Comparison of Five Major Wind Codes with Sri Lankan Context

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ABSTRACT: The design manual "Design building for high winds – Sri Lanka" is the only mandatory document available for wind load design in Sri Lanka. This document extensively covers the design and construction of low rise buildings. However, the evolution of tall building construction requires advances for Sri Lankan wind loading standards, which cannot be prepared yet due to lack of available data and technology. Therefore, designers have used different international standards for wind design for medium and high rise buildings without understanding the impact of many different standards in a particular design fully. Hence, some of these common international wind standards such as CP 3 Chapter V – Part 2:1972, BS 6399.2:1997, AS 1170.2:1989, AS/NZS 1170.2:2002 and EN 1991-1-4:2005 are evaluated in this paper by considering loads exerted at ultimate limit state on the structural members such as beam and column members, shells such as shear walls and supports of a building by wind loads to understand the suitability of these codes for Sri Lankan context. The serviceability limit state behavior of a building is also evaluated according to the methods given in wind loading standards. The effective increase of the return period due to the use of load factors for wind loads is also important in order to achieve adequate risk level for different type of buildings. Finally conclusions have been drawn about the selected wind loading standards, which can be used with Sri Lankan context until country could produce its own wind code.
KEYWORDS: Design manual, Wind loading standards, Return period.

1 INTRODUCTION

Wind engineering is not an advance branch of engineering in Sri Lanka compared to the other engineering works. The only available mandatory document on wind engineering in Sri Lanka is design manual "Design Building for High Winds – Sri Lanka", which is primarily based on CP 3 Chapter V – Part 2:1972. This useful document only covers the design and construction of low rise buildings with enhanced wind resistant abilities by using simple techniques that can be practiced even in today. Apart from design manual, CP 3 Chapter V –Part 2:1972 is the most common practice for designing buildings against wind loads. However, like in many other countries in Asia- Pacific region, there is a national trend to build tall buildings at city centres in Sri Lanka since last two – three decades (Karunarathne, 2008). Due to inadequate provisions given in both design manual and CP 3 Chapter V – Part 2: 1972 for designing tall buildings, many Sri Lankan designers and engineers are using various international wind loading standards. However, use of these standards without fully understanding methods and factors, may lead to arise some problems like inconveniences about understanding and comparing wind load calculations, lack of harmonization among wind load design of structures etc,. Therefore, Sri Lankan researchers (Premachandra, 2008, Wijerathne, 2001) have done some comparative studies about international wind loading standards to select most suitable wind codes for Sri Lanka.

However, some issues like the use of load factors with different standards, effective increment of return period resulted from use of load factors, appropriate wind velocity values for designing tall buildings and low rise buildings, most suitable wind loading standards for the Sri Lankan context etc. are still need to be addressed. In this study above issues are being discussed through a comparative study of five major wind codes; with special refer to tall building design with prevailing conditions in Sri Lanka.

2 PRESENT CONDITIONS IN SRI LANKA

According to the Design manual, Sri Lanka was divided in to three wind zones (Figure 1). Two types of wind speeds are defined which can be named as wind speeds for normal structures and wind speeds for post disaster structures as shown in Table 1. All these wind speeds are 3 second gust wind speeds with 50 years return period. These wind speeds were not derived from the proper collection of data or by using proper statistical analysis. Thus, they would not have sound theoretical basis. However, wind speeds were derived by using some alternative measurements such as damages sustain along the cyclone path, historical data and Australian practices instead of using proper statistical methods (Clarke et al, 1979).

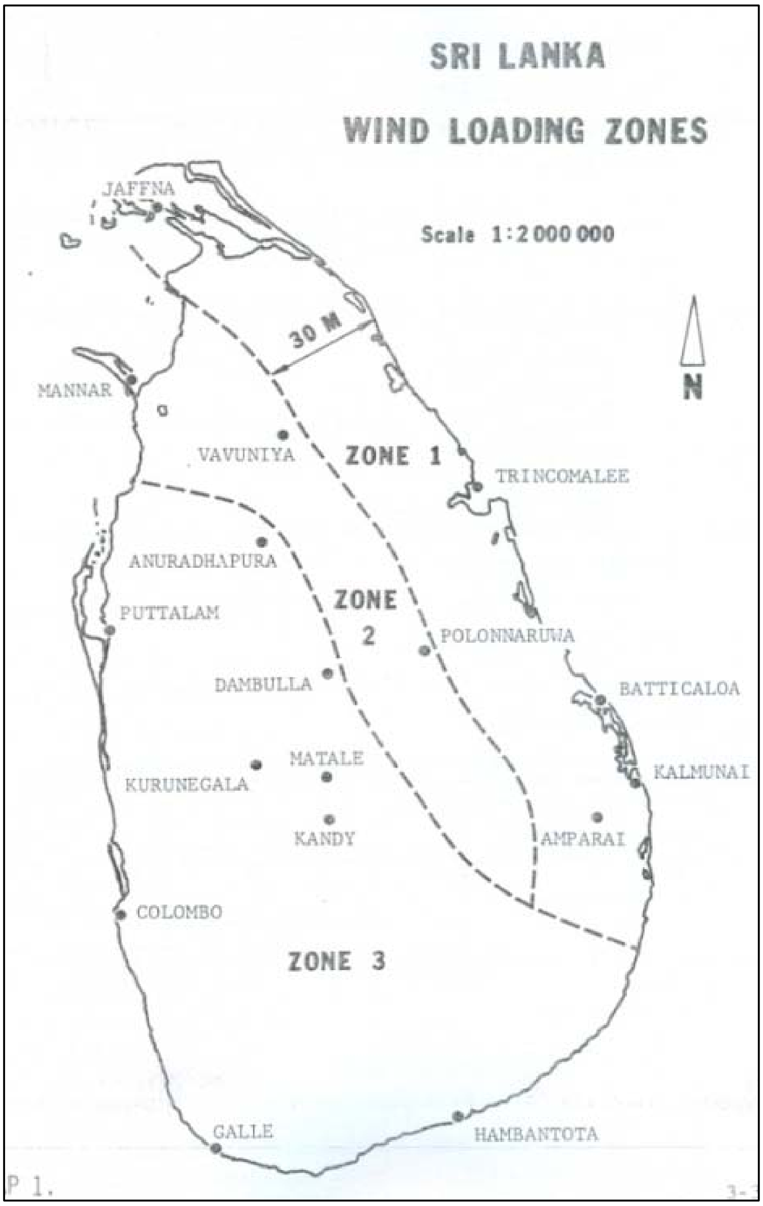


Figure 1: Wind Zones in Sri Lanka (Design manual “Design building for high winds”, 1978)

Table 1. Three second gust velocities used for different areas of Sri Lanka (Design manual "Design building for high winds", 1978)

Wind Zone	Post disaster structures (ms^{-1})	Normal structures (ms^{-1})
Zone 1	54	49
Zone 2	47	42
Zone 3	38	33

3 SELECTING CODES AND STANDARD FOR THE STUDY

Due to the lack of information in the design manual to address the issues of tall building design, many Sri Lankan engineers have used different international wind loading standards as their preferred options. These preferred options may vary from some old code of practise like CP 3 Chapter V – Part 2 to newest codes like Euro code. By considering all of current practises recently found in Sri Lankan civil engineering sector, following codes and standards were chosen for the comparison purpose; CP 3 Chapter V – Part 2:1972, BS 6399.2:1997, AS 1170.2:1989, AS/NZS 1170.2:2002 and EN 1991-1-4:2005.

CP 3 Chapter V-Part2:1972 uses quasi-static method to calculate wind loads on a building. This quasi static approach more suitable for evaluating wind loads on low rise buildings rather than to evaluate the performance of a high rise building. Many like to continue to CP 3 Chapter V- 1972 because of its simplicity and familiarity about the code. BS 6399.2:1997 is the newer version of the British standard and capable of handling both static and dynamic behaviour of a building. Gust Load factor is a more popular method to calculate wind load by considering both fluctuating wind speeds and dynamic behaviour of a structure. AS 1170.2:1989 use this method and it generally uses 3 second gust velocity as basic wind speed. AS/NZS 1170.2:2002 has changed some factors and methods used in previous Australian standard and make it as a simple document to use. Apart from these reasons, Australian standards cover wide spectrum of wind including cyclones and it can be more suitable for small island nations like Sri Lanka that may face devastating actions of high winds. EN 1991-1-4:2005 is the newest code and not only it compromise many aspects present in other codes such as BS 6399.2:1997, AS/NZS 1170.2:2002 but also it allows to adjust methods and factors which are suitable for own country by means of a national annex.

4 BASIC WIND SPEED VALUES WITH DIFFERENT AVERAGING TIMES

Wind loading standards use different averaging time wind speeds to calculate wind induced loads on structures. When it comes to Sri Lankan context, available wind speed data is only the 3 second gust wind speed. Therefore, it is needed to convert 3 second gust wind speed values into wind speeds based different averaging times by using some conversion factors. The value of 1.06 was used as conversion factor to convert mean hourly wind speeds to 10 minute mean speed, as proposed by the Institute of Civil Engineers in United Kingdom (ICEUK). The method proposed by Cook (1999) was used to convert 3 second gust wind speeds to mean hourly wind speed. The wind speeds in all three zones with different average times are shown in Table 2.

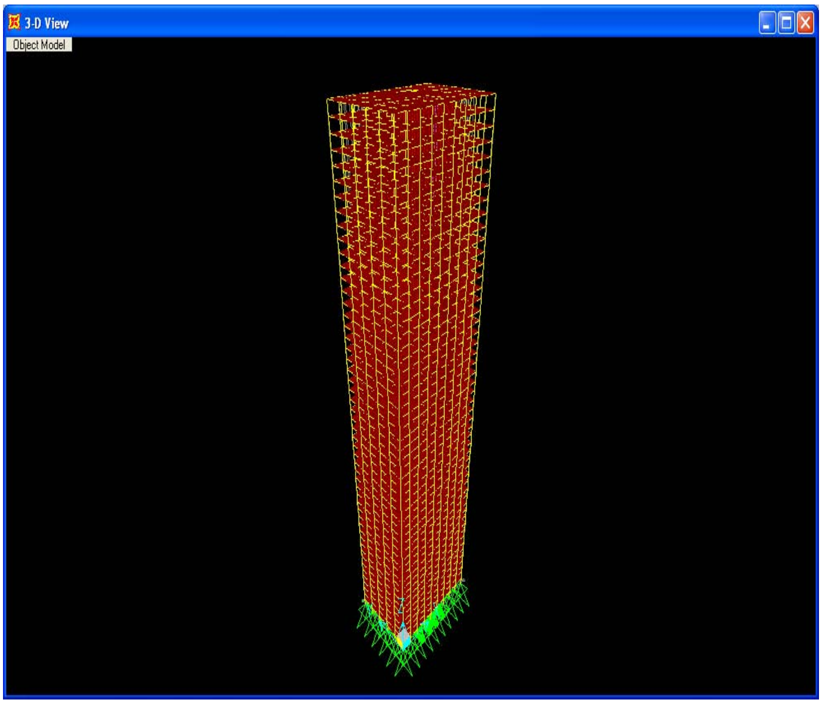
Table 2. Basic wind speeds with different averaging time

	Zone 1 (ms^{-1})		Zone 2 (ms^{-1})		Zone 3 (ms^{-1})	
	Normal structure	Post disaster structure	Normal structure	Post disaster structure	Normal structure	Post disaster structure
CP 3 : Chapter V - Part 2 : 1972 (3 second gust wind speed)	49	54	43	47	33	38
BS 6399 - 2:1997 (Mean hourly wind speed)	27	30	24	26	18	21
BS EN 1991-1-4:2005 (10 minutes mean wind speed)	28	32	25	28	19	22
AS 1170.2 -1989 (3 second gust wind speed)	49	54	43	47	33	38
AS/NZS 1170.2:2002 (3 second gust wind speed)	49	54	43	47	33	38

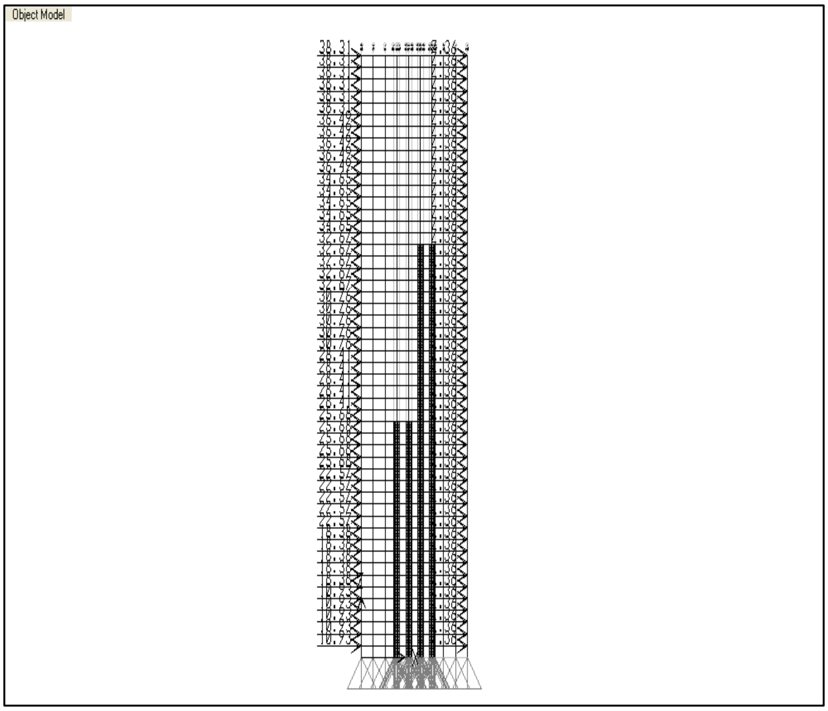
5 A CASE STUDY FOR COMPARISON

A 183 m high rectangular shaped building modelled and analysed by using SAP 2000 software, in order to determine dynamic behaviour of a tall building and the effect of using various standard to calculate the wind induced behaviour. The plan dimensions of the building are 46 m x 30 m (Figure 3(a)). The building is typical column - beam frame structures with service core of shear walls. Within the service core, all lifts, ducts and toilets are located. The hard zoning lift system was used for the building to simulate a more actual scenario. The diaphragm constraint was used for slabs to move all points of the slabs together. Other than the dead load of the structural members, super imposed and live loads were applied in the model according to the BS6399: Part 1: 1996. Wind loads on the building were calculated for all three wind zones as given in design manual and applied with respect to two orthogonal directions as joint loads at the column - beam junctions on the wind ward and leeward faces separately as shown in Figure 3 (b).

Wind forces were calculated as provisions given in different wind loading standards by encountering different factors and methods. For British and Euro codes, wind loads were calculated according to the division –by –parts rule. Only for wind zone 1, importance factor 1.1 used with special terrain – height multiplier as given in AS 1170.2:1989, but to calculate wind loads according to the AS/NZS 1170.2:2002 only higher terrain –height multiplier for cyclonic region was used



(a)



(b)

Figure 3: (a) Finite element 3 – D model of 183 m high building (b) Wind loads applied in windward and leeward side of the 183 m high building

6 RETURN PERIOD VS LOAD FACTORS

Various return period values are used for design of different types of buildings that are vulnerable for different levels of risk. These levels of risk can varying from country to country for the same type of building, due to various country specific factors. According to the

Building Code of Australia, recommended values of annual probability of exceedence for different types of structures are shown in Table 3.

Table 3: Importance level of buildings and structures (Building Code of Australia, 2007)

Importance levels of buildings and structures		Annual probability of risk exceedence at ULS
Level	Building type	
1	Building with low degree of hazard	1:200
2	Houses	1:500
3	Building with large number of people	1:1000
4	Essential to post disaster recovery / associated with hazardous facility	1:2000

To compare the effective increment in return period for the factored wind load combinations as defined in British standards, arisen from 50 year return period wind, un-factored wind loads, which were arisen from a higher return period wind, were used. Wind speeds for different return periods were obtained by using probability curve from BS 6399.2:1997 and wind velocities are shown in Table 4.

Table 4: 3 second gust wind speeds for different return periods, obtained from probability curve for BS 6399.2:1997

Zo	Wind Speed (ms^{-1})				
	Normal Structure	Post Disaster Structure	100 years return period	500 years return period	1000 years return period
1	49	54	51	55	74
2	42	47	44	47	63
3	33	38	34	37	50

All the above wind speeds values were calculated according to the following Equation (1). The factors in this equation other than the probability factor (S_p) were assumed to be constant.

$$V_s = V_b \times S_a \times S_d \times S_s \times \quad (1)$$

Where,

V_b – Basic wind speed in ms-1(10 minutes mean wind speed)

S_a – Altitude factor which is given as $S_a = 1 + 0.001$

- Site altitude

S_d – Directional factor; if the orientation unknown or ignored $S_d = 1.0$

S_s – Seasonal factor; for permanent building $S_s = 1.0$

S_p – Probability factor; with standard risk

Therefore, the wind loads, which were arisen from higher wind speeds corresponding to different return periods, are directly proportional to square of the probability factor $\left(S_p^2\right)$.

According to the Table 2, wind speeds for post disaster structures are approximately equal to the 500-years return period wind speeds values. However, these wind velocities cannot relate to each other by using load factors unless they yield same load effects with and without load factors. Therefore, finite element analysis was used to determine the existence of the relationship between load factor and $\left(S_p^2\right)$ values of wind speed with different return periods.

The results were compared for eight cases such as 1.2G+1.2Q+1.2W, 1.0G+1.4W, 1.4G+1.4W 1.2G+1.2Q+Wu, 1.0G+Wu, 1.4G+Wu and the wind load cases W and Wu alone. The Wu denotes the ultimate wind speed at different return periods like 500 years and 1000 years return period. Each case was compared for two orthogonal wind directions named as X and Y. The results were compared with 50 years return period wind speeds for both normal structures and post disaster structures, together with load factors as defined in British Standards. The results are displayed in Figure 4 to 7. WOLF1 / WLF is the ratio between 500 years return period wind load without load factors and 50 years wind load with load factors, calculated for normal structures. WOLF2 / WLF is the ratio between 1000 years return period wind load without load factors and 50 years wind load with load factors, calculated for post disaster structures.

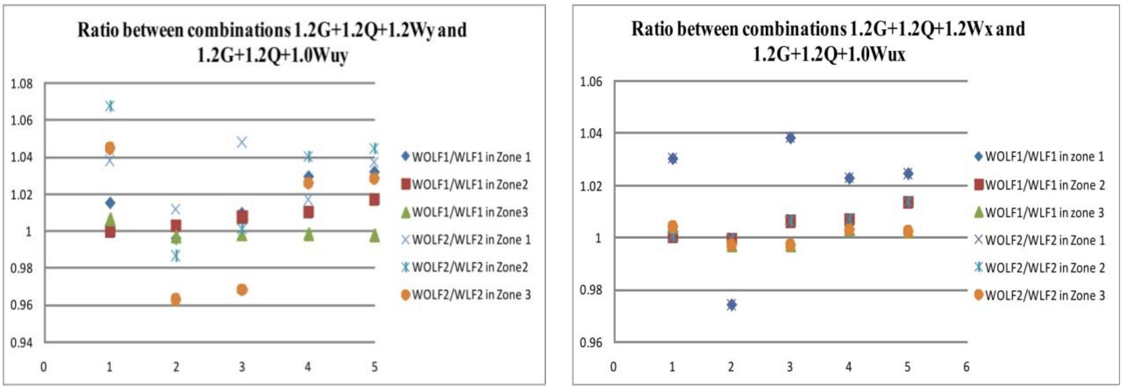


Figure 4: Comparison of results obtained for 1.2G+1.2Q+1.2W load combination with and without load factor

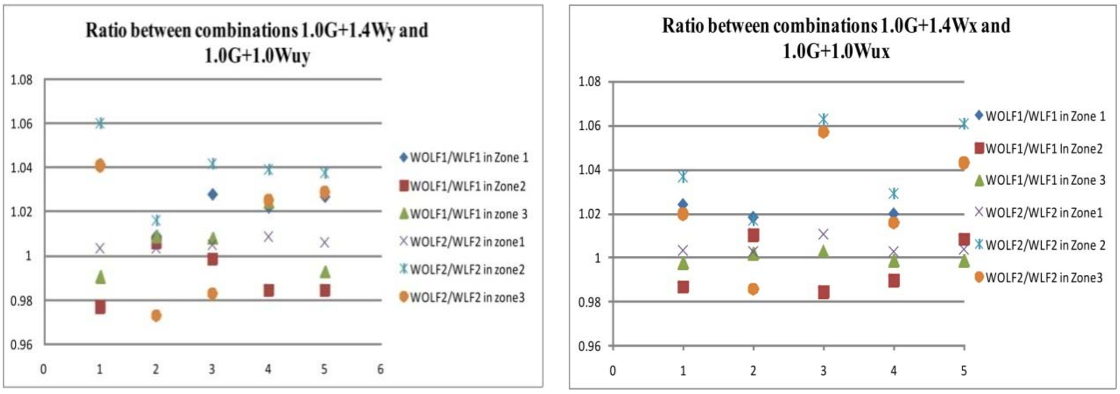


Figure 5: Comparison of results obtained for 1.0G+1.4W load combination with and without load factor

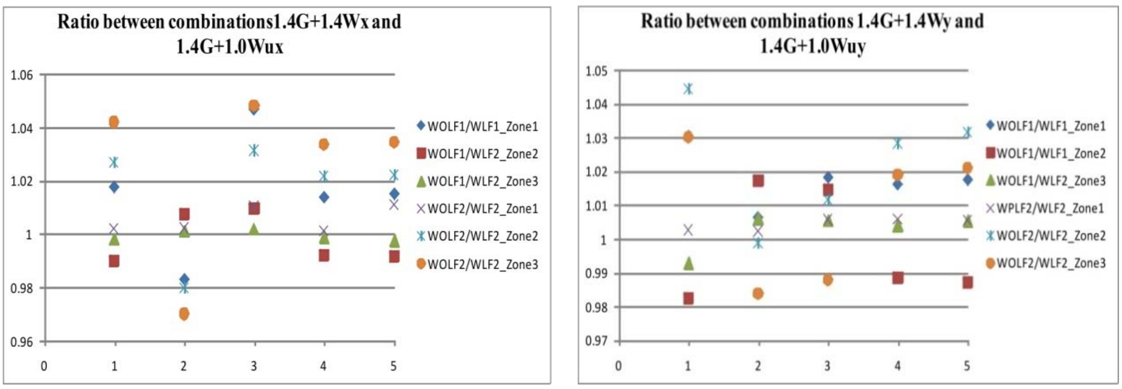


Figure 6: Comparison of results obtained for 1.4G+1.4W load combination with and without load factor

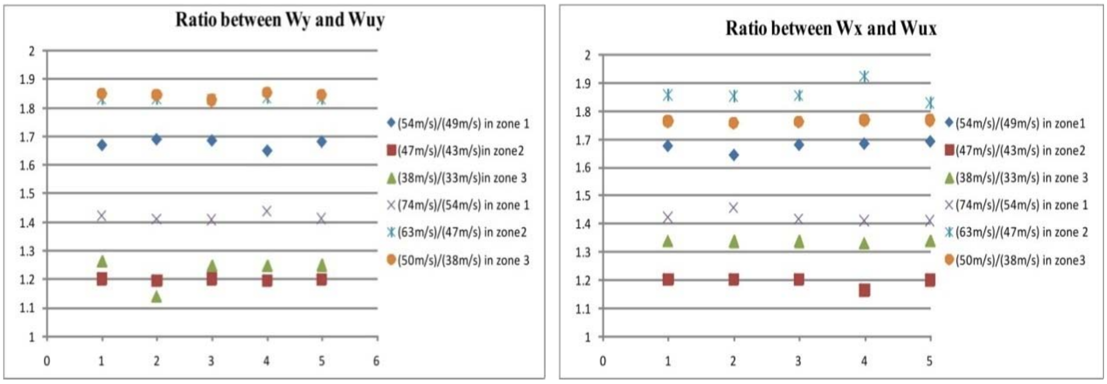


Figure 7: Comparison of results obtained for 1.0G+1.4W load combination with and without load factor

It can be seen from the above graphs that there is close relationship for 50 years return period wind speeds for normal structures and post disaster structures with 500 years return periods and 1000 years return periods respectively. The results obtained for load combinations with load factors and without load factors are approximately the same. The variations of the results are varying in the order of +5% to -5%. However, the ratios for the wind load cases are much greater than the predicted values of $\left(S_p^2\right)$and the load factor value 1.4. Hence, it is noticeable that a relationship among $\left(S_p^2\right)$values and load factor 1.4 and the actual load effects on the structural members derived from wind loads are an unlikely event. Therefore, by using load factors with 50-years return period wind speeds for normal structures and post disaster structures, a designer can achieve wind speeds with 500 years return period and 1000 years return periods respectively.

According to Table 3 and results shown in Figures 4 - 7, dwellings should be designed to withstand against wind loads derived from 50-years wind speed for normal structures with load factors as defined in British standard or for the loads that are directly obtain from 500-years return period wind speeds, without any load factors. This is applicable for all three wind zones. For high-rise office buildings, which accommodate large number of people, designs should be based on wind loads derived from wind speed for post disaster structures with load factors or 1000 -years return period wind speeds.

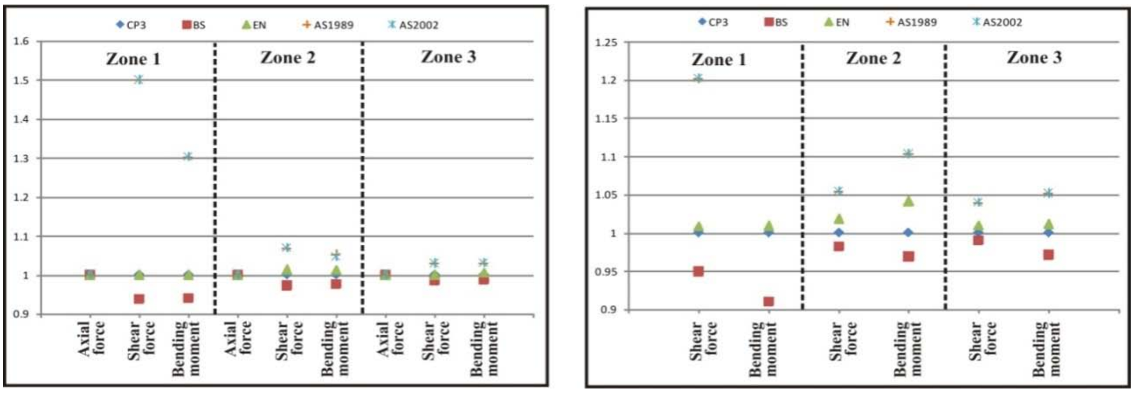
7 WIND INDUCED FORCES

Wind loading standards only facilitate to calculate wind pressures at different heights of the building. Multiplying these values by contributory areas will available the calculation of wind forces at a particular height. However, it is not the actual force experienced by the structural members such as beam, columns, etc. due to the various behaviours of a structure like load sharing among structural members. These actual member forces are necessary to design structural members against lateral loads such as wind load. Actual member forces can be obtained by using finite element 3-D model by applying forces derived from different standards. For the purpose of comparison, the results, obtained are shown as normalised forces. This is a ratio obtained by dividing the force obtained by using any standard divided by same kind of force obtained by using CP 3 Chapter V- Part 2:1972 most common practice in Sri Lanka.

The member forces used for comparison in this study are maximum values of axial forces, shear forces and bending moments in columns, shear forces and bending moments in beams, base moment and base shear at the support level and maximum compressive stresses in shear wall.

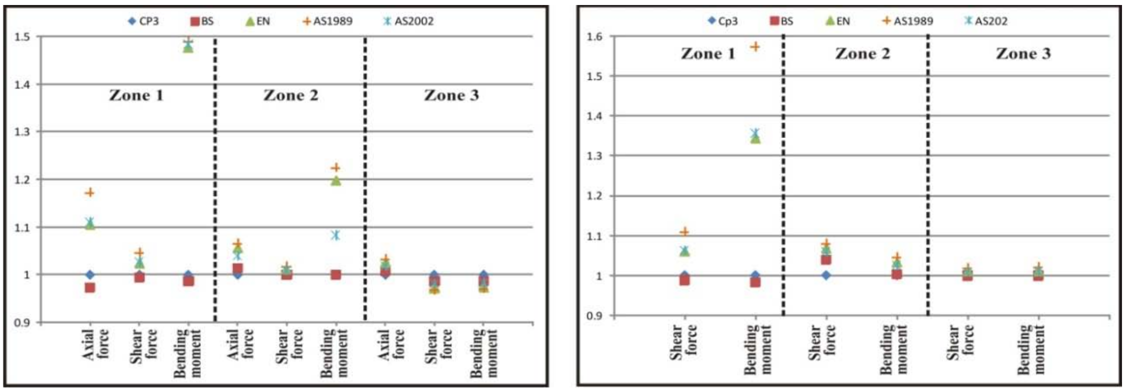
Wind induced forces in columns, beams, supports and walls on 183 m high building in all three zones are shown in Figure 8 to 15. The member forces are calculated for the following load combinations:

1. 1.2(Dead loads) +1.2(Live load) +1.2(Wind load)
2. 1.0 (Dead loads) + 1.4(Wind load)
3. 1.4 (Dead loads) + 1.4(Wind load) and
4. Wind load only.



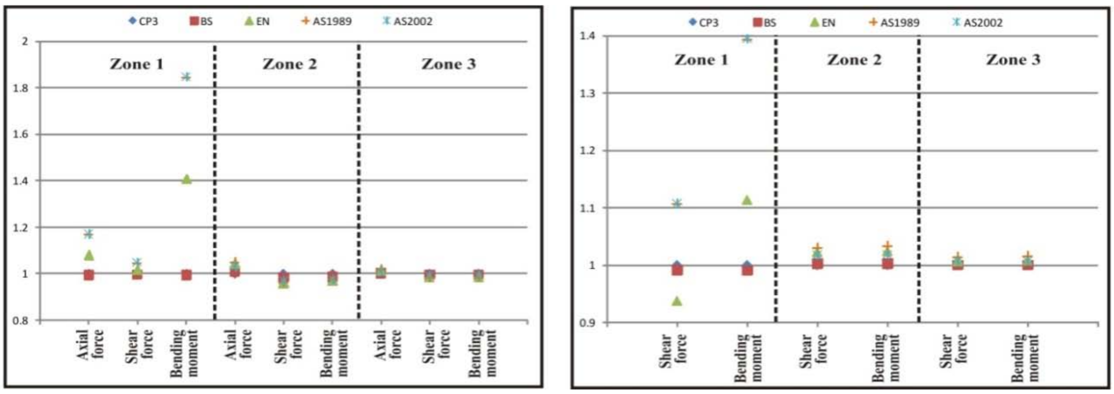
(a) (b)

Figure 8: (a) Column loads (b) beam loads for load combination 1.2G+1.2Q+1.2W (wind flow perpendicular to 46 m side)



(a) (b)

Figure 9: (a) Column loads (b) beam loads for load combination 1.0G+1.4W (wind flow perpendicular to 46 m side)



(a) (b)

Figure 10: (a) Column loads (b) beam loads for load combination 1.4G+1.4W (wind flow perpendicular to 60 m side)

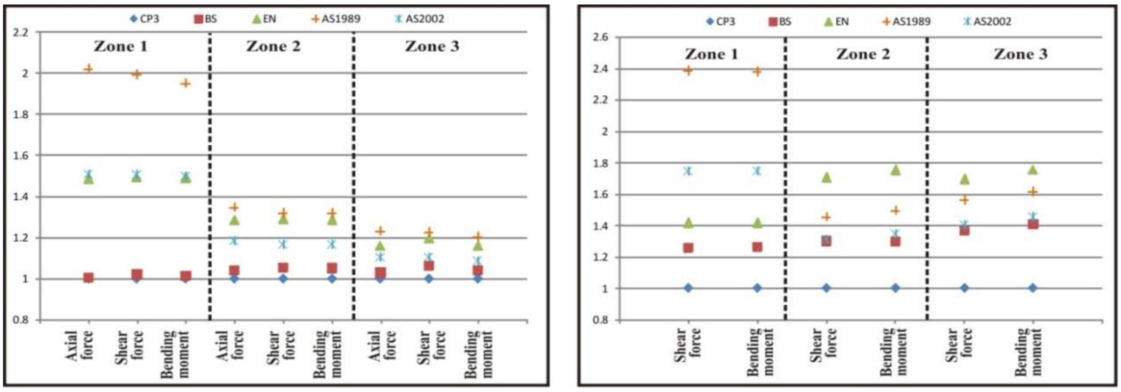
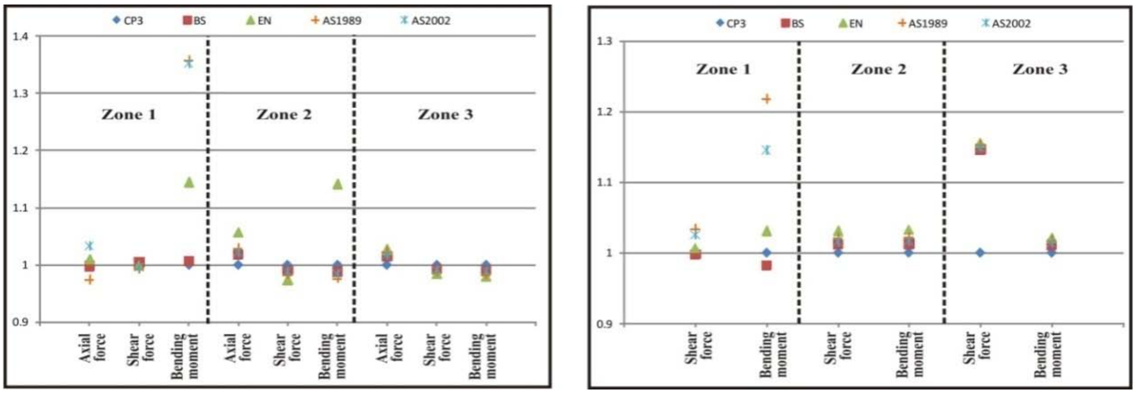
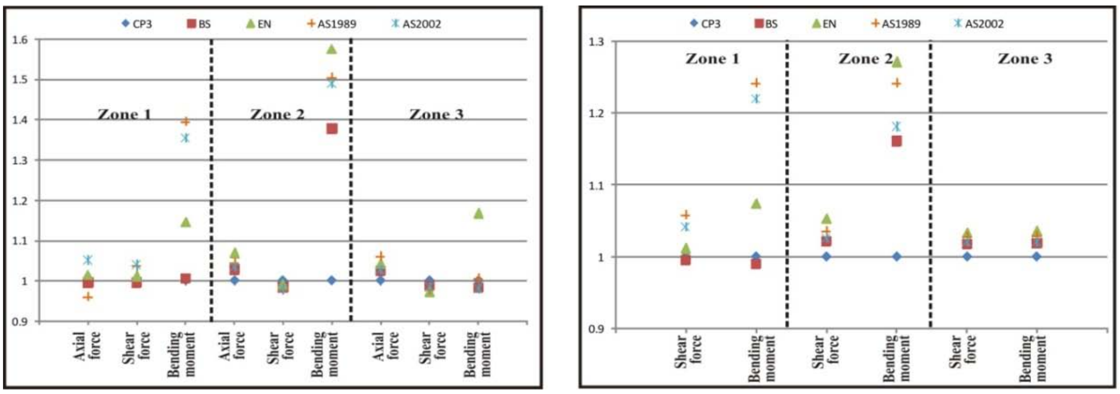


Fig-ure 11: (a) Column loads (b) beam loads for wind load only (wind flow perpendicular to 60 m side)



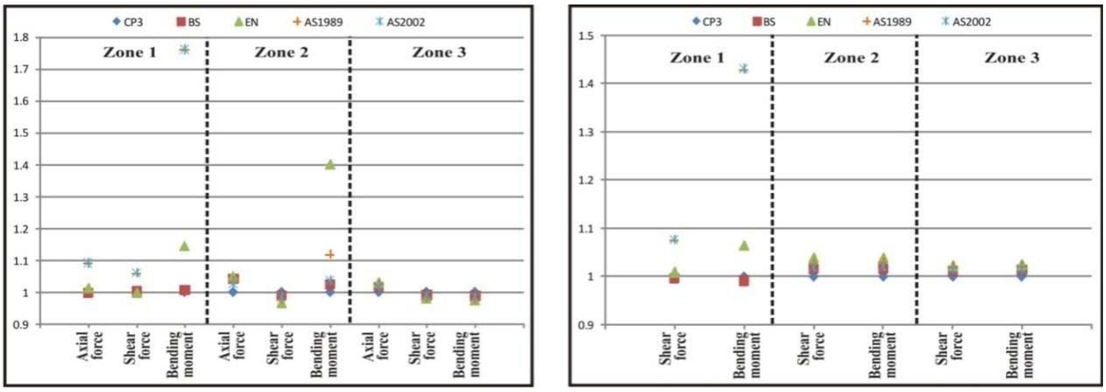
(a) (b)

Figure 12: (a) Column loads (b) beam loads for load combination 1.2G+1.2Q+1.2W (wind flow perpendicular to 30m side



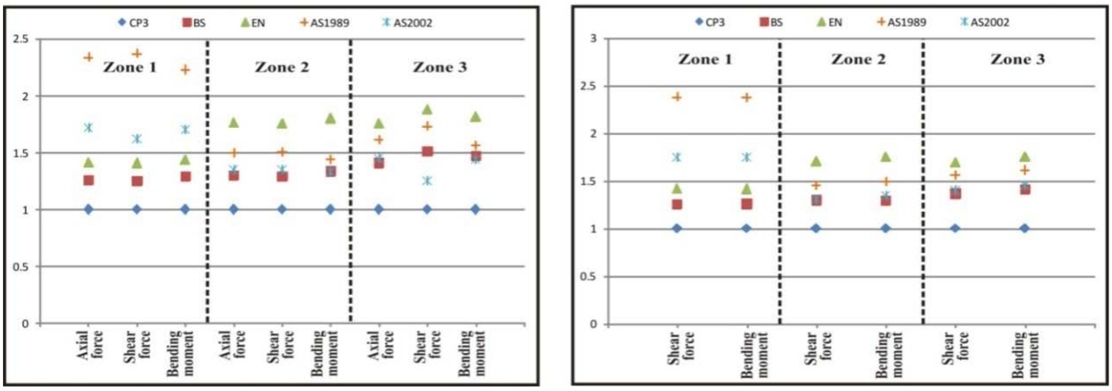
(a) (b)

Figure 13: (a) Column loads (b) beam loads for load combination 1.0G+1.4W (wind flow perpendicular to 30m side)



(a) (b)

Figure 14: (a) Column loads (b) beam loads for load combination 1.4G+1.4W (wind flow perpendicular to 30m long side)

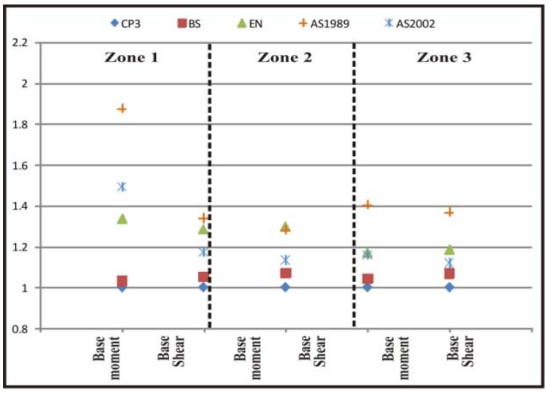


(a) (b)

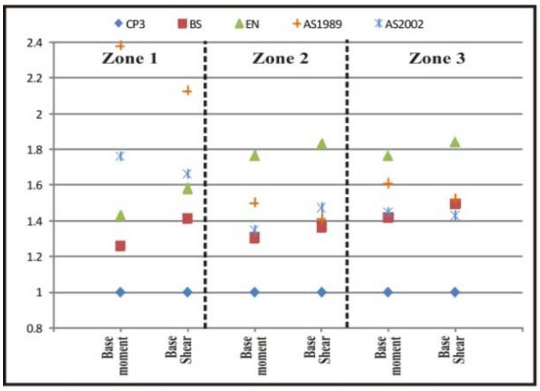
Figure 15: (a) Column loads (b) beam loads for wind loads only (wind flow perpendicular to 30m long side)

The 183 m tall building is more susceptible to wind loading due to its exceptional height. However, the governing load can be observed for load combination of 1.2G+1.2Q+1.2W. The variation in the zone 1 is much larger due to higher wind loads derived from Australian standards, especially for AS 1170.2:1989, which uses an importance factor. Normalised bending moment has maximum variation about 35% in column and about 48% for the beams. However, column maximum axial load variation is in the range of 10%. This value is as high as 17% when wind load is governing as in load combination 1.0G+1.4W. The bending moment value is higher as 50% for the column and more than 55% for beam bending moments for load combination 1.4G+1.4W, higher normalised bending moments can be obtained for column such as value of a 1.8 for wind in both directions in zone 1. However, for the other zones normalised forces are very close to 1. When wind flow perpendicular to 46 m side, normalised forces for Australian standards are higher as 2.0-2.4 in zone 1. These higher values are getting lesser from zone 2 to zone 3. When wind flow perpendicular to 30 m side, normalised forces in zone 1 are as high as 2.5

8 BASE REACTIONS



(a)

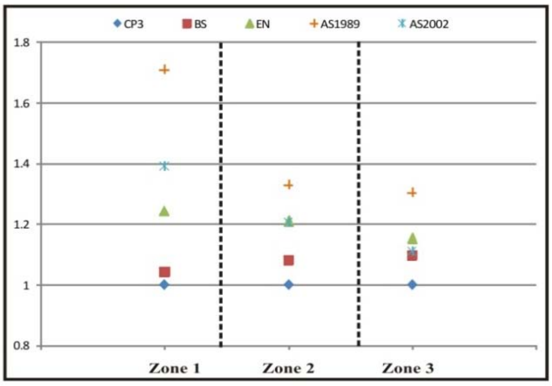


(b)

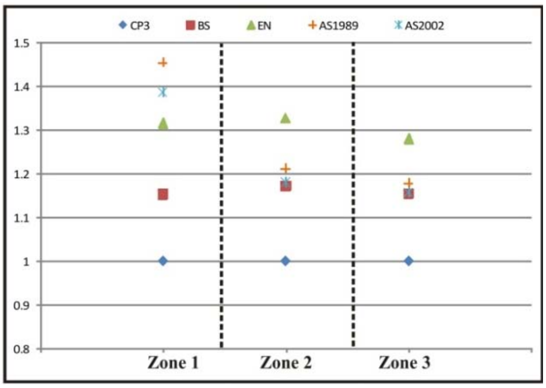
Figure 16: Base moment and base shear of the 183m building (a) wind flow perpendicular to 46 m side (b) wind flow perpendicular to 30 m side.

In zone 1, maximum base moment and base shear can be observed for Australian codes, because of their higher wind speeds resulting from special terrain-height multiplier used in zone1. These values are almost twice the valued derived from CP 3 Chapter V: Part2:1978. However, these codes have a difference for the building due to importance factor used by AS 1170.2:1989. In the zone 2 and zone 3, Euro codes yield higher base moment as well as base shear values. The maximum value 1.6 can be observed in zone 2 for wind flow perpendicular to 30 m side for both 48 m and 183 m building. BS 6399.2:1997 has almost same values for base moment and base shear for 183 m building when wind flow perpendicular to 46 m long side.

9 MAXIMUM SHELL STRESS IN SHEAR WALLS



(a)



(b)

Figure 17: Maximum shell stress in shear wall of the 183m building (a) wind flow perpendicular to 46 m side (b) wind flow perpendicular to 30 m side

Maximum shell stresses can be observed in reinforced concrete shear walls for wind load derived by using AS 170.2:1989 for building. The maximum normalized value is 1.7 in 183 m high building. For 183 m high building wind loads derived from Euro code exert maximum shell stress in zone 2 and 3.

10 DRIFT LIMIT

Wind loading standards and design codes limit the allowable wind drift of the buildings in order to prevent damage to the cladding, partition and interior finishes, to reduce effect of motion perceptibility and to limit the P–Delta or secondary loading effects (Mendis et al, 2007). Therefore, drift limit is checked for 183 m height building in order to determine the buildings would exceed the drift index limit or not. The maximum values of deflection in serviceability limit condition were obtained by wind loads applying to the finite element 3-D model for all three zones. According to the BS 8110-Part 2: 1985, the maximum allowable deflection is calculated as $h_s/500$, where h_s is the storey height for single storey building. Therefore, maximum allowable deflection value calculated for 183 m height building is 366 mm. The average drift index is defined as a ratio between maximum deflections to total height of the building. The calculated drift index values are shown in Table 5.

Table 5: Drift index for 48m and 183 m height buildings in zone 1, 2 and 3

Wind loading standard	Average drift Index		
	Zone 1	Zone 2	Zone 3
CP 3 Chapter V - Part 2:1972	1/961	1/1250	1/1785
BS 6399.2:1997	1/935	1/1219	1/1754
AS 1170.2:1989	1/425	1/862	1/1471
AS/NZS 1170.2:2002	1/565	1/1020	1/1562
BS EN 1991-1-4:2005	1/561	1/1010	1/1538

The generally acceptable average drift index limit for the high rise building is 1/500 (Mendis et al., 2001). By reference to Table 2, only the building model with wind loads derived from AS 1170.2:1989 in zone 1 exceeds the generally accepted drift limit because of it uses both importance factor and the cyclonic terrain-height factor. However, rest of the cases satisfies the drift index requirement. In zone 3, all models have lower drift values, which are approximately half of the threshold value.

11 ALONG WIND AND CROSS WIND ACCELERATION

Only Australian and Euro code facilitate the calculation of the acceleration at top of the building. Euro code provides a method to calculate only along wind acceleration, while both Australian standards provide methods to calculate both along wind and cross-wind accelerations. Australian standards use 5 years return period wind speeds for calculate serviceability limit state conditions, which is obtained by using probabilistic method proposed in BS 6399.2:1997 as shown in Table 6.

Table 6: Wind speeds used for acceleration calculations

Return period	Wind speed (ms^{-1})		
	Zone 1	Zone 2	Zone 3
50 – years	54	47	38
5 - years	46	40	32

According to Table 6, it is clear that the service wind speed values of 5 years return period are closer to wind speed proposed for normal structures in design manual “Design building for high winds-Sri Lanka”. Five years return period wind speed values for zones 2 and 3 have good agreement with values proposed in AS/NZS 1170.2:2002. However, the derived value for zone 1 is higher than the value suggested in Australian standard.

The acceleration values obtained from the calculations are shown in Table 7.

Table 7: Acceleration values at 183 m height in zone 1, 2 and 3

Zone	Wind direction	Acceleration type	Acceleration (ms-2)		
			AS 1170.2: 1989	AS/NZS 1170 .2: 2002	BS EN 1991-1-4: 2005
Zone 1	Normal to 46 m side	Along wind	0.155	0.156	0.134
		Cross wind	0.239	0.233	-
	Normal to 30 m side	Along wind	0.109	0.107	0.094
		Cross wind	0.227	0.221	-
Zone 2	Normal to 46 m side	Along wind	0.076	0.078	0.080
		Cross wind	0.173	0.166	-
	Normal to 30 m side	Along wind	0.051	0.052	0.058
		Cross wind	0.168	0.159	-
Zone 3	Normal to 46 m side	Along wind	0.034	0.034	0.033
		Cross wind	0.118	0.116	-
	Normal to 30 m side	Along wind	0.024	0.026	0.025
		Cross wind	0.106	0.093	-

Euro code yields higher along wind acceleration values than predicted by Australian standards. However, these along wind acceleration values are much less than across wind acceleration values due to slenderness of 183 m height building. Most of the cases, these across-wind acceleration values are exceed the threshold value set for human comfort that is 0.15 ms^{-2}. Due to higher wind speed value in zone 1, along wind acceleration values exceed the threshold value set for human comfort.

12 CONCLUSION

For the governing load case 1.2G + 1.2Q + 1.2W, all wind loading standards gave almost the same wind load except wind loads for the Australian standards in zone 1. Australian Standards gave higher wind loads in zone 1 because of the use of higher terrain-height multiplier and an importance factor for cyclonic region, zone 1. The use of higher terrain height multiplier in cylconic region can be justified because of higher risk level are required to design buildings in cyclonic regions. However,the use of importance factor 1.1 may leads to more conservative wind load design and thus it is recommended not to use it with higher terrain height multiplier. Euro code also derived higher wind loads due to higher pressure coefficient values used by the code.

Neither Design manual “Design buildings for high winds in Sri Lanka” nor CP 3 Chapter V:1972 is adequate to addressed wind loads design for tall buildings. By considering facts like available wind speed data, Australian standards are more suitable than other standards. Between two Australlian standards, newer version AS/NZS 1170.2:2002 is much more convenient due to the simplicity. Therefore, Australian standard AS/NZS 1170.2:2002 can be used with the Sri Lankan context until Country produce its own wind loading standard. As an interim measure, Sri Lanka can prepare a national annex for Euro code , which has adequate flexibility to adjust to country specific factors, which facilitates the adoption to real condition prevailing in Sri Lanka while adhering to an international wind code.

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