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THE OPTIMUM LOCATION OF OUTRIGGER IN REDUCING THE ALONG-WIND AND ACROSS-WIND RESPONSES OF TALL BUILDINGS

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Abstract: Outrigger is one of the many tall building structural systems that are used to reduce the building responses to wind. However, it is not known where the outrigger should be placed so that the responses of the tall building due to wind can be minimized. Thus, 64-story reinforced concrete buildings with the ratio of height to the breadth of 6:1 are studied in order to determine the optimum location to construct the outriggers to minimize the along-wind and across-wind responses. Buildings with different location of outriggers are analysed by a structural analysis software in order to determine the natural frequencies and eigenvectors in the along-wind and across-wind direction. The along-wind responses are determined by employing the procedures from the ASCE 7-02 while the across-wind responses of the buildings are calculated based on the procedures and wind tunnel data available in a data base of aerodynamic load. The database is comprised of high-frequency base balance measurements on a host of isolated tall buildings models. Results from the analysis shows that the optimum location to construct the outriggers is between one third to two third of the height of the building.

Keywords: *outrigger, wind, tall building, building response*

1.0 Introduction

Excessive deflection and acceleration of a building can cause inconveniences to the occupants of the building. Drift index or the ratio of the maximum deflection at the top of the building to the total height, that equals to 1/300 may cause cracking of the reinforced walls and visual annoyance while drift index of 1/200 may cause improper drainage and damage to the windows and finishes (Balendra, 1993).

Design drift index limits that have been used in different countries range from 0.001 to 0.005. Generally, lower values should be used for hotels or apartment buildings than for office buildings, since noise and movement tend to be more disturbing in the former. Malaysian code (MS 1553:2002) limits the total drift of wind force resisting system to 1/500 of the height, and the inter-story drift to 1/750 of the height. The ASCE7-

02 states that the drift limits in common usage for building design are on the order of 1/600 to 1/400 of the building or story height (ASCE Task Committee on Drift Control, 1988). Furthermore, acceleration is the predominant parameter that affects human perception to motion and vibration (Irwin, 1986). Acceleration of 0.5 m/s^2 may cause people having difficulty to walk naturally and to lose balance when standing (Yamada and Goto, 1975). ASCE7-02 requires excessive structural motion to be mitigated by measures that limit building or floor accelerations to levels that are not disturbing to the occupants or do not damage service equipment.

Outrigger system is one of the many structural systems used to reduce the drift of tall building. Outrigger-braced high-rise structure consists of reinforced concrete or braced-steel frame main core connected to the exterior columns by flexurally stiff horizontal cantilever beams. When horizontal loading acts on the building, the column-restrained outriggers resist the rotation of the core, causing the lateral deflections and moments in the core to be smaller than if the free-standing core alone resisted the loading. The result is to increase the effective depth of the structure when it flexes as a vertical cantilever, by inducing tension in the windward columns and compression in the leeward columns (Taranath, 1997). In order to stiffen the outriggers adequately in flexure and shear, they are designed as either one or two stories deep.

Question arises on what level the outriggers should be placed so that the responses in the along-wind and across-wind directions can be reduced most effectively. Is it better to place the outriggers at top of the building or is it better to place the outriggers near to the bottom of the building. Thus, the objective of this research is to find the best location to construct the outriggers to reduce the along-wind and the across-wind responses of tall buildings most effectively.

2. 0 Properties of Buildings

The building studied is a flexible office reinforced concrete building that has a square plan of 48 m x 48 m with height of 288 m. The ratio of height to the horizontal dimension of the building is 6:1. The building has 64 stories and each storey height is 4.5 metre. The plan of the building studied is shown in Figure 1.

The lateral system of the building studied is of reinforced concrete consisting of a central core, columns, diagonal beams and perimeter beams using concrete strengths of 80 MPa and modulus of elasticity 4.83 x 10^7 kPa. Typical floor system consists of wide flange beams with section of UB 457 x 191 @ 98 kg/m, UB 610 x 229 @ 125 kg/m and UB 610 X 305 @ 179 kg/m which span from the core wall to the perimeter beams. A composite metal deck system with concrete topping completes the floor system. The total thickness of the floor is 110 mm.

Three sizes of core walls are studied: 12 m x 12 m, 18 m x 18 m and 24 m x 24 m. Buildings with two different core wall thickness that are 350 mm and 800 mm were analysed. The thickness of the core wall is uniform from the base to the top of the building. The sizes of the internal columns of the building are 1300 mm x 1300 mm, 1100 mm x 1100 mm and 700 mm x 700 mm for floor 1 to 25, 26 to 50 and 51 to 64 respectively. However, the four internal columns exist only for building with core wall size of 12m x 12 m and 18 m x 18 m. The 24 m x 24 m core wall has no internal columns. The size of both perimeter and diagonal beams is 300 mm x 1000 mm.

Buildings with the outrigger systems have four outriggers with the width of 400 mm and depth of one story height. Each outrigger spans from the corner of the core wall to the perimeter column which is located at the corner of the building. In other words, the four outriggers are lying exactly on the four diagonal reinforced concrete beams. These outriggers are placed at five different positions which are at 25%, 33%, 50%, 75% and 100% of the height of the building; i.e. outriggers that are located at the top of the building has position 100% of the height of the building.

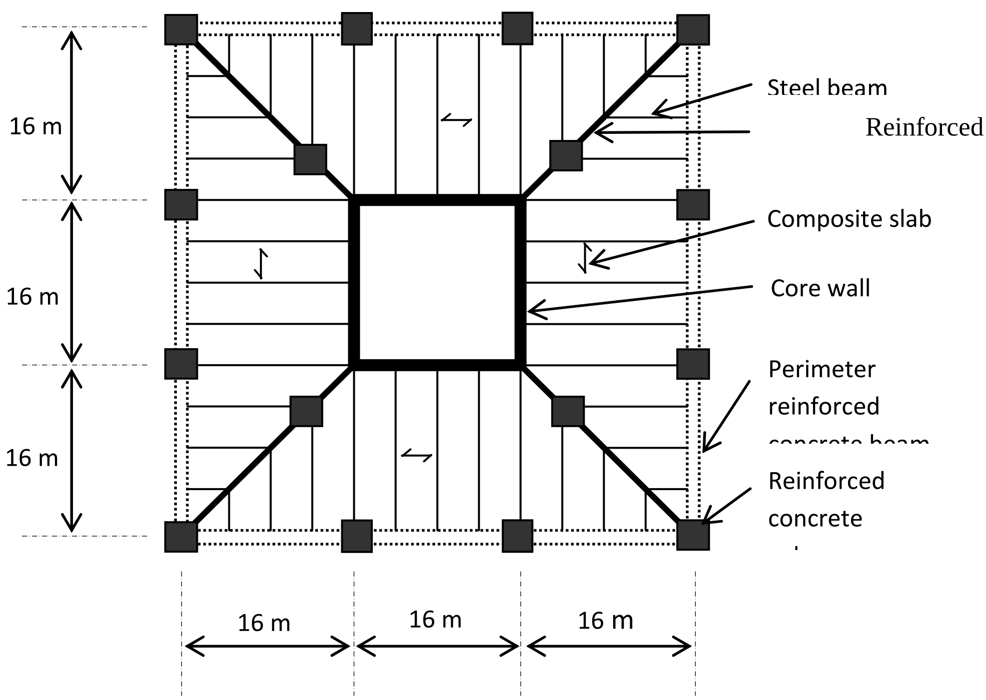


Figure 1: Typical plan view of the building studied.

3.0 Methodology

The procedure employed in this project consists of several steps as shown in Figure 2.

3.1 Modeling of the Building

The structural analysis software used to analyze the buildings is GTSTRUDL. All columns and beams are modeled as space frame members while the core walls, outriggers and belt walls are modeled as solid tridimensional finite element with 20 nodes which are called as isoprametric quadratic solid (IPQS). Each node of the IPQS element will have three translational degree of freedom, u_1,u_2, u_3. Figure 3 shows the model of outrigger building systems used for the eigenproblem analysis in GTSTRUDL. The outputs required from the eigenproblem analysis are the frequency in the along-wind and across-wind directions, as well as the eigenvectors in the across-wind direction.

3.2 Wind Speed

This project is a serviceability design, which requires 10-year return period to be used (ASCE 7-02). The wind speeds is converted from 50 year recurrence interval to 10 year recurrence interval, and also from the 3 second gust wind speed to 1-hour mean wind speed by using the procedures in the ASCE7-02. Power law is applied to convert the wind speed at 10 m height to the wind speed at the building height.

The final wind speeds which are used to obtain the across-wind response are one-hour averaging time wind speeds for 10-year return period at building height in urban area. The wind speeds used are 25.27 m/s, 32.56 m/s and 38.14 m/s for Malaysia, New York and Hong Kong wind environment, respectively. On the other hand, the computation of the along-wind responses uses the 3-second gust wind speeds for 10-year return period at 10 m height in open terrain which are 28.14 m/s, 36.26 m/s and 42.48 m/s for Malaysia, New York and Hong Kong wind environment, respectively.

3.3 Along-Wind Response

Once the natural frequency is obtained and the wind speeds are determined, the along-wind response is calculated immediately by using FORTRAN program which is written based on closed form formula provided in the ASCE7-02. The formula given in the ASCE7-02 for the along-wind responses are:

1. The maximum along-wind displacement $X_{max}(z)$ as a function of height above the ground surface is given by

$$X_{\max}(z) = \frac{\phi(z)\rho bhC_f\hat{V}_{\bar{z}}^2}{2m_1(2\pi n_1)^2}KG \tag{1}$$

where

$\phi(z)$ = fundamental mode shape = $(z/h)^{\xi}$;
ξ = mode exponent;
ρ = the air density;
C_f = mean along-wind force coefficient;
m_1 = modal mass = $\int \mu(z)\phi^2(z)dz$;
$\mu(z)$ = mass per unit height;
K $= (1.65)\hat{\alpha}/(\hat{\alpha}+\xi+1)$;
$\hat{V}_{\bar{z}}$ = the 3 sec gust speed at height $\bar{z}$ $= \hat{b}(z/33)^{\hat{\alpha}}\hat{V}_{ref}$
$\hat{V}_{ref}$ = 3 s gust in exposure C at reference height (obtained from the Figure 6-1, ASCE 7-02);
$\hat{b}$ and $\hat{\alpha}$ = value given in Table 6-2 in the ASCE 7-02
n_1 = building natural frequency in Hz
G_f = gust factor

2. The maximum along-wind acceleration as a function of height above the ground surface is given by

$$\ddot{X}_{\max}(z) = g_{\ddot{x}}\sigma_{\ddot{x}}(z) \tag{2}$$

where

$\sigma_{\ddot{x}}(z)$ = rms along-wind acceleration as a function of height above the ground surface

$$= \frac{0.85\phi(z)\rho bhC_{fx}\overline{V}_z^{\,2}}{m_1} I_{\bar{z}} KR$$

$$g_{\ddot{x}} = \sqrt{2\ln(n_1 T)} + \frac{0.5772}{\sqrt{2\ln(n_1 T)}}$$

T = 3600 seconds

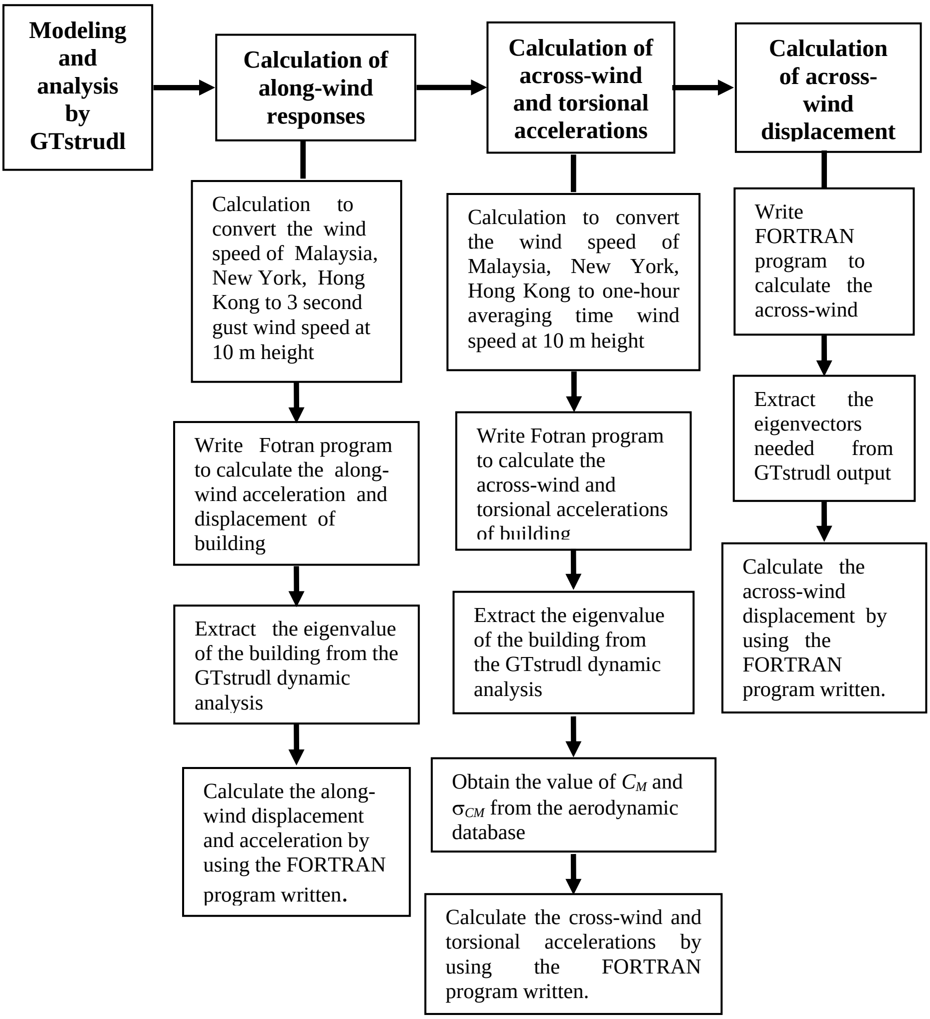


Figure 2. The procedure for the analysis of buildings in this research

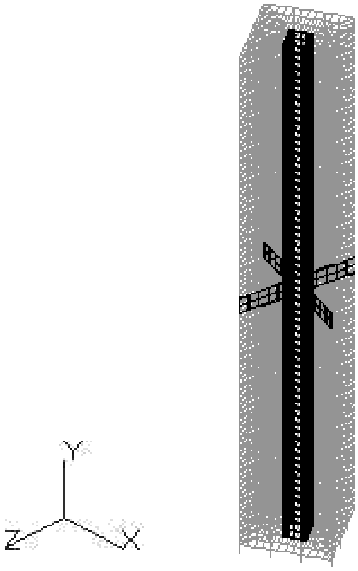


Figure 3.Outrigger and core wall system;

3.4 Across-Wind Response

The across-wind response computed in this project is based on the aerodynamic data base of the University of Notre Dame. Results from wind tunnel test of different models were used to calculate parameters such as the non-dimensional moment coefficient,C_M, The data is stored in the aerodynamic data base at the University of Notre Dame that can be accessed by any user with Microsoft Explorer at the URL address http://www.nd.edu/~nathaz/.The cross section of the models as shown in Table 1 were tested on an ultra-sensitive force balance. Each cross section was made of rigid balsa wood and was constructed with three different height: 406, 508 and 610 mm (16, 20 and 24 inches). Photograph of some of the balsa wood models is shown in Figure 4.

Table Model cross sections .

Model	1	2	3	4	5	6	7	8	9
Shape									
D:B	**2:6**	**3:6**	**4:6**	**4:4**	**6:4**	**6:3**	**6:2**	**4:4 (60°)**	**4:6**

B indicates the width of the model normal to the oncoming flow; while *D* the depth.

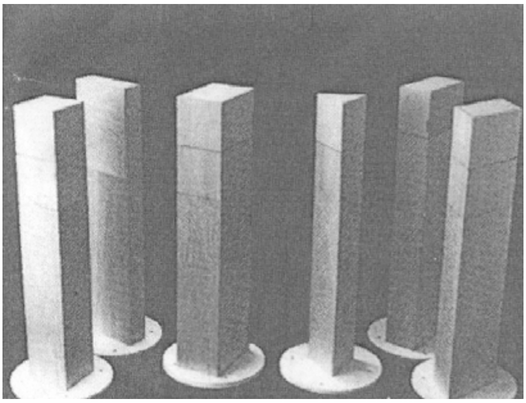


Figure 4 Balsa wood models tested (after Zhou,Kijewski,Kareem,2003)

Each of the balsa models was tested in a boundary layer wind tunnel with a 3 m (10 ft) x 1.5 m (5 ft) cross-section and 18 m (60 ft) length (Kareem, 1990). The turbulent boundary layers simulated in this study were generated by the natural action of the surface roughness added on the tunnel floor and the upstream spires. According to Zhou, Kijewski, and Kareem (2003), two typical boundary layers: boundary layer similar to the conditions of open and boundary layer similar to urban flow environments were simulated in this study. A high-frequency force balance was then used in conjunction with each balsa wood model to determine the dynamic wind-induced structural loads. The output of the sensitive, multicomponent force balance was recorded over a 5-minute interval at sampling rate of 300 Hz. The resulting time histories were segmented into blocks, analyzed by a 4096-point fast Fourier transform, to yield over 20 raw spectra, which were subsequently ensemble averaged. The theoretical development and background of the wind-induced response (displacement and acceleration in the along-wind, across-wind and torsional directions) procedure of the aerodynamic data base are given in the website.

The flowchart given in Figure 2 shows that before commencing the calculation of the across-wind or torsional responses, the values of RMS base coefficient, C_M (f) and the non-dimensional power spectral density (σ_M) have to be obtained from the aerodynamic database at the the same url address. The calculation of the across-wind acceleration is expedited by using a FOTRAN program. Only the maximum across-wind acceleration that is across-wind acceleration at the top of the building is computed. The computation of the across-wind displacement is performed by using another FOTRAN program. Eigenvectors and lumped mass at selected points are required for this computation.

4.0 Results and Discussion

The results from the analysis shows that the along-wind responses and the across- wind responses (displacement and acceleration) reach at their minimum values when the outrigger is positioned at about half of the height of the building (Figure 5, 6, 7 and 8). However, there is not much difference in the values of these responses if the outrigger is placed at any level located between one-quarter to two-third of the height of the building. This behaviour is observed for all buildings that are studied. Furthermore, placing the outrigger at the top of the building also reduces the value of the along-wind and across-wind responses, but, the reduction of the responses are the least compared to placing the outrigger at other positions.

Comparison of the graphs of the along-wind and across-wind responses against the different position of the outriggers in Figure 5, 6, 7 and 8 with the graphs of the natural frequencies of the buildings against the different position of the outriggers in Figure 9 shows that the responses are inversely proportional to the natural frequency of the buildings. This is as expected, as the value of the natural frequency is required in the calculation of the responses, and is further illustrated in equation (1) where the along-wind displacement is inversely proportional to the natural frequency. According to Buchholdt (1997), the natural frequency of a general *N* degree of freedom system can be obtained by using the Rayleigh quotient, which is

$$\omega^2 = \frac{\phi^T K \phi}{\phi^T M \phi} = \frac{K^*}{M^*} \tag{3}$$

in which ω^2 is the eigenvalue, K is the stiffness matrix, M is the mass matrix, ϕ is the eigenvector matrix, K^* is the modal stiffness and M^* is the modal mass. The output from GTSTRUDL analysis of the building provides the values of eigenvectors, ϕ of each joint of the structure and the eigenvalue, ω^2 of the building. By calculating the mass of each floor, the lumped mass matrix of the building can be constructed. Then, the modal mass, which is $M^* = \phi^T M \phi$ is computed.

Both mass and eigenvector matrices are constructed by using 9 points that are located at different height from the ground and are equally spaced. The selected points are as close as possible with the centroidal axis of the building to avoid any local deformation. Knowing the values of the modal mass and the eigenvalues allows the calculation of the modal stiffness.

The value of the modal mass, M^* increases slightly, as the higher the position of the outrigger. However, the modal mass, M^*, is considered as constant as the differences of the values of the modal mass, M^* for different position of the outrigger is less than 0.5 percent (Table 2). It is observed that the modal stiffness, K^* increases as the outrigger is placed higher from ground and achieves its maximum value when the outrigger is placed at about mid height of the structure. Then, the modal stiffness, K^* decreases as the outrigger is placed higher from the mid height of the building and achieves its minimum value when the outrigger is placed at the top of the building (Table 2). As the modal mass remains almost constant no matter where the outrigger is placed, the behaviour of the natural frequency in the along-wind and across-wind direction is caused primarily from the variation of the modal stiffness of the building when the position of the outrigger is changed.

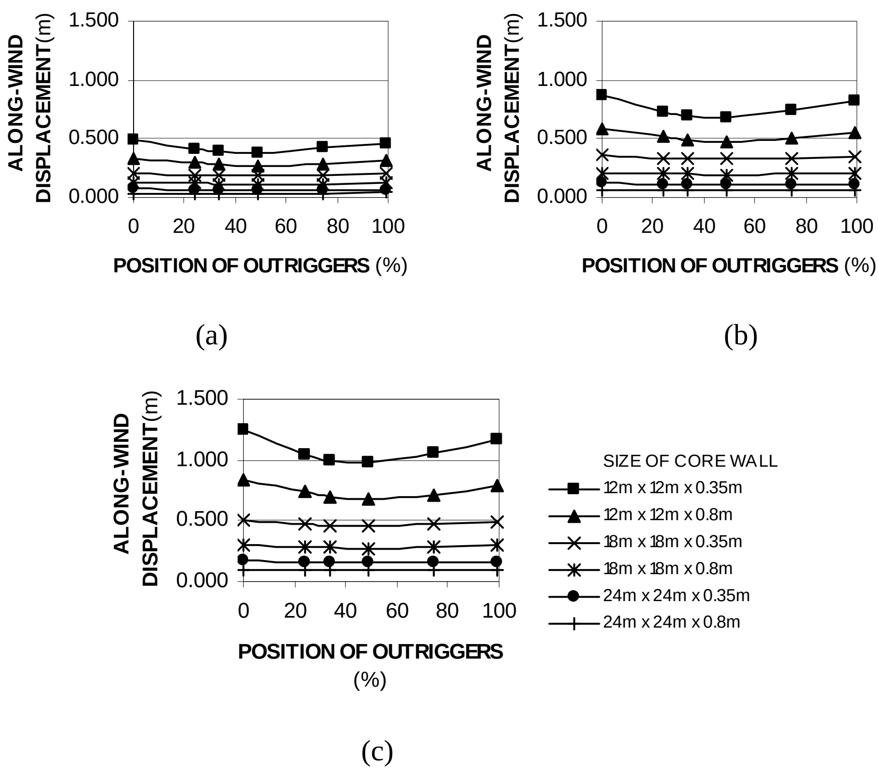


Figure 5. Variation of along-wind displacement in (a) Malaysia (b) New York (c) Hong Kong when the position of the outriggers is altered.

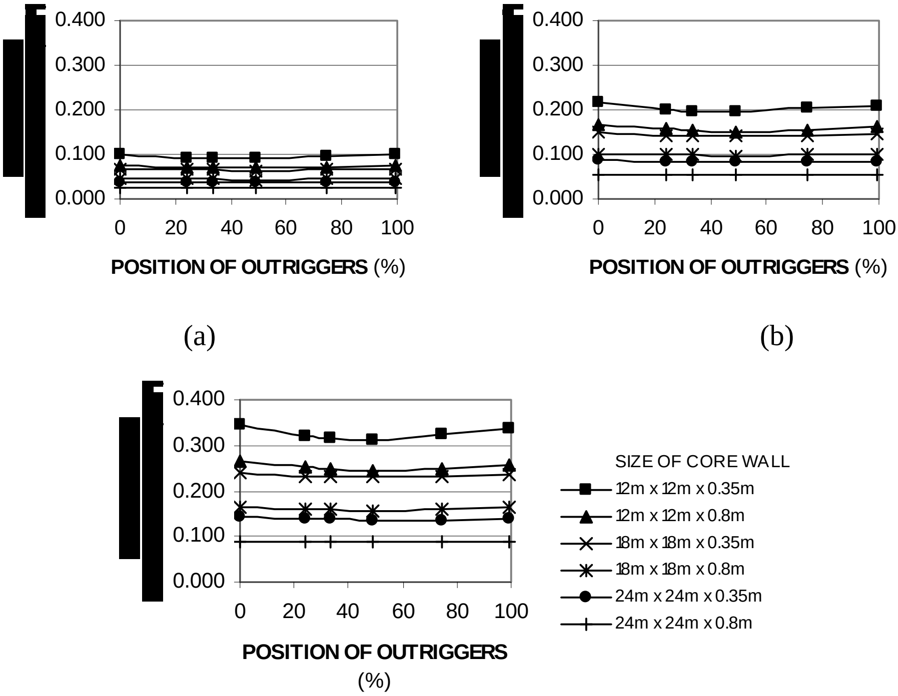
In order to confirm the findings from this research, three cantilever beams with different varying cross sections were studied. Each of the cantilever beam has the modulus of elasticity of 30 GPa , length of 12 metres and breadth of 800 mm. In order to depict the change of the cross-section of the columns of the tall building with height, the depth of the beams is varied as shown in Figure 10.

Furthermore, the depth of a section of the beam is altered to 2500 mm to represent the outrigger. The length of the section is 500mm and is placed at different point for each run. The locations of the points are 3m, 4m, 6m, 8m and 11.75m that correspond to positions of 25%, 33.3%, 50%, 66.67%, 75% and 97.9% of the height of the structure from ground, respectively. Then, the GTSTRUDL software is used to calculate the displacement at the tip of the beam when the beam is exerted by a concentrated force of 15000 kN at the tip of the beam. The stiffness of the beam is obtained by dividing the applied force with the displacement.

The result from the analysis shows that the maximum value of stiffness is obtained when the 500 mm section is placed at 50%, 33% and 25% of the height of the structure from ground level for trial 1, 2 and 3, respectively. This indicates that the exact location of the altered section for the maximum value of stiffness to be achieved depends on how the cross section of the beams is changed with length as trial 1, 2 and 3 have different variation of cross section with length. In other words, the exact location of the outrigger to achieve maximum stiffness or minimum responses depends on how the size of the column is changed with height.

Furthermore, the values of the stiffness of the beams are very close to the maximum value of the stiffness when the altered section is placed between one-quarter to two-third of the beams. Figure 5,6,7 and 8 also shows that the values of the responses of the buildings are close to the minimum value of the responses if the outrigger is placed between one-quarter to two-third of the buildings. Thus, as the exact location of the outrigger that will cause minimum responses of any building to occur depends on how the size of its columns is changed with height and as the values of the responses are close to the minimum value of the responses if the outrigger is placed between one-quarter to two-third of the building, it is concluded that the optimum location to construct the outrigger is between one-quarter to two-third of the height of any building.

The result from the beam analysis also proves that the stiffness values of the beam decrease as the 500 mm section is placed further away from the mid span of the beam and reach its minimum value when the 500 mm section is located at the tip of the beam for the all trials. In other words, when the outrigger is placed at top of the building, the building will achieve minimum lateral stiffness which causes maximum responses of the building when it is exerted by wind.



(a) (b)

(c)

Figure 6. Variation of along-wind acceleration in (a) Malaysia (b) New York (c) Hong Kong when the position of the outriggers is altered.

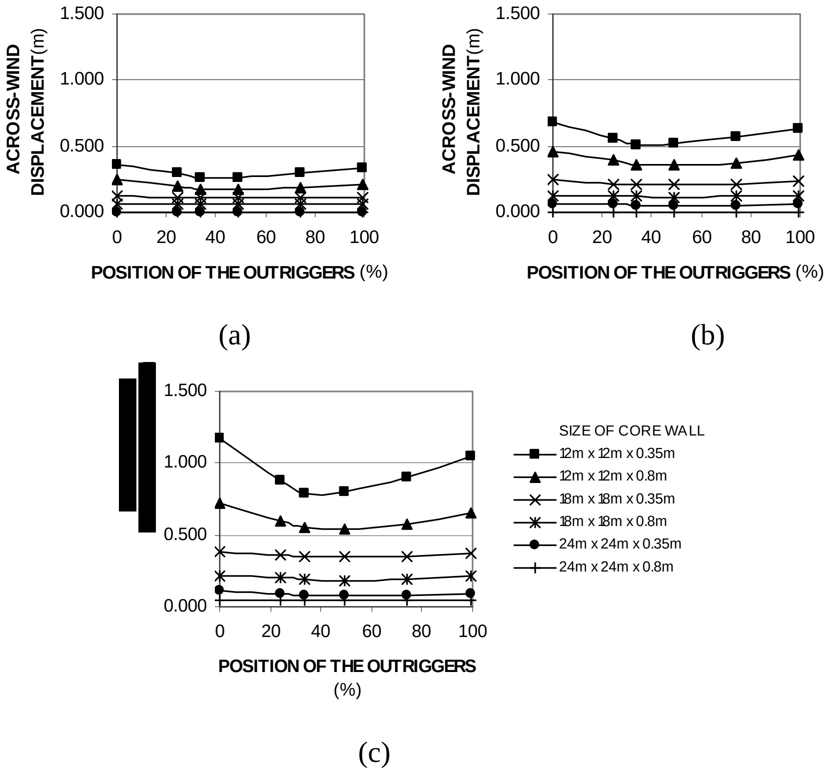
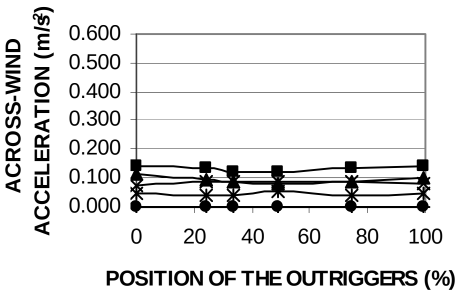
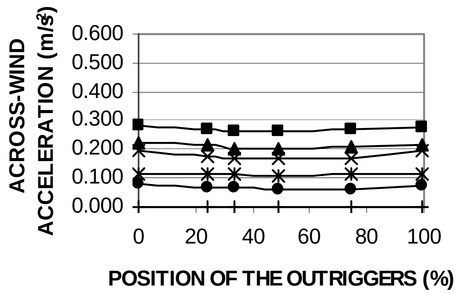


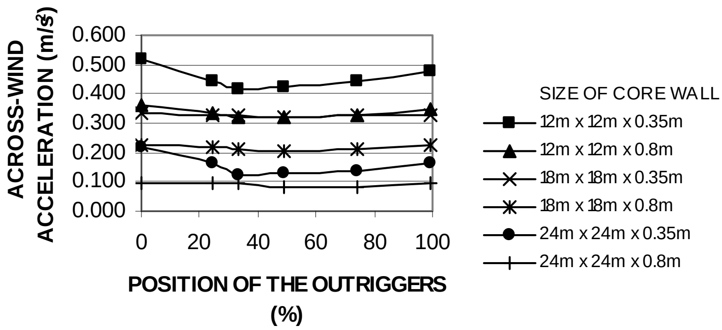
Figure 7. Variation of across-wind displacement in (a) Malaysia (b) New York (c) Hong Kong when the position of the outriggers is altered.



(a)

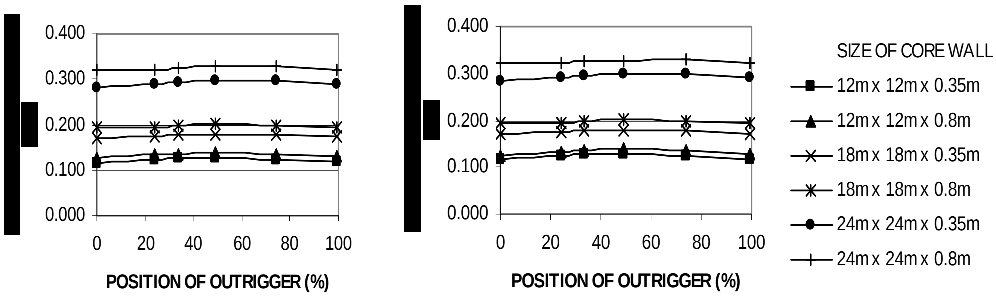


(b)



(c)

Figure 8. Variation of across-wind acceleration in (a) Malaysia (b) New York (c) Hong Kong when the position of the outriggers is altered.

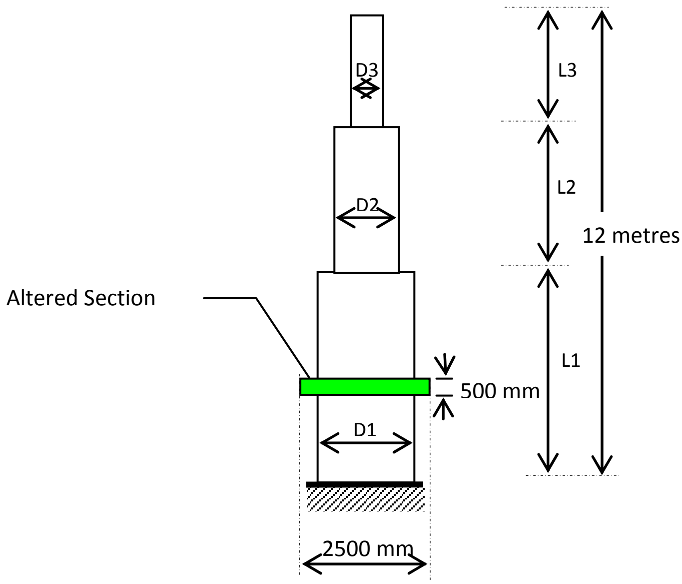


(a) **(b)**

Figure 9. Variation of natural frequency in (a) along-wind direction (b) across-wind direction with different position of outriggers.

Table 2 The Variation of Modal Mass, M^* and modal stiffeness, K^* with Different Position of Outrigger of A Building Having an 18 x 18 Size and 350 Thickness of Corewall.

Position of Outrigger (%)	Modal Mass, M^* (kg)	Modal Stiffness, K^* (N/m)
No Outrigger	107390751	122525
24.2	107396551	131334
33.6	107413098	134356
49.2	107444774	134396
74.2	107566499	133040
99.2	107656939	125736



Properties	L1 (m)	D1 (mm)	L2 (m)	D2 (mm)	L3 (m)	D3 (mm)
TRIAL 1	4.69	2100	4.68	1800	2.63	1500
TRIAL 2	4	2100	4	1800	4	1500
TRIAL 3	12	2100	-	-	-	-

Figure 10: The properties of the beams for trial 1, 2, and 3.

Furthermore, the graphs in Figure 5, 6, 7 and 8 become more horizontal as the size and the thickness of the core wall are increased. The values of the responses that correspond to position 0 percent on the x-axis of the graphs represent the values of the responses when the building has no outrigger at all. When the graphs become almost flat, it indicates that the responses of the buildings having the outrigger at any location are about the same as the responses of the buildings having no outrigger at all. In other words, the reduction of the responses due to the addition of the outrigger to a building is small, and this occurs when the building has large size and thickness of the core wall.

In order to find out why the reduction of the responses decreases as the core wall becomes larger or thicker, two cantilever beams with different variation of the cross sections are studied. As the core wall becomes larger and thicker, the moment of inertia of the core wall increases. Thus, the cross sections of the beam in Trial 2 is made larger than the beam in Trial 1 in order to depict the increment of the moment of inertia of the core wall when the core wall size and thickness are increased. Both of the beams have the breadth of 800 mm, length of 12 metre and modulus of elasticity of 30 GPa. The depth and moment of inertia of the beams are as shown in Table 3. Furthermore, the depth of a section of the two beams with length of 500 mm is changed to 2500 mm. The centroid of the altered section is located exactly at mid span of the cantilever beam. The purpose of having the altered section is to depict the outriggers that are located at the mid height of a building. Each beam is exerted by a concentrated load, P=1500 kN, at its tip.

Figure 11 (a) and (b) are plots of M/EI diagrams for the cantilever beams before the 500 mm sections are placed at the mid span of the beams for Trial 1 and 2, respectively. These plots depict the M/EI diagram for the buildings without any outriggers. Figure 11 (c) and (d) are plots of M/EI diagrams for the cantilever beams with the altered section. These plots depict the M/EI diagrams for the buildings with outriggers at the mid height of the buildings.

According to Beer and Johnston (1981), the displacement of a cantilever beam equals to the area under the M/EI diagram multiplied by the distance between the centroid of the area and the point where displacement is required. Line abcd in Figure 11(c) and (d) form a cut or groove in the M/EI diagram. The altered section of the cantilever beam has larger moment of inertia, I, and thus causes a sudden drop of the value of M/EI diagram as depicted by line ab. Line cd represents the sudden jump due to the change of I, from the large I value of the altered section to the smaller original moment of inertia, I of the beam. Thus, the location of line abcd is the location of the altered section. Lines abcd in Figure 11(c) and (d) represent how much the reduction of the displacement when the altered section is at position 50%. It is observed that the length of ab and cd is smaller for Trial 2 that have larger moment of inertia than the ones in Trial 1. This shows that the reduction of the displacement becomes smaller or, the increment of the stiffness becomes smaller as the moments of inertia of all the cross sections of the cantilever beam are increased. As a result, the increment of the natural frequency in both the along-wind and across-wind becomes smaller, while the reduction of the responses in the along-wind and across-wind directions decreases as the size and the thickness of the core wall are increased

Table 3. The depth and moment of inertia of the two cantilever beams studied.

	Trial 1		Trial 2	
Distance from Fixed Support (m)	Depth (mm)	Moment of Inertia (m^4)	Depth (mm)	Moment of Inertia (m^4)
0 - 4.69	2100	0.617	2400	0.922
4.69 - 9.37	1800	0.389	2100	0.617
9.37 - 12	1500	0.225	1800	0.389

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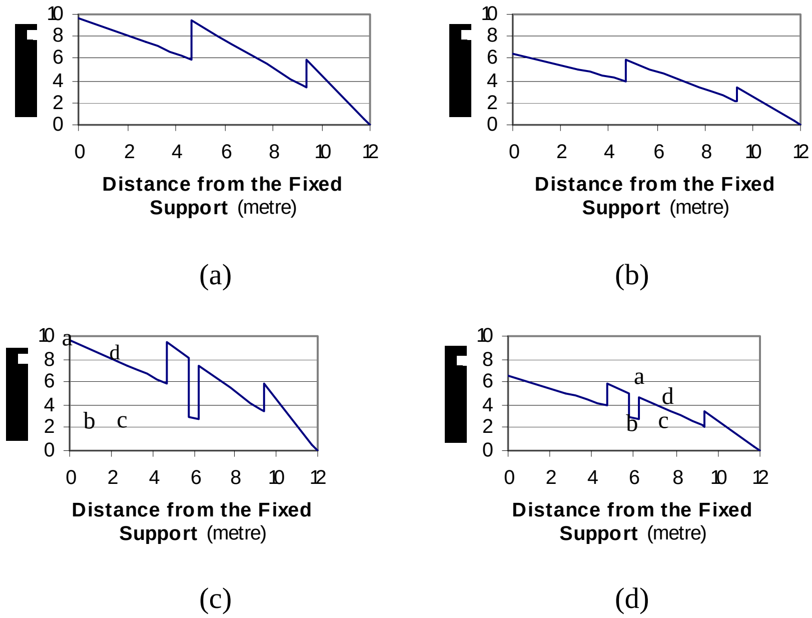

Figure 11. M/EI diagram without the altered section for (a) Trial 1; (b) Trial 2. M/EI diagram when thealtered section is at position 50% for the cantilever beam (c) in Trial 1 (d) in Trial 2.

5.0 Conclusion

The best location to construct the outrigger is between one-quarter to two-third of the height of the building in order to minimize both the along wind and the across wind responses. Constructing the outriggers on the top of the building do reduce the responses in both the along wind and across wind direction. However, the reduction of the value of the responses is the least when the outriggers are placed at this position. Furthermore, the benefit of using the outriggers to reduce the along-wind and across-wind responses is significant only for buildings with small dimension of core wall and diminishes as the size of the core wall becomes larger.

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