Study of the effectiveness of Outrigger system for high-rise composite buildings for Cyclonic Region.

S. Fawzia, A. Nasir and T. Fatima

Abstract**—** The demands of taller structures are becoming imperative almost everywhere in the world in addition to the challenges of material and labor cost, project time line etc. This paper conducted a study keeping in view the challenging nature of high-rise construction with no generic rules for deflection minimizations and frequency control.

The effects of cyclonic wind and provision of outriggers on 28-storey, 42-storey and 57-storey are examined in this paper and certain conclusions are made which would pave way for researchers to conduct further study in this particular area of civil engineering.

The results show that plan dimensions have vital impacts on structural heights. Increase of height while keeping the plan dimensions same, leads to the reduction in the lateral rigidity. To achieve required stiffness increase of bracings sizes as well as introduction of additional lateral resisting system such as belt truss and outriggers is required.

Keywords**—** Cyclonic wind regions, dynamic wind loads, Along-wind effects, Crosswind response, Fundamental frequency of vibration.

I. Introduction

This paper discusses the deflection control and frequency optimization by using belt truss and outrigger system for various height of building structure. Similar study is conducted before by Fawzia et al [1], where; authors have compared deflection variation by using up to three belt truss and outrigger system for same height. However; current paper will outline the comparison of belt truss and outriggers using 28-storey, 42-storey and 57-storey high building models i.e. 98 m, 147 m, and 199.5 m respectively. The lateral loads used are Wind Cyclonic conditions as outlined in Australian Standard [2]. These prototypes are constructed in Strand7 [3] and an initial model is run for natural frequency of vibration. The frequency values of basic models (i.e. model without outrigger systems) are used to calculate along-wind and crosswind actions on building. The deflection variations under these loads are analyzed by providing various combinations of bracings systems (i.e. core walls, outriggers and belt truss).

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II. Background

In last few decades, city growth has become a significant trait of urban development worldwide. These demographic changes have influenced the life style of common people that include livelihood standard, approach to amenities, eating habits, economic levels and living spaces etc. The present trend of people moving toward metropolis has caused scarcity of living space within cities and thus; demanding them to grow upwards i.e. triggering the construction of taller and taller structures.

Mendis et al [4], proposed that this demand is always auxiliary to a multitude of variables, such as strength, durability, forming techniques, material characteristics, nature and extent of reinforcement, aesthetics and much more. Thus; design intent has always been to accomplish an understandable necessity through communities to erect or re-erect structures deemed to be affordable and safe during their life span. The structure or building must be converted into or remain a natural part of the developed milieu.

Gabor [5], states that the main aim of structural design is to provide a safe load path during any stage of construction, building lifespan and while its demolition, under all possible loads and effects and within acceptable risk limits set up by society.

Nevertheless; a progressively aggressive construction industry stipulates cost effectiveness besides architecturally challenging structures that compel engineers to devise newer techniques to innovate and apply mix and match approach to the available material and resources. As mentioned by Ali [7], that an innovative system of casting square, twisted, steel bars with concrete as a frame with slabs and concrete exterior walls was used in the Ingalls Building in Cincinnati, Ohio, the first 15-story concrete "skyscraper" built in 1903 by Elzner. The introduction of composite construction to tall tubular buildings, first conceived and used by Fazlur Khan of Skidmore, Owings & Merrill (SOM) in the 1960s, has paved the way for super-tall composite buildings like the Petronas Towers and the Jin Mao building in the present era. Consequently; tall building construction is promptly transforming and its frontiers are continually being assessed and extended. The super tall buildings such as the Burj Khalifa, under construction 151 storey Incheon Tower in South Korea and proposed 1 km tower in Saudi Arab are all instigated by such innovations [4].

The fundamental design criterions for high-rise building are strength, serviceability and stability whereas Jayachandran [7] also includes human comfort into these. According to the guidelines of Australian Standards [2,8,9] and [10], Stability and Strength are covered by Ultimate limit state design while

Serviceability limit state applies to short and long term deflections (that includes creep and shrinkage) of whole structure as well as its components.

Outrigger systems are generally very effective in fulfilling the serviceability requirements of tall buildings. Rahgozar et al [11], states that in this system, columns are tied to the belt trusses. Therefore, in addition to the traditional function of supporting gravity loads, the columns restrain the lateral movement of the building. When the building is subjected to lateral forces, tie-down action of the belt truss restrains bending of the shear core by introducing a point of inflection in its deflection curve. This reversal in curvature reduces the lateral movement at the top. The belt trusses function as horizontal fascia stiffeners and engage the exterior columns, which are not directly connected to the outrigger belt truss.

Outriggers have been in constant used in various high-rise developments as mentioned above, however; their use and provision is specific to a particular construction or building structure. Usually structural engineers have to conduct a rigorous analysis with trial and error approach before a conceptual set of information can be achieved, that enable them to estimate certain primary information required by developers or clients, before beginning of a project. Hence; certain generic rules and principals are needed that can help structural designer to compute requirement of bracings (i.e. core walls, outriggers, belt truss etc.) based on structure height and plan dimensions (i.e. width and length). These interns would be helpful in the approximate judgment of various quantities and cost (i.e. material, labor coast, project time line etc.) without indulging in rigorous analysis and wind tunnel testing at initial stage of project design work.

This investigation tries to move the academic research towards fulfilling the gap in this essential and critical aspect of civil engineering.

III. Loadings

The actions or loads acting on tall buildings can be broadly classified into two types;

Gravity Loads

Lateral loads

A. Gravity loads;

The loads acting downwards because of the effect of acceleration due to gravity are termed as gravity loads. These intern generally classified in three types as:

Inherit self weight of structural elements depending on member size and material properties.

Superimposed dead loads.

Live loads.

1. Self weight of structure;

Structural self weight is adopted for the prototype as follow;

i) Composite Slab;

Slab overall depth is 120mm including 1.0 BMT Lysaght Bondek [12] metal sheeting. Equivalent Elastic modulus and density is entered in the Strand7 [3] model. The overall depth is selected as per the loads assumptions given in onesteel table [13] for primary and secondary beams sizes.

ii) Secondary beams and Primary beams ;

Structural steel secondary beams and primary beam are provided with approximate sizes as given in the Onesteel tables [13]. These sizes are adopted readily based on assumptions of superimposed loads and live loads provided in the table for typical office floor and given span lengths.

iii) Composite column ;

The weight of a column is a characteristic of its cross-sectional area and material density. The cross-sectional area depends on the loads carried by column as per its tributary area on each level/floor.

The size of column provided other variables remain constant, is directly proportional to the load it carries, hence; Cross-sectional area reduces as building height increases.

iv) Reinforced concrete (RCC) wall;

Reinforced concrete wall self weight is also a characteristic of its cross-sectional area, however; the gravity loads they carry are usually far below their capacities. The walls are mainly treated as lateral load resisting elements and effective in controlling the lateral deflections and fundamental frequency of structure. The thicknesses satisfy the minimum requirement of Building code of Australia [14], for fire and durability as well as to provide enough rigidity in order to keep the fundamental frequency of vibration under certain limits so that the Australian Standard prescribe limits [2] could be applicable.

2. Super Imposed Dead loads (SDL);

Superimposed dead loads consist of loads of permanent fixtures and fittings such as ceilings, air-conditioning ducts, floor finishes, partitions etc.

In this model approximate value of superimposed dead loads i.e. 1.5 kN/m^2 as describe in Onesteel tables [13] for a typical office building is adopted.

3. Live loads (LL);

Live loads mainly correspond to human loads and they are highly variable. Australian standard [15], recommend live load reduction based on tributary areas to account for their wavering effects.

Typical office LL is adopted for this paper is 3 kN/m^2.

B. Cyclonic wind load

The structure is tested under worst wind loads for Cyclonic region D as per Australian wind standard [2], whereas; guidelines of Australian standard for general principals [8] are followed for the return period for serviceability limit state.

The model is tested for X and Y wind direction initially to establish the direction of worst loads effects. In this instance it came out to be the Y-direction. The models are then checked in Y- direction wind loads for Along-wind, Crosswind and combine load effects.

The main parameters are;

Basic Wind Speed = 53 m/s

Cyclonic Wind Region = D

Average recurrence interval (R) = 25 yrs

Terrain Category = Category 1

The site wind speed is given by;

$V_{sit,\beta} = V_R\, M_d\, (M_{z,cat} \times M_s \times M_t)$ m/s

Where:

β = cardinal direction clockwise from true north.

M_d = wind direction multiplier
M_s = shielding multiplier
M_t = topographic multiplier
$M_{z,cat}$ = terrain/height multiplier. (It varies with structural height however; for calculating the dynamic load factor "z" is taken equal to "h").
Design wind pressure is given as:
p = (0.5 ρ_{air}) $[V_{des,\theta}]^2 C_{fig} C_{dyn}$ (kPa)
Where:
ρ_{air} = density of air
V_{des},θ = building orthogonal design speed
C_{fig} = aerodynamic shape factor. It is calculated assuming an effectively sealed environment within the building.
C_{dyn} = The dynamic response factor .

i) Along-Wind response:

Dynamic response factor (C_{dyn}) represents Along-wind response in wind sensitive structures such as high rise buildings, where fundamental frequency of vibration falls between the range of 0.2 Hz to 1.0 Hz. It is calculated as:

$$C_{dyn} = \frac{1 + 2I_h\sqrt{g_v{}^2 B_s + \dfrac{H_s g_r{}^2 S E_t}{\zeta}}}{(1 + 2g_v I_h)}$$

ζ = ratio of structural damping to critical damping of a structure.
I_h = Turbulence intensity obtained by setting "z = h" & Terrain category 1.
g_v = peak factor for the upwind velocity fluctuations.
B_s = Background factor given as follow:

$$B_s = \frac{1}{1 + \dfrac{\sqrt{0.26\,(h-s)^2 + 0.46B_{sh}{}^2}}{L_h}}$$

h = average roof height of structure above ground (m).
s = height of the level where action effects are calculated (m).
b_{sh} = average breath of structure between height h and s (m).
L_h = a measure of the integral turbulence length scale at height h given as: **85 $(h/10)^{0.25}$** (m).
H_s = height factor for the resonant response which equals to: **$1 + (s/h)^2$**
g_R = peak factor for resonant response (10 min period), given by : $\sqrt{2\log_e(600n_a)}$
n_a = first mode natural frequency of vibration in along wind direction obtained from computer analysis (Hz).
S = size reduction factor given as:

$$S = \frac{1}{\left(1 + \dfrac{3.5n_a h\,(1+g_v I_h)}{V_{des,\theta e}}\right)\left(1 + \dfrac{4n_a b_{0h}\,(1+g_v I_h)}{V_{des,\theta}}\right)}$$

b_{0h} = average breath of structure between height 0 and h (m).
$V_{des,\theta}$ = design wind speed determined at building height h.
E_t = (p/4) times spectrum of turbulence in the approaching wind stream, given as:

$$E_t = \frac{\pi N}{(1 + 70.8N^2)^{5/6}}$$

N = reduced frequency (non dimensional) given as:
$n_a L_h\,[1 + (g_v I_h)] / V_{des,\theta}$

ii) Crosswind Response:

The equivalent static crosswind force per unit height is given by:
weq (z) = 0.5ρ_{air} $[V_{des,\theta}]^2$ d (C_{fig} C_{dyn}) N/m
Where;
d = horizontal depth of structure parallel to wind stream
(Cfig Cdyn) = the product of effective aerodynamic shape factor and dynamic response factor is given as;

$$1.5\, g_R \left[\frac{b}{d}\right] \frac{K_m}{(1 + g_v I_h)^2} \left[\frac{z}{h}\right]^k \sqrt{\frac{\pi C_{fs}}{\zeta}}$$

k = mode shape power exponent for the fundamental mode.
I_z = turbulence intensity at **2h/3** of building height (use interpolation if required).
z = reference height on structure above average ground level (m).
K_m = mode shape correction factor for crosswind acceleration, given by; **K_m = 0.76 + 0.24k**
V_n = reduced velocity (m/s), calculated as:

$$V_n = \frac{V_{des,\theta}}{n_c b(1 + g_v I_h)}$$

n_c = first mode natural frequency of vibration in the cross wind direction obtained from computer analysis (Hz).
b = breath of structure normal to wind direction (m).
C_{fs} = crosswind force spectrum coefficient.

As the wind actions are trapezoidal in nature i.e. varies with height, these are generated by using Excel sheet for each building type .These are then applied in Strand7 [3], as uniformly distributed horizontal force in kN/m to each storey.

IV. Framing Layout

The model layout selection is primarily dictated by Australian Standards limitations and applications. The current Australian standard [2] is only applicable for building heights up to 200 m and frequency range from 0.2 Hz to 1.0 Hz. Therefore the maximum model height is chosen within these limitations.

The model layout selected in this instance is L-shaped with walls on the right and left hands as well as top left corner of building (Fig. 1). The height of each storey level is 3.5 m which a typical office level used in the country and it can accommodate the service ducts etc.

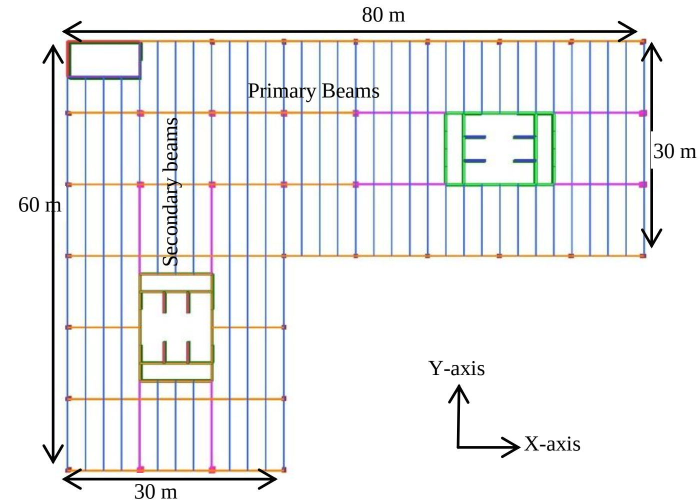

Fig. 1 Plan of typical floor level

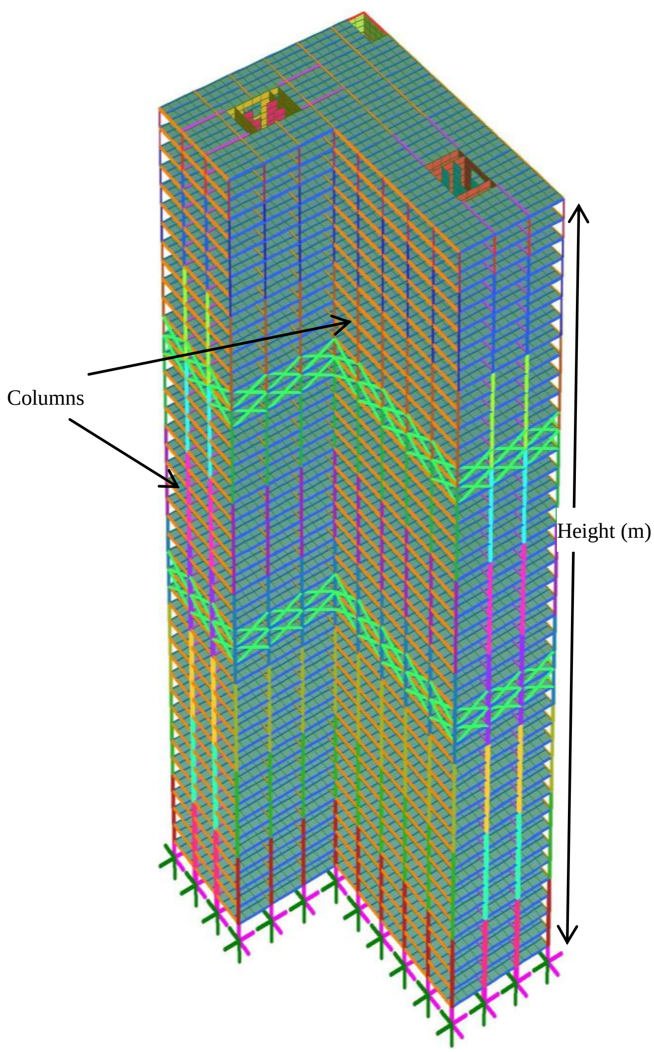

Fig. 2 Three Dimensional Elevation

The model main features are:

i) Core wall layout:

The office building falls in building classification "Class 5" of Building code of Australia [14] therefore; stairs well and lifts are provided to satisfy the minimum access and egress requirement.

Fig 3 shows wall layout in three models i.e. Corner wall (CW), main right wall (RW), main left wall (LW). However: right side wall (RSW) and left side wall (LSW) are only provided in 57-storey model.

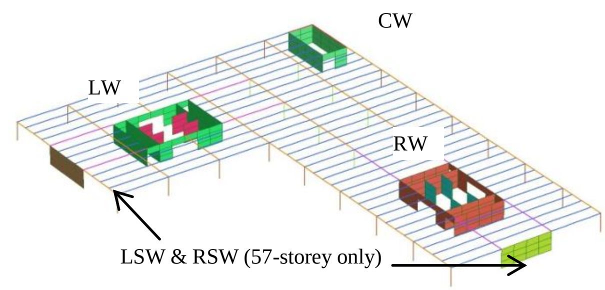

Fig. 3 Wall Designation

ii) Beams Layout:

Secondary beams are running along the shorter dimension and primary beams are along the longer dimensions.

iii) Column positions:

Columns are at 10 m centre to centre spacing. This spacing is chosen as desirable open space criteria for office buildings.

iv) Construction type:

Simple construction is adopted according to the definition of Australian Standard [9] and frame moment releases are provided for primary and secondary beams (Fig. 4). Braced core frame is provided for the lateral load resistance. The outriggers, however; are provided when structure reaches at height where deflections and frequencies are out of the prescribe Australian code limits [2,8].

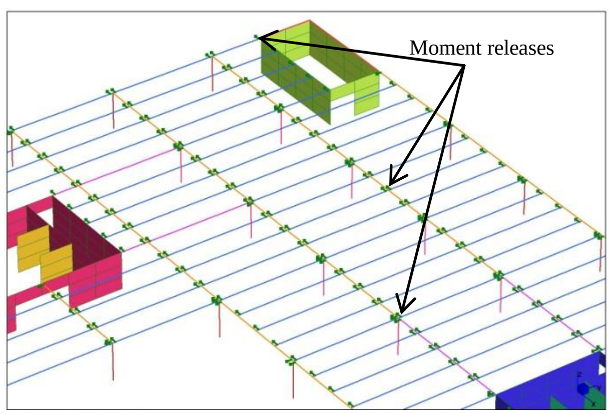

Fig. 4 Partial Plan for Moment Release in Beams

v) Support at base:

Column and core both are fixed at the base (Fig. 5). Columns attract very less lateral load due to their small rigidities comparing with the massive core wall structure [16]. The pinned and fixed base usually does not change the lateral load attracted to the column. However; in case of pinned base, column must be sufficiently strong to resist all the lateral moments by themselves without transferring them to the base. This kind of setup is usually not common in real world where most bases especially for tall structures are either designed as raft/mat or deep foundations.

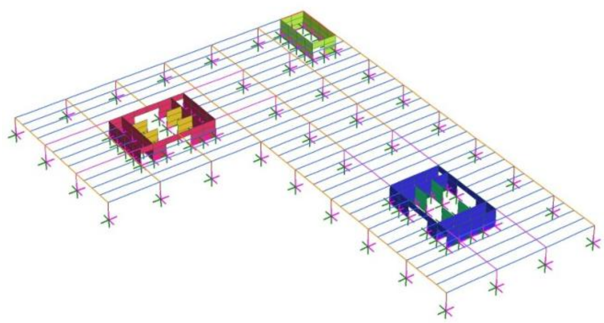

Fig. 5 Fixed supports at base

V. Element Properties

Horizontal elements (i.e. Slab and beams) sizes are fixed as these are only meant for carrying the local floor loads, however the sizes of vertical elements varies with height. Columns sizes are dominated by the gravitational loads whereas; core wall thickness is mainly dictated by the lateral loads.

A. Composite Columns sizes:

In this model the Columns are grouped for each 5-storey i.e. same size is provided for each 5 levels. This is based on load variation each five levels.

The cross-sectional size is calculated by applying the guidelines of Australian code for Concrete Structures [3].

Typical Floor loads for composite column = Slab self weight + (Secondary Beams + Primary Beams) self weight + SDL + LL + Column Self Weight

The composite effect is provided by using Structural Steel I-Sections (UC and WC) from AISC capacity tables [17], within the prescribe steel percentage limits of Australian standard [10] (see Table I, II, II). The transformed properties of composite columns are entered into the finite element software [3] for analysis.

TABLE I

COLUMN SIZES FOR PROGRESSIVE 28-STOREYS (98M)

Levels group	Interior Column				Exterior Column		
	f'_c (MPa)	Size (Sq.mm)	E_T (MPa)	γ_T (kg/m^3)	Size (Sq.mm)	E_T (MPa)	γ_T (kg/m^3)
L26-L28	65	325	41987	2600	250	42317	2610
L21-L25	65	525	41895	2596	375	41863	2595
L16-L20	80	600	44679	2618	425	44839	2624
L11-L15	80	700	43659	2584	525	45024	2630
L6-L10	100	725	47454	2627	525	47536	2630
L1-L5	100	800	47156	2617	575	47641	2633

f'_c = Concrete strength, E_T = Transformed elastic modulus,
γ_T = Transformed density

TABLE II

COLUMN SIZES FOR PROGRESSIVE 42-STOREYS (147M)

Levels grouped	Interior Column				Exterior Column		
	f'_c (MPa)	Size (Sq.mm)	E_T (MPa)	γ_T (kg/m^3)	Size (Sq.mm)	E_T (MPa)	γ_T (kg/m^3)
L41-L42	65	350	39909	2531	300	40815	2561
L36-L40	65	500	42356	2612	350	42524	2617
L31-L35	65	625	42145	2605	450	42137	2604
L26-L30	80	675	44881	2625	500	43385	2575
L21-L25	80	775	44273	2604	575	44122	2599
L16-L20	80	850	45172	2635	625	44692	2619
L11-L15	100	850	47682	2635	625	47209	2619
L6-L10	100	925	47180	2618	650	47802	2639
L1-L5	100	975	47064	2614	700	47031	2612

f'_c = Concrete strength, E_T = Transformed elastic modulus,
γ_T = Transformed density

TABLE III

COLUMN SIZES FOR PROGRESSIVE 57-STOREYS (199.5M)

Levels group	Interior Column				Exterior Column		
	f'_c (MPa)	Size (Sq.mm)	E_T (MPa)	γ_T (kg/m^3)	Size (Sq.mm)	E_T (MPa)	γ_T (kg/m^3)
L56-L57	65	300	40815	2561	300	40815	2561
L51-L55	65	475	42891	2630	350	41355	2579
L46-L50	65	625	42145	2605	450	42748	2625
L41-L45	65	750	41736	2591	550	42410	2614
L36-L40	80	775	44273	2604	550	45645	2651
L31-L35	80	850	44706	2619	625	44692	2619
L26-L30	80	925	45093	2632	675	44881	2625
L21-L25	80	1000	44877	2625	725	44940	2627
L16-L20	100	1000	47392	2625	725	47454	2627
L11-L15	100	1025	47367	2624	750	47839	2640
L6-L10	100	1075	47075	2614	775	47481	2628
L1-L5	100	1125	47013	2612	800	47871	2641

f'_c = Concrete strength, E_T = Transformed elastic modulus,
γ_T = Transformed density

B. RCC wall sizes:

The initial wall thickness are supplied as per BCA [14], fire and durability requirements. These are progressively changed during the course of structural analysis for serviceability limit state (see Table VI).

VI. Optimization Procedure

The three dimensional modeling is carried out in Strand7 [3], which is a finite element based software. The properties for column and walls as outlined in Table I, II and III are input in the model for three different models heights, in addition to the Slab and beams properties. To achieve a structural arrangement that satisfies frequency criteria and deflection limits of relevant standards is a repeated task and can also be termed as “trial and error” procedure, as Jayachandran [7] outlines that overall optimization of tall building frame is complex and time consuming.

The steps that have been done repeatedly for the optimizations are:

i. Input of minimum required wall thickness, column sizes and Slab and beam properties for first run of model.

ii. The first Run is “Natural Frequency Analysis” that gives the fundamental frequency of vibration of structure. This frequency is check against the recommended frequency by Australian standard [6]. The model is run and re-run and for each solver cycle the wall thickness are adjusted (usually increased) in order to get the appropriate lateral rigidity, until the required frequency value is achieved. The introduction of outrigger system is also beneficial during this procedure to attain needed lateral stiffness, however; this option is used only for 57-storey structure.

iii. The acquired frequency is then utilized in cyclonic wind load calculations in order to get Dynamic Along-wind and Crosswind effects on building. Although in this paper, frequency of basic model is used for cyclonic wind calculations.

iv. These wind loads are applied on to the structure in software [3]. Australian Standards advocate using Along-wind and Crosswind effects simultaneously on the structure, therefore: load combinations (Comb 1) is used in Strand7 [3] to get such effects.

v. Serviceability (i.e. deflections) limits are checked to measure structural capability of lateral load resistance for Along-wind (Fig. 6), Crosswind (Fig. 7) and combination of both as follow.

Load 1 - Along-Wind Actions
Load 2 – Crosswind Actions
Comb 1 – (Along-wind Actions + Crosswind Actions)

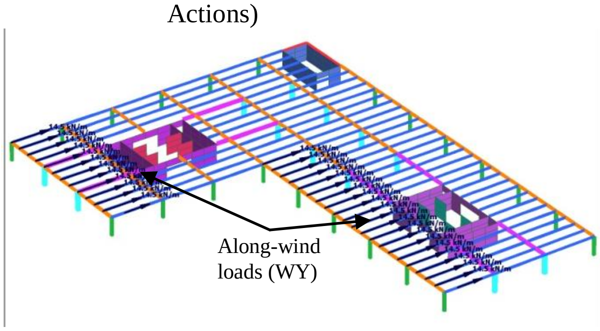

Fig. 6 Along-wind actions (WY)

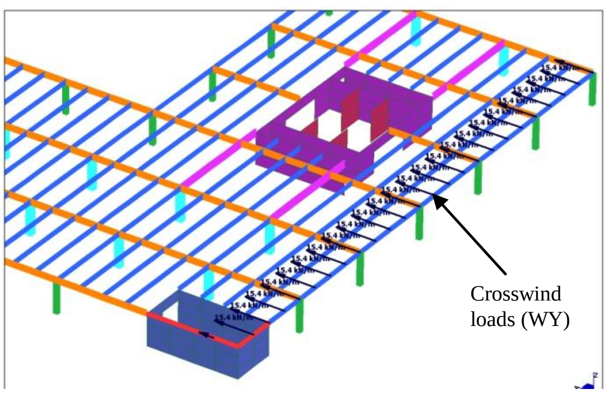

Fig. 7 Crosswind actions (WY)

Above steps are performed repeatedly by adjusting wall thicknesses and introduction of outriggers at various levels until desired results are achieved.

VII. Modeling Arrangements

Several models have been developed and run that cannot be presented here. However few of the representative model arrangement are list in Table VI.

The below models are inspired by the previous work of Fawzia et al. [1], where she has used two and three outriggers arrangements. Similar ratios are adopted for 42-storey and 57-storey model for providing two and three outrigger levels. As well as guidance is drive from the work of Wu et al [18], in which study has done on the optimum location of outriggers when structure is subjected to trapezoidal horizontal loads.

TABLE VI
Representative Models

Wall thickness	Description	Model Name
28-Storey		
RCW= LCW=CW : L1-L10 = 300 mm, L11-L20=300 mm, L31-L28 = 300 mm	Without outriggers.**	28M1
RCW= LCW=CW : L1-L10 = 350 mm, L11-L20= 300 mm, L31-L28 = 300 mm	Without outriggers.	28M2
42-Storey		
RCW= LCW=CW : L1-L10 = 500 mm, L11-L20= 450mm , L21-L30 = 400 mm, L31-L40 = 350 mm, L41-L42 = 300 mm	Without outriggers.**	42M1
	21^{st} level outriggers.	42M2
	21^{st} and 42^{nd} level outriggers.	42M3
RCW= LCW=CW : L1-L10 = 550 mm, L11-L20= 500mm , L21-L30 = 450 mm, L31-L40 = 400 mm, L41-L42 = 350 mm	21^{st} and 42^{nd} levels outriggers.	42M4
RCW= LCW=CW : L1-L10 = 600 mm, L11-L20= 550mm , L21-L30 = 500 mm, L31-L40 = 450 mm, L41-L42 = 400 mm	21^{st} and 42^{nd} level outriggers.	42M5
	Double outriggers at 20^{th} , 21^{st} and 41^{st} , 42^{nd} levels.*	42M6
	18^{th} , 30^{th} and 42^{nd} levels outriggers.*	42M7
57-storey		
RCW= LCW=CW : L1-L10 = 700 mm, L11-L20= 650 mm , L21-L30 = 600 mm, L31-L40= 550 mm, L41-L50 = 500 mm, L51-L57 = 450 mm,	Without outriggers.**	57M1
	57^{th} and 34^{th} level outriggers.	57M2
RCW= LCW=CW : L1-L10 = 800 mm, L11-L20= 750 mm , L21-L30 = 700 mm, L31-L40= 650 mm, L41-L50 = 600 mm, L51-L57 = 550 mm,	57^{th} and 34^{th} levels outriggers.	57M3
	Double outriggers at 57^{th} ,56^{th} and 34^{th} , 33th levels.	57M4
	57^{th} , 40^{th} and 24^{th} levels outriggers*	57M5
	Double Out riggers at 57^{th} ,56^{th} , 40^{th} , 39^{th} and 24^{th} , 23^{rd} levels.*	57M6
RCW= LCW=CW :L1-L10 = 800 mm, L11-L20= 750 mm , L21-L30 = 700 mm, L31-L40= 650 mm, L41-L50 = 600 mm, L51-L57 = 550 mm, LSW=RSW: L1-L10 = 550 mm, L11-L20 =500 mm, L21-L30 = 450 mm, L31- L57 = 350 mm.	Double Out riggers at 57^{th} ,56^{th} , 40^{th} , 39^{th} and 24^{th} , 23^{rd} levels.*	57M7

*Three levels of out riggers. **Basic model

VIII. Modeling Validation

The comparisons of various values are given in Table V, Table VI and Table VII for 28-storey, 42-storey and 57-storey models respectively. The manual calculations are performed through Excel spread sheets as well as hand calculating of some values and compared with the computer generated results of Strand7 [3].

TABLE V
MODELING VALIDATION FOR 28- STOREY

Items	Manual Cals	Strand7	Difference
Interior Column load (kN)	10800	10934	1.24 %
Exterior Column load (kN)	5180	5307	2.45 %
Structure Self weight (kN)	441974	439907	4.7 %
Base shear - Along wind response (kN)	27384	27384	0.0
Base Shear Cross wind (kN)	23609	23616	0.0

Column load = self weight of structure/ column tributary area.
Difference = {(Manual load – strand7 output)/ Manual Load} x 100

TABLE VI
MODELING VALIDATION FOR 42- STOREY

Items	Manual Cals	Strand7	Difference
Interior Column load (kN)	16988	16406	3.4 %
Exterior Column load (kN)	8140	8522	4.7 %
Structure Self weight (kN)	763488	787936	3.2 %
Base shear - Along wind response (kN)	44848	46254	3.13 %
Base Shear Cross wind (kN)	36720	36700	0.0

Column load = self weight of structure/ column tributary area.
Difference = {(Manual load – strand7 output)/ Manual Load} x 100

TABLE VI
MODELING VALIDATION FOR 57- STOREY

Items	Manual Cals	Strand7	Difference
Interior Column load (kN)	24078	22245	7.6 %
Exterior Column load (kN)	11547	11963	3.6 %
Structure Self weight (kN)	1307800	1258228	3.8 %
Base shear - Along wind response (kN)	66776	66776	0.0
Base Shear Cross wind (kN)	51854	51960	0.2 %

Column load = self weight of structure/ column tributary area.
Difference = {(Manual load – strand7 output)/ Manual Load} x 100

IX. RESULT

The results that are achieved are presented in following tables and graph.

A. 28-Storey model:

TABLE VIII
RESULTS FOR 28-STOREY

Model Name	Frequency		Deflection (mm)		
	1st Mode	2nd Mode	DY (Along-wind)	DY (Cross wind)	DXY (Comb1)
28M1	0.415	.485	170	120	170
28M2	0.475	0.511	150	110	160

This model has a lowest depth to height ratio therefore; stiff enough to lateral loads. It does not require any additional rigidity to achieve frequency and deflection limits.

B. 42-storey model:

The analysis results of various 42-storey models are given in Table IX and a comparison of Along-wind, crosswind and combination of both is given by Fig. 8.

TABLE IX
RESULTS FOR 42-STOREY

Model Name	Frequency (Hz)		Deflection (mm)		
	1st Mode	2nd Mode	DY (Along-wind)	DY (Cross wind)	DXY (Comb1)
42M1	0.265	0.286	520	290	630
42M2	0.281	0.302	480	260	590
42M3	0.291	0.312	450	230	550
42M4	0.298	0.320	410	210	500
42M5	0.304	0.326	380	200	460
42M6	0.323	0.352	340	170	420
42M7	0.314	0.339	360	180	440

The trend of deflections is downward till 42M6 and rises in 42M7 as seen in Fig. 8. 42M6 has double outrigger one at mid-height and other at the top of structure whereas 42M7 has three outrigger levels approximately one third and two third heights, in addition to one at the top. From the deflection curve it is evident that two double levels outriggers are more effective than three single levels outriggers.

The combination deflection is dominating whereas: deflections in along-wind and comb 1 are greater than Australian standard [2] limits of height/500. The crosswind though imparts fewer effects on this building and deflection is within the prescribe value.

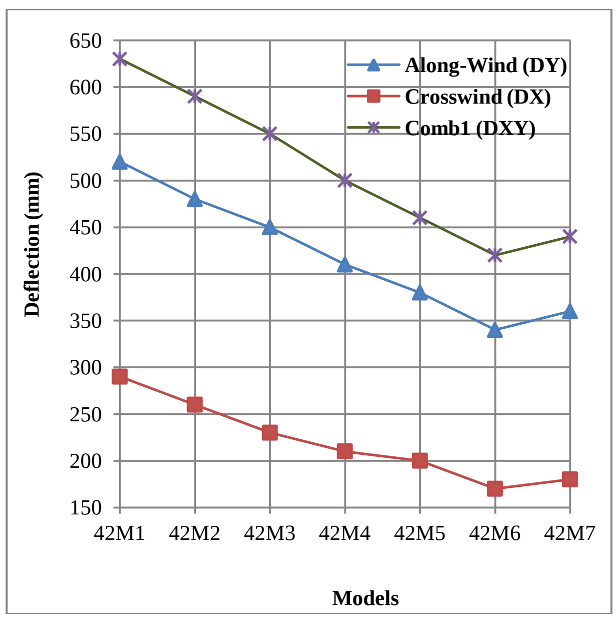

Fig. 8 Deflection comparison for 42-storey

The first and second mode frequency shows similar trend as deflection graph (see Fig. 9). Frequency values gives somewhat predicted results, there is a marked difference between the frequencies of two single outrigger system and two double outriggers due to increase rigidity, the three outriggers gives the value between the above two.

Frequency values (see Table IX) are however; within limits in the first model, therefore; wind effects are the critical in this instance hence serviceability limits need to achieve.

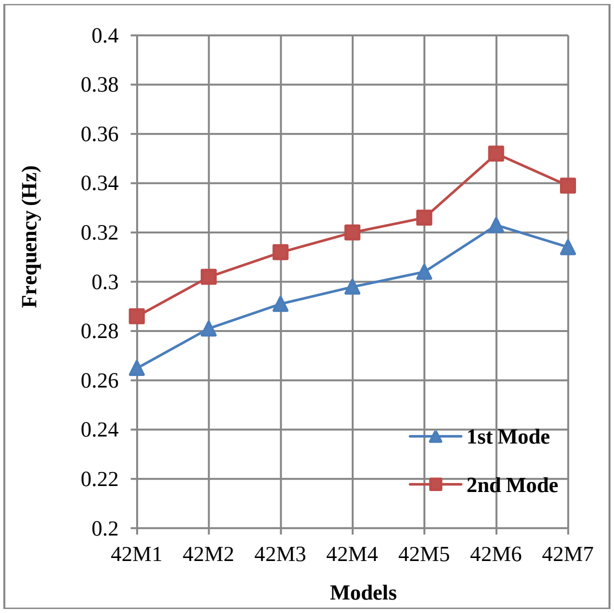

Fig. 9 Frequency comparison for 42-storey

C. 57-storey

This is the tallest prototype and is leaner/cylinder due to increased height to plan ration as compared to other two models. Therefore the lateral rigidly has reduced which will appear by comparing deflection values in Table X to 28-storey and 42-storey models in Table VIII and Table IX.

TABLE X
RESULTS FOR 57-STOREY

Model Name	Frequency (Hz)		Deflection (mm)		
	1st Mode	2nd Mode	DY (Along-wind)	DY (Cross wind)	DXY (Comb1)
57M1	0.166	0.180	1210	730	1450
57M2	0.184	0.197	1020	600	1260
57M3	0.188	0.201	910	550	1120
57M4	0.202	0.216	810	470	1010
57M5	0.196	0.208	860	510	1070
57M6	0.212	0.228	750	430	940
57M7	0.219	0.232	660	390	820

Reduced frequency and higher deflections corresponds to reduced rigidity. Fig. 10 shows; that in comb 1 (i.e. combine action of along wind and crosswind) have the maximum deflections. the graph in Fig. 10 follows a steady downward trend till 57M5, where a notable increase of deflection value occurred, which shows that providing double outriggers at two locations provide more stiffness as altogether they are four outrigger levels instead of providing three levels of truss at various levels (see Table IV).

The 57M7 models is supplied with three levels of double storey outriggers with side walls, still deflections values (see Table X) are far less than the Australian standard [2] confinement of height/500. This means than the structure requires further stiffness in terms of more shear walls and bracings for limiting values of lateral deflections.

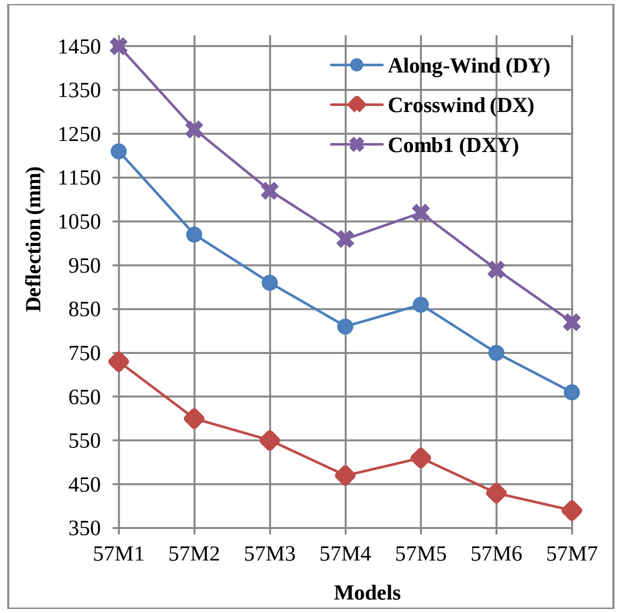

Fig. 10 Deflection comparison for 57-storey

The frequency trend is similar to the deflection as seen in Fig. 11.Minimum requirement of 0.2 Hz can be achieved in 57M4, which has two levels of double outriggers of and again dropped down in 57M5 with three levels of outriggers. Using six outrigger levels i.e. 57M6 however show a very sharp increase in frequency.

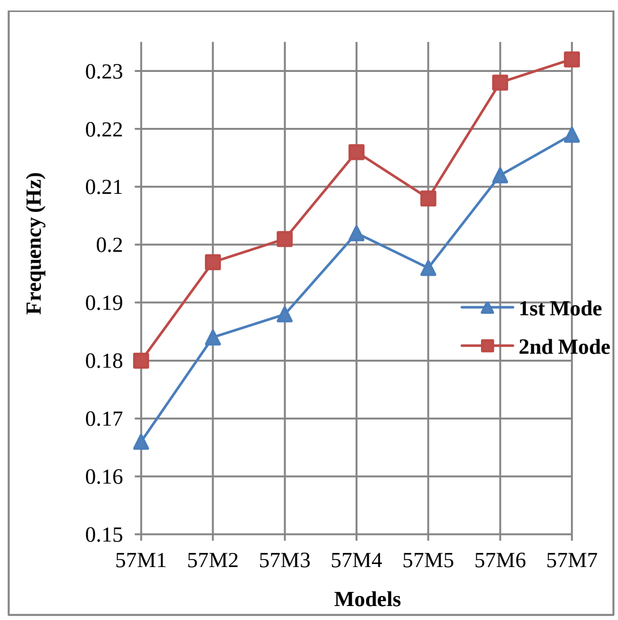

Fig. 11 Frequency comparison for 57-storey

X. CONCLUSION

The above investigation comes to the conclusion that rigidity/stiffness of composite high-rise building is inversely proportional to its height i.e. the lateral stiffness decreases with increase in height of structure while keeping the other variable constant. Therefore introduction of additional bracing system is required to keep up with the serviceability limits.

28-storey model has b: h and d:h equal to 1: 1.633 and 1: 1.225 respectively. There is not a marked difference of vertical and plan dimensions and in this case frequency and deflections limits could be readily attainable (see Table IX).

42-storey model has b:h and d:h equal to 1: 2.45 and 1: 1.84 respectively. Here the vertical height has exceeded more than double in one plan dimension. Frequency limits could be accomplished without belt truss and outrigger system but to attain deflection limits truss system is required. This system provides a reverse curvature and consequently reduces the deflection at the top of structure.

57-storey model has b:h and d:h equal to 1: 3.325 and 1: 2.5 respectively. This model has vertical dimensions almost three times of its plan dimension and as a result; it requires truss system as well as additional stiffness in terms of shear walls to accomplish the criteria of frequency and deflection.

Introduction of outriggers and belt truss proved to be more efficient in deflection minimization then achieving the required value of fundamental frequency of vibration. Since composite buildings usually have structural steel bracings truss and these do not have appreciable locally stiffness rather can be very useful in providing a tie down effects between shear walls and columns.

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