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A Semi Heuristic Investigation: Bracing Effect of Diagrid

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A B S T R A C T

Given the increasing need for wind-resistant structures in the current period of fast urbanization and population increase, a thorough knowledge of how different cross-section geometries affect wind load distribution and structural behavior overall is crucial. We investigate the impact of both a rectangular and twisted section design in this study. Using ETABS modules, this study focuses on the aerodynamic characteristics of both configurations, such as pressure distribution, drag-lift coefficients, and vortex shedding frequencies. The research demonstrated the impacts of wind drag in relation to a structure's cross-section layout by intensively simulating a wide variety of realistic wind conditions in order to replicate real-world scenarios. The purpose of the study is to give structural engineers some guidance on how to choose the best cross section shape to minimize wind-related damage and displacement, improve building safety, and take aerodynamic variables into account. In conclusion, this study provides important insights into the dynamics of wind-structure interaction and is a vital resource for developing stronger structures and buildings that can withstand increasing wind loads. By providing a comprehensive evaluation of the impact of different cross sections on wind load responses, it creates opportunities for innovative design solutions that strike a compromise between structural economy and aerodynamic performance.

Keywords: wind load, structural cross-section, aerodynamic parameters, structural design, vortex shedding

1. Introduction

In order to comprehend how wind load affects a structure, we encounter a wide range of factors that cover every possible scenario of events that may be seen in many ways and tend to produce an infinite number of aerodynamic outflow patterns that can subsequently be inferred to fit into a generalized notation. Steel and RCC are typically used in building construction to generate rigid structures that can bend to controlled magnitudes before failing. Eddy currents are believed to exist on these structures, nevertheless, because the wind force is obstructing them. Due to its fluid nature, the wind is made up of circular eddies that circulate in various directions and with varying radii. The wind is given a blustery or turbulent distribution by these eddies, which significantly affects anything that stands in its course. Drag and blockage prevent the fluid from flowing freely across the building surface. The fluid's response force as it passes the structure is known as drag.

The wind also changes the shape of the body that the fluid comes into contact with, which puts a lot of strain on the structure in addition to creating vortex shedding in the building in the opposite direction of the wind flow. The Reynolds number will not be consistent for roughly the whole figure when the flow is turbulent. Non-streamlined body shapes, such as cubes and cylinders, produce much more resistance or drag at the same fluid velocity, as known facts declare. Since high-speed wind forces are known to have a significant role in the vibrations and oscillations of tall building RCC constructions, it is thus suggested that the building's shape and specific geometries with lower drag coefficients be used when building tall buildings. All structures undergo several modal form changes while under stress.

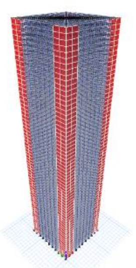

Fig. 1 A perspective view of the SWMRF structure

Fig. 2 A perspective view of the Diagrid structure

1.1 Objective

The goal of the study is to improve knowledge of how wind pressure and its dispersion affect structures with different cross sections. aimed to investigate the load transfer mechanism in different parts in order to come up with solutions for improved load distribution.

1.2 Background

Because skilled workers from all over the nation have been relocating to active urban areas due to localized industrialization since the late 1800s, taller and taller buildings have been built to house more people in a smaller urban area while still being close to the work site. By the early 2000s, the issue had gotten much worse, and as a result of the population boom, skyscrapers and towering structures were commonplace in the majority of urban areas.

At this time, nearly all nations established regulatory standards to provide safety procedures for the planning and building of tall buildings in general. Following an experimental and mathematical assessment, the procedures were released as national and international codes of practice. The extreme sway and shock abatement rules are taken into consideration in these codes. describing the proper cross section of the members, the reinforcing details, and external support measures like dampeners to prevent hazardous or undesired sway and stress development that might potentially harm the structure. Consequently, it is safer and more resilient to wear-causing natural factors.

1.3 Significance

The general design procedure of a tall building involves consideration of the following concepts:

- Structural Integrity
- Proximity to other buildings
- Safe evacuation
- Hazard mitigation and Economic Viability

The most significant implication of the aforementioned aspects is that a structure must maintain its structural integrity in the face of natural agents. This is especially true for tall, high-rise buildings, where even the most prestigious and well-designed structures must frequently withstand the damaging effects of wind pressure. Sometimes, in cyclone-affected areas, it may reach an astounding 150 kmph. This makes it imperative to establish a uniform procedure for evaluating wind load. In an effort to determine the best cross section to produce the least amount of wind drag, towering structures were subjected to wind load analyses using ETABS.

Because there are so many people living in the building at any given moment, the process of constructing a safe structure is crucial. It must be built to retain its integrity and usefulness until all money is received to at least partially offset the cost of construction because it also has a high financial and environmental cost.

1.4 Methodology

Arunachalam et al. (2011) suggested a semi-heuristic way to diagnosis the mode spectrum versus quantized wind pressure across a large region. By incorporating a dimensionless parameter, designated as "fact" in the equation, which falls between zero and one and is consistent with Ansys' lock-in excitation capability—which would subsequently be used to aid in the evaluation of the wind force diagram—the suggested solution was reinforced even further. The following presumptions were taken into account to produce the outcome:

- $exp\,(-c\Delta z/D_e) \approx 1$, due to the direct relation of vortex shedding at the upper one-third of the structure.
- As turbulence intensity variation in the upper 1/3rd of the structure is negligible, $I_{z1} = I_{z2} = I_{zref}$ and I_{zref} are taken to be equal to the turbulence intensity at 5/6 H above ground level.
- $f_{sh,z1} = f_{sh,z2} = f_o$ in the lock-in area because the shedding frequency tends f_{sh} to resonate into the structure's natural frequency, f_o.
- In the lock-in zone, the eddies are more concentrated, converging their frequencies in a particular region around the natural frequency, f_o.
- The top one third section of the building is considered to have approximately the same diameter, i.e. $D_{z1} = D_{z2} = D_e$.

And therefore, the preceding assumptions lead us to the following equation:

$$S_{mf,overall}\,(f) = \left(0.5\rho\overline{V_{ref}^2}D_eH\right)^2\left[\frac{\{(0.089)(3.4)\}^2}{B\sqrt{\pi}(fact)}\cdot\frac{1}{H^{2\beta+2}}\cdot\frac{1}{\left(\frac{5H}{6}\right)^{4\alpha}}\right]$$

$$\int_{2H/3}^{H}\int_{2H/3}^{H}\left(\frac{1}{f_0}\right)exp\left[-\left\{\frac{1-\frac{f}{f_0}}{B.fact}\right\}^2\right]\cdot\left\{\frac{1}{3.4-0.12\left(I_{\frac{5H}{6}}-7.5\right)}\right\}^2(z_1z_2)^{2\alpha+\beta}.dz_1dz_2 \qquad ...(1)$$

Where,

ρ= density of air,

V_{ref} = mean velocity at z_{ref}

D_e = effective dia at 5/6H

H =height of the structure

B =spectral bandwidth

I(5H/6) = turbulence intensity at height 5H/6

∝ = power law exponent of mean velocity profile

β = fundamental mode shape.

Since the upper third of the structure is thought to be primarily responsible for the vortex shedding, as was previously described, the limits of integration are taken from H to two thirds of H, and equation (1) therefore becomes:

$$\int_0^\infty S_{mf,overall}(f)df = \left\{\left(0.5\rho\left(\overline{V_{ref}}\right)^2 D_e H\right)^2 \{(0.089)(3.4)\}^2.\left(\frac{6}{5}\right)^{4\alpha}\right\}\left\{\frac{1}{3.4-0.12\left(I_{\frac{5H}{6}}-7.5\right)}\right\}^2 \frac{1}{(2\alpha+\beta+1)^2} \left\{1-\left(\frac{2}{3}\right)^{(2\alpha+\beta+1)}\right\}^2 \quad \text{....(2)}$$

Going by the principles of random vibration theory and mechanical admittance function, taking only structural damping, η, the variance of the across

$$\sigma_y^2 = \frac{1}{k^2}\int_0^\infty S_{mf,overall}(f).|H(f)|^2.df \quad \text{...(3)}$$

wind response becomes:

where σ_y^2 = variance of tip deflection with wind direction due to vortex shedding and |H(f)| is the mechanical admittance function. Eq. (3) can then be re-written as:

$$\sigma_y^2 = \frac{1}{k^2}\left[\int_0^{fo} S_{mf,overall}(f)df + S_{mf,overall}(f_0).\int_0^\infty |H(f)|^2.df\right] \quad \text{...(4)}$$

$$\sigma_y = \frac{1}{k}\left[\left\{\int_0^{f0} S_{mf,overall}(f)\,df\right\}^{1/2} + \left\{\left(\frac{\pi}{4\eta}\right)\left(f_0 S_{mf,overall}(f_0)\right)\right\}^{1/2}\right] \quad \text{...(5)}$$

As $\sqrt{x+y} \approx \sqrt{x} + \sqrt{(y)}$ when y >> x, Eq. (4) can be given as:

where η is the structural damping,

$\sigma y = Response\ 'A' + Response\ 'B'$.

Take the 2nd term in Eq.(4) representing the response from eddies with frequencies centered around f_o, from Eq. (1) shows:

$$\text{Response 'B'} = \left(\frac{\sigma_y}{D_e}\right) = P\sqrt{\frac{1}{B.fact.\eta}} \quad \text{.....} \quad (6)$$

where P is:

$$P = \left\{\frac{(0.089)(3.4)}{8}\right\}.(\pi)^{-\frac{7}{4}}\left(\frac{6}{5}\right)^{2\alpha}.\frac{1}{\left(\frac{2m_{ei}}{\rho D_e^2}\right)}.\frac{1}{S^2}.\left(\frac{(2\beta+1)}{(2\alpha+\beta+1)}\right)\left(\frac{1}{3.4-0.12\left(I_{\frac{5H}{6}}-7.5\right)}\right)\left\{1-\left(\frac{2}{3}\right)^{(2\propto+\beta+1)}\right\}..(7)$$

S = Strouhal number,

$$S = \frac{f_0 D_e}{V_{ref}} \quad \text{....}(8)$$

Similarly, taking the 1st term in Eq. (4), representing the response due to vortex shedding from eddies with frequencies far from natural frequency, f_o,

$$\left(\frac{\sigma_y}{D_e}\right) = P.\left(1+\frac{1}{\sqrt{(B.'fact'.\eta)}}\right) \quad (9)$$

we can show that:

1.5 Description of the buildings:

The study makes use of simulated models of two buildings: a diagrid structure supported by CFST (Concrete Filled Steel Tube), and a typical moment-resisting frame with a shear wall at the core. The architectural design is briefly described as follows:

Table 1 – Description of the building

Sr. No.	Remarks	Model-A	Model-B
1	Model Type	Shear Wall Reinforced Moment Resisting Frame	Diagrid Model Moment Resisting Frame with inner core
2	Need for design	Sway determination using only shear wall	Sway determination using diagrid bracing technology
3	Threats to design	Seismic Vibration Live Load Wind Load	Seismic Vibration Live Load Wind Load
4	Nominal Column Section	Upto floor 34: (SWMRF-external)2.5x2.5 m^2 (SWMRF-interior)2x2 m^2 Floor 34- 65: All column sections assume the size of 2x2 m^2	Upto floor 34: (Diagrid-external)2.5x2.5 m^2 (Diagrid-interior)2x2 m^2 Floor 34-65: All column sections assume the size of 2x2 m^2
5	Nominal Beam Section	Upto floor 34: (SWMRF-external)2.0x0.8 m^2 (SWMRF-interior)1.5x0.8 m^2 After floor 34: All beam sections assume the size of 1.50 x 0.8 m^2	Interior corner beams: 2.0x0.8 m^2 All other beams have been taken as 1.5x0.8 m^2
6	Column to Column Spacing	7 m	7 m

The models have been designed on Performance Based Optimized Section Approach, and a 5% reduction factor has been followed

The Shear Wall used is 600 mm thick

The above stated data was then utilized for further analysis of the structure.

2. Results

The report can then be compiled on the basis of the following data output, taking a cyclone approach in y axis into account:

- Mean variance of moments in x,y and xy directions.
- Floor displacement

Table 2 – Mean variance of moment in x direction

Floor	Mxx (kNm/m)(Model B)	Mxx (kNm/m)(Model A)
55	12.36	18.96
50	19.14	30.51
45	52.37	74.96

40	69.22	115.57
35	94.21	170.64
30	122.58	199.26
25	142.66	233.82
20	164.19	276.47
15	243.85	423.07
10	298.12	533.46
5	327.44	566.87

Table 3 – Mean variance of moment in y direction

Floor	Myy (kNm/m)(Model B)	Myy (kNm/m)(Model A)
55	-205.43	-267.29
50	-288.71	-312.53
45	-376.94	-440.89
40	-593.26	-605.62
35	-715.36	-835.15
30	-847.09	-980.79
25	-1186.38	-1231.36
20	-1294.12	-1356.13
15	-1380.01	-1478.48
10	-2187.36	-2258.42
5	-2258.16	-2372.77

Table 4 – Mean variance of moment in xy direction

Floor	Mxy (kNm/m)(Model A)	Mxy (kNm/m)(Model B)
55	185.14	276.35
50	294.93	347.86
45	488.52	529.54
40	651.27	725.18
35	789.28	956.13
30	951.61	1159.94
25	1163.02	1376.81
20	1457.79	1683.63
15	1762.22	1807.67
10	1896.23	2028.02
5	2168.19	2232.67

3. Conclusion:

It can therefore be deduced from the aforementioned data that, while the SWMRF tends to face a more direct wind pressure which may count upwards of **7 kN/m^2** in a cyclone in the entire face, the twisted face of the diagrid makes it possible to **reduce the area under stress by 66%**. Breaking the force, which is then taken up by the CFST, already known to perform better in tension, compared to concrete.

The maximum storey displacement in X direction in control model was **63.235 mm**, while the same experienced by the diagrid model was ruled out to be **28.048 mm**. The time period of both the buildings also confirms the same as the control model tends to possess a natural time period of **15.77 seconds** while the diagrid mitigates the same at a lower band of **9.53 seconds**. All of these evidences indicate the presence of better drift management by the diagrid model.

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