STRATEGIES FOR ADOPTING NEW TRENDS IN WIND LOAD EVALUATION ON STRUCTURES

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Abstract: The advancement of knowledge of wind engineering introduces lots of changes to wind loading standards. There are many differences in old codes of practices compare to the newer standards by means of factors, methods and ultimately wind induced forces in structural members. Since tall buildings are more susceptible for wind loads and thus, require more close consideration when they are designed for wind loads. In this study, five major wind loading standards, 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 compared with respect to the CP 3 Chapter V – Part 2, for designing of a 183 m tall building. From one standard to another, factors like basic wind speeds, terrain height multiplier and procedures like analysis methods are different because of strategies set by the conditions and requirements of the country of its origin. The serviceability limit state behaviour of tall building is equally important like ultimate limit state behaviour and hence it discussed with this paper by the means of drift index and along and cross wind accelerations.

Key Words: *Wind loading, Dynamic response*

1. Introduction

The assessment of wind loads on buildings requires knowledge of complex interaction between meteorological, aerodynamic and structural aspects of the problem. The physical modelling of the structure is only viable mean to obtain information about along-wind, cross-wind and torsional effects of the structure resulted from wind loads. However, requirements time, cost and resources discourage the designers to carry out physical modelling. Therefore, as an alternative, wind loading standards have included empirical relationships to produce an estimation procedure to evaluate the dynamic wind response (Kareem and Kijewski, 2001). Basically wind loading standards enable estimating safety and serviceability of a structure with information supplied by meteorology and aerodynamics together with basics of structural theory (Kulousek, 1984). However, due to continuous research work done on wind engineering during last two to three decades lots of improvements have been included in wind loading standards. Old codes were suppressed by the new standards, which are capable of supporting dynamic analysis rather than assuming only quassi –static behaviour of the building and better strategies to assess risk for different types of buildings. The evolution of the tall building design and construction may also enforce the wind loading standards to have methods to predict complex behaviour of the building not only at the ultimate limit state but also at the serviceability limit state as well.
The first mandatory document on wind engineering, the design manual “Design buildings for high wind – Sri Lanka” was published by the Sri Lankan Government in 1980. The design manual was based on the previous code of practise CP 3 Chapter V – Part 2: 1972; this extensively covers the design and construction of low rise buildings (Clarke et al, 1979). However, in recent times, there has been a national trend to build tall, slender tower type high rise buildings in Sri Lanka especially in Colombo city limit (Karunarathne, 2001). Not Only due to their heights but also light materials used as building materials for both super structure and the inner partition walls and some complex architectural features, these buildings may be prone to excessive dynamic motion induced by winds. Neither design manual nor CP 3 Chapter V-Part 2: 1972, adequately address this kind of complex situations. This means that there is a need to go for a wind loading standard, which can cover more complex wind spectrum as well as dynamic effects arising from the wind. Therefore, designers and structural engineers of Sri Lanka have been looking for advance wind loading standards, which are capable to evaluate more complex dynamic behaviours. International wind loading standards such as Australian, British, American, Japanese and Euro codes have been used by Sri Lankan engineers.

However, use of different wind loading standards in a given design may lead to severe problems such as poor understanding about the use of country specify factors in conjunction with Sri Lankan context, some inconveniences about understanding and comparing wind load calculations, lack of harmonization among wind load design of structures, etc. Therefore, it is necessary to have a broad and clear idea about strategies adopted by different wind loading standards before carrying out any wind load design of a building.

2. Different strategies adopted by wind codes and standards

2.1 Selecting codes and standards for the study

Due to the incapability of design manual to address the issues of tall building design, many Sri Lankan engineers 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 that are found in Sri Lankan civil engineering sector, following codes and standards were chosen for the comparison purpose. The selected codes and standards are 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.

There are different strategies that can be clearly identified from these selected codes and standards. CP 3 Chapter V-Part2:1972 uses quasi-static method to calculate wind loads on a building, this quassi static approach is 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 with this code because of its simplicity and familiarity of the code. BS 6399.2:1997 is the newer version of the British standard and capable to handle 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 Gust factor 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 made it as a simple document to use. Apart from these reasons, Australian standards cover wide spectrum of wind, including cyclones and it is used by many island nations such as Fiji, Solomon Island, etc. EN 1991-1-4:2005 is the newest code and not only it compromises many aspects present in other codes such as BS 6399.2:1997, AS/NZS 1170.2:2002. However, it allows to adjust the methods and factors which are suitable for own country by means of a national annex.

2.2 Basic wind speeds

The design manual defined two types of 3 – second gust wind speeds for three wind zones in Sri Lanka as shown in Table 1. The basic wind speed can vary with different average times. The averaging time depends on some facts such as long enough to allow the non stationary phenomena to decrease to a minimum, long enough to allow recording of steady vibration during a simultaneous examination of the structural response, short enough to provide a true picture of wind gusts of short duration and long enough to allow the application of the wind speed measurement methods used in meteorology. The conversion between two averaging times can be done by using some graphical method like Durst method or by using some empirical relationship like one proposed by Cook (1999) to convert 3 second gust wind speed to mean hourly wind speeds or use a constant value as a conversion factor as ICEUK proposed 1.06 for convert mean hourly wind speeds to 10 minute mean wind speed. The basic wind speeds used for different standards is shown in Table 2.

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

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

2.3. Pressure coefficient

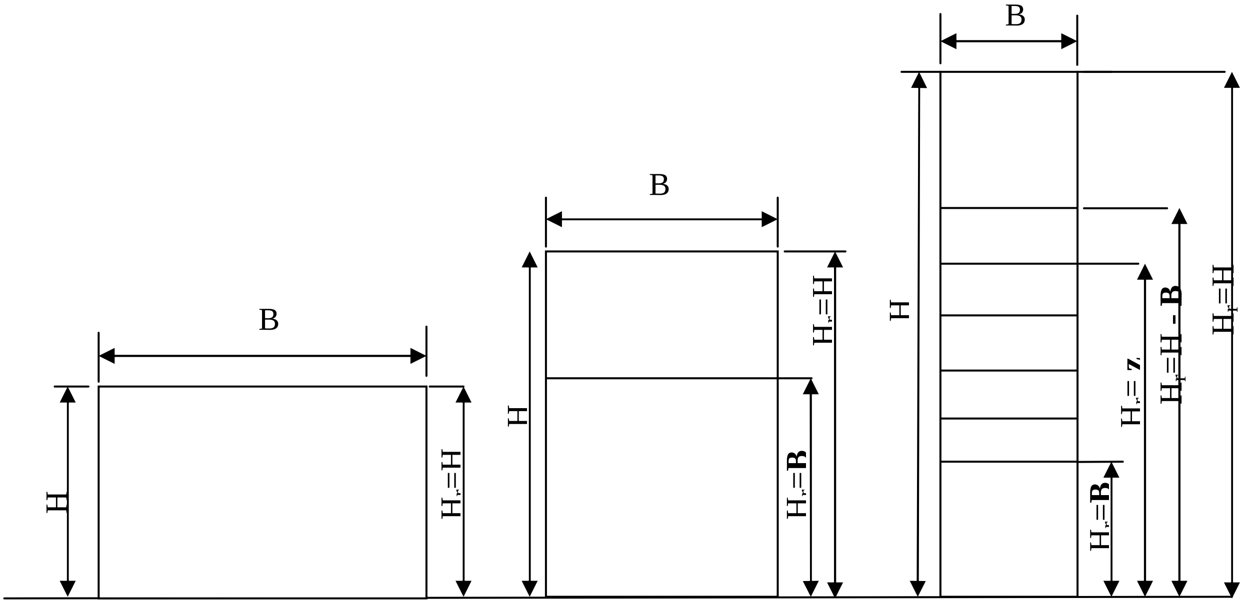
The total pressure mainly depends on three factors namely external pressure coefficients, internal pressure coefficients and wind speed at that height. The external pressure coefficients (C_{pe}) used in international standards are different from one another due to their own methods of determinations and the policies adopted by the country. Internal pressure coefficient values are also not same in codes due to their national practices. The external and internal pressure coefficient values used for 183 m high building with rectangular plan dimension of 46 m x 30 m are shown in Table 3.

Table 3. External and internal pressure coefficient

Standard		$C_{pe,windward}$	$C_{pe,leeward}$	C_{pi}
CP3 Chapter V-Part 2: 1972	46 m side	+0.70	-0.40	+0.2 or -0.3
	30 m side	+0.80	-0.10	
BS 6399.2:1997	46 m side	+0.80	-0.30	-0.3 or +0.2
	30 m side	+0.80	-0.30	
BS EN 1991-1-4:2005	46 m side	C_f= 1.3		
	30 m side	+0.80	-0.65	-0.3 or 0.0
AS 1170.2:1989	46 m side	+0.80	-0.50	-0.2 or 0.0
	30 m side	+0.80	-0.39	
AS/NZS 1170.2:2002	46 m side	+0.80	-0.50	-0.2 or 0.0
	30 m side	+0.80	-0.39	

2.4 Pressure distribution along the building height

Newer codes like British and Euro codes use 'division – by - parts' rule to distribute wind pressure along the building height as shown in Figure 1. However, in CP 3 Chapter V and Australian standards use dynamic wind pressure continuously change with the building height. The suction pressure variation in leeward side is not defined in many codes except Australian codes defined it as uniform pressure and value is equal to the pressure at top of the building.



One part when H<B   Two parts when B<H<2B   Multiple parts when H>2B

Figure 1. Division of buildings by parts for lateral loads (BS 6399.2:1997)

2.5 Analysis methods used in different standards

CP 3 Chapter V – Part 2:1972 uses quassi –static method to assess the wind loads on the building, which is more suitable When the structure is very stiff, the deflections under the wind loads would not be significant and the structure is said to be 'static' (Dyrbre, 1999). However, slender structures are more susceptible to dynamic motion in both parallel and perpendicular to the directions of the wind. Dynamic analysis used in AS 1170.2:1989 is the gust factor method, which uses stochastic dynamics theory to translate the dynamic amplification of loading, caused by turbulence and the dynamic sensitivity of the structure, into an equivalent static loading (Kijewski and Kareem, 2001). However, new version of the Australian Standards AS/NZS 1170.2:2002 uses a dynamic factor which encounters factors such as background factor, resonant factor, etc. The dynamic analysis method used in BS 6399.2:1997 is based on equivalent static method with dynamic augmentation factor which depends on building type and this method limits to use with building less than 200 meter high. Euro code defines a factor called structural factor should take into account the effect on wind actions from the non simultaneous occurrence of peak wind pressures on the surface together with the effect of the vibrations of the structure due to turbulence.

3. Case study for comparison

A 183 m high rectangular shaped building was modelled and analysed by using SAP 2000 software, in order to determine dynamic behaviour of tall building and the effect of using various standards to calculate the wind induced behaviour. The plan dimensions of the building are 46 m x 30 m (Figure 2(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 the 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 2 (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 has used with special terrain – height multiplier as given in AS 1170.2:1989, but only higher terrain –height multiplier used for cyclonic region to calculate wind loads according to the AS/NZS 1170.2:2002.

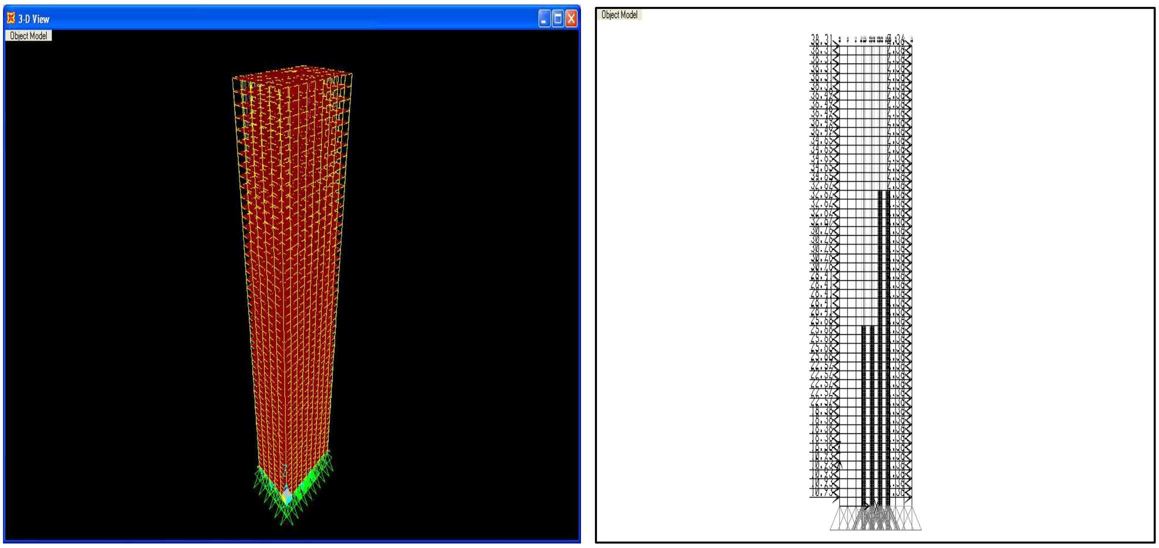


Figure 2: (a) Finite element 3 – D model of 183 m height building (b) Wind loads applied in windward and leeward sides of the 183 m high building

4. Wind Pressures and Wind induce forces

4.1 Comparison of wind pressure

The calculated wind pressure values by using different standards are showing in Table 4 and the difference of those pressure values compared with respect to the CP 3 Chapter V – Part 2:1972, which uses quassi –static approach to calculate wind pressure.

Table 4: Comparison of wind pressure at 183m height in zone 3

	CP3 Chapter V-Part2:1972	BS 6399.2 : 1997	BSEN 1991-1-4 :2005	AS 1170.2: 1989	AS 1170.2: 2002
Basic wind speed (ms^{-1})	38	21	22	38	38
Terrain height multiplier	1.172	2.18	1.71	0.806	1.23
Design wind speed (ms^{-1})	44.54	45.78	37.69	33.69	46.74
Dynamic wind speed (Nm^{-2})	1190	1257	1227	680	1311
Dynamic response factor	-	1.125	0.972	2.085	0.918
External pressure coefficient-Windward	+0.7	+0.8	+0.8	+0.8	+0.8
External pressure coefficient-leeward	-0.4	-0.3	-0.65	-0.5	-0.5
Internal pressure coefficient	0.2	0.2	-0.3	-0.2	-0.2
Total Pressure at 183 m height (kNm^{-2})	1.309	1.335	1.663	1.802	1.564
% difference at the top with respect to CP3 Chapter V-Part 2:1972	0	2.0	27.0	37.7	19.5

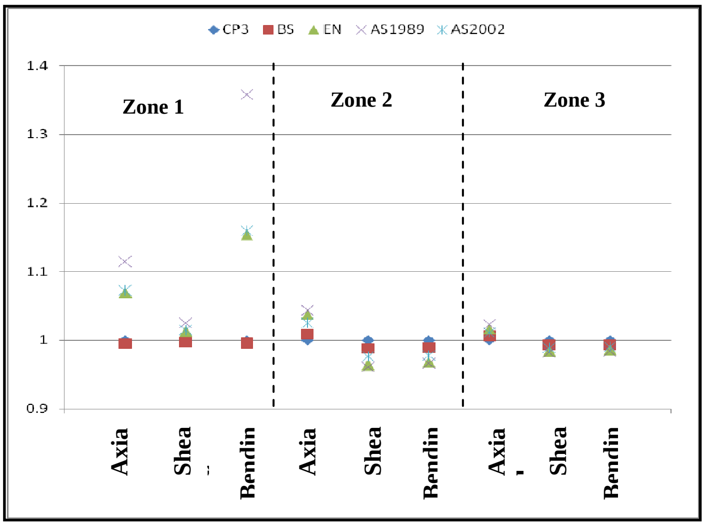
According to the Table 5, it can be seen that there is only 2% difference in pressure, when it calculated with CP 3 Chapter V-Part 2 and BS 6399.2:1997. The AS 1170.2:1989 standard has much larger pressure difference because the use of importance factor for its calculation. A higher percentage of pressure difference in Euro code is primarily resulted due to its higher negative pressure coefficient in leeward face of buildings. The pressure difference at the top most level of the building for AS/NZS 1170.2:2002 is about 20%, compared with the CP3 Chapter V-Part 2:1972, where both codes use 3 second gust wind speed for its calculations.

4.2. Comparison of structural member forces

Wind loading standards only facilitate to calculate wind pressures at different heights of the building. Multiplying these values by contributory areas will enable 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 obtained results are shown as normalised forces. This is the ratio between a force obtained from a particular standard and the same force obtained from 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.
The member forces are calculated for the following load combinations:

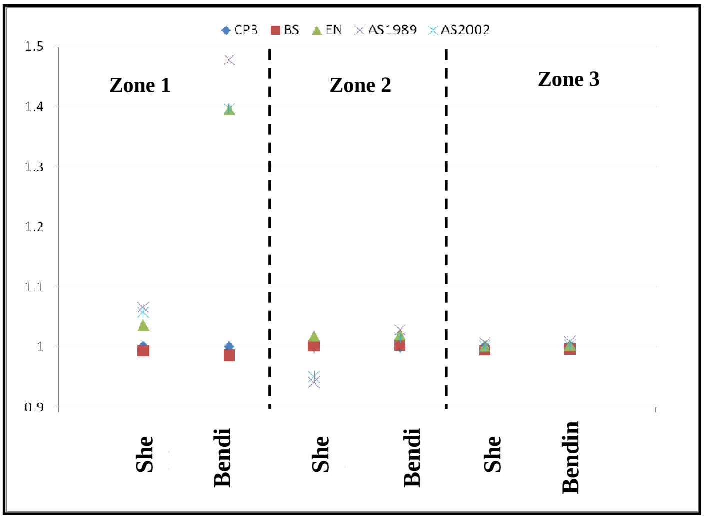
1. 1.2(Dead loads)+1.2(Live load)+1.2(Wind load)
2. 1.0 (Dead loads) + 1.4(Wind load)
3. 1.4 (Dead loads) + 1.4(Wind load) and
4. Wind load only.

Wind induced forces in columns and beams, on 183 m high building for governing load case 1.2G+1.2Q+1.2W in all three zones are shown in Figures 3(a) and (b), respectively.



(a)

Figure 3(a): Column loads for load combination 1.2G+1.2Q+1.2W
(wind flow perpendicular to 46 m long side)

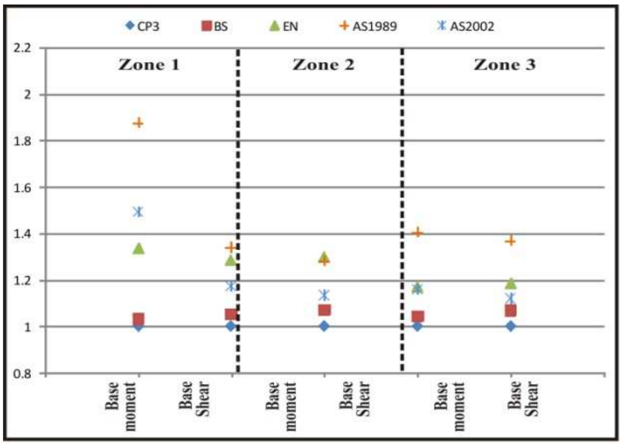


(b)

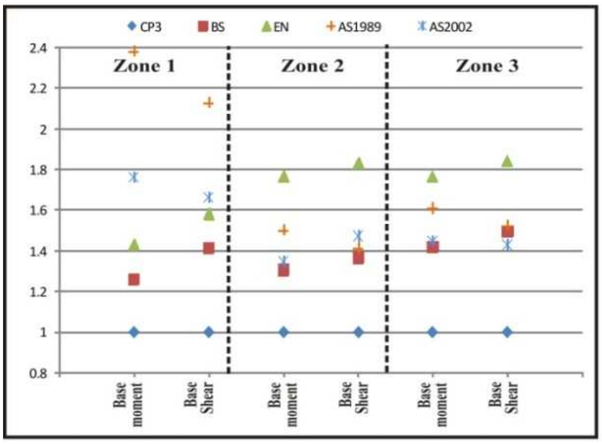
Figure 3(b): Beam loads for load combination 1.2G+1.2Q+1.2W
(wind flow perpendicular to 46 m long side)

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 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 as importance factor 1.1 in zone 1. 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. For wind load only case, the variation is much larger as 200% to 250% in zone 1 for Australian standards. It should be noted that although this variation is obtained with 1.0G + 1.4W and 1.4G + 1.4W cases, still the significance of this variation on design calculation would be to a lesser degree.

5. Base reactions



(a)

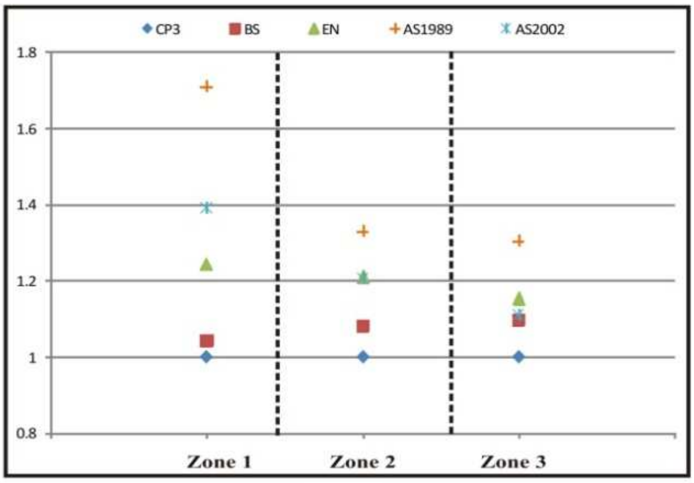


(b)

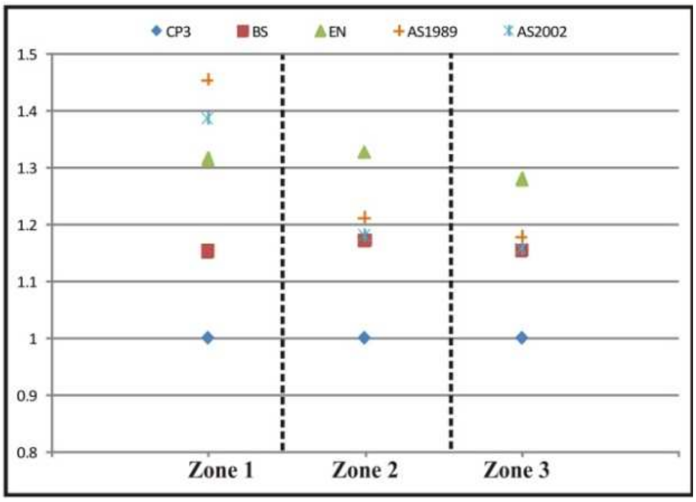
Figure 4: Base moment and base shear of the 183m building (a) wind flow perpendicular to 46 m wall (b) wind flow perpendicular to 30 m wall.

According to the Figures 4(a) and 4(b) 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 in 183 m 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 shears values. The maximum value 1.6 can be observed in zone 2 for wind flow perpendicular to 30 m side of the 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.

6. Maximum shell stress



(a)



(b)

Figure 5: 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

The absolute maximum principal stresses in the shear walls, which are induced by the dead live and wind load were used for the comparison purpose and results are shown in Figures 5(a) and 5(b). Maximum shell stress can be observed for wind load derived by using AS 1170.2:1989 for the 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.

7. Drift index

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 tall building in order to determine whether 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 183 m height building 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 it uses both importance factor and the cyclonic terrain-height multiplier. 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.

8. Along wind and cross wind acceleration

Only Australian and Euro codes facilitate the calculation of acceleration at top of the building. Euro code provides a method to calculate only the 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

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 Australian codes. 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 could exceed the threshold value set for human comfort that is 0.15 ms^{-2}. Even for higher wind speed value in zone 1, along wind acceleration values do not reach the threshold value set for human comfort.

9. Conclusion

International wind loading standards have their own prefferences over choice of different basic wind speeds with averaging time and analysis methods and pressure coefficient values. Therefore, for the same building design, wind pressures obtained from different standards are not the same. The dynamic analysis methods give higher wind pressure values compared to the values obtained from the quassi- static method. However, wind pressure values obtained from the dynamic analysis are varying because of strategies adopted by standards such as pressure distribution according to the "Division –by- Parts" rule, higher pressure coefficient values, etc. Ultimately the building design should have sound safe and satisfactory behaviour in both serviceability and ultimate limit states. Therefore, the use of higher terrain height multiplier can be justified for the wind zone 1 in Sri Lanka for ultimate limit state design, which has higher probability of being hit by a cyclone. However, use of both terrain-height multiplier and an importance factor may lead to a more conservative design and thus, it is recommended not use both in one design. The cross-wind acceleration is more important than the along wind acceleration for tall slender buildings. According to the case study, for the wind loads derived from previous Australian standard exceeds the threshold value in zone 1, because it used both importance factor cum higher terrain height multiplier for its calculation. The same trend can be observed in drift index calculation that allowable drift index value was exceeded by the AS 1170.2:1989 in zone 1 but for other standards they are within the limit. Hence, an international standard used in any other country than its origin, it is recommended to carry out a detail analysis in order to determine the strategies adopted by those standards.

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References

1. Australian standard for wind loads ; AS 1170.2:1989, Standards Australia, New South Wales, Australia
2. Australian and New Zealand standards: Structural design actions Part 2: wind actions; AS/NZS 1170.2:2002, Standards Australia, New South Wales, Australia
3. British Standard: Eurocode 1: Actions on Structures – Part1- 4: General actions - wind actions; BS EN 1991-1-4:2005 , British Standard Institution, London
4. British Standard: Loading for buildings - Part 2: Code of Practice for wind loads; BS 6399- 2:1997, British Standard Institution, London
5. Clarke, A.G., Swane, R.A., Schneider, L.M, Shaw, P.J.R., Technical assistance to Sri Lanka on Cyclone Resistant Construction, Vol 1, Part 1 -4, 1979
6. Cook, N. J., Wind loading, A practical guide to BS 6399-2 Wind loads for buildings, Thomas Telford, 1999
7. CP 3 Chapter V: 1972, Code of Basic data for the design of buildings chapter V. Loading, Part 2 Wind Loads, British Standard Institution, London
8. Cyclone Events 1900 – 2000, Available Source, www.meteo.gov.lk, last acces on 03/09/2010
9. Dyrbye, C., Hansen, S., O., Wind Loads on Structures, John Wiley & Sons, 1999
10. Karunaratne, S.A., High rise buildings in Colombo (Structural Engineer's View), Proceedings of the international conference on "Advances in Continuum Mechanics, Materials Science, Nano science and Nano technology: Dedicated to Professor Munidasa P. Ranaweera", University of Peradeniya, Sri Lanka, 2008. pp 291- 302
11. Kijewski, T., Kareem. A., Dynamic wind effects, A comparative study of provision in codes and standards with wind tunnel data, 2001
12. Kolouske, V., Pirner, M., Fischer, O., Naprstek, J., Wind effects on civil engineering structures, Elsevier, 1984
13. Mendis, P., Ngo, T., Hariots, N., Hira, A., Samali, B., Cheung, J., Wind Loading on Tall buildings, EJSE special issue; Loading on structures, 2007. pp 41-54
14. Report on the Calibration of Euro code for wind loading (BS EN 1991 - 4) and its UK National Annex against the current UK wind code (BS 6399: Part 2:1997)

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