unpublished report by Shiff (1999) from a survey conducted on earthquake performance of telecommunication towers it was concluded that tall broadcast towers and large building-supported microwave towers are the most vulnerable to earthquakes although none of these towers has been a direct threat to life safety during any event.

There are several codes or guidelines being used globally for these towers. Modern codes and standards such as International Code Council (ICC) 2000, National Research Council Canada (NRCC) 2005, ANSI/Electronic Industry Association (EIA)/ Telecommunication Industry Association (TIA) 2005 have recently addressed the seismic analysis of telecommunication towers on building rooftops by either proposing a simplified method for the estimation of seismic base shear forces. The TIA/EIA-222-G is aimed at the communication industry and has been developed by the Telecommunication Industry Association in association with the Electronic Industry Association to provide minimum criteria for specifying and designing steel towers and antennae support structures. This standard applies to all classes of communication service such as: AM CATV, FM, Microwave, Cellular, TV, UHF, VHR etc. It is a recognised standard by the major building codes (UBC and IBC) and therefore can typically be used in most jurisdictions. The wind loads resulting from the antennae and support platforms are due to their relatively large, concentrated areas. Due to the nature of the equipment, the overall deflection, twist and sway of the structure are a concern. Many antennae especially microwave dishes, operate on a line of sight principal with very narrow beam widths. A relatively small deflection of the tower can seriously degrade or interrupt the performance of the communication system.

3.0 Scope of Research

In this study the focus had been narrowed down to only on structural related issues for assessment on four legged self-supporting steel towers. Six (6) steel telecommunication towers of different height categories have been selected for this purpose. The research work includes investigating, accessing, analysing and evaluating, including modeling the selected towers located in various seismic zones in the country. The categories of towers are tabulated in Table 1.

Table 1: Heights of Towers for Modeling and Analysis

Tower Categories	Height of Tower (meter)
Lower Rise (< 19.81 meter)	Nil (no tower in this category)
Medium Rise (19.81 ≤ H ≤ 73.15)	30 45
High Rise (H> 73.15)	90 120 140

Location of towers is selected in the various seismic zones as indicated in the CIDB 500 Years Return Period map for Peninsular and the PWD 500 Years Return Period for East Malaysia map. The zones are Zone 1, Zone 2A and Zone 2B. Refer Table 2 for summary of seismic zones in Malaysia.

Table 2: Seismic Zones in Malaysia

Peak Ground Acceleration (gals)	Seismic Zone	Seismic Zone Factor (Z)
0 - 40	0	0.0
41 - 80	1	0.075
81 - 100	2A	0.15
101 - 150	2B	0.20
151 - 300	3	0.30
301 - 500	4	0.40

Various values of peak ground acceleration have been selected for the purpose of analysis as shown in Table 2. Since the study is carried out in the 500 year return period, three (3) ground accelerations in Zone 1 and three (3) in Zone 2 are selected as tabulated in Table 3.

Table 3: Peak Ground Acceleration for Tower Analysis

No	Seismic Zone	Peak Ground Acceleration (gals)
1	Zone 1	0.04
2	Zone 1	0.06
3	Zone 1	0.08
4	Zone 2	0.08
5	Zone 2	0.10
6	Zone 2	0.12

Besides that various ground type has been selected for each seismic zone for all the towers. These ground types that falls under the normal Malaysian condition are considered and being used in the analysis work where the ground condition that has been applied in Euro Standards (EC8) and International Building Code (IBC) are referred. The ground types are ground Type A, B, C and D. For this study, two numbers of 4 legged self supporting steel tower from medium rise category and 4 numbers from high rise category are selected. In total there are be six (6) numbers of towers in the respective category to be modeled in four (4) types of ground conditions and analysed on the different seismic zones.

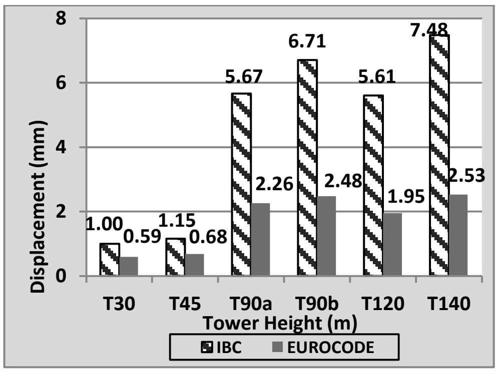
3.1 Results of Analyses

Analysis that has been done on the selected towers is using two types of codes, namely IBC and EC8. Analysis involving Joint Displacements and Base Shear has been carried out for all the modeled towers. Axial forces experienced by the structures too have been obtained. There are two zones involved i.e. Zone 1 and Zone 2, while four ground types involved i.e. ground type A, type B, type C and type D. For zone 1 there are three types of peak ground acceleration being adopted, namely 0.04 gals, 0.06gals and 0.08 gals. For zone 2, also three types of peak ground acceleration are used for the analysis on all the ground types; namely 0.08gals, 0.10 gals and 0.12 gals. Results for all the tower analysis are described in the following paragraphs.

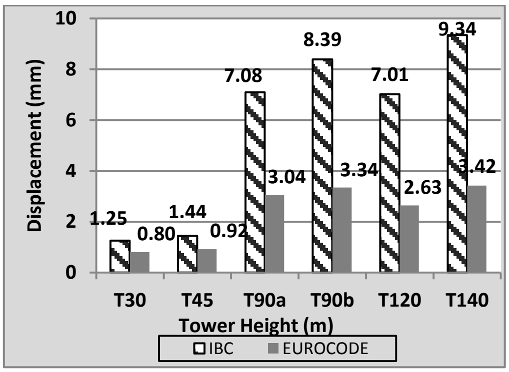
3.1.1 Joint Displacements Analysis

For joint displacement, there are twenty four (24) analysis have been done. Results are shown in Figure 1 to Figure 6

(a) Zone 1- Acceleration 0.04 gals

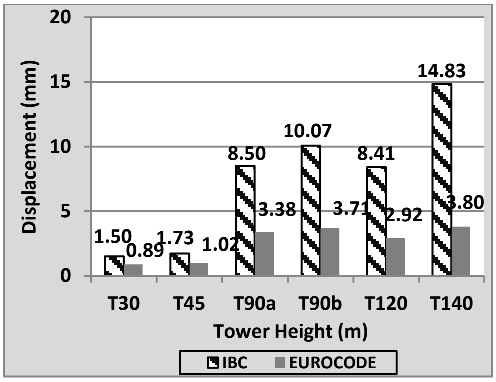


(a)
Ground Type A

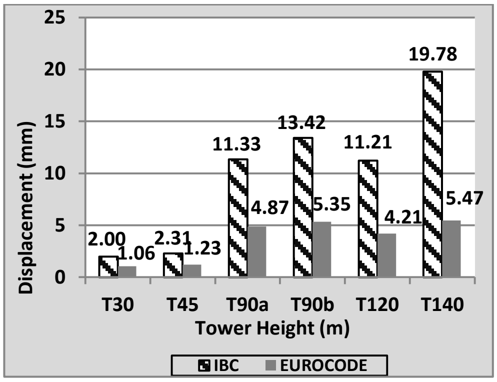


(b)
Ground Type B

Figure 1(a): Joint displacement for towers in Zone 1, 0.04 gals



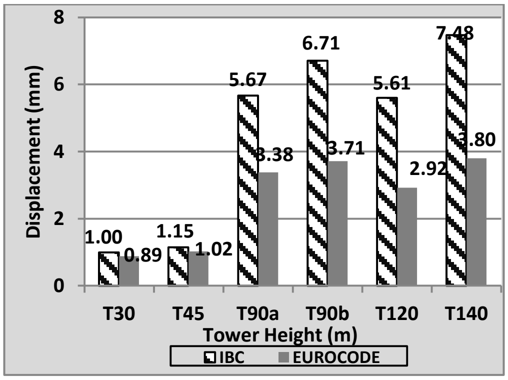
(c)
Ground Type C



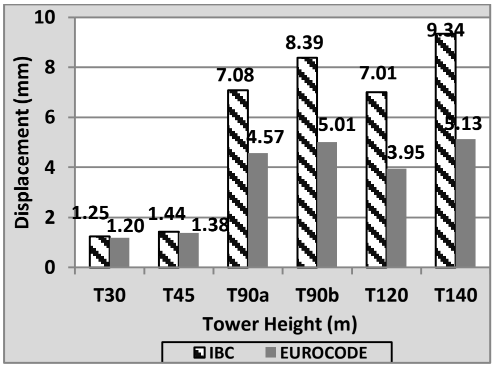
(d)
Ground Type D

Figure 1(b): Joint displacement for towers in Zone 1, 0.04 gals

(b) Zone 1- Acceleration 0.06 gals

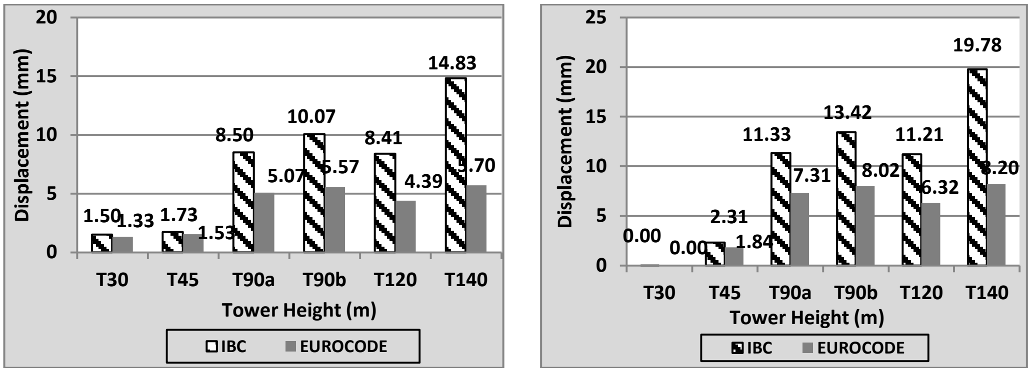


(a)
Ground Type A



(b)
Ground Type B

Figure 2(a): Joint displacement for towers in Zone 1, 0.06 gals

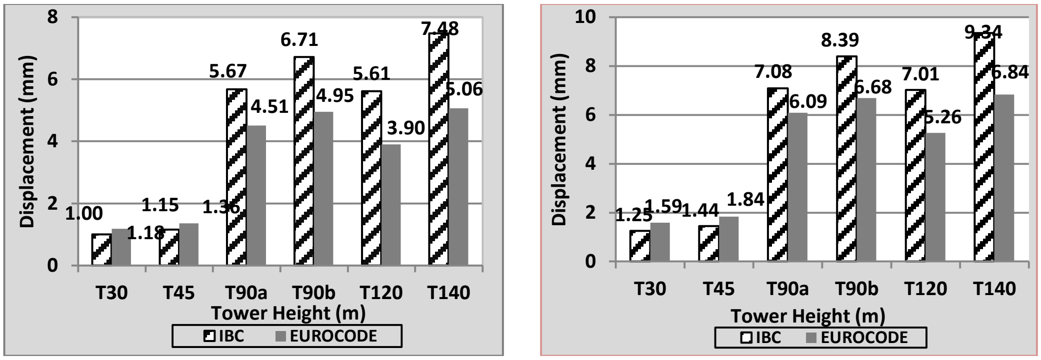


(c)
Ground Type C

(d)
Ground Type D

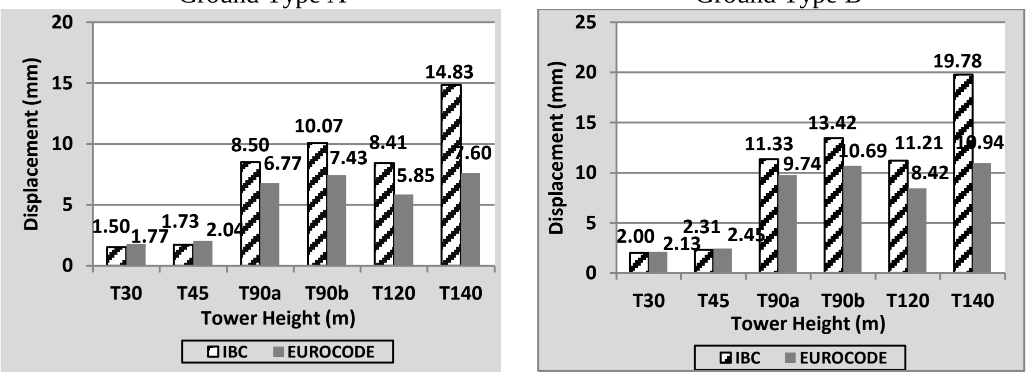
Figure 2(b): Joint displacement for towers in Zone 1, 0.06 gals

(c) Zone 1- Acceleration 0.08 gals



(a)
Ground Type A

(b)
Ground Type B

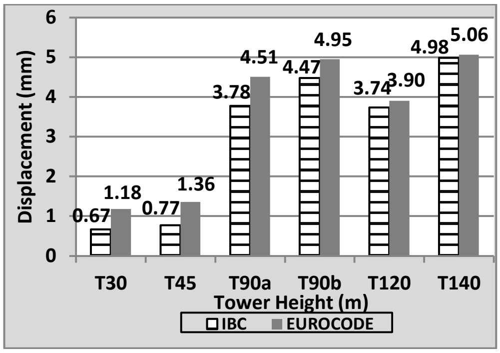


(c)
Ground Type C

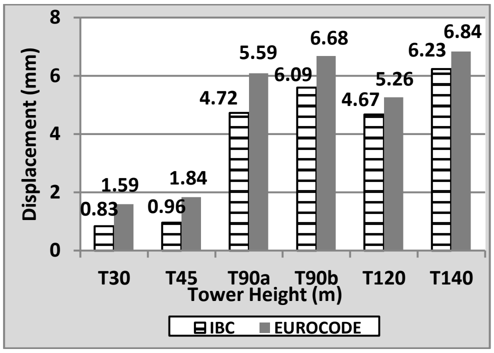
(d)
Ground Type D

Figure 3: Joint displacement for towers in Zone 1, 0.08 gals

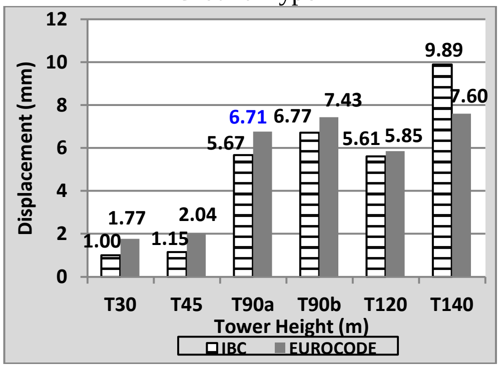
(d) Zone 2- Acceleration 0.08 gals



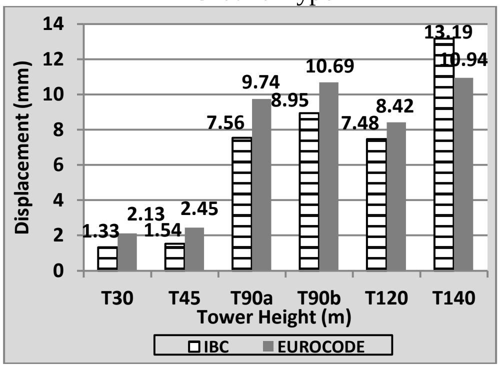
(a)
Ground Type A



(b)
Ground Type B



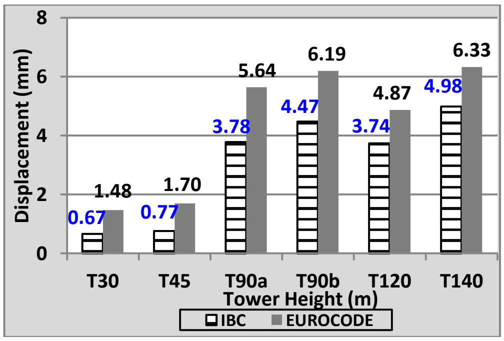
(c)
Ground Type C



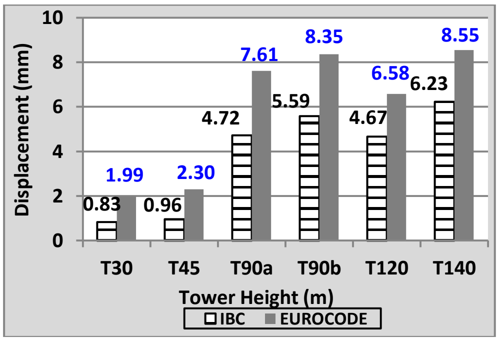
(d)
Ground Type D

Figure 4: Joint displacement for towers in Zone 2, 0.08 gals

(e) Zone 2- Acceleration 0.10 gals

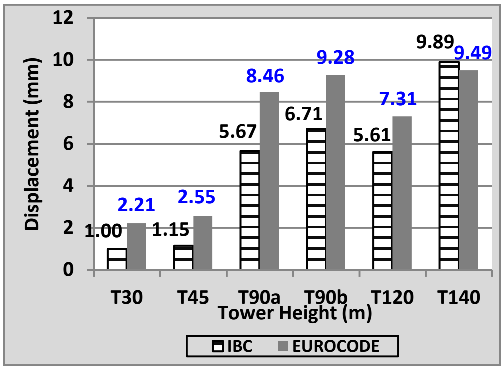


(a)
Ground Type A

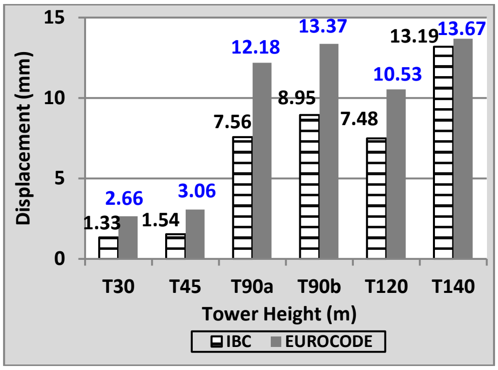


(b)
Ground Type B

Figure 5(a): Joint displacement for towers in Zone 2, 0.10 gals



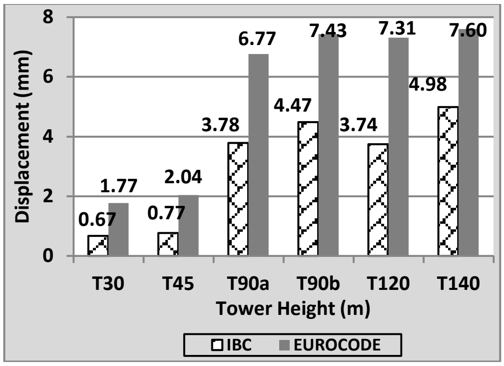
(c)
Ground Type C



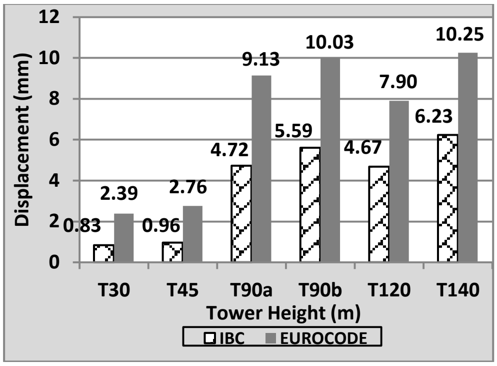
(d)
Ground Type D

Figure 5(b): Joint displacement for towers in Zone 2, 0.10 gals

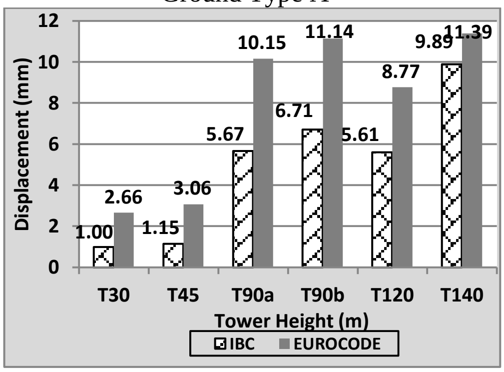
(f) Zone 2- Acceleration 0.12 gals



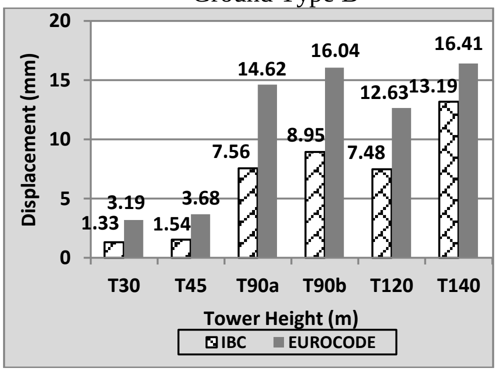
(a)
Ground Type A



(b)
Ground Type B



(c)
Ground Type C



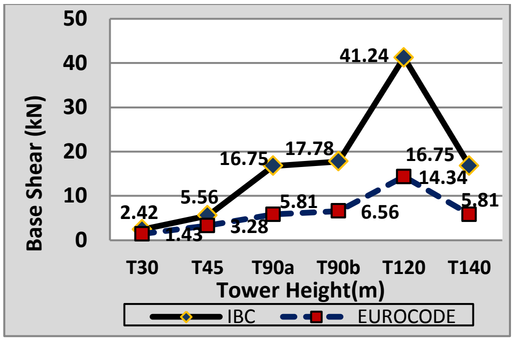
(d)
Ground Type D

Figure 6: Joint displacement for towers in Zone 2, 0.12 gals

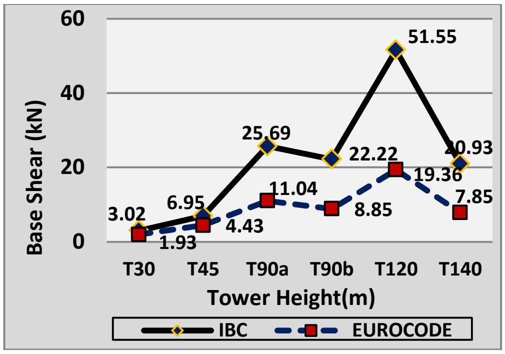
3.1.2 Base Shear Reaction Analysis

For base shear reactions, there are also 24 types of analysis that has been done in the different types of soil conditions and peak ground acceleration. Results are shown in Figure 7 to Figure 12

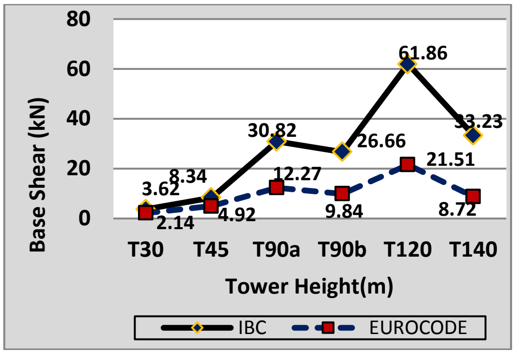
(a) Zone 1- Acceleration 0.04gals



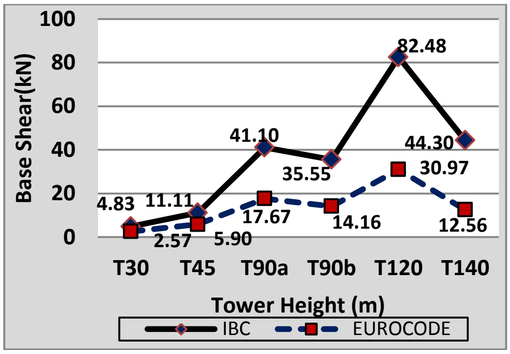
(a)
Ground Type A



(b)
Ground Type B



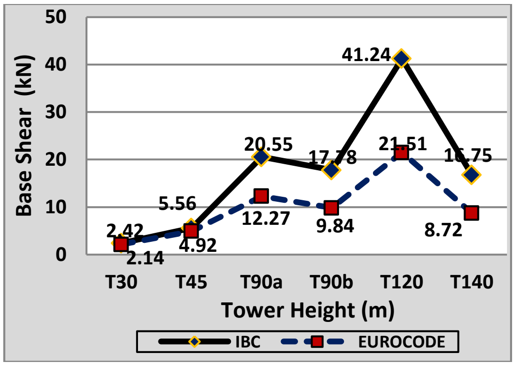
(c)
Ground Type C



(d)
Ground Type D

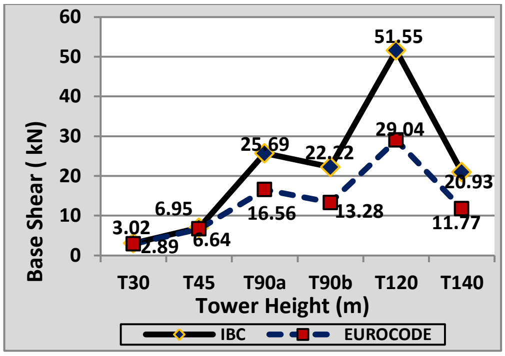
Figure 7: Base reactions for towers in Zone 1, 0.04 gals

(b) Zone 1- Acceleration 0.06gals



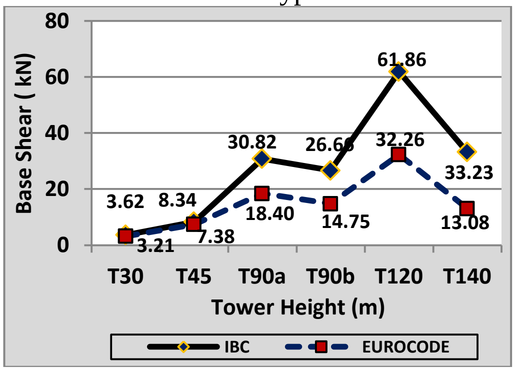
(a)

Ground Type A



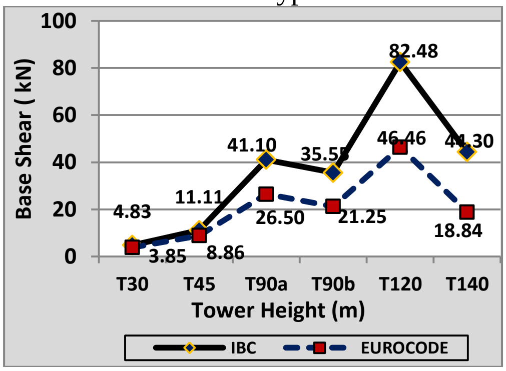
(b)

Ground Type B



(c)

Ground Type C

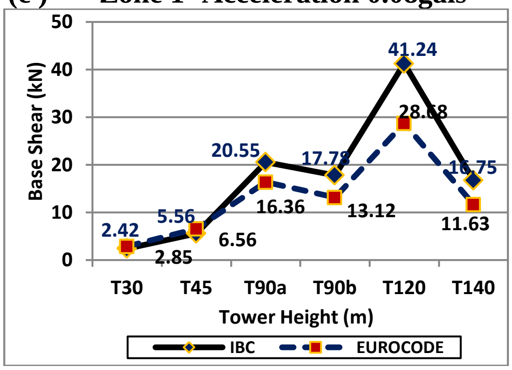


(d)

Ground Type D

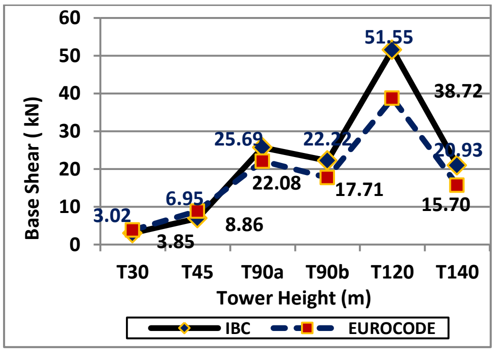
Figure 8: Base reactions for towers in Zone 1, 0.06 gals

(c) Zone 1- Acceleration 0.08gals



(a)

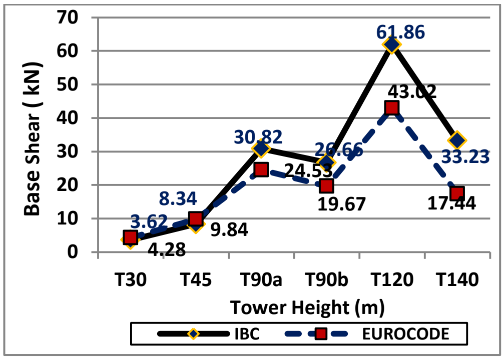
Ground Type A



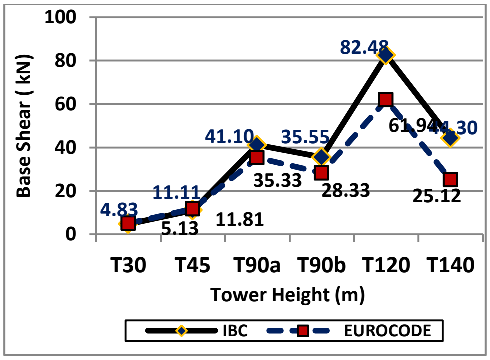
(b)

Ground Type B

Figure 9(a): Base Reactions of Towers in Zone 1, 0.08 gals



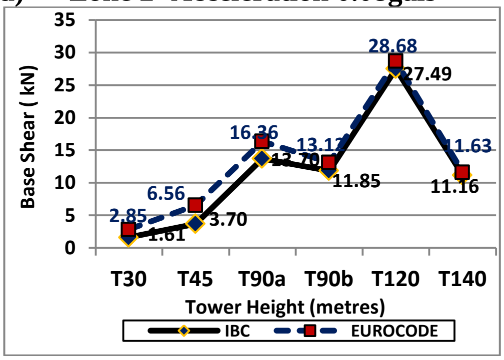
(c)
Ground Type C



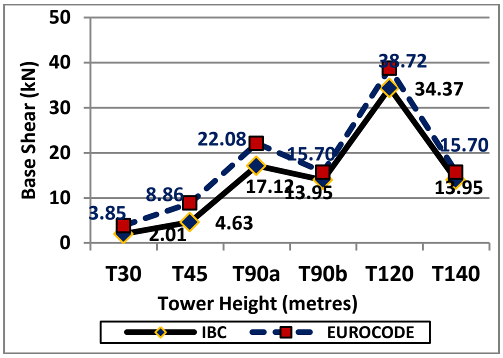
(d)
Ground Type D

Figure 9(b): Base Reactions of Towers in Zone 1, 0.08 gals

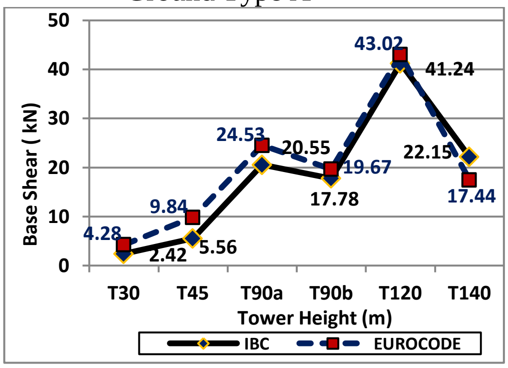
(d) Zone 2- Acceleration 0.08gals



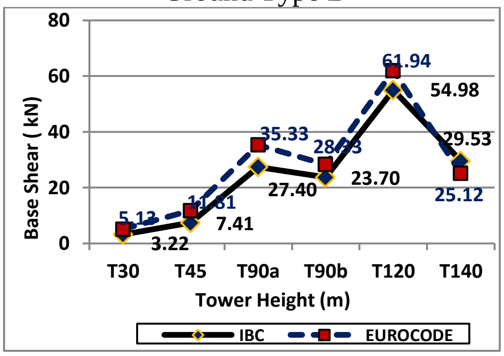
(a)
Ground Type A



(b)
Ground Type B



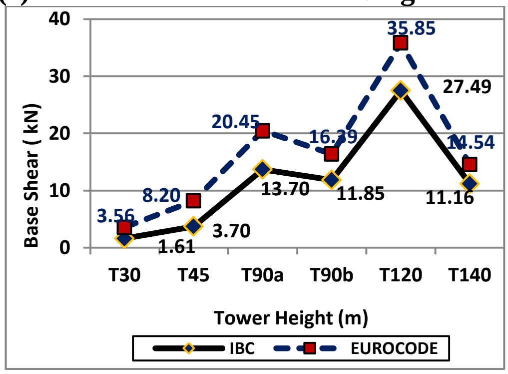
(c)
Ground Type C



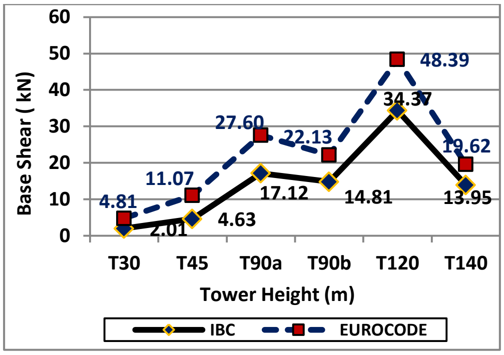
(d)
Ground Type D

Figure 10: Base Reactions of Towers in Zone 2, 0.08 gals

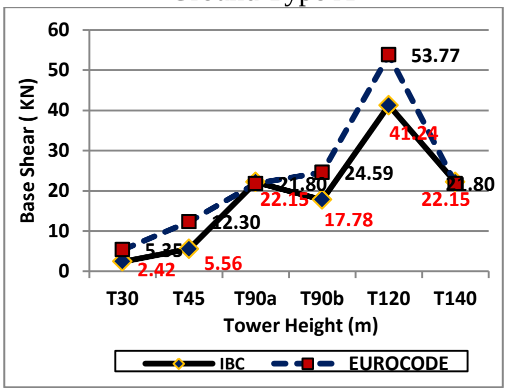
(e) Zone 2- Acceleration 0.10gals



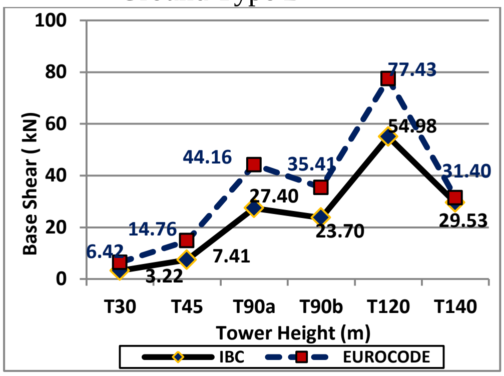
(a)
Ground Type A



(b)
Ground Type B



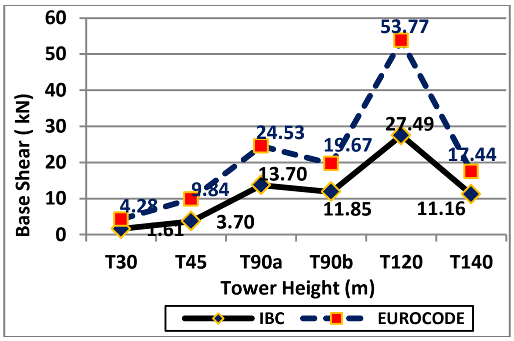
(c)
Ground Type C



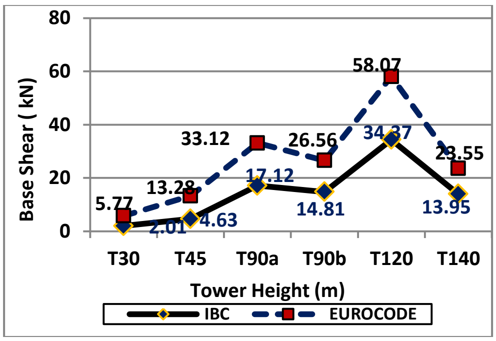
(d)
Ground Type D

Figure 11: Base reactions for towers in Zone 2, 0.10 gals

(f) Zone 2- Acceleration 0.12gals

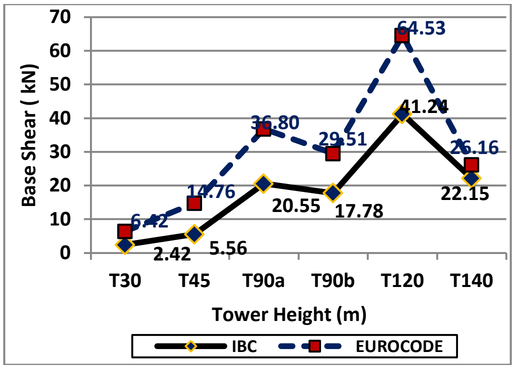


(a)
Ground Type A

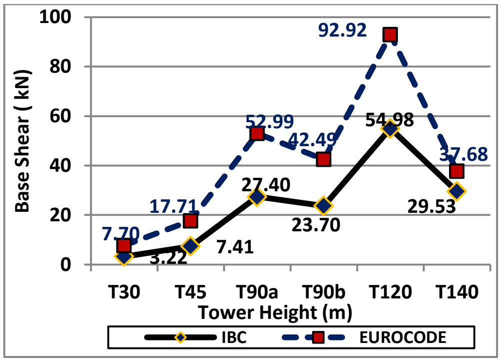


(b)
Ground Type B

Figure 12(a): Base reactions for towers in Zone 2, 0.12 gals



(c)
Ground Type C



(d)
Ground Type D

Figure 12 (b): Base reactions for towers in Zone 2, 0.12 gals

3.1.3 Results and Discussion on Joint Displacement and Base Shear Reaction Analysis Output.

From the analysis, it is noticeable that in most of the cases the highest tower do not show the logical trend of results, where higher towers should produce higher values for joint displacements and base shear.

In the joint displacement analysis that has been done for all the towers in various types of peak ground accelerations, different ground types and also different seismic zones conditions, shows that the results of tower T120 which is only 120 meters high exhibit the highest values of displacement compared to the highest tower T140 which is 140 meters high. While for base shear analysis, all output of shear values shows that the highest tower T140 also does not produce the highest shear or showing the worst. Logically it should. But it shows that T120 is more dominant in all output of base shears reactions results. There are several factors that can justify about the obtained results. They are:-

(i) Weight of Towers

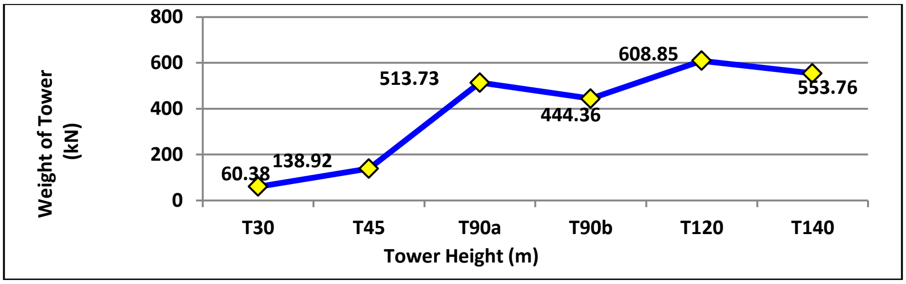


Figure 13: Weight of towers (kN) against tower height (m)

As seen from Figure 13, tower T30 has the least weight followed by tower T45, T90b, T90a, T140 and T120. Tower T120 weighs the heaviest compared with all the other towers including the highest tower i.e. tower T140. Logically, the highest tower will be the heaviest and will also produce the highest values in all analysis. But this is not so for tower T140 and for tower T120.

Results shown earlier in the analysis that has been carried out, exhibits that joint displacement analysis is increasing based on the increasing height of tower and this seems logical. Logically the higher tower will produce higher displacement values. Exceptionally, in this analysis it seems that for tower T120 which is higher but has lower displacement values compared to tower T90 which is only 90 meters high. The reason arises from this fact that the stiffness of tower T120 is several times higher than that of tower T90. From the above graph we observed that the weight of tower T120 is heavier, which is 608.8kN and this is almost one and half times more than the weight of tower T90b which is only 444.36kN. So, this is one of the reasons why such characteristic is seen in the output of joint displacement and base shear analysis.

(ii) Natural Period and Frequency Characteristics of Tower

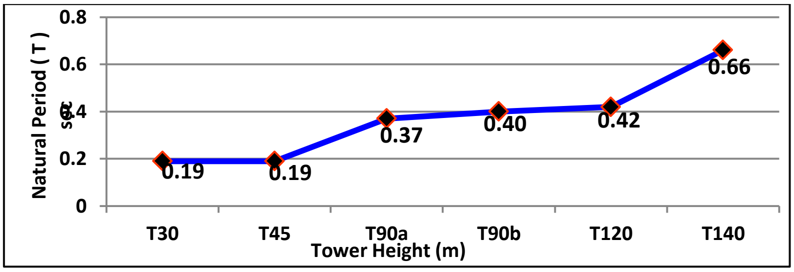


Figure 14: Height of towers (kN) vs natural period (sec)

As seen in Figure 14 above, in the first mode versus the tower height graph it clearly shows that the value of natural period increases with increasing tower heights. The highest tower, T140 shows the highest natural period among all with 0.66 seconds while for tower T30 being the shortest height shows the least value i.e. only 0.19 seconds. Tower T45 also records a natural period of similar to T30.

In addition it is seen that although towers, T90a, T90b and T120 have different heights, their natural period have approximately the identical natural period i.e. almost to 0.4 seconds. This is also one of the reasons why such characteristic is encountered in the output of joint displacement and base shear analysis results. Looking back at the above, i.e. from factors of weight and natural period we can prove this with the following equations

$$W = \sqrt{k/M} \quad (1)$$

$$2\pi/T = \sqrt{k/M} \quad (2)$$

where k = stiffness, W = weight, T = natural Period and M=Mass
From equation (2) , we obtain

$$T = 2\pi \sqrt{k/M} \quad (3)$$

For tower T120, where M = 608.8 KN and T = 0.42 and for tower T90b, where M = 444.36 KN and T = 0.40

By inserting these values in equation (3) ;

For T120,

$0.42 = 2\pi \sqrt{608.8/k_{120}}$

Therefore $k_{120} = (0.42 / 2\pi)^2$
$\mathbf{= 4.47 \times 10^{-3}}$

For T90b,

$0.40 = 2\pi \sqrt{444.36/k_{90b}}$

Therefore $k_{90b} = (0.40 / 2\pi)^2$
$= \mathbf{4.05 \times 10^{-3}}$

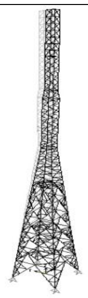
From the above it clearly justified that**,** $\mathbf{k_{120}}$ >>> $\mathbf{k_{90b}}$. This shows that the higher the towers, the bigger will be the value of its stiffness, **k**, and thus the higher will be the displacement differences shown.

(iii) Shapes / Arrangement of Tower

As seen from Table 4, the shapes of the entire analysed towers are not identical. Some having a regular shape throughout the whole stretch while others having a broad shape in the lower part but more tapered towards the peak. The sort of 'irregular' shape and arrangement that varies from each towers also contributes to the inconsistent trend in the output of displacement and base shear reaction values.

From these analyses, the researcher has come to a suggestion that it is a good point that for the erection of towers in future, a standardised or by optimizing the shape should be practiced so that tower owners can improve the seismic performance of their towers.

Table 4: Shapes of modeled towers

Tower Name	T30	T45	T90a
			
	T90b	T120	T140
			

From the above discussions, we can conclude that we have been able to retrieve the displacement and base shear reactions for towers under earthquake conditions using both IBC and EC8 analysis. Generally, the value of lateral displacement and base shear reactions are higher when the PGA value increases. Moreover the same trend is observed also when the ground type changes from ground type A to ground type D.

In addition, factors such as the weight of towers, shapes or arrangements of members and the natural period also contributes to the obtained results. But still we could not confirm as yet whether the existing towers are able to sustain the earthquake effects experienced in Malaysia or not. To response to this question we should compare the displacements and reactions already obtained with allowable values. We need to refer to codes or regulations related to telecommunication tower structures as outlined in TIA/EIA-222G and ASCE 7-10.

In TIA/EIA-222G, there are some limitations for sway displacement and torsional rotations of telecommunication towers. However these are for the case when the tower is under wind action. In case of earthquake, since the telecommunication towers do not have non-structural elements that will get damage due to seismic action, seismic codes like ASCE 7-10, do not put limitation for lateral displacement of non-building structures like telecommunication towers.

Due to this limitation if we want to compare the lateral displacements, we can use the limitations that we have for common buildings that are found in building codes. Since these limitations can be considered restrict for telecommunication towers, but if the towers could pass these limitations then we can assure that they are safe. The displacement limitation in section 12.12.1 of ASCE 7-10, is stated as 0.02 heights of structures. In order to prove this we should calculate the towers top displacement through section 12.8.6 of ASCE 7-10. The displacement at Level x (δx) (in. or mm) used to compute the design story drift, Δ, shall be determined in accordance with the following equation:

$$\delta_x = \frac{C_d\, \delta xe}{I_e} \tag{4}$$

where:

C_d = the displacement amplification factor, δxe = the deflection at the location required by this section determined by an elastic analysis I_e = the importance factor. In the analysis carried out earlier, $\mathbf{C_d = 3}$ and $\mathbf{I_e = 1.5}$

Then the obtained result can be compared with 0.02 heights of the towers. It should be mentioned that, TIA/EIA-222G limits the maximum displacement to 0.03 height which is 1.5 times more than the proposed value by IBC.

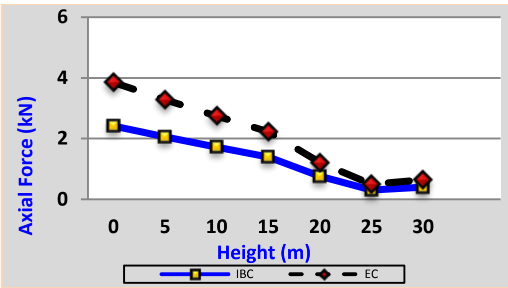
From the displacements limitation, it clearly verified that all maximum displacements of all the towers is less than the permissible limit as that has been outlined in ASCE/IBC. This shows that the structural integrity of the self supporting steel towers are in good condition and suspected to be able to withstand seismic ground movements due to earthquakes.

Besides the above factors, the researcher will explore more on the Axial Force characteristics of the towers that has been modeled. Since these towers are restrained as hinge therefore the obtained shear force and moment are negligible. So, in this study only the axial forces in the members are considered. Since only samples of the 489 towers owned by TM are being analysed, there seems that many more numbers of towers to be thoroughly inspected to determine their state of structural integrity especially for those towers located near to the more hazardous zone.

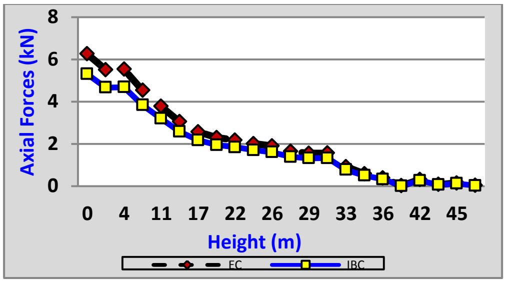
3.1.4 Analysis of Axial Forces for Modeled Towers

For axial forces analysis only the maximum peak ground acceleration in each zone is selected besides two ground types. In Zone 1, peak ground acceleration of 0.08 gals with ground type A and ground type D has been carried out on all the modeled towers. For Zone 2 only the maximum peak ground acceleration i.e. the 0.12 gals has been selected for the axial shear analysis on both ground type A and D. The total number of analysis carried out for the axial forces are twenty four (24). The results for the axial forces are as shown in Figure 15 to Figure 18.

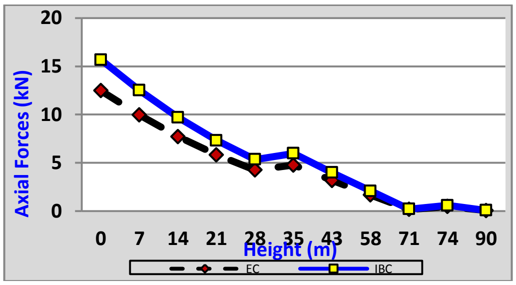
(a) Axial Forces in Zone 1, Peak Ground Acceleration 0.08 - Ground Type A



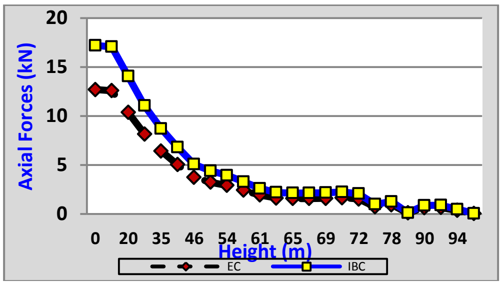
(a)
Axial Forces on Tower T30



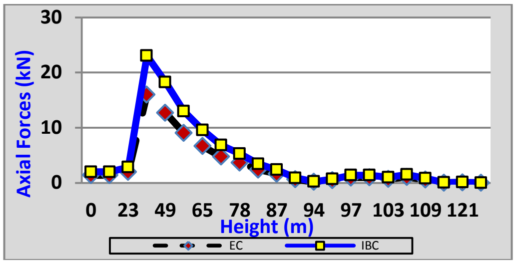
(b)
Axial Forces on Tower T45



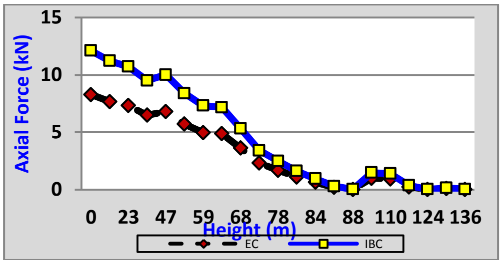
(c)
Axial Forces on Tower T90a



(d)
Axial Forces on Tower T90b



(e)
Axial Forces on Tower T120

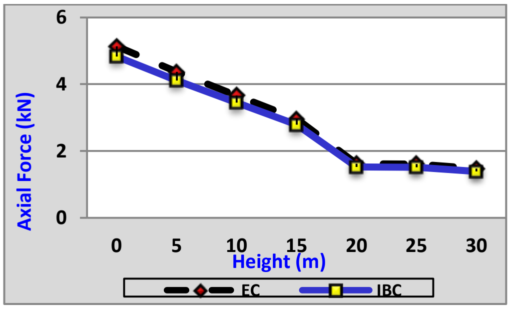


(f)
Axial Forces on Tower T140

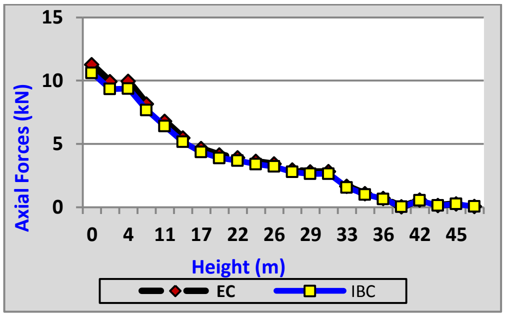
Figure 15: Axial forces for towers in Zone 1 with 0.08 gals on ground type A

In zone 1 with ground acceleration of 0.08 gals, it is observed that for axial forces in ground type A shows that for lower rise tower i.e. T30 and T45, the EC8 estimates the axial force of elements higher than IBC. Refer Figure 15 (a) and (b). However, for towers T90a, T90b, T120 and T140 the IBC overestimated the axial force of elements in comparison to the EC8. Refer Figure 15 (c) to (f).

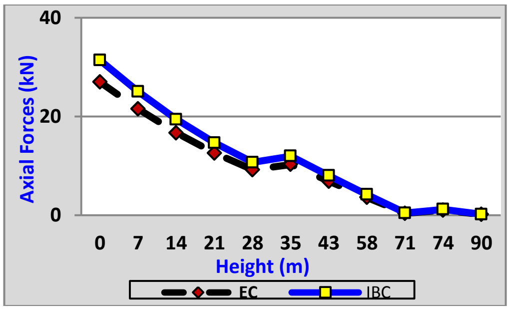
(b) Axial Forces in Zone 1, Peak Ground Acceleration 0.08 - Ground Type D



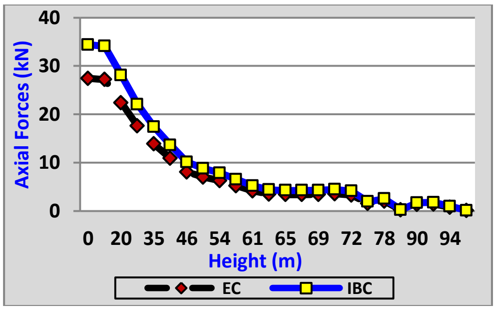
(a)
Axial Forces on Tower T30



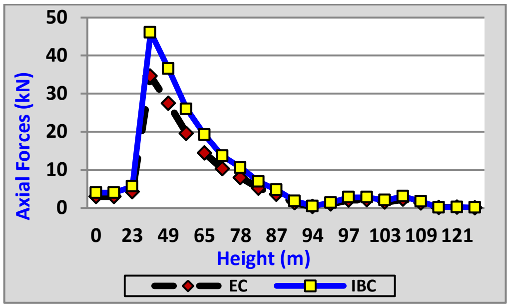
(b)
Axial Forces on Tower T45



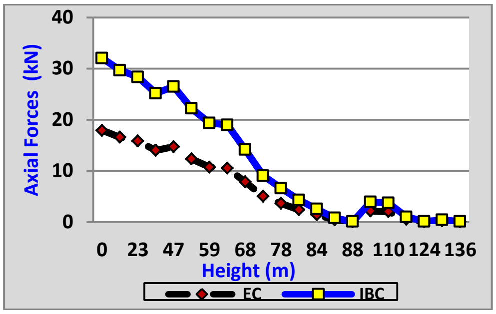
(c)
Axial Forces on Tower T90a



(d)
Axial Forces on Tower T90b



(e)
Axial Forces on Tower T120



(f)
Axial Forces on Tower T140

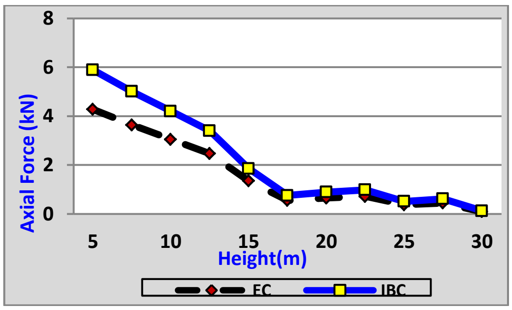
Figure 16: Axial forces for towers in Zone 1 with 0.08 gals on ground type D

In Figure 16 (a), T30 shows that the axial force of tower's leg in EC8 is higher compared to IBC. In ground type D the axial force experienced here is much higher compared to that shown in ground type A earlier. This trend can also be seen for T45 where the axial forces shown in ground type D is higher in EC compared to IBC with double values of axial forces. Refer graph 16 (b).

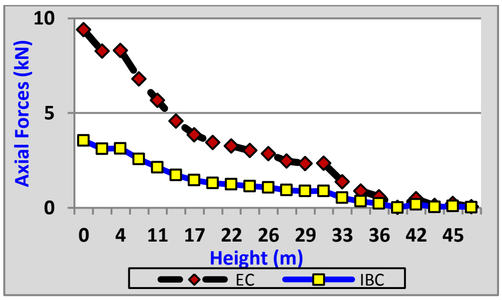
For higher tower, T90a and T90b show that axial forces in IBC is more dominant than EC8. Results obtained in ground type D is almost double than those obtained in ground type A. Also for T120 and T140 towers resemble the same characteristics with IBC recorded higher values. In all towers it is observed that when height increases, the difference between IBC and EC8 decreased.

Comparing results in zone 1, for all towers excited with the same 0.08 peak ground acceleration clearly indicates that results in both analysis shows different character's in the axial forces shown.

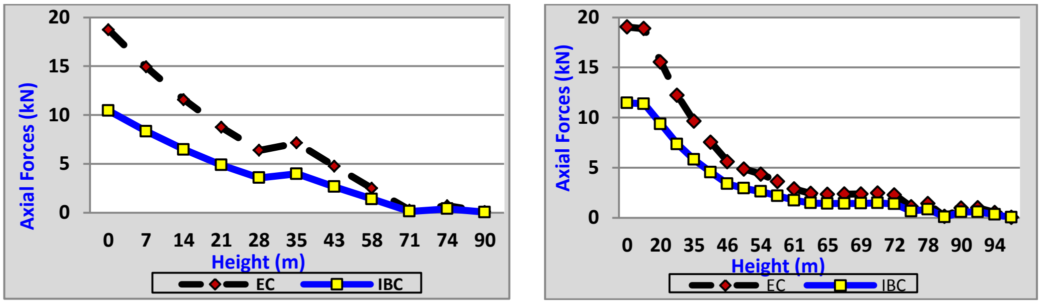
(c)Axial Forces for Zone 2, Peak Ground Acceleration 0.12 - Ground Type A



(a)
Axial Forces on Tower T30



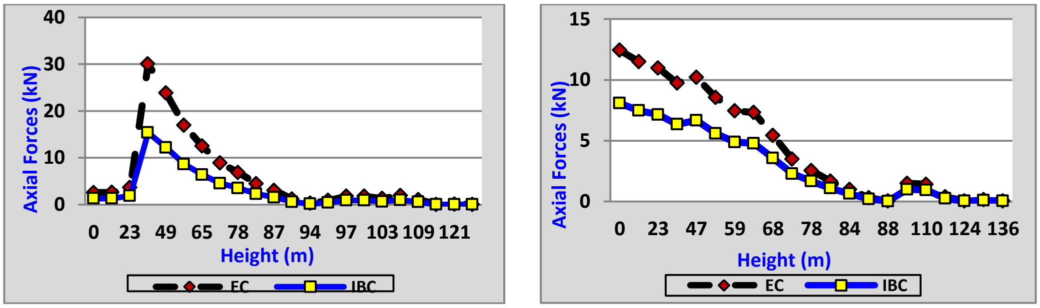
(b)
Axial Forces on Tower T45



(c) Axial Forces on Tower T90a

(d) Axial Forces on Tower T90b

Figure 17(a): Axial forces for towers in Zone 2 with 0.08 gals in ground type A



(e) Axial Forces on Tower T120

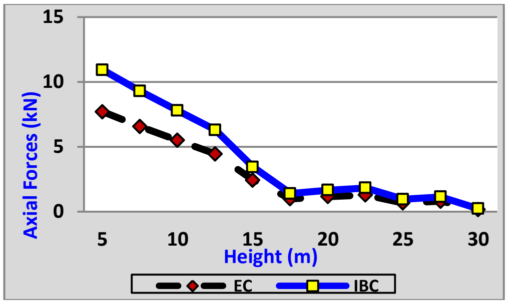
(f) Axial Forces on Tower T140

Figure 17(b): Axial forces for towers in Zone 2 with 0.08 gals in ground type A

Results in zone 2 with higher peak ground acceleration and soil type A, indicate a variation of output in the analysis. In graph 17 (a) T30 shows a change of results where IBC shows a higher reading. T45 still maintains the same trend as in earlier analysis where axial forces in EC8 are more dominant compared to IBC.

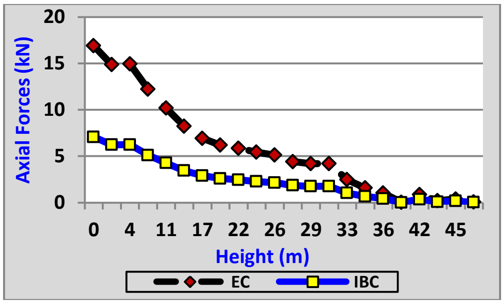
In graph 17(c) and 17(d) both T90a and T90b indicates that axial forces experienced by the towers are more dominant in EC8 analysis. Again for towers T120 and T140 resemble the same trend as in tower T90 where the axial forces experienced in IBC is lesser than EC8.

(d)Axial Forces in Zone 2, Peak Ground Acceleration0.12 - Ground Type D



(a)

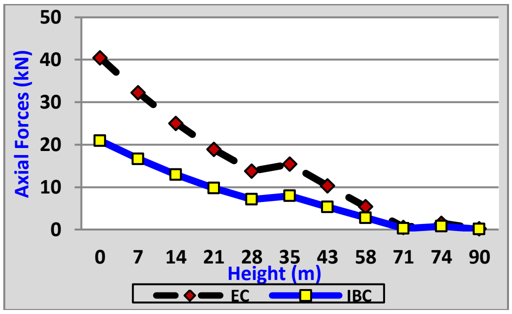
Axial Forces on Tower T30



(b)

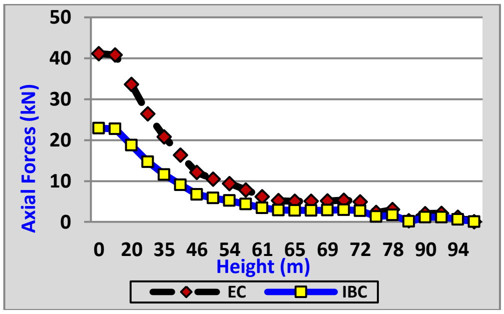
Axial Forces on Tower T45

Figure 18(a): Axial forces for towers in Zone 2 with 0.08 gals in ground type D



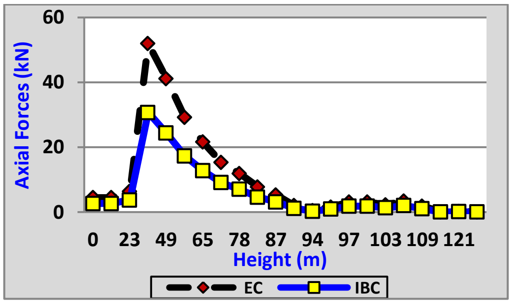
(c)

Axial Forces on Tower T90a



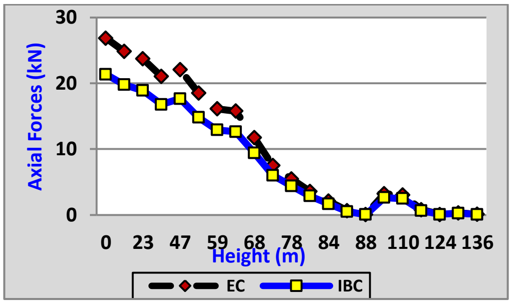
(d)

Axial Forces on Tower T90b



(e)

Axial Forces on Tower T120



(f)

Axial Forces on Tower T140

Figure 18(b): Axial forces for towers in Zone 2 with 0.08 gals in ground type D

In this analysis, the same characteristics of towers are observed as in ground type A. As seen in graph 18(a) the axial forces experienced by T30 shown in IBC is higher compared to EC8. For T45 a reversal trend shows where EC8 is more conservative. Refer Figure 18 (b), where T45 indicates a higher value in axial compared to T30 which is lower in height. As seen in Figure 18(c) and 18(d) above , all higher towers T90a, T90b, T120 and T140 shows a much more significant values in axial forces experienced. The values obtained in ground type D is double than that shown in ground type A.

4.0 Conclusions

From the results obtained and discussions made the researcher has come to some conclusions and can be summarised as follows:

(i) Results on twenty four analysis (24) of towers for joint displacement has shown that the displacement is totally increasing based on the height increasing and this seems logical except for tower T120 which is less than tower T90b. Ground type D presents the biggest displacements for all the towers, this generally shows that in the more hazardous zones it has increased the lateral displacement of towers. Both codes IBC and EC8 did not show a consistent trend in this analysis. In certain cases, IBC is more conservative compared with EC8 and vice versa.

(ii) For base shear analysis, results from all the twenty analysis (24) on all towers also present the same trend as in joint displacements. Generally in the more hazardous zones it has increased the base shear reactions where again ground type D shows the biggest value. In this analysis the researcher found that tower T120 which is only 120 meters high posses the biggest base shear values in all ground types and peak ground acceleration compared to tower T140 which is 140 meters high. Results clearly indicate that the values of base shear for the tallest tower is not the worst.

(iii) There are several factors that could justify the obtained results. The first factor is weight of tower T120 is highest compared to tower T140. Secondly is the natural period and frequency of the tower and finally the shape or arrangement of members for the tower structure. It is expected that the shape and good performance of T120 has contributed with such trend of output in the analysis.

(iv) Displacements limitation for all the towers analysed does not exceed the permissible limit as outlined in ASCE/IBC where it clearly verified that all maximum displacement of all the towers is less.

(v)Also analysis of axial forces performed on the towers clearly shows that the axial forces exhibited by the towers which are in the more hazardous zone shows a much higher value to almost double even though the same peak ground acceleration is used. Comparing between ground type A and D, with 0.08 peak ground acceleration clearly shows that axial forces experienced in ground type D is highest. The same trend is also seen in the 0.12 peak ground acceleration results.

(vi)The axial forces results shows that there is an indication of a greater sensitivity of the leg members to the peak ground acceleration of the earth movement and the type of ground it is situated.

(vii)All the towers studied behaved within serviceability limits.

Though only six samples of towers are being used for the analysis, it does show signs that there are seismic behaviors acting on the telecommunication towers in Malaysia due to the earthquake effects. Seismic amplifications of displacements, base shear reactions and axial forces may affect the top part of the tower where the antennas are attached, but they should not result in any local permanent deformation after the earthquake. Such deformations may results to a loss of serviceability resulting in unacceptable signal attenuation or failure. Therefore it is recommended more studies should be carried out to further determine the safety and reliability of all towers in the country especially those located in the more hazardous zones.

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