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Deterministic Wind Load Dynamic Analysis of High Rise Steel Buildings Including P-Delta Effects

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Abstract— This study concerns with the investigation of the second-order geometric nonlinearity effects of P-Delta analysis on the dynamic response of high rise steel buildings due to deterministic wind load. Linear and nonlinear time history analyses were conducted to analyze different tall steel building models adopted in the study. Five steel building models ranging from 10 to 50 stories were numerically modeled and analyzed using finite element code ETABS (version 16.0.3). Deterministic dynamic wind load per ASCE 7-10 is applied to the buildings as a main lateral load. Comparative study between linear and nonlinear time history analyses reveals that nonlinear time history analysis including P-Delta effects displayed larger values of buildings lateral sway than those of linear time history analysis. Generally, including P-Delta effect in the nonlinear analysis increases the flexibility of the building structure, and thus increases response peak values and that peak values occur at a longer time periods indicating lesser response oscillations. The study recommends that P-Delta effect need to be addressed by any dynamic wind analysis for tall steel buildings with 20 story height or more.

Keywords— Finite element, High-Rise buildings, P-Delta, Time history analysis, Wind load.

1. Introduction

There has been a major move towards the construction of tall buildings in recent years. In tall buildings, stability becomes more prominent since effects such as, P-Delta effect would occur and weaken the effectiveness of tall buildings to lateral loads and thus impair the stability of tall buildings [8].

There have been so many occasions in which structures failed due to instability, thus a special type of analysis should be carried out to avoid instability issues. Engineers typically use linear elastic static analysis to determine design forces and moments resulting from loads acting on a structure. In a first-order elastic analysis, equilibrium and kinematic relationships are based on the non deformed geometry of the structure. When lateral loads, such as wind loads, are applied to the structure, it often assumes a configuration which deviates quite noticeably from its non deformed configuration requiring a second order analysis. A second order analysis, which applies equilibrium and kinematic relationships to the deformed structure, is always necessary for the stability consideration of structures [3].

The P-Delta effects are the second order effects seen in slender structures due to additional moments developed due to excessive lateral sways. According to AISC 360-10 [1], two types of secondary effects can be identified; The P-δ and the P-Δ effect. The P-δ is the effect of loads acting on the deflected shape of a member between joints and nodes, whereas P-Δ is the effect of loads acting on the displaced location of joints or nodes in a structure. **Fig.1** shows both types of P-Delta effects [3].

Wind creates inward and outward pressures acting on building surfaces, depending on the orientation of the surface. This pressure produces uplift on some parts of the building, forcing the building apart if it is too weak to resist the wind loads. Therefore, it is crucial to overcome this problem by selecting an appropriate connection between beams and columns in a frame such as rigid or pin ended, moreover, a suitable bracing system must be introduced to withstand any additional lateral loads [6].

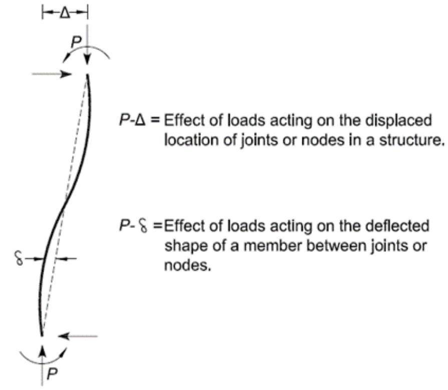

Figure 1: P-Delta effects [3]

The issue of P-Delta effects was an area of extensive research in recent years, chiefly for seismic excitation. Most recently, Mallikarjuna and Ranjith [6], and Mosa [7] investigated the effect of P-Delta on the dynamic response of tall buildings due to wind loading.

This research aims to investigate the effects of P-Delta on the dynamic response of tall steel buildings under wind load. Linear and geometric nonlinear with large displacement dynamic time history analysis was conducted for different building models with variable heights. ETABS software (Ver. 16.0.3) is used throughout the study.

2. Description of the Models

Models adopted throughout the present study are essentially multi-story braced steel frame buildings with different number of stories. Mainly, all models are square in their plane and divided into 9 bays in each direction (X and Y), each panel has a span of 4 meter. The story height is fixed at 4 meter, and the number of stories is ranging from 10 to 50 with 10 stories increment (i.e. 5 models in total). The floor system is set to be composite concrete deck slab with properties conforming to the stipulations stated in AISC 360-10 [1]. Deck total depth is assumed to be 100 mm which includes both slab and rib depths. Constant number of shear studs connecting the deck slab to secondary steel beams (joists) to simulate full composite action. Default meshing of floor system is selected, where auto cookie cuts the horizontal floors at beams and walls, and consequently 3 elements for each panel are created. The models were braced in the X-axis direction along which wind loading was applied. X-bracing system was implemented for its efficiency to resist lateral loads as shown in **Fig.2**. Buildings are symbolized or labeled **"B"** according to their height starting from 10 story building and up to 50 stories.

Load cases and wind coefficients adopted in the present study are listed in **Table 1**. Sections of the different structural components incorporated in the building models such as, girders, joists, bracing, columns are given in **Table 2**. The selection of the members' properties was to satisfy strength and serviceability requirement as per AISC 360-10 [1].

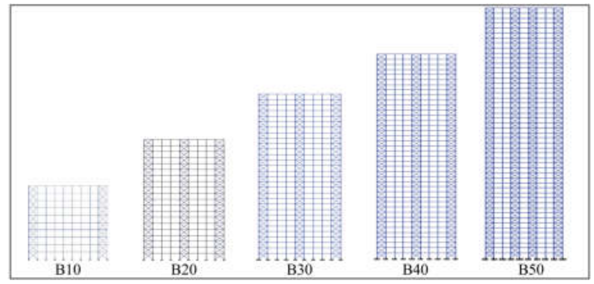

Figure 2: Building models with the adopted X-bracing system.

Joints at ends of columns and girders are assumed to be continuous, whereas pinned connections are assigned for cross bracing and secondary beams. The superstructure model was isolated from its substructure, and placed upon idealized rigid supports. The support under each column is assumed fixed. **Fig.3** shows the plan for the main study models, a square shape (36m by 36m) with the columns' major axes along the X-axis.

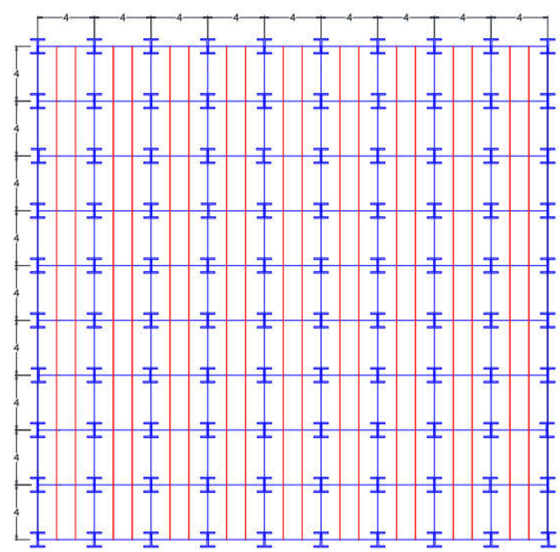

Figure 3: Building model floor plan.

Table 1: Load cases and wind parameters.

Case	Value
Super Dead Load	$2\ \mathrm{kN/m^2}$
Live Load	$2\ \mathrm{kN/m^2}$
Line Load	$1.5\ \mathrm{kN/m}$
Wind Speed	100 mph
Exposure Type	B, C, D
Topographical Factor	1
Gust Factor	0.85
Directionality Factor	0.85

Table 2: Structural component sections used in the building models

B10					
Story Numbers	1 to 10				
Column Section	HP10×42				
Bracing Section	HSS6×6×1/4				
Girder/ Joist Section	W14×22 / W10×12				
B20					
Story Numbers	11 to 20	1 to 10			
Column Section	HP12×53	HP14×73			
Bracing Section	HSS6×6×1/2				
Girder/ Joist Section	W16×26 / W10×12				
B30					
Story Numbers	21 to 30	11 to 20	1 to 10		
Column Section	HP12×53	HP14×73	HP16×121		
Bracing Section	HSS8×8×3/8				
Girder/ Joist Section	W16×36 / W10×12				
B40					
Story Numbers	31 to 40	21 to 30	11 to 20	1 to 10	
Column Section	HP12×53	HP12×84	HP16×121	HP16×183	
Bracing Section	HSS8×8×3/8				
Girder/ Joist Section	W18×40 / W10×12				
B50					
Story Numbers	41 to 50	31 to 40	21 to 30	11 to 20	1 to 10
Column Section	HP12×74	HP14×102	HP16×121	HP18×135	HP18×204
Bracing Section	HSS10×10×5/8				
Girder/ Joist Section	W21×44 / W10×12				

3. Deterministic Wind Load Modeling

Some analyses in ETABS (Ver. 16.0.3) demand two components of wind loading to run properly, but it is not always necessary [4]. Time history analysis cannot be carried out unless the two components are specified. The following subsections are dedicated to outline the two components of wind loading.

3.1 *Spatial Component*

ETABS software is capable of generating automatic wind loads using various international design codes. In this study, ASCE 7-10 [2] was implemented and the exposure was chosen to be from area objects.

Area objects which enclose the four aspects of the models were modeled as cladding with zero area mass and without any section properties. An option in ETABS to draw auto cladding using any of the following,

1. Floors.
2. Beams.
3. Columns.

In the present study, the first option was selected. Wind loads were only applied in global X direction, and thus a pressure coefficient of 0.8 was used for windward direction, and 0.5 for leeward direction in accordance with ASCE 7-10 stipulations [2].

3.2 *Temporal Component*

Wind speed at any time and height may be written in terms of the product of vertical profile (spatial component) and a time function (temporal component) as shown in Eq. (1) [5],

$$V_{(z.t)} = V_{(z)} \times V_{(t)} \tag{1}$$

where, $V_{(z)}$ is the vertical profile, and $V_{(t)}$ is the time function.

The time varying feature of wind could be described by employing a half sine wave as shown in Eq. (2) below [5]

$$V_{(t)} = \sin\left(\frac{\pi}{t_d}t\right) \tag{2}$$

where, t_d is pulse duration of the excitation.

The equation above may not represent the exact time variation of winds, but it is capable of capturing the underlying feature responsible for the enhanced loads in a typical gust-front [5].

To avoid confusion, the two components of wind are implemented in time history analysis, where load name and function stand for the spatial and temporal components of wind, respectively. Temporal function could be easily defined using Eq. (2) and then imported into ETABS by means of a text file which contains readings of 5 seconds with an increment of 0.1 second as shown in Fig.4.

4. Results and Discussions

4.1 *Linear Time History Analysis Results*

This subsection summarizes models' responses in terms of top displacement and base shear analyzed under linear time history as illustrated in Fig.5. Temporal function period used in this analysis is of 5 seconds, and damping ratio was selected according to the recommendations stated in ASCE 7-10 under wind chapters, that is 2%. The figures for all models show that taller buildings display fewer oscillations than their shorter counterparts and that peak values for the response are, generally, greater for taller buildings. Moreover, peak responses occur at longer time periods for tall buildings as opposed to shorter buildings.

Finally, it seems appropriate to consolidate all buildings' responses in one graph to conceive a better insight of their behavior as presented in Fig.6.

4.2 *Nonlinear Time History Analysis Results*

As in subsection 4.1, the same procedures are applied here with only one exception, a comparison of linear time history and nonlinear time history with P-Delta analyses are illustrated in the same graph to substantiate the difference between the two analyses procedures and to focus on the effect of P-delta analysis on the response values.

Below are the graphs in Fig.7 of all five building models depicting results of top displacement and base shear analyzed under nonlinear time history with P-Delta analysis and compared with linear time history analysis.

Top displacement results presented in these figures illustrate the superiority as to which analysis yields larger response values. Nonlinear P-Delta analysis imparts larger numbers than linear analysis in terms of top displacement. Also, top displacement results indicate that the dynamic response of buildings under both types of analyses occurs at almost the same time for B10 building, i.e. building with 10 stories, since the peak values occur at the same time. On the other hand, for taller building models, Peak values due to P-Delta analysis occur at a longer time periods due to the fact that P-Delta effects increase the flexibility of a given structure which leads to an increase of the time required to complete a cycle of lateral movement.

As for base shear results, these figures demonstrate that base shears for taller buildings due to nonlinear analysis are, generally, smaller than that of linear analysis, and the difference increases as the number of stories increases, because the higher the building, the more flexible it becomes due to nonlinear behavior and the more time it requires to complete a cycle of lateral sway which leads to decrease of base shear values, and thus base shear are inversely proportional to building height.

Finally, it is worthwhile to consolidate P-Delta effects on the dynamic response of the adopted building models as shown in Fig.8. In this figure, increase in the buildings peak sway is presented as a function of building height. It is obvious that buildings sway rapidly increases as building height increase, especially for building with 20 stories or more. This result validates the importance of P-Delta effect on the dynamic wind response of such buildings.

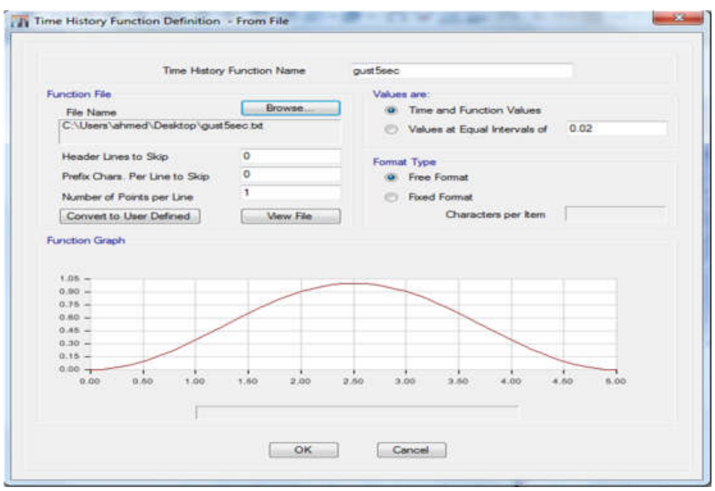

Figure 4: Temporal variation of wind pressure

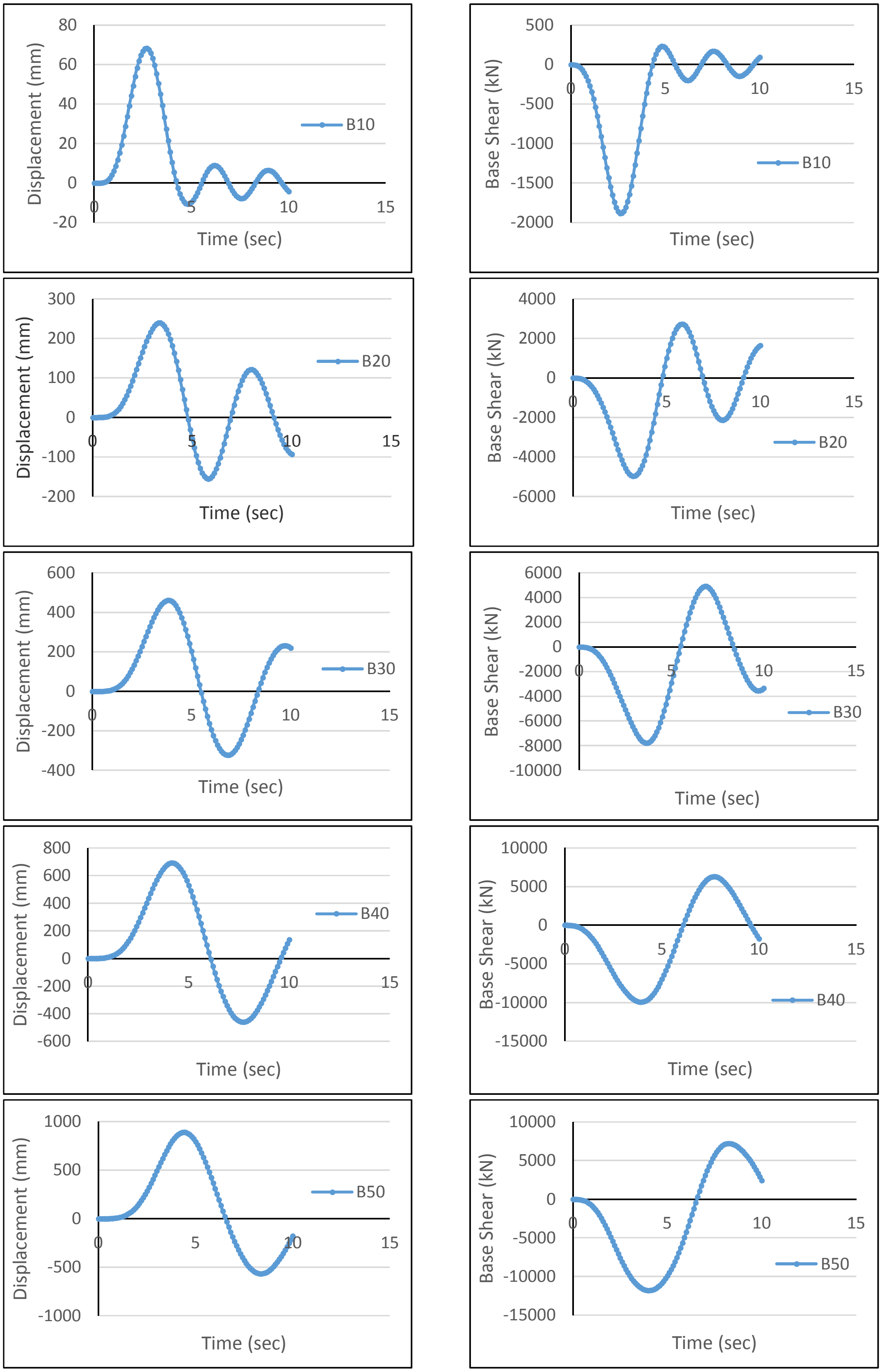

Figure 5: Displacement and base shear time history for building models (Linear Time History Analysis)

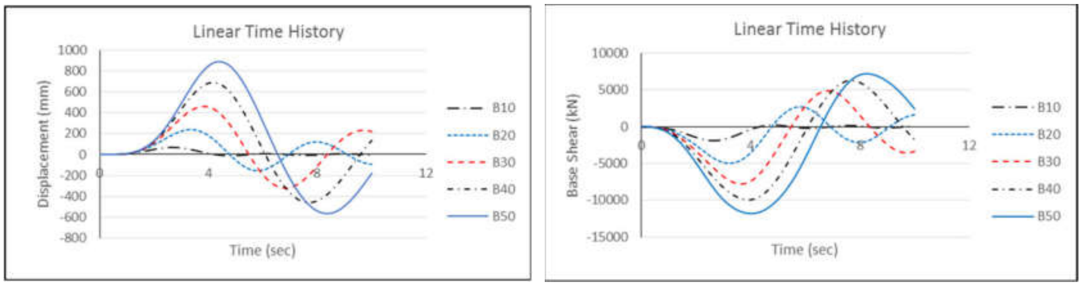

Figure 6: Top displacement and base shear time histories for all building models.

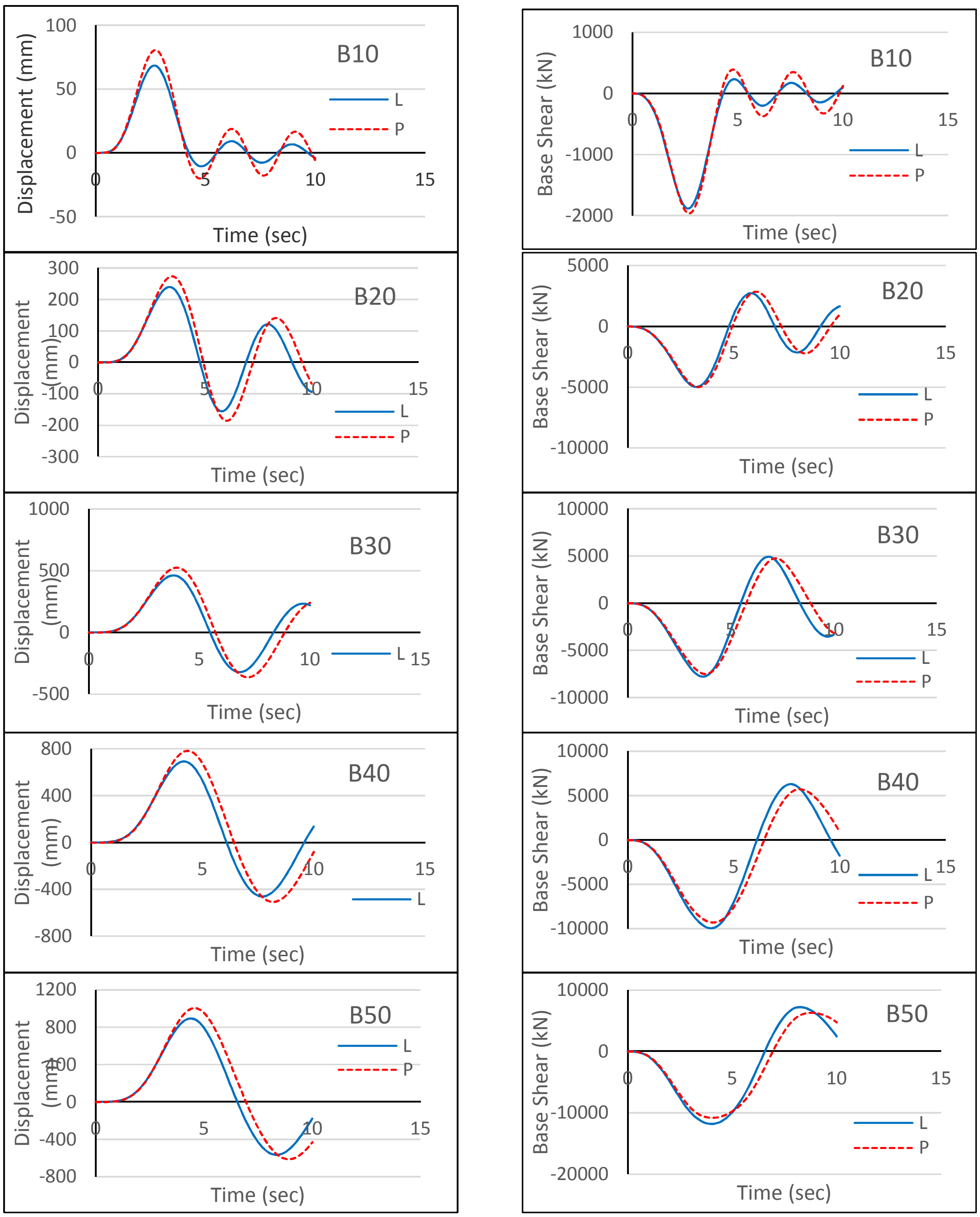

Figure 7: Comparison of linear (L) and nonlinear P-Delta (P) analyses in terms of top displacement and base shear.

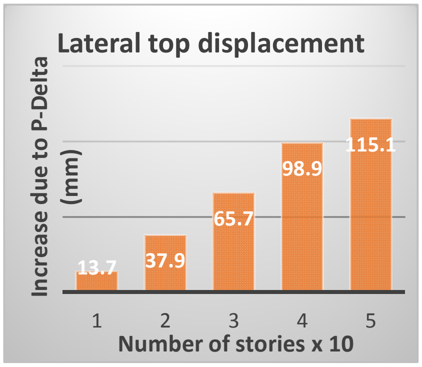

Figure 8: Increase in buildings peak top sway due to dynamic P-Delta effect.

5. Conclusions

In this paper, an attempt has been carried out to investigate the effects of P-Delta (nonlinear geometric analysis) on the dynamic response of tall steel buildings, and thus the following conclusions are drawn in below.

1. Taller buildings exhibit less oscillations due to wind load than shorter counter parts, as shorter buildings damp the lateral motion quicker than those of taller heights.
2. Response peak values with P-Delta analysis occurs at longer time periods due to the fact that P-Delta effects increase the flexibility of a given structure which leads to an increase in the time period of lateral sway or vibration.
3. Nonlinear time history analysis including P-Delta effect indicates results for lateral displacements larger in value than those of linear time history analysis, and thus it is imperative to carry out nonlinear time history analysis, since it yields larger and more significant response values.
4. Results show that the effects of P-Delta on the dynamic response of tall buildings with 20 story height or more are significant and must be addressed by any dynamic wind analysis for high rise steel buildings.

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التحليل الحركي لاحمال الرياح الحتمي للابنية الفولاذية الشاهقة متضمنا تاثير بي-دلتا

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الخلاصة – تتعلق هذه الدراسة بالتحقيق في التأثيرات اللاخطية الهندسية من الدرجة الثانية لتحليل بي-دلتا على الاستجابة الحركية للمباني الفولاذية العالية الارتفاع بسبب احمال الرياح الحتمية. تم الاخذ بالاعتبار التحليل الخطي والاخطي لتحليل الاستجابة الزمني لتحليل نماذج مختلفة لابنية عالية الارتفاع تم تبنيها في الدراسة. تم نمذجة خمسة نماذج من الابنية الفولاذية تتراوح من 10 إلى 50 طابقا عدديًا وتحليلها باستخدام برنامج التحليل الانشائي ETABS (الإصدار 16.0.3). تم تطبيق حمل الرياح الحركي الحتمي على المباني بموجب المواصفة الامريكية ASCE 7-10 كالحمل الجانبي الرئيسي. تكشف الدراسة المقارنة بين تحليلات الاستجابة الزمني الخطية واللاخطية أن تحليل الاستجابة الزمني الاخطي المتضمن تأثير بي-دلتا أظهر قيمًا أكبر لازاحة المباني الجانبية من قيم تحليل الاستجابة الزمني الخطي. بشكل عام ، تضمين تأثير بي-دلتا في التحليل اللاخطي يزيد من مرونة بنية المبنى ، وبالتالي زيادة قيم ذروة الاستجابة وأن قيم الذروة تحدث في فترات زمنية أطول مشيرا الى تذبذبات استجابة اقل. توصي الدراسة بضرورة ان يؤخذ تأثير بي-دلتا من خلال أي تحليل رياح حركي للمباني الفولاذية عالية الارتفاع التي يبلغ ارتفاعها 20 طابقًا أو أكثر..

الكلمات الرئيسية – طريقة العناصر المحددة، المباني الشاهقة، تأثير بي-دلتا، تحليل الاستجابة الزمن، احمال الرياح.