Comparison between Approximate and Programmed Methods for Structural Systems of Tall Buildings

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ABSTRACT

In this research work a comparison was made between approximate and exact methods of analysis for tall buildings. One representative tall buildings system was chosen for comparison purposes, namely the Rigid Frame System. The proposed structure was modelled in a square plan area of 35×35m². The models were solely subjected to wind loads. Then, models were analysed using both approximate manual methods and a Finite Element based computer program. An Excel sheet was formulated and developed to carry out the approximate method procedure. Results for manual and computerized analysis were compared for three different building heights. The same structure was subjected to gravity loads in addition to wind loads, and was analysed and designed using the Finite Element based computer program. The design results were compared for three different heights. Obtained results indicated that manual analysis methods are reliable for buildings with heights lower than 25-storeys, especially for the higher storeys. Developed model would help in quick structural preliminary analysis, procedures and preparations of tall buildings.

Keywords: ETABS, Tall buildings, preliminary analysis, manual analysis, computer model, design comparison.

Introduction

Tall buildings have fascinated mankind since beginning of civilization. Commercial and residential tall structures have been a large part of tall building construction. The rapid growth of population in the capital city of Sudan, Khartoum, and the pressure on limited land area have influenced tall residential buildings development. High cost of land is a major reason to drive residential buildings upward. Tall commercial buildings emerged as a result to the need of business activities to be as close as possible to each other and to the city centre, thereby putting intense pressure on available land space.

In preliminary stages one can select a system for the structure from alternative proposed systems based on their performance. Deflections and major member forces could be determined as well for the chosen system. The model that is formed in the preliminary analysis should represent fairly well the principal modes of action and interaction of the major structural elements (Smith et al, 1991).

The selection of a structural system for buildings is influenced primarily by intended function, architectural considerations, internal traffic flow, nature and magnitude of horizontal loading, height, aspect ratio, influence of material, method of construction, planned location, and to a lesser extent intensity of loading (Taranath, 2010).

Tall buildings in Sudan majorly fall within 15 storeys up to 20 storeys - with only one 29 high rise building (The NTC Tower). Choosing the most cost effective structural system for a high rise building and relying on results of preliminary analysis with regard to wind loading constituted the main work studied in this research.

Approximations and simplifications adopted in making a preliminary analysis are sometimes huge, concerning loading distribution, plastic hinge formation, or when representing a complex bent as a simple cantilever.

Even with gross approximations made in simplifying the structure and affiliated loadings, it is generally expected that a preliminary analysis should give results for deflections and main member forces that are dependable within about 15% of the values of the accurate analysis (Smith et al, 1991).

Studies on frame analysis were not targeted as it is not an economical system for super tall buildings. Other structural systems were studied. Kurc and Lulec (2011) investigated the axial force in columns and walls of a 37-storey model. Their results indicated that the column and wall axial load might vary up to 45% depending on the type of analysis and effects that were considered. Sutjiadi and Charleson (2012) investigated the potential of double-layer space structures to be applied vertically as a new structural system in super-tall buildings. They concluded that compared with other current tall structures, vertical double-layer space structures are structurally relatively efficient. Kamath and Rao (2012) examined the outrigger system using ETABS software and found that performance of the outrigger is most efficient for a relative height of the outrigger equal to 0.5m. Zalka (2009) developed closed-form formulae for the static deflection of symmetric multi-storey buildings braced by moment-resisting (and/or braced) frames, (coupled) shear walls and core and the proposed closed-form solution for the top deflection is simple and reliable.

For simple building, Ajwad et al. (2015) observed that ETABS software gives more economical structure than STAAD software and the modelling process is easier.
This research addressed a verification of loading percentage variation, and a definition of the level of accuracy of approximate methods with increase in height of the structure, when subjected to a pure wind load.

The research focuses on cost effect (volume of concrete and area of steel for the structure main elements) upon choosing the structural system of a defined building. Two structural systems were compared with height increase for both a frame system, and a dual one.

Materials and Methods

The case study chosen is a building of a plan area of 35×35m², divided into 7 panels 5 meters each, both ways. Buildings’ models were constructed on ETABS for 15 storeys, 20 storeys, and 25 storeys to evaluate the best choice of a structural form -with respect to cost efficiency- from two forms: rigid frame form, and combined rigid frame-shear wall form. The models were subjected to both gravity and wind loadings.

Floor slabs were designed as 20 cm thick slabs and were subjected to a dead load of 7.5 kN/m² that consists of: 4.5 kN/m² partitions loads, 1.5 kN/m² finishing, and 1.5 kN/m² for the electrics and air conditioning equipment, and to a live load of 2.5 kN/m² as the model was considered to be an office building (BS, 1996).

Initial columns and girders sizes were adopted as 300×1000 mm and 300×700mm respectively for all floors to minimize the effect of the section variation in wind load distribution. Likewise initial columns’ orientations were adopted as shown in Fig. (3) to minimize the change in stiffness effect on wind load distribution.

Wind loads were calculated using principles outlined in the British Standard CP3 Chapter 5 Part 2. The basic wind speed was taken as 45 m/s and the building territory was taken as 3. The Topography Factor (S₁) and the Statistical Factor (S₃) were taken as 1 with reference to Table (2) and Figure (2) of the code. Ground roughness, building size and height above ground Factor (S₂) was calculated for the different heights with reference to Table (3) of the code. The design wind speed (Vₛ) and wind pressure (q) were then calculated. Results were as presented in Table (1). Wind pressure per meter height is referred to as Q₁ in the table. F is the wind force per storey height, and F₁ is the wind force per storey per bent.

Results from these calculations were assigned to the model for analysis.

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Table (1): Wind loading calculations.
Columns and girders were assigned to the selected 35×35m² plan area. Initial frames were adopted parallel to the applied wind direction. The model was analysed using a Finite Element based computer program (ETABS 9.7.2) under solely wind loads. Results were then compared with a hand calculated model. Two methods for wind loading calculation were adopted for the hand calculations: The Portal Method and The Cantilever Method (Smith et al., 1991). Results were compared with the computer results for different heights, Figures (5), (6), (8) and (9) show the results of the 20-storey model for external column B/1 and internal column B/2 as referred to Figure (1). Figure (7) shows comparison of results for bending moment of the girder at the top, middle, and low levels. Perpendicular frames were then added to account for the change in wind direction.

**Portal Method**

The shear force in each storey was allocated to the columns in proportion to the aisle width they support, as indicated in equation 1.

\[ V_{Ci,j} = V_j \times a_{i,j} / \sum L_{Gi,j} \]  

Where:

- \( V_{Ci,j} \): Shear force in column i at storey level j
- \( V_j \): Shear force at storey level j
- \( a_{i,j} \): Aisle width supported by column i
- \( L_{Gi,j} \): Length of girder i at storey level j

From the columns' shears and storey height, maximum moments at the columns were found as presented in equation 2.

\[ M_{Ci,j} = V_{Ci,j} \times H_j / 2 \]  

Where:

- \( M_{Ci,j} \): Bending moment for column i at storey level j
- \( H_j \): Height of storey j

Girder-end moments were found from the equilibrium of the joints, and girder shear were found by dividing the girder-end moment by half the span (Smith et al., 1991).

**Cantilever Method**

The second moment of the column areas about their centroid was found using equation 3.

\[ I = \sum i a_i c_i^2 \]  

Where:

- \( I \): Second moment of column areas
- \( A_i \): Area of column i
- \( C_i \): Distance of column i from the centroid

Column axial forces in each storey were found using equation 4

\[ F_i = \frac{M_i A_i}{I} \]  

Where:

- \( M_i \): The storey moment resulting from wind load.

Girder shears were found from the vertical equilibrium, girder-end moments were then found. Column maximum moments were found from the equilibrium of joints, then column shear were evaluated (Smith et al., 1991).

**Results and Discussion**

For manual analysis purpose Shear forces and bending moments due to wind loading for each storey were calculated. Results for the 20-storey model are shown in Figures (1) and (2). The selected area is shown in Figure (3), a section of which is shown in Figure (4) for 20-storey model.

![Figure (1): Wind Bending Movement](image-url)
Figure (3): Rigid Frame Model Plan

Figure (4): Rigid Frame Model, Section at Grid B

Figure (5): 20-Storey Model, Column (B/1) Wind Load Moments

Figure (6): 20-Storey Model, Column (B/2) Wind Load Moments

Comparison between manual calculations and ETABS model for column bending moments are shown in Figures (5) and (6). Two columns are chosen for the comparison; internal column and external column. Columns are defined with reference to Figure (3).

Taking the FEM as the reference, manual results for bending moment are shown to be giving approximately accurate results, and can be used to estimate the bending moments on columns for top storeys. The results at the bottom storey are shown to vary from the FEM results. Hence the manual methods seem to under-estimate the bending moments applied to columns at the bottom storeys.

The Cantilever method appears to give results approximately similar to the program results. Though it shows little negative difference in the inner columns, the Portal method appears to over-estimate bending moments in the outer columns.

As the building height increases, the Cantilever method appears to be giving more accurate results, and hence can be used to estimate the bending moments for tall buildings for preliminary design.

![Figure (7): beam bending Moments](image-url)
Figure (7) Cont.: beam bending Moments

Comparison between manual calculations and ETABS model for girder bending moments are shown in Figure (7). One girder (girder at grid A, Figure (3)) was chosen for the comparison, from the 20th storey down to the ground level storey.

Taking the FEM as the reference, the girder's moments are more accurately estimated by the Portal Method at the middle storeys. The method tends to overestimate the girder moments at both end bays and under-estimates them at the middle bays for the top storeys.

The Cantilever Method appears to slightly over-estimate the bending moments on the girders at all levels.

The actual bending moment diagram appears to be completely different from the diagrams estimated by both manual methods for the top two storeys. This was justified by the shear deflection mode of the rigid frame, which tends to reverse the girder moments at the top storeys, and this wasn't taken into account in the approximate methods.

Comparison between manual calculations and ETABS model for column shear forces are shown diagrammatically in Figures (8) and (9). The two columns compared for moments are chosen for shear comparison for the 20-storey model. Columns are defined with reference to Figure (3).

The over-all trend of the shear force change over the height seems to be well demonstrated by both manual methods.

The Portal Method gives identical shear forces at ground level and at base, which is because the applied wind load was not given a value at the ground level considering it as the zero level of wind loading. The portal method calculates the shear force on each column by allocating the storey shear to the columns in proportion to their aisle widths, as the storey shear did not increase at the ground level the shear force on columns should stay the same, as shown by the results.

The Cantilever Method also gives identical shear forces at the ground level and at the base, which could be explained by the fact that the method transforms the wind moments into shear forces at the girder, then to girder moments, subsequently column moments, and finally column shears. This implies that the method depends on girders to receive wind shear forces. This may lead to the idea that the slight increase in wind shear force at the base is taken by the girder not by the columns. This does not apply to the 25 storey model, as it may be shown that there is a remarkable change in shear force at the base for both inner and outer columns, this may be explained by the fact that the increase in wind moment is significant.

The significant drawback in shear force at the bottom storeys that is shown by ETABS results, can be justified by the racking behaviour of the frame. This generates moments resulting in shear forces that applies to the opposite direction of those caused by the wind loads, resulting in a diminution in shear forces at that level. This effect is more recognizable at the outer columns which act as the outer fibres of the vertical cantilever.

In an over-all view, the Cantilever method gives approximately accurate results, slightly over-estimated, for the outer columns for all heights, and slightly under-estimated at bottom storeys for inner columns for all heights. The Portal method gives over-estimated shear force results in columns. Cantilever method can be a useful tool for preliminary analysis.

**Design of Structural Members**

Floor slabs were meshed 1 meter each way. Frames perpendicular to the assumed direction of wind loading application were added to the model, and column directions were changed to account for change in wind direction. The new column arrangement is shown in Figure (10). The wind load direction is shown by the arrows. Only one direction was considered in the analysis and design procedure.

Design procedure consisted of several trials, where the sections of the columns were changed in every trial step to obtain optimum design sections. Hence the best combination-with respect to quantities-of concrete sections/reinforcement steel was chosen. The design procedure focused on designing the columns because the floor framing does not change much with height; therefore it remains constant for all models.

![Figure (10): Rigid Frame Model Plan (Modified Frames)](image-url)
Figure (11) shows the change in reinforcing steel and column section with height for column A-1 as referred to in Figure (10). The X-axis indicates the reinforcement area in squared millimetres, the Y-axis indicates the storey number, and the bubble size indicates the column section. Each column section used in the design was assigned to a number as follows:

- Section 50×25cm² assigned to number 1
- Section 60×25cm² assigned to number 2
- Section 70×25cm² assigned to number 3
- Section 80×25cm² assigned to number 4
- Section 90×25cm² assigned to number 5
- Section 100×25cm² assigned to number 6
- Section 110×25cm² assigned to number 7
- Section 100×30cm² assigned to number 8
- Section 110×30cm² assigned to number 9
- Section 120×30cm² assigned to number 10
- Section 120×35cm² assigned to number 11
- Section 130×30cm² assigned to number 12
- Section 130×40cm² assigned to number 13

Figure (11): Column A-1 Comparison of Quantities
Comparison charts show that as height increases the required reinforcement increases at top levels for outer columns. This may be attributed to the effect of wind overturning moment which is not suppressed by gravity loads at high levels. The required reinforcement and concrete section increases at bottom levels for inner columns due to the increase in gravity loading.

The 25-storey building shows major increase in material quantities when compared with 20-storey building. The 15-storey building shows minimal material quantities, which makes the rigid frame system a good candidate for storeys up to 20.

Conclusions

From the research work undertaken herein the following conclusions emerged:

1. Manual methods of analysis for rigid frames were found to give reliable results for the preliminary stages of the design process.
2. The final design for rigid frames should be made consulting the finite element method, FEM, results of analysis. This is because the manual methods tend to under-estimate forces on structural members for buildings higher than 20 storeys.
3. The rigidity and coherence of the rigid frame render it a very suitable system at the proposed building with height lower than 25-storeys. This is due to the tendency of used materials to increase when height exceeds aforementioned value.

References

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