UNIVERSITY OF GAZİANTEP
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A NUMERICAL STUDY ON P-DELTA EFFECT IN HIGH-RISE REINFORCED CONCRETE BUILDINGS SUBJECTED TO SEISMIC EXCITATION

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IN
CIVIL ENGINEERING

By
ASAAD MOHAMMED HUSSEIN KADHIM
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A Numerical Study on P-Delta Effect in High-Rise Reinforced Concrete Buildings Subjected to Seismic Excitation

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Supervisor

Assoc. Prof. Dr. Esra METE GÜNEYİSİ

By

Asaad Mohammed Hussein KADHIM

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Asaad Mohammed Hussein KADHIM

ABSTRACT

A NUMERICAL STUDY ON P-DELTA EFFECT IN HIGH-RISE REINFORCED CONCRETE BUILDINGS SUBJECTED TO SEISMIC EXCITATION

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M.Sc. in Civil Engineering

Supervisor: Assoc. Prof. Dr. Esra METE GÜNEYİSİ

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Lateral displacement caused by main lateral loads such as earthquake, wind, and blast loadings is the key factor that controls the structural design of multistorey buildings and plays important roles in P-delta effect consideration. The present study aimed to evaluate the dynamic response of high rise reinforced concrete structures with P-delta effect. For this, five different frame structures were studied. They had four bays and similar floor plans, however, number of storeys varied as 10, 15, 20, 25, and 30. The nonlinear dynamic analysis was performed for all frame models. In the analysis, two cases were adopted. The first one was conducted with ignoring P-delta effect while in the second one, the P-delta effect was introduced for all. Three ground motion records, namely, 1979 Imperial Valley, 1987 Superstition-Hills, and 1992 Landers were utilized in the dynamic analysis. As a seismic hazard level, 10% probability of exceedance in 50-year period was taken into account. The results of the dynamic analysis showed that P-delta effect were more pronounced in the case of the structures having higher storeys. Moreover, the response of the case study structures having P-delta were influenced with the characteristic of the earthquakes used.

Keywords: Dynamic analysis, Earthquake, High rise building, P-delta effect, Reinforced concrete frame

ÖZET

SİSMİK HAREKETE MARUZ YÜKSEK KATLI BETONARME BİNALARDA P-DELTA ETKİSİ ÜZERİNE NÜMERİK BİR ÇALIŞMA

KADHIM, Asaad Mohammed Hussein
Yüksek Lisans, İnşaat Mühendisliği Bölümü
Danışman: Doç. Dr. Esra METE GÜNEYİSİ
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Deprem, rüzgar ve patlama yükleri gibi temel yanal yüklerden kaynaklanan yanal yer değiştirme, çok katlı binaların yapısal tasarımını etkileyen ve P-delta etkisinin dikkate alınmasında önemli rol oynayan bir faktördür. Sunulan çalışmada, yüksek katlı betonarme binaların dinamik tepkisinin, P-delta etkisi de düşünülerek değerlendirilmesi amaçlanmaktadır. Bunun için, beş farklı betonarme çerçeveli yapı incelenmiştir. Yapılar 4 açıklıklı ve benzer kat planlarına sahip olmakla birlikte; 10, 15, 20, 25 ve 30 kat olmak üzere farklı yüksekliktedirler. Doğrusal olmayan dinamik analiz tüm çerçeve modelleri için yapılmıştır. Analizlerde, iki durum düşünülmüştür. İlk durumda, P-delta etkisi ihmal edilirken, ikinci durumda ise, bütün binalarda P-delta etkisi dikkate alınmıştır. Dinamik analizlerde, 1979 Imperial Valley, 1987 Superstition-Hills ve 1992 Landers depremlerine ait üç yer hareketi kaydı kullanılmıştır. Araştırmada, sismik tehlike seviyesi, 50 yılda aşılma olasılığı %10 olarak dikkate alınmıştır. Analiz sonuçları P-delta etkisinin daha yüksek katlı olan yapılarda daha belirgin olduğunu göstermiştir. Ayrıca, kullanılan depremin özelliklerinin P-delta etkisindeki yapıların sismik tepkilerini etkilediği gözlenmiştir.

Anahtar Kelimeler: Dinamik analiz, Deprem, Yüksek katlı yapı, P-delta etkisi, Betonarme çerçeve

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To Dear My Father, Mother, Sister, and Brothers

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LIST OF SYMBOLS

ACI	American concrete institute code
AISC	American institute of steel construction
ASCE	American society of civil engineers
B.L	Blast loading
Cd	The deviation factor amplification
CP	Collapse prevention
CSA	Canadian standards association
C_d	Deflection amplification factor, whose value ranges from 1.25 to 6
D.L	Dead load
DOF	Degree of freedom
DSD	Displacement seismic design
E	Modulus of elasticity
EF	Element Frame
EF-1	First RC test structure
EF-2	Second RC test structure
EF-5	Fifth RC test structure
EQ	Earth quake load
EVD	Equivalent viscous damping
F	Lateral load

$f'c$	Concrete compressive strength
FEMA	Federal Emergency Management Agency
f_u	Ultimate strength of steel
f_y	Yield strength of the steel
h_{sx}	Story height below level x
H	Vector of the actual horizontal loading
H	Height of the column
H1	Horizontal force in first storey
Hn	Horizontal force in every storey
I	Moment of inertia
IBC	International Building Code
IO	Immediate occupancy
K	First order stiffness matrix
KG	Geometric stiffness matrix
KG	Geometric stiffness properties
L	Span length
L.L	Live load
LS	Life safety
M	P-Δ moment
M3	Plastic hinge moment capacity
MDOF	Multi degree of freedom
MRCBF	Modern design of RC-MRCBFs
MRF	Moment resisting frame

M_A	P-delta moment in point A
M_E	Elastic stiffness contribution
M_G	Geometric stiffness contribution
M_T	Total moment
M_w	Magnitude
NALD	Nat Haz Aerodynamic Loads Database
NZS	New Zealand Loadings Standard
P	Axial load
PGA	Peak ground acceleration
PGD	Peak ground displacement
PGV	Peak ground velocity
P-M3	Axial force- moment interaction
P-δ	P-"small-delta"
P-Δ	P-"big-delta"
P_x	Total vertical design load at and above level x with load factor ≤ 1.0
Q	Applied load
QY	Yield load
R	Response modification factor
RC	Reinforced concrete building
RUAUMOKO	Software package
SBC	Standard Building Code
SDOF	Single degree of freedom
SFRS	Scottish Fire and Rescue Service

UBC	Uniform Building Code
US	United State of America
V_x	Seismic shear force acting between level x and x-1
V_A	Lateral load in the point A
W	Distributed load
W.L	Wind load
x	Levels x and x-1
$\boldsymbol{\alpha}$	Alpha angle of inclination
β	Ratio of shear demand to shear capacity for the story between
Δ	Design story drift occurring simultaneously withV_x
Δ	P-delta displacement
Δ*	Vector of the total lateral displacements
Δ_u	Lateral deflection
Δ1	First displacement of P-delta
Δ2	Second displacement of P-delta
θ	Maximum limit on stability coefficient
ω	Longitudinal distributed load

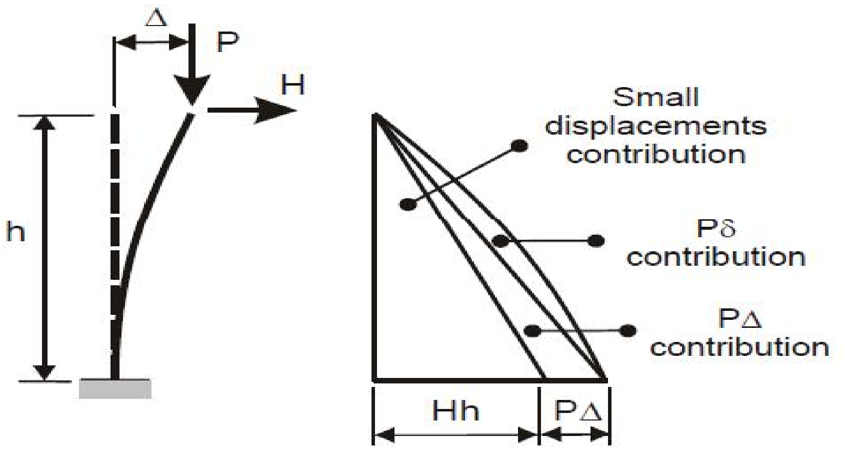
CHAPTER 1

INTRODUCTION

1.1 General

P-delta effect can be defined as the application of gravity load on laterally displaced multistory building due to many types of lateral loads, which involves in the equilibrium and compatibility relationships of a structural system loads and the redistribution of the moments, which magnifies story drift and certain mechanical behaviors while reducing deformation capacity. If deformations become sufficiently large as to break from linear compatibility relationships, then large-displacement or large-deformation analyses become necessary. However, in particular concern, in first-degree analysis of the structures, so that this effect is neglected for traditional purposes (Moghadam and Aziminejad, 2004). The two sources of P-delta effect are illustrated and described in Figure 1.1 (Powell, 2010).

P-delta effect depends on applied load and building characteristics. In addition, parameters such as length and stiffness of the building, and the degree of contrast would have been interested. Moreover the inequality of the buildings often due to the architectural purposes causes unbalances distribution of the masses, stiffness and/or the total strength. Usually they cause distortions resulting from the displacement of torsional load varying between lateral resistance elements and thus, concentration of damage in some of them. Therefore, buildings and unbalanced torsion are usually more vulnerable to earthquake damage. The distortions caused by torsion can be affected the results of P-delta effect. Consequently, it is expected that the torsion and P-delta are interacted in the seismic behavior of some buildings. It is likely to be effective in this interface and, there are long list of parameters. Some of the parameters are lateral and torsional stiffness of building; the properties of loading and ground motions, mass moment of inertia, the level of its eccentricity, and height are some of these parameters. To include the effect of P-delta in buildings, a symmetric analysis of some of the measures has been proposed in the literature such as the ones by Rutenberg (1982), Wilson and Habibullah (1987). In addition, Wynhoven and Adams (1972) studied the effect of torsion on the inelastic lateral stability of frame-shear wall building systems.



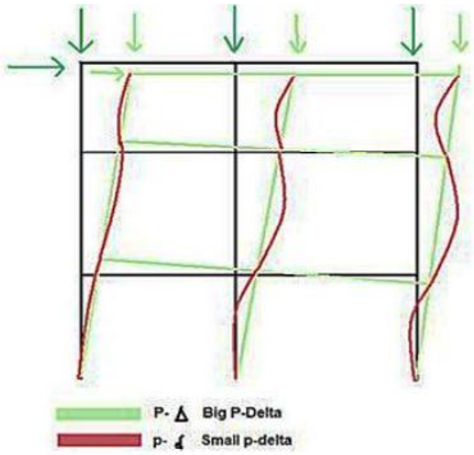
a) Column b) Bending moment

Figure 1.1 P-Δ about column (Powell, 2010)

1.2 Types of P-delta effect in multistory buildings

(i) P-"small-delta" or P-δ effect, this type of effect is associated with local deformation relative to the element chord between end nodes. Typically, P-δ only becomes significant at unreasonably large displacement values, or in especially slender columns. So long as a structure adheres to the slenderness requirements pertinent to earthquake engineering, it is not advisable to model P-δ, since it may significantly increase computational time without providing the benefit of useful information. An easier way to capture this behavior is to subdivide critical elements into multiple segments, transferring behavior into P-Δ effect (Powell, 2010).

(ii) P-"big-delta" or P-Δ effect, or, generally associated with displacements that relative to member ends. Unlike the former P-δ, this type of P-delta effect is critical to nonlinear modeling and analysis. As indicated intuitively by Figure 1.2, gravity loading would influence structural response under significant lateral displacement. P-Δ may contribute in losing the lateral resistance, ratcheting of residual deformations, and dynamic instability (Deierlein et al., 2010). As shown in Figure 1.3, the effective lateral stiffness decreases by reducing strength capacity in all phases of the force-deformation relationship (PEER/ATC, 2010).



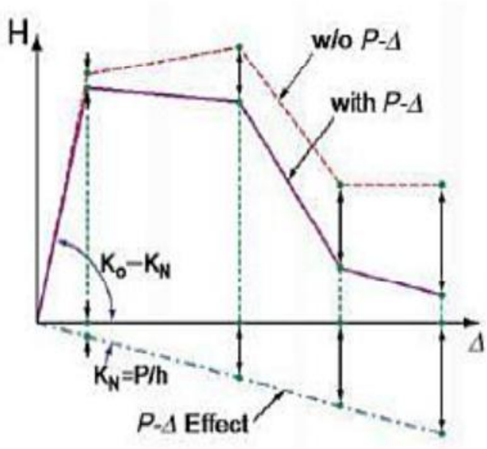


Figure 1.2 P-delta effect on structure (Deierlein et al., 2010)

Figure1.3 P-delta effect on H- Δ curve (PEER/ATC 2010)

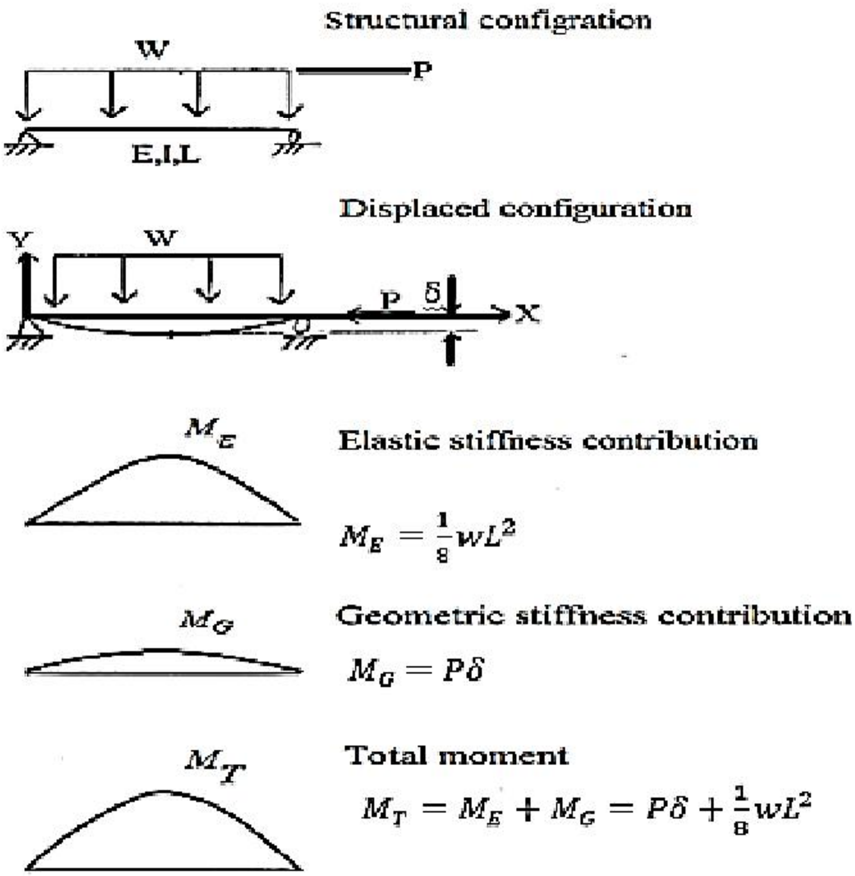
1.3 Types of P-Δ analyses in high-rise building

(i) Simply supported beam P-δ ;

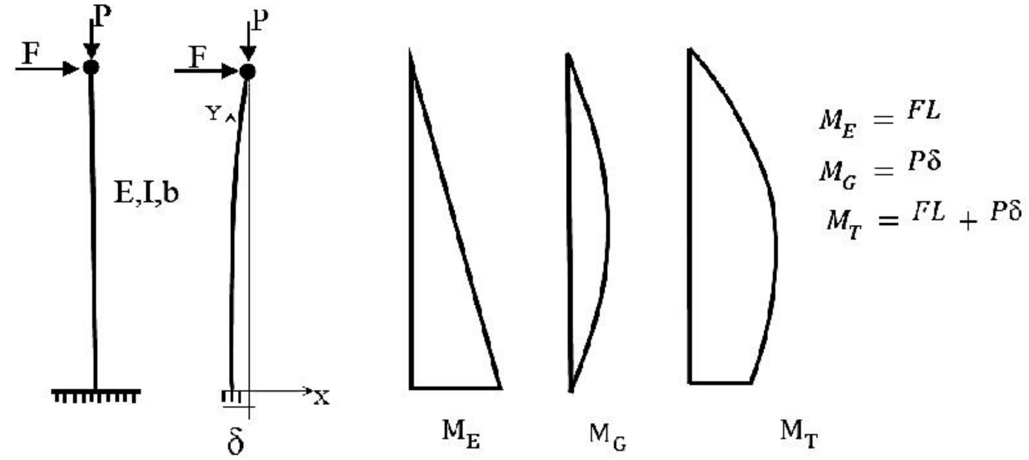
P-δ is a local effect associated with axial load on displacement relative to element chord extending between end nodes. Figure 1.4 illustrates the values for the maximum flexural response, which occurs at mid-span of a simply supported beam. Herein, the longitudinal distributed load ω correlates with elastic bending-stiffness properties KE to induce vertical displacement δ. An additional flexural contribution comes from the relationship between this deformed configuration and axial load P. The geometric stiffness properties KG, which dictate this relationship, are discussed further (Wilson et al., 2004).

(ii) Cantilevered column P-δ ;

P-Δ effect should be implemented during design, whether static or dynamic, linear or nonlinear. When considering large-displacement effect, smaller lateral displacements result. There is no geometric limitation for the application of P-delta effect, which projects laterally from the column tip in a straight line. Large-displacement effect, however, is bound by column length. As column rotation increases, the tip displaces along a curvilinear profile. As a result, for a given large-displacement effect, axial displacement should be larger (Wilson, 2004). When observing P-δ effect on a cantilevered column, response which is shown in Figure 1.4 (b). The columns seldom displace with single curvature. More commonly, with multi-story-building analysis and design, columns deform according to a third-order (cubic) displacement pattern under double curvature. As shown in Figure 1.4 (c), P-δ effect is much less pronounced because an inflection point intersects the element chord near mid span, previously where displacement from chord was greatest. However, what is often of significance, given this loading condition and double-curvature displacement pattern, is P-Δ effect. Although displacement deviates from element chord are small, the lateral displacement associated with story drift is significant. With increasing levels of drift, gravity load has a greater effect on mechanical behavior, as shown in Figure 1.4 (d).



(a)



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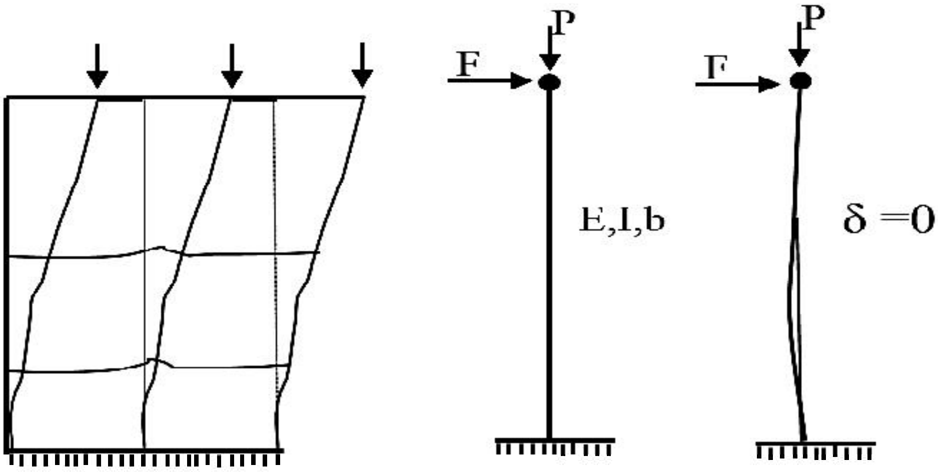

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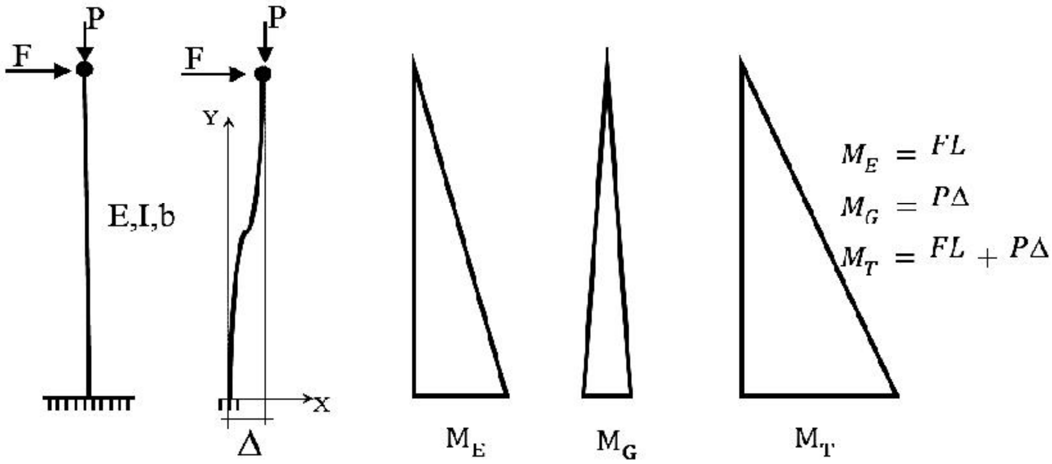
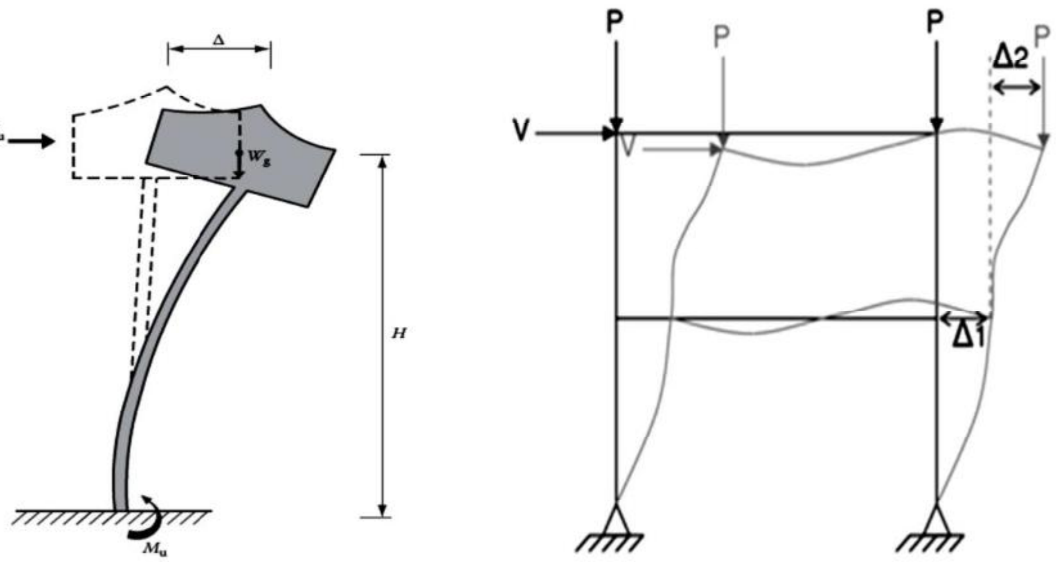


(d)

Figure 1.4 Types of P-δ analysis applied to (a) simply supported beam, (b) cantilevered column (single curvature), (c) cantilevered column (double curvature), and (d) cantilevered column by Wilson (2004)

Multistory structures may be subjected to different types and forms of lateral loadings through its life period. Examples of these loads are earthquake, wind or blast loading. The displacements or base moments causes by these mentioned loads and the normal vertical gravity loads (self-weight, other dead loads and live loads) should be interfaces in calculation So the new developed displacement would adds new forces to the system because the axes of the active gravity has been changed from its original cases, this procedure would be frequently take place, until the value reaches a non-reasonable value that can be neglected, where it is minimized or the difference is less than 5 %. This can be explained in simple model as shown in Figure 1.5 (a) and (b) (Taranath, 2009).



(a) (b)

Figure 1.5 P-Δ explanations (a) simple elevated tank pole and (b) simple frame (Taranath, 2009)

1.4 Importance of P-Δ effect analysis on high-rise building

Research on structural inelastic response has shown that P–Δ effects are significant on flexible structures and amplify the lateral displacements (Davidson et al., 1992; Gupta and Krawinkler, 2000). The additional deformations result in an increase in ductility demand as explained by Bhowmick et al. (2009). Most of code of practice in United State have intended to take into account the P-Δ effect. For example, ACI code indicates that in the design of the column, the slenderness effect can be accounted for using two different approaches. First it is called the moment magnifier approach, which uses some code-based equations to approximate these second order effects, and the other approach is to perform a P-Δ analysis. In most cases of high-rise buildings (more than 25 stories) horizontal displacement is high and thus P-Δ effect value is significant, and neglecting it may cause collapse or damage the structure as it is shown in Figure 1.6. Economic design of multi-story structures depends on more than one factor that includes property, stiffness, ductility, etc. P-Δ effect is one of the important factors that requires in many cases changing dimension or property or system of structures and this is money expense. As structure becomes more slender or has a small cross sectional area for the base plan and less resistant to deformation, or it is exposed to high intensity lateral load such as wind load or seismic than the need to consider the P-Δ effect increases. Recent studies show that P-Δ effects are reasonable and have to be included in designing calculations of high-rise building for safe and stable structures (Taranath, 2009).

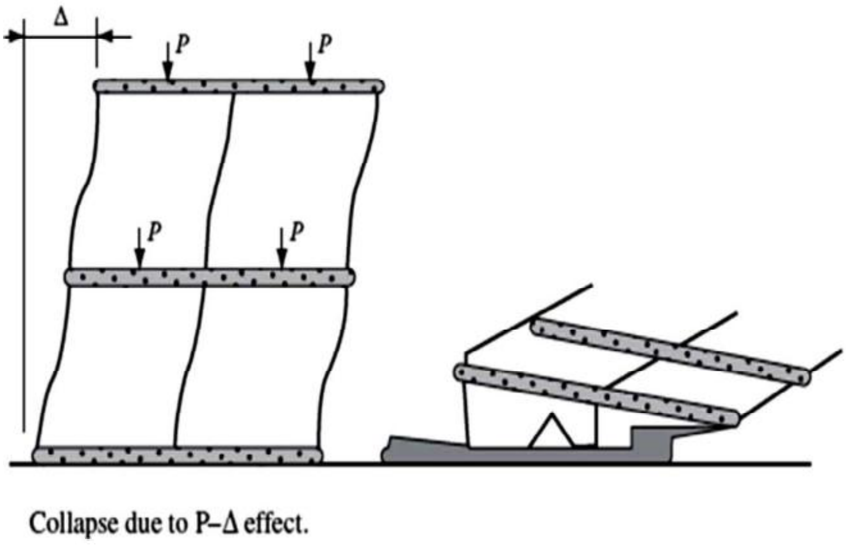
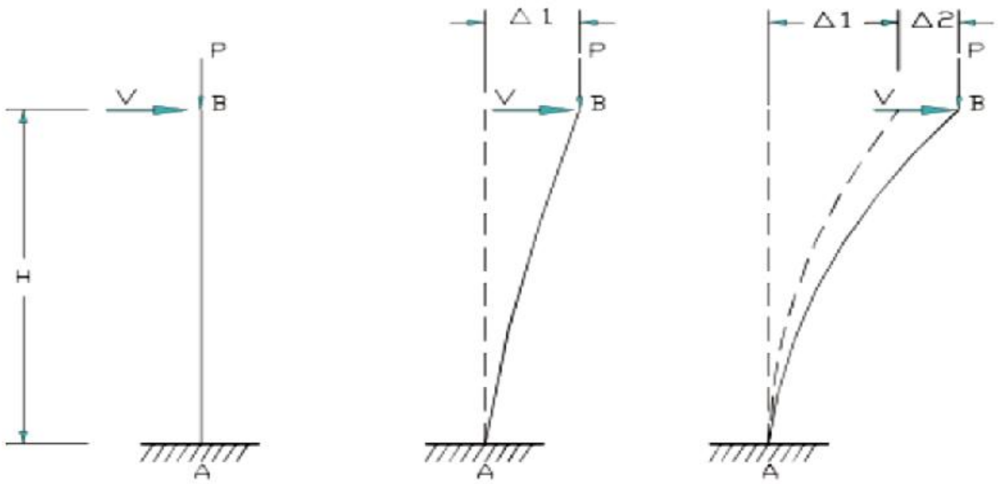


Figure 1.6 Collapse due to P-Δ effect (Taranath, 2009)

Alternatively, more P-Δ excessive effects eventually enter the characters to the solution, pointing to the lack of physical structure stabilization. Such behavior is a clear indication of bad design structure that is in need of additional stiffness. Nevertheless, if the weight of the structure is high in ratio to the lateral stiffness of the structure, and contributions from the effects of P-Δ would be amplified to a large extent, and under specific circumstances, it can be change the displacement force members by 25%. P-Δ would create an additional base moment as from its definition in Eqn. (1.1) (Wilson, 1997).

$$\mathrm{M} = \mathrm{P} \times \Delta \tag{1.1}$$

As Δ increases the amount of P-Δ moment (M) would be increased with repeating the iteration. This is clear in Figure 1.7. The iteration has to be repeated until relative Δ would reach small and negligible value (Naeim, 2001).



$\boldsymbol{V_A = V}$ $\boldsymbol{V_A = V}$ $\boldsymbol{V_A = V}$

$\boldsymbol{M_A = V.H}$ $\boldsymbol{M_A = V.H + P\Delta 1}$ $\boldsymbol{M_A = V.H + P(\Delta 1 + \Delta 2)}$

(a) (b) (c)

Figure 1.7 The P-delta effect (a) equilibrium in the under formed state, (b) immediate P-delta effect, and (c) accumulation of the P-delta effect (Naeim, 2001)

P-Δ can be calculated to see whether the values are within the 5 %, if it is out of this limitation the P-Δ effect should be considered Otherwise control should be done by changing the system of structure or adding some additional members to increase the stiffness or distributing the mass in order to decrease the tensions effect that will produce additional P-Δ effect. Research that had been done on P-Δ recommended the designers to choose a new systems of structure and using advance calculation for determining P-Δ to get the exact value that is important to get economic structures with high resistance (Andrews, 1977). It is highly recommended by researchers to increasing the frame rigidity by reducing the erosion, and the influence of implications P-Δ can be ignored. The more practical to control of the excessive displacement in the framework of structures, that is affected by P-Δ are to increasing the strength instead of hardness. This is an important subject to decide when P-Δ has to be considered in structural designing. In addition, it depends upon the experience of engineer. The P-Δ effects to be included are as follows (Black, 2011):

(i) High rise building with more than or equal to 25 story with normal story height and member stiffness,

(ii) A structure that would be constructed in the high intensity earthquake or max ground motion that may be occurring concurrently with the other stresses (such as gravity), at a particular instant the stresses may all be additive,

(iii) Weak structure with low stiffness or stability due to large spans and story height or with not enough shear wall or bad arranged shear walls, causing large displacement and then large P-Δ would occur,

(iv) Multi story building exposed to very strong winds and especially in open areas and as by increasing the height of the structure the intensity would increase in addition that the resultant of lateral loads would be increased as in Figure 1.8,

(v) Multi story or high rise building with very small width due to its length and lateral force is perpendicular to its width,

(vi) Type of support and rigidity (fixed, pin, spring), and

(vii) Ratio of completely horizontal crosses dimension to the height of the structure.

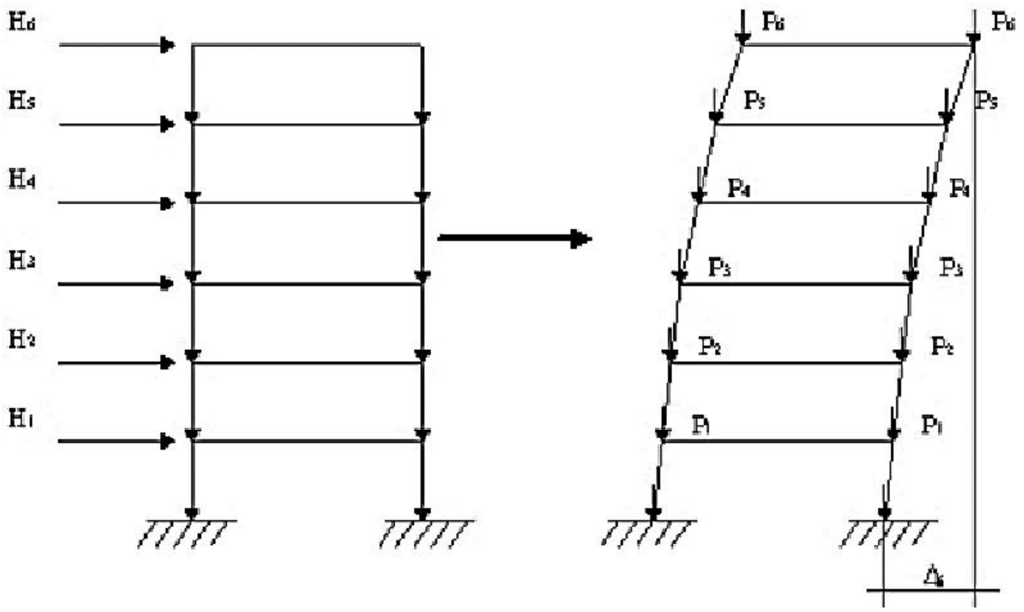


Figure 1.8 Distribution of gravity and lateral loads with P-Δ in multistory building (Moy, 1974)

1.5 Treatment of P-Δ effects in seismic design for multistory building

Several previous investigations of the effects of P-Δ have been pointed that the combination of large loads of gravity and lateral displacement, especially in the medium and high-rise buildings, can cause second-degree influence to become great. It has expressed the importance of influencing the level of P-Δ in terms such as stability indicators, drift indicators and the ratio of the base shear to the total mass. When it becomes a big P-Δ, it has to be considered significant increases in displacement, curvature ductility, plastic hinges and rotation deviation in order to maintain stability and service delivery structures (Tjondro, 1990).

A displacement seismic design (DSD) methods based on displacement, because of its simplicity and effectiveness, and have been more and more recognized in the seismic structural research communities during the past few years. Nevertheless, dynamic effect of P-Δ, and has long been well aware that a major cause of earthquakes in the structure that can amplify the seismic responses structures or even lead to instability structure, and is still not well resolved in practice because of the complex non-linear mechanism engineering. Therefore, the objective of achieving a practical solution for the general purpose that consider the effects of P-Δ in various modes of DSD systems and single-degree of freedom (SDOF), First, the current curriculum evaluation consider the effects of P-Δ in the current seismic analysis and design through the implementation of a wide range of non-linear time history analysis, and second, theme and design of the new recommendations on the effects of ignoring P-Δ threshold formulas and has promoted thresholds permitted design on the basis of statistical data. In the work, they explained the procedure proposed by the example in seismic design (Wei et al., 2011).

Asimakopoulsos et al. (2007) suggested action to address the impact of P-Δ in the list to direct the design seismic displacement of regular RC moment resisting frames with ideal behavior of flexible plastic materials. Derived a simple formula to yield displacement amplification factor as a function of the softness and the coefficient of stability because of the seismic response of the degree-of-freedom flexible one system taking into account the effect of P-Δ. Parametric seismic analysis and extensive inelastic steel structures from the moment the plane resistance is produced in a simple formula for the coefficient of dynamic stability as a function of a number of stories from the context of the column and beam stiffness ratio. Therefore, the effect of P-Δ can easily be taken into account in the design based on the direct displacement through seismic stability coefficient and the yield displacement amplification factor.

1.6 Objective of the study

The aim of this thesis was to carry out a numerical study on the behavior of the high-rise reinforced concrete (RC) structures having P-delta effects under seismic loading. For this purpose, five different RC buildings were analyzed by considering different numbers of stories (10, 15, 20, 25, and 30 stories). In the analysis, two cases were adopted. The first case was normal analyzing of the frames with ignoring P-delta effect, and the second case was analyzing of the same frames with considering P-delta effect, using the well-known commercial structural program SAP2000 v14 (CSI, 2011). The obtained results were evaluated and discussed comparatively for each frame type.

1.7 Outline of Thesis

This thesis is organized into five chapters with the following contents.

Chapter 1: Introduces a brief definition and shows the importance of P-delta effects, moreover, the objective of the thesis is provided.

Chapter 2: Contains a review of the relevant literature that covers previous studies conducted on high-rise buildings, analysis of the buildings under lateral loads, different structural forms of the buildings, and the impact of the P-delta analysis on the building.

Chapter 3: Deals with the description of studied building, method of analysis, and the characteristic of the ground motion used.

Chapter 4: Provides the results and discussion of the study.

Finally, **Chapter 5:** Summarizes the main conclusions achieved in this study.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

An efficient and economical high-rise building cannot be designed without a thorough understanding of the significant factors affecting the selection of the structural system and knowledge of how the structural system will interrelate with architectural, mechanical and electrical aspects. High-rise buildings structural system can be classified into four basic groups; rigid and semi-rigid frames, shear wall or braced frame structures, shear wall or truss-frame interactive structures, and tube structures. Tubular structures can be further categorized into frame tube systems and high efficiency tube systems. High efficiency tube systems evolved from the basic frame tube. Figure 2.1 shows a comparison of high-rise building systems versus the number of stories (Paulino, 2010).

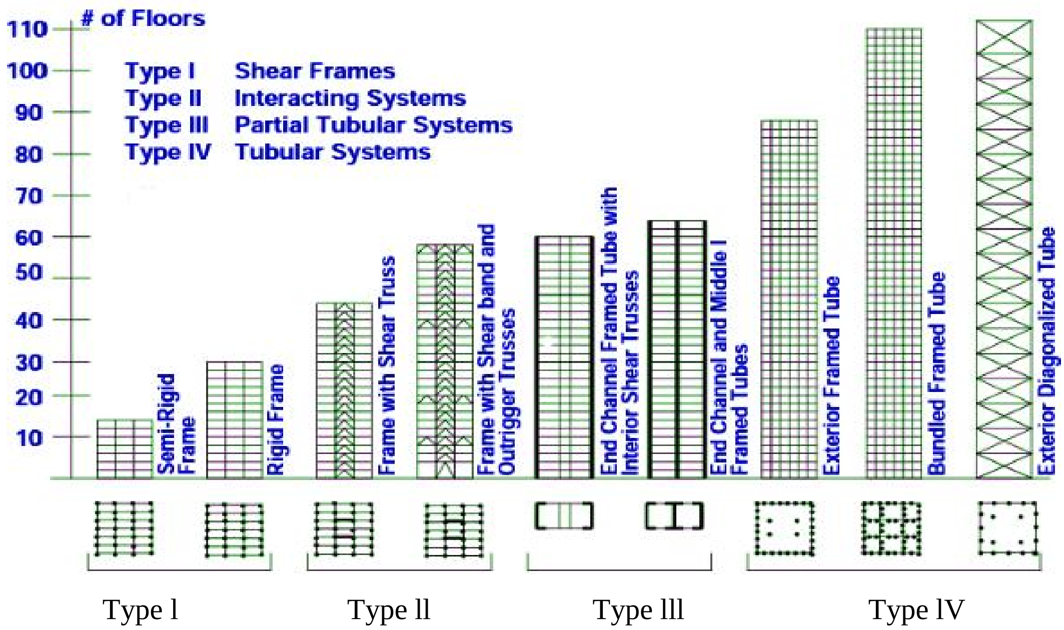


Figure 2.1 Comparisons of Structural Systems (Paulino, 2010)

In order to control building response to lateral loading, structural engineers may utilize efficient shapes and improve stiffness of the system and building weight. One of the most rapid and remarkable recent technologies is the use of the computer to analyze complex structural systems of tall buildings and produce construction documents. However, digital tools to assist in structural analysis to generate innovative tall building forms have not progressed at a comparable rate (Paulino, 2010).

2.2 High-rise building

Several papers and reports have been researched and published regarding the history of high-rise building in the technical literature. These studies have been summering below:

Ning (1998) searched an exact numerical technique for the analysis of tall reinforced concrete constructions in the usableness boundary state. This process did not consider the ultimate strength boundary states. It emphasized chiefly located in the appraisal of the lateral bending and valuation of the stiffness decrease of the structures due to the crack establishment within the members, which was the basic data required by the engineers.

In the study of Lee et al. (2002), an effective technique was suggested to analyze tall box system structures, taking the effects of floor slabs. The suggested technique would decrease the computational time and memory in the analysis by utilizing the sub structuring method and matrix condensing. Through the suggested technique, it would be more effective to enquire the seismic response of box system structures with getting into account the effect of the flexural stiffness of slabs.

Zhou (2004) studied the vibration-based seismic damage identification of high-rise building structures. His study focused on the data regarding the locations of the damage, and recognition of the rigorousness, as well as a general valuation of the post-earthquake damage.

Neidl-Comejo (2004) concluded that the strength of a building was not just getting from the material’s severity, it also got from the technique the building was organized and how its elements were set up, therefore the system made as an all.

Ali and Moon (2007) reviewed the development of high-rise building's structural systems and the technical driving force behind high-rise building evolutions. For the basic constructive systems, a modern categorization - inside structures and outside structures was presented.

Panagiotu (2008) studied the evolution of a modern displacement-based seismic design technique for applying within performance-based. The capability to design was applied to ensure the mechanism of inflexible deformation. Established in rules of plastic analysis and structural dynamics that modern formulation admitted the calculation of the effects of the system over strength and of the higher modes of response. Equivalent emphasis was applied to displacement, force, and acceleration demands parameters.

Hoogendoorn (2009) carried out an extensive structural analysis with regard to the along-wind response in the serviceableness boundary state. Also, he compared the lateral load resisting systems of the studied buildings. An arranged of comparing standards was drawn up, including the structural response, to find the attractiveness of each alternate from the financial point of view of an actual estate investor.

The investigation also executed to verify the most common structural systems that were used for the reinforced concrete high rise building under the action of gravity and wind loads. These regulations contained “Rigid Frame”, “Shear Wall/Central Core”, “Wall-Frame Interaction”, “Outrigger”, and “Tube in Tube” systems (El-Leithy et al., 2011).

Lee et al. (2012) presented an effective procedure for finding the optimum solution for construction of high-rise buildings. This procedure achieved optimum design solutions and decreased the bias induced by the rework that usually a raised in point based and set based design procedure. These proposed procedures and its applications for construction of high rise buildings was comprehensively examined.

Carpinteri et al. (2013) presented a study on the evaluation of global displacement and lateral load distribution of external effects on tall buildings. A proposed analytical method that evaluated the behavior of tall buildings retained types of bracings was reviewed with regarding to the Intesa Sanpaolo tower

2.3 Design codes of high-rise building

Every region in the world has a special design code, the code for every region is supported on the rules and conditions of that region. Therefore, there are many codes in the world; some of them are American code, Eurocode, and New Zealand Loadings Standard, Draft NZ/Australian Standard, etc. This section presents the previously studied that used different codes to design tall buildings.

In the study of Fenwick et al. (2000), series of reinforced concrete ductile moment resisting frame structures were designed according to the earthquake design codes. Aforementioned codes are draft version of New Zealand/Australian Loadings Standard, New Zealand Loadings Standard-NZS 4203-1992, Uniform Building Code-UBC1997, the International Building Code-IBC2000-1998, and Eurocode8-1998. Reinforced concrete ductile moment resisting frame structures were evaluated on high and low seismic areas. For higher seismic areas, New Zealand Loadings Standard-NZS 4203-1992 and draft version of New Zealand/Australian Loadings Standard gave low results compared the other codes. They also recommended to increase design strengths at draft version of New Zealand/Australian Loadings Standard.

Eduardo and Burgos (2006) presented a numerical study in which the displacement-based seismic design was introduced and evaluated with emphasis on the reinforced concrete shear wall buildings. The proposed method was intended to be applied to the reinforced concrete shear wall buildings with a regular plan configuration. Gravity loads were considered according to the provisions of the National Building Code of Canada 2005 (NBCC 2005).

Gabbai et al. (2008) deduced that the use of Minimum Design Loads for Buildings and Other Structures (ASCE 7-05) wind load factors for the design of high rise flexible building resulted in safety levels that could be significantly lower than safety levels typical of common, rigid structures. Wind load factors incorporated in the ASCE 7-05 standard was based on rough approximations of wind effects and the uncertainties inherent in them.

Panagiotou (2008) presented an experimental research program, with extensive shake table tests, of a full-scale 7-story reinforced concrete wall building slice, that was conducted at the University of California, San Diego. The base shear coefficient obtained by the proposed method was 50% of that required by the equivalent static method prescribed by the Minimum Design Loads for Buildings and Other Structures ASCE-7 code.

El-Leithy et al. (2011) aimed a comparative analysis to choose the structural system optimized for high particular building. The adequacy of the structure was measured by the volume of concrete of major components, structural period, and base shear values in this analysis, design considerations were made according to building code requirements for structural concrete (ACI 318-05) and (ASCE 7-05) standards.

Moreover, El-Leithy et al. (2011) analyzed the rigid frame system and they recommended that only 10-storey building of 35 m high had admissible wind drift. While, 20-storey structure of 70 m high and more, had a deflection more than the allowable limits, also a comparatively high lateral elasticity calls for economically large members. In addition, it was not possible to contain the required deepness of beams within the normal roof space in a high-rigid frame.

Tuna (2012) focused on examining the behavior, response, and modeling of shear walls, with the objective of improving our ability understand failure/collapse of reinforced concrete shear wall buildings under earthquake loading. The wall test database was used to assess the validity of the ACI 318- 11 (S21.11.9) equation used to compute the strength of the shear wall.

Kwon and Kareem (2013) examined the differences and similarities about wind loads and their influence on high-rise buildings according to international wind design codes. Aforementioned these codes are ASCE, AS/NZ, AIJ, CNS, NBCC, EU, ISO, IWC, and Nat Haz Aerodynamic Loads Database (NALD), which was a database, enabled method.

2.4 Structural forms in high-rise building

There are many structural forms that can be used in high-rise buildings, according to the needs and the condition of the structures. For example, Kyungha Park (2007) evolved bettered design lateral load models for the abstract design of moment-resisting frame structures. These design lateral load models were based on inflexible demeanor and were a basic element of a suggested seismic design methodology to bounds the extent of structural damage in the system and spread this damage uniformly along the tallness. These load models were exportable to supply a consistent distribution of floor ductility ratios. The regular distributions of floor drift ratios when likened to the distributions got with moment-resisting frames designed based on the code-compliant design lateral load patterns.

Modirzadeh et al. (2012) identified and used a reliable system input-output relation. To evaluate the performance criteria at untried design points (i.e., buildings with different modifier values) using a design of experiment technique. The proposed method of the performance-based evaluation was illustrated through consideration of the different structural deficiencies on a typical six-storey reinforced concrete building in Vancouver. Through the designed experiments, the main and interaction effects of the performance modifiers were also examined in their study.

Nollett and Smith (1998) proposed a new concept to define the lateral stiffness of wall-frame high-rise structures by rigidification a floor of the frame system either at the upper or at a mediate optimized stage. The shear inflexibility of the frame system was modified in a floor level by infilling one or many bays of the frames on that floor with concrete or masonry boards, or supplying bracing to the floor, or expanding the size of the columns and girders encircling the floor. The efficiency of the conception and the value of the parameters regarded in the demeanor of a stiffened-story structure were incontestable with the serve of a continuing pattern solution. It was displayed that in a few structures the lateral stiffness could be raised by as much as 70%. The method was then utilized to a model structure, which was examined with both the continuing pattern and a stiffness matrix solution.

Oztorun et al. (1998) presented a three-dimensional finite element computer analysis of high-rise building structures, created by perforating shear walls of open and/or unopened cross-sections and flat plates, the commercial software evolved for this function supplies a particular and effective mesh creation procedure. A graphic program was also improved to make the data interactively by using a screen graphic selection. The structural pattern could be produced or expanded absolute easy with the utilize of the shown mesh creation program. The beams or columns could be added or, deleted without any difficulties at all. The plate finite element evolved can exemplify the membrane as well as the deflection demeanor of shear wall and story elements.

These programs was improved to find the results to a few existent constructions and to find the limits of the simplifying suppositions generally created for the analysis of high-rise building structures. The program was also adequate of doing analysis by utilizing formal easy patterns of high-rise structures and of affirming the limits placed for the suppositions.

Kim et al. (2005) proposed an effective technique that might be applied to the analysis of a high-rise building construction with shear walls heedless of the number, size and position of openings in the wall. The proposed technique applied super components, substructures and assumed beams. Static and dynamic analyses of model structures with different cases of holes were made to affirm the effectiveness and precision of the suggested technique. It was affirmed that the suggested technique could supply results with great accuracy needing importantly decreased computing effort.

Kai-Huang (2009) developed a simplified model termed as a continuum multi-degree of freedom (MDOF) model for seismic analysis as well as for seismic evaluation of reinforced concrete wall-frame structures, which was one of the most popular structural forms of high-rise buildings.

Resatoglu et al. (2010) presented a static analysis of out of plane unsymmetrical-coupled shear walls, applying uninterrupted joining techniques in conjunction with Vlasov's theory of thin-walled beams. The technique of analysis exhibited was likened with commercial structural analysis software SAP 2000 by frame technique. The results displayed effective accord on affirming the validity of the suggested technique which could be effectively utilized in the preliminary calculations of high-rise buildings.

Boivin (2012) proposed for a new capacity design method by considering higher mode amplification effects for determining, the capacity design envelopes for flexural and shear strength design of regular ductile RC cantilever wall structures used as the Scottish Fire and Rescue Service (SFRS) for multistory buildings. The research concentrated on cantilever walls because higher mode amplification effects are usually much more important in cantilever walls than in coupled walls. In addition, the researcher studied the influence of various parameters on the higher mode amplification effects, and hence on the seismic force demand, in ductile cantilever walls.

Tuna (2012) concentrated on examining the demeanor, response, and modeling of shear walls with the aim of ameliorating ability realize failure/collapse of reinforced concrete shear wall constructions under seismic loading. He followed two studies. For example, the first work concentrated on seismic performance of an RC double system high-rise building (core wall and moment frames) designed following different design approaches whereas the other study concentrated on modeling and demeanor of a four-story RC building examined on the E-Defense. Two additional studies were carried to inquire and realize the failure of shear walls: one that focused on specifying shear strength and deformation capability of the constrictive walls by modernizing a comprehensive test database, and another that concentrated on potential causes of collapse of a 15-story shear wall building (Torre Alto Rio) in the Chile earthquake 2010.

Jiang (2013) submitted an analytical investigation on the demeanor and retrofit of aging medium-rise massive cast-in-place reinforced concrete shear walls under lateral loads. An adjust of paradigm and parametric walls was designed to exemplify building structures from the 1960s and first 1970s in areas of the US with great seismicity. Analytical patterns of the walls were made utilizing a micro-plane fiber component that could catch the axial–flexural–shear fundamental interaction in the nonlinear ambit. ASCE 41-06 was applied by considering these patterns to lead nonlinear lateral load analyses to measure the next three walls retrofit techniques advisable: (1) decrease of flexural strength; (2) increase of concrete confinement; and (3) raised shear strength. The solutions display that the reinforced concrete shear walls were probable to showing bounded lateral deformation capacitance without retrofit or with the utilize of an individual retrofit went up. A compounding of dissimilar retrofits techniques, perhaps, were required for meliorated demeanor.

2.5 High-rise buildings under wind loads

Gu and Quan (2004) tested fifteen-typical high-rise building models of basic cross-sections and aspect ratios from 4 to 9. A high-frequency technique used the balance of power in the wind tunnel for the first part on the dissemination of dynamic forces across the wind. The investigation into the implications of the case of the terrain, and aspect ratio and the aspect ratio of a modified cross-section and the corner of the building on the wind forces across models in detail. New formulas of the power spectra of the crosswind dynamic forces and the coefficients of base moment and shear force were derived.

Mingfeng (2008) developed a design optimization technique that automatically found out the most cost efficient design solution while satisfying all specified ultimate safety, serviceability and habitability design performance criteria formulated as deterministic and probabilistic design constraints. Time-variant reliability investigated of wind-excited building structures using extreme value statistical analysis. To identify and model the major uncertainties involved in wind loading conditions and structural systems for assessing the reliability of high rise buildings against wind-induced motion. In addition, the reliability, performance-based optimal design framework was developed to solve the design optimization problems of wind-sensitive, tall buildings subjected to both deterministic drift and probabilistic acceleration performance constraints.

Chan et al. (2010) presented an analysis of equal stable wind loads on high-rise buildings with 3D ways supplied that the wind tunnel calculated aerodynamic wind load spectra were applied. Then a merged wind load updating analysis and optimum stiffness design method was evolved from the lateral displacement design of high a regular constructions requiring twinned lateral torsional movements. The solutions of a virtual 40-stories building example, with important swaying and torsional effects were exhibited. Not just is the method capable to create the most cost effective component stiffness dispersion of the structure satisfactory multiplex unstableness wind drifts design standards, but an expected benefit of decrease the wind-induced loads could also be attained by the stiffness design optimization technique.

El-Leithy et al. (2011) performed a comparative study by concerning the efficiency of five structural systems and the capacity of any system in the reduction of wind drift to a certain rise of the building. Under the effect of wind loads, as the rise over structure and lateral deviation and the moment of the coup in the base increase. A major reduction in the wind drift in a high rise building was obtained by altering the structural form of the building to something more consistent and stable to limit deformation and increase stability.

Zhao (2011) presented aerodynamic optimization studies were conducted to reduce the correlation of vortex shedding along the building height, and thus to reduce the crosswind building response. The results showed that the optimum wind load could be reduced through effective in the formation of a certain building. Detailed wind tunnel studies including high Reynolds number tests and flexibility aero model tests were conducted, to accurately capture wind load on the building.

A framework for designing the performance-based component of wisdom was proposed by Spence and Gioffre (2012). The method was based on the concept of the fragility of the direction that combines the rigor aerodynamics building climate trends and information model. Then it was suggested an optimal design based on reliability, efficiency of the plan, on the basis of the separation of overlapping loops optimization traditionally reliability of the analysis carried out by the proposed framework design based on performance. It was optimized inseparable from the problem by identifying a series of approximations explicit sub-problems in terms of statistics response from the second division of the functions and constraint solution.

Li et al. (2013) investigated the effect of the wind loads of the high-rise building. The wind velocity factors in the tunnels for wind power generation were considered in the wind tunnel and the wind clime information analysis. Wind induced pressures, the total forces on the building pattern with a geometrical scale of 1:150, including the average and unsteady elements, were specified, and the wind velocity amplifications in the tunnels were calculated in the wind tunnel tests. Comparative analysis and discussions of the results for four examples were conducted. The wind velocity amplifications were measured in the tunnels for wind-power generation through the installing of wind turbines and to gain a better understanding of the wind effects on such a high rise building with open holes. The results showed that the wind tunnel test provided critical design parameters for the high-rise building.

2.6 High-rise buildings under seismic loads

Earthquakes are the main cause of structural damage and collapse in the world, which then results in huge economic losses and serious accidents. The rapid development of earthquake engineering has accumulated more and more experience on what structural failure mechanism is and how to make the structures have the best earthquake capability in order to mitigate and reduce the earthquake disaster, and even some of the difficulties still need to be resolved. It has been built and there is a large amount of new structures such as high-rise buildings and isolated structures, but should improve the earthquake resistance design, according to data monitoring and realistic for the purpose of strengthening the development of earthquake engineering (Wang et al. 2009).

Tarjan et al. (2004) presented an analysis of approximate earthquake to build multi-storey structures. The building is stiffened by an arbitrary combination of lateral load-resisting subsystems (shear walls, frames, trusses, coupled shear walls, cores). The analysis is based on the continuous method. The spatial vibration question of the replacement beam is solved approximately. Simple formulas are given to calculate the periods of vibration and internal forces of a building structure subjected to earthquakes.

Li et al. (2013) investigated the wind loads of the high-rise building and the wind velocity up factors in the tunnels for wind power generation founded on wind tunnel exams and wind clime information analysis. Wind induced pressures, the total forces on the building pattern with a geometrical scale of 1:150, including the average and unsteady elements, were specified, and the wind velocity amplifications in the tunnels were calculated in the wind tunnel tests. Comparative analysis and discussions of the results for four examples were carried. The wind velocity amplifications measured in the tunnels for wind-power generation through the installing of wind turbines and to gain a better understanding of the wind effects on such a high rise building with open holes. The results showed awaited to be of significant interest and virtual utilize to engineers and investigators involved in the design of high-rise buildings integrating wind turbines for ability generation.

Assessment of the seismic vulnerability of high-rise buildings in the Mid-America region using fragility analysis was presented by De-Leon (2010). Pushover analysis and nonlinear dynamic analysis were performed on a case study structure designed under the provisions of the current International Building Code (IBC, 2003), Standard Building Code (SBC, 1999), and IBC with local Shelby County amendments, to evaluate its seismic performance. A probabilistic demand model was constructed using simulated data from structural analysis to develop fragility curves for the case study structure. Sets of proposed fragility curves were developed using spectral acceleration as an intensity measured.

Dominguez et al. (2012) summarized the resultants of a survey consecrated to appraise. By utilizing nonlinear dynamic analyses, the seismic demeanor of six reinforced concrete moment resisting chevron braced framed buildings were studies. 2-D patterns that calculate for the fundamental interaction amid frames were applied for the nonlinear dynamic analyzes of the capacity-designed buildings utilizing the RUAUMOKO software package. A lot unreal reads comparable to the maximal believable earthquake affiliated to the design spectra were used to carry out the nonlinear dynamic analysis. From the outcomes found, they were ended that if content, design rules, and particular design parameters for the modern design of RC-MRCBFs were utilized, proper international ductility capabilities and over strength requirements were found, and an acceptable structural functioning was attained.

Epackachi et al. (2012) analyzed the linear and nonlinear demeanor of one of the highest RC constructions, a 56-stories structure, placed in a high seismic area in Iran. In this tower, shear wall systems with asymmetric openings were used under both gravity and lateral loads and might result in a few particular events in the demeanor of structural components specified shear walls and coupling beams. The analytical methodological analysis and the resultants found in the valuation of life-safety and collapse prevention of the building were also talked about. The frail area of the structure established on the resultants was presented, and an elaborated talked about of a few significant structural views of the multi-stories shear wall system considerately of the concrete time dependence and constructional succession effects were also admitted. Esmaili et al. (2008) also studied the structural aspects of one of the tallest RC buildings, located in the high seismic zone, with 56 stories.

Lu and Jiang (2011) investigated some research achievements on the response of the high-rise buildings under wind and earthquakes in mainland China. Through the integration of the design of the seismic performance based law in the current seismic design, it became much more possible for designers to control the levels of damage intentionally structures within the acceptable range during earthquakes varying degrees. In mainland China, the shaking table tests were used extensively scalable model for evaluating the overall performance of complex seismic high-rise buildings, and consequently reconsider the structural design to meet performance targets. Structures could be protected from earthquakes and wind with the help of structural control technologies. There was no steady progress in the field of research and development of technologies for structural control in mainland China. They combined their work in general with the application of engineering and can turn out to meet the engineering practice of actual needs. Most of the results of the research were successfully applied in engineering practice.

2.7 Static and dynamic seismic analysis of high-rise building

Dewobroto (2011) studied the effect of structural modeling on the analysis of P-delta effects. According to AISC (2010), P-delta effect would be consisted of brace framed component (P-δ) which was affected locally in element structure, and sway framed components (P-Δ) which were effective globally in structure. For that reason, it would be evaluated several case models to be analysis by a commercial second order analysis program SAP 2000 and seen the effects of adding the number of nodal and place to influence the accuracy results. The results of this study would be useful as a reference to find an appropriate structural model in considering the influence of P-delta effects, particularly on the steel structures. In his study, the second order analysis with SAP 2000 version 7.4 and 14 computer programs were used to predict the critical buckling load. The buckling load on a structure occurred with a certain pattern or model. In the previous case simulation, a symmetrical load pattern that opposed to the pattern of the buckling was used. Therefore, it was required to consider the structure with load patterns similar to the pattern of their buckling. The difference only to the load W orientation as explained in Figure 2.2. If then it was associated with the condition of how to build the element stiffness matrix in SAP 2000, namely Element Frame EF, which is based on the DOF at the tip nodal points only, it was predicted that the way of placement element in modeling the structure will influence to the result of P-delta effects. As shown in Figure 2.3, for the load patterns of W (Case-1), that were contrary to the pattern of buckling, the effect was positive, the forces results almost constant in each axial load condition, even near the critical buckling load. These conditions would be different, when the pattern of a given load W (Case-2 and Case-3) producesd deformation similar to modes of buckling, apparently due to P-delta affect yield to the magnification of internal force in a very significant, especially if the axial force approaching critical buckling load. Although the load of W almost equal in value. The phenomenon was almost similar to the effect of resonance. For all cases, the critical buckling load did not changed.

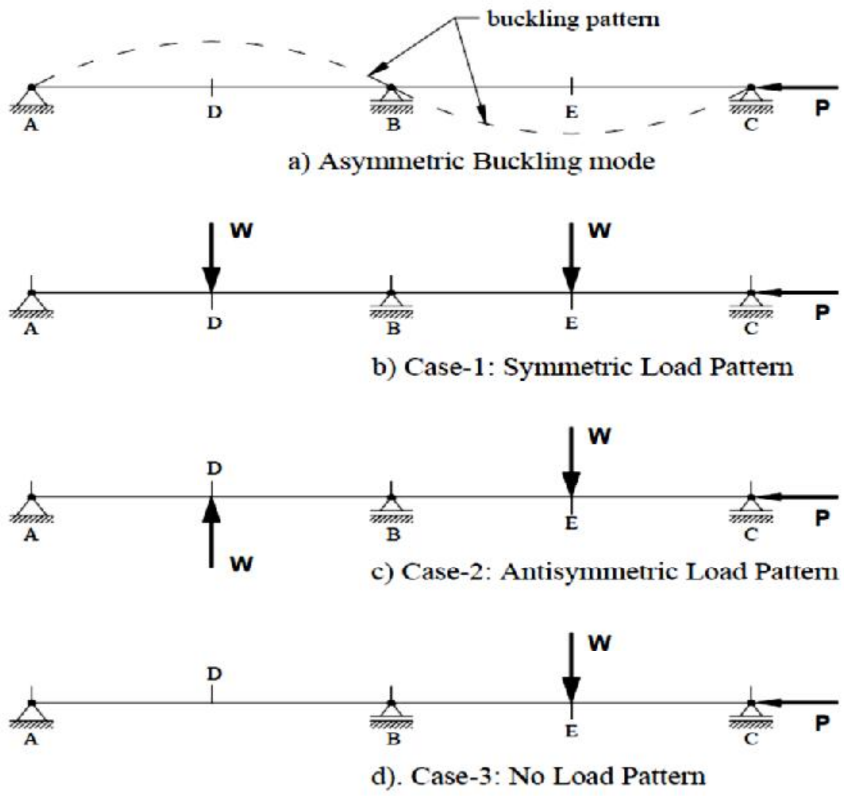


Figure 2.2 Load and buckling pattern for three study cases by Dewobroto (2011)

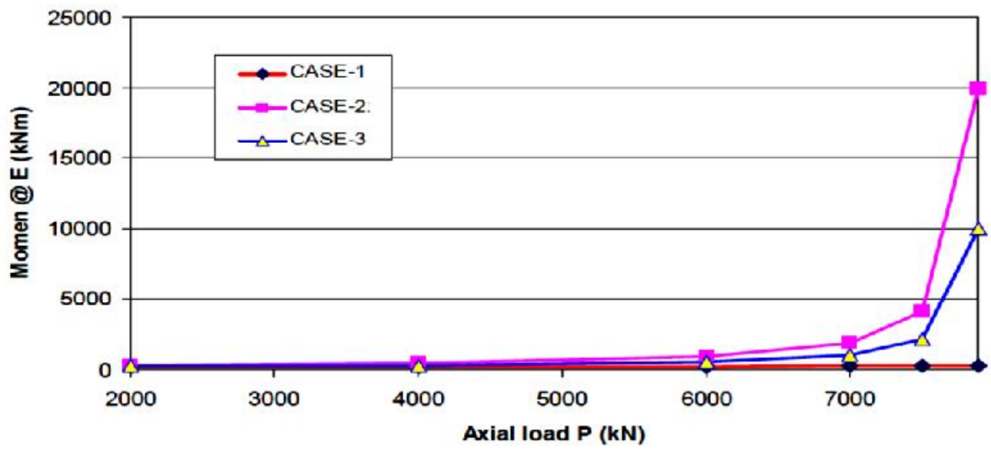


Figure 2.3 The relation of axial load and bending moment at point E (Dewobroto, 2011)

Patil et al. (2013) studied equivalent static analysis of high-rise building with various conditions of lateral stiffness system. Some models are prepared that was pure frame structure, brace frame, shear wall frame, and they found that the axial forces are decreased from bare frame models to shear wall models. Shear wall models attracts minimum axial forces. Reduction of axial forces is due to a provision of the lateral resisting system and Comparing the top storey drift in the longitudinal direction. The models with shear wall located on exterior frame of X & Z direction throughout height was found most effective in resisting lateral loads because it shows least deflection as compared with another model. A significant amount of increase in the lateral stiffness was observed in all models of brace frame and shear wall frame as compared to bare frame.

Hassaballa et al. (2013) studied seismic analysis of multi-storey RC frame in the town of Khartoum under a mild earthquake load as an application of the seismic hazard analysis. The frame analysis using the response of spectrum mode for the calculated displacement of the seismic and pressure. Horizontal movement had the greatest impact on the pivotal compression loads external columns as compared with the interior pressure and pressure columns on ground floor columns were approximately 1.2 to 2 times the tensile stresses. He was found in the values of shear forces to be approximately four times the values because of the L / C1. The maximum values of the pressure and tensile stresses in the beam nearly equal. Bending moments in the beams and columns due to seismic excitation showed a significantly greater value when compared with that in view of constant loads, and they concluded that the interior columns in all floor levels were hardest hit by the pressure forces resulting from all cases of load combinations, And bending moments in the beams and columns due to excitation seismic showed a much larger values than it was due to loads of static also pressure the pressure generated from all cases of loads on the ground floor columns were greater than the tensile stress in those columns whereas at other levels was a slight difference. Pressure on the ground floor columns and the pressure were about 1.2 to 2 times the tensile stresses. The pressure and tensile stresses in nearly equal studied the packets and the frame is not sufficient to resist the seismic load applied.

Li and Chen. (2009) studied how commercial software could be used to produce much realistic simulations of dynamic response of structures under earthquake loading. By examining the amplification of the earthquake wave in the free field and near structures, they also illustrated the effect of soil-structure interaction in the dynamic analysis. They found that if the ground acceleration at ground level could reach more than 0.2g, the seismic design should be carried out separately and could not be simply replaced by wind load design. Unfortunately, out ideal building was not sophisticated and representative enough for us to make further recommendation about the design value based on the time history analysis. Nevertheless, the analysis does highlight the amount of amplification that could be expected in rather typical geological settings.

Gupta and Joshi (2001) made a random method response spectrum to calculate the seismic response of multi-degree of freedom (MDOF) structures, based on stochastic approach through the consideration of the implications of each of the random nature of the ground motion and uncertainty in accurate identification the structural properties (storey masses, storey stiffness, and modal damping ratios). For the consideration of the random nature of the structural response, described by equations different response function and the spectral energy density, defined in terms of media characteristics (Medium) that supposed to be known precisely, and fixed formula of power spectral density function excitement earthquake input. To examine the impact of uncertainty in the structural characteristics, approximate expression of the first class has been developed for the variations of the root mean square from where the contrast of different model parameters for the structure of the response capacity. The variation in the definition of model parameters in terms of the parameters corresponding to the conditional mean the structural characteristics of median, the specific values of the differences of the masses and the story stiffness. It was found on the numerical results calculated and wide ranges of uncertainty in the structural characteristics and several different widely thrills inputs to be in very good agreement with the values of Monte Carlo simulation. The quantities of the different response in general, significantly affected by the uncertainties that might normally be present in determining the structural characteristics. The proposed formula provides a simple and accurate way to reasonably assess the efficiency and impact of uncertainty in the structural characteristics of the response. Moreover, this can be considered as equivalent to traditional methods of spectrum overlay because it was based PSDF compatible response spectrum analysis and conditional structure.

Fan et al. (2009) provided a detailed examination of the dynamic properties of the seismic replies of Taipei 101 longest a skyscraper in the world. The constitutive relationships for rectangular CFT columns were established based on the unified theory, and then were verified through comparison between the shaking table test data and numerical analysis results. In addition, create a 3-D finite element model of the structure of Taipei 101 on the basis of constituent relationships were verified columns CFT rectangular selecting and types of specific elements of the structural members. The seismic analysis results of the high-rise building indicated that the structural system, with belt trusses at every eighth or tenth story, provides equal stiffness along the height of the building, which can decrease the lateral deformation efficiency. The same time, such a system structural massive frame with a basic set of Central columns prepared on the perimeter of the building contact upon all the face of the building, it is converted total of dead and live loads on each floor to the external columns sloping, and thus the ability to withstand the structural side pregnancy strengthens. The results have also shown that Taipei 101 has a comparatively high resistance to earthquakes and could ensure structural safety under the guise of seismic with a moderate seismic immunize, as provided for in domestic law seismic design. Nevertheless, it was shown that there were sudden changes in shear strength in the columns near the floors with outrigger belts. It should address this issue sufficiently in high-rise structures earthquake resistant design of this type.

Wen et al. (2002) studied the shear wall structure 21-floors, built in the 1960s in Hong Kong, and an example to discuss these two effects: (that the damage in structures caused by earthquakes is highly dependent on the state of the site and epicentral distance). High-rise buildings are more likely to get a situation of damage to the site ductility, and the damage is much more severe earthquakes field of earthquakes near the field. The intensity of the quake, which occurred in the eighth, and the possibility of a complete breakdown (P) more than 1-24% of the earthquakes near the field, and 1-41% from a lot of earthquakes field if the building was moved from site to site A80 rocks consists of soft thick mud. The intensity of the ninth, pin apartments 6-69% earthquake near field, and 14-79% of earthquake-term field if the building was moved back from the site of the rock site soft soil. Therefore, the influence of the site is very important and not be neglected. This was the first time that these effects are included in the DPM follow the methodology and analytical approach, instead of using "adaptive experts approach" more personality and more specifically. The seismic forces computed in each floor by using the base shear method. In addition, it includes the effects of the natural state the location and the period of construction automatically by using the place or the specific source of the earthquake, which depends on the design response spectrum. Then use the maximum average ductility modified to form a logarithmic distribution of factor ductility. Through the integration of these distributions, or DPM and the fragility of the building under types and income, levels of various seismic curves were obtained. The results showed that the damage to the high-rise buildings was the most severe of the earthquake field was far from an earthquake near field because of the richness of the contents of the low-frequency. Therefore, high-rise buildings in soft soil are subject to earthquakes remote domain is the damage more appropriate than the rocks on the site and subject to earthquakes near field. This was consistent with the phenomenon of the field infield, usually of a high degree of selectivity-damage. Expect to the same result as can be extracted even when we use the other high-rise building and follow a little different analytical method. At the end, it should be all of the effects of status and distance epicentral analytical included in the DPM account or fragility curves. The location and distance epicentral important implications and not be neglected.

Wilkinson et al. (2006) provided material non-linear model plane framework is able to analyzing high-rise buildings that exposure to strong earthquake. On each floor represents a model for the building of the assembly of the elements of the vertical and horizontal beam. Description displacement through translation (the effect) of each floor and turn over all intersections beam columns. Moreover, associated block only with translations, and therefore the analysis may be implemented as condensation constant rotation, alongside with the integration of the dynamic equations to be translated. It was here carried out the vital integration used the (Runge-Kutta) planner. This approach allows the building to be modeled by degrees of freedom m (n + 2) (where m is the number of storeys and n is the number of bays). Rank matrix stiffness shortcut is only m. Construction, which requires a reverse rotation, rank m (n +1), and hardening of the matrix, only steps required in-time changed the pattern of return of previous time step. This model is particularly attractive to the non-linear response analysis of the history of high-rise buildings as it effectively allows each floor and have multiple repetitions, and each connection to be modeled with a moment's turnover appropriate relationship.

Moreover, Wilkinson et al. (2006) compared to the results of the task analysis easy with a fixed date, time, and results from a simplified model present three examples of verification. Results verify that the model is able to perform non-linear analysis of history in response to the regular high-rise buildings. The model has been shown to be able to analyze the simplest structures to within less than 1% of the results that have been obtained by finite element analysis. It gives the model accurately modes higher than the vibration and thus can be used to consider the impact of these on the collapse of the buildings. Using a simple push on the analysis of structures with a conditional second large mass and accelerated the collapse of the sequence may produce an error, so the collapse of the pregnancy is not true. To accurately determine the collapse loads of structures, P–δ effects need to be considered as does the true moment rotation relationship of the connections especially if stiffness degradation over successive cycles.

Moehle (2006) used the non-linear dynamic analysis as an instrument to verify the seismic performance of significant structures. The software instruments available, and research results, and experience gained through the building real applications and provide a basis for the effective application for non-linear analysis procedures. Significant the considerations definition of performance targets, the selection of input ground movements, and model building non-linear analysis was appropriate, and the interpretation of the results. Carried out correctly, the non-linear dynamic analysis particular structural earthquake system and the environment was the best way to learn about the non-linear dynamic response characteristics, including yielding mechanisms, internal forces associated with them, the demands of the deformation, which describes the requirements. Could be determine the dimensions and details superior to those obtained using the mandatory requirements of the law before the construction of such an analysis, leading to increased confidence in the construction of performance characteristics including safety. Care must be exercised to specify, and implement, reinforcement details that will perform as intended. Peer review remains an essential part of performance-based design of high-rise buildings.

2.8 P-delta effect on high-rise building

P-delta effect is defined as “The secondary effect on shears and moments of structural members due to the action of the vertical loads induced by horizontal displacement of the structure resulting from various loading conditions” according to ASCE 7 (2005). P-delta effect is the second-order effect of gravity loading (Smith and Coull, 1991) and is essentially a stability problem (Tarnath, 1998). It has been termed the P-delta effect because the additional overturning moment on the building are equal to the sum of story weights (P) times the lateral displacements (Delta) (Wilson, 2002). For most building structures, the P-delta effect occurs in the columns because of gravity load. Columns in buildings are subjected to simultaneous bending caused by lateral loads, and axial compression due to gravity loads. Hence, they are in effect (beam-column) (Tarnath, 1998). The column axial forces being compressive make the structure more flexible against lateral loads. Gravity load considered for P-delta effect consists of dead load plus a fraction of live load. The concept of P-delta effect could be illustrated by a simple cantilever subjected to vertical load, Pg due to gravity and horizontal load, F due to earthquake or wind as in Figure 2.4.

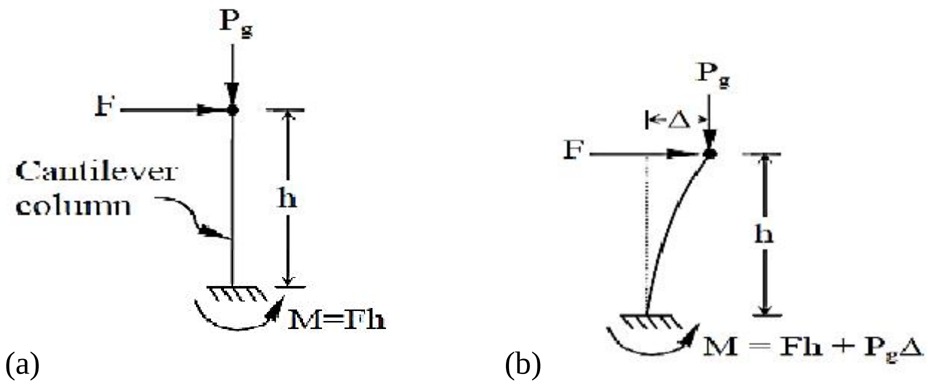


Figure 2.4 Illustration of P-delta effect (a) unreformed configuration and (b) deformed configuration (Tarnath, 1998)

When the building was subjected to horizontal forces, the resulting horizontal displacements lead to additional overturning moments because the gravity load Pg is also displaced. Therefore, in addition to the overturning moments produced by lateral force F, the secondary moment Pg. Δ must also be resisted. This moment increment in turn produces additional lateral displacement, and hence Δ increases further.

Of the many methods available, three methods were widely used to account for the P-delta effect in buildings. These were moment magnifier method, iterative P-delta analysis method and Non-iterative P-delta analysis method (Tarnath, 1998).

In the traditional moment magnifier method, moments obtained from first-order elastic analysis were multiplied by the moment magnifier that was the function of factored axial force and the critical buckling load for the column. The problem was usually classified into sway frame and non-sway frame. The frame was considered non-sway if the increase in the lateral moments resulting from P-delta effects does not exceed 5 percent of the first-order moments (ACI 318, 2005). The details of this method were given in ACI 318 (2005).

In the Iterative P-delta analysis method, an initial first-order analysis of the structure was made with the external horizontal loading. The resulting horizontal deflections were then used in conjunction with the gravity loading to compute at each floor level an equivalent increment of horizontal load. This increment was added to the initial horizontal load and the analysis was repeated. The resulting increased deflections were then used in conjunction with the gravity loads to compute another set of equivalent horizontal increments, which again are added to the initial horizontal load for a reanalysis. The iterations were continued until increases in the deflections become negligible (Smith and Coull, 1991).

The third method was the Non-iterative P-delta analysis method, in which the second-order effects were accounted for by directly modifying first-order stiffness matrix so that, when analyzed for the actual horizontal loading, the resulting values of drift and member forces include the P-delta effects. The matrix equation for this case was given by (Smith and Coull, 1991):

$$(H) = (K - KG)(\Delta^*) \tag{2.1}$$

In which (H) = vector of the actual horizontal loading, K = first order stiffness matrix, KG = geometric stiffness matrix and was the function of the gravity loading, and (Δ*) was vector of the total lateral displacements, which includes P-delta effects. The details of the formulation of the last two methods were given elsewhere (Smith and Coull, 1991; Wilson, 2002).

However, Smith and Coull (1991) were discouraged the use of traditional moment magnifier method, especially in the case of heavy gravity loading or of a flexible structure, because of the deterioration of accuracy.

Moreover, the inclusion of P-delta effect in the analysis stage eliminates the need to determine the column effective length factors, since the P-delta effects automatically produce the required design moment amplifications (Wilson, 2002).

Normally, the maximum moment in columns occurs at the ends. However, in a very slender column or columns bent in single curvature, maximum moment may occur between its ends (ACI 318, 2005), this was the local P-delta effect.

The decision as to whether to include P-delta effect in the analysis was provided by the stability ratio, θ given by (ASCE 7, 2005):

$$\theta = (P_x \,.\, \Delta)/(V_x h_{sx}\, C_d) \qquad (2.2)$$

Where, P_x= total vertical design load at and above level x with load factor ≤ 1.0, Δ is design story drift occurring simultaneously with V_x, V_x is seismic shear force acting between level x and x-1, h_{sx} is story height below level x, C_d is deflection amplification factor, whose value ranges from 1.25 to 6 depending on the efficiency of the seismic force resisting system. The higher value of $C_d$refers of more ductile systems. ASCE 7 (2005) states that if the stability ratio, θ computed from Eq. (2.3) is less than 0.10, inclusion of P-delta effects may be waived. The standard (ASCE 7, 2005) also imposes the maximum limit on stability coefficient, θ as:

$$\theta_{max} = \frac{0.5}{\beta\, C_d} \leq 0.25 \qquad (2.3)$$

Where β = ratio of shear demand to shear capacity for the story between levels x and x-1. β is permitted to be conservatively taken as 1.0. Where the stability coefficient, θ is greater than 0.10 but less than or equal to θ max, the incremental factor related to P-delta effects of displacement and the member forces are to be determined by rational analysis. Where θ was greater than θ_{max}, the structure was potentially unstable and should be redesigned. FEMA450 (2003) encourages the inclusion of P-delta effect in the analysis as it effectively checks the stability of a structure based on its initial stiffness. Because high-rise building design results in larger computer analysis models as compared to low-rise building design, the most important thing to keep in mind was fundamental behavior and to provide “sanity checks” along the way that ensure analytical modeling is accurately depicting the real structural behavior (Zils and Viise, 2003).

Noise and Maison (1983) presented a matrix formulation based on the concept of geometric stiffness, for calculating P-Δ impact computer in seismic analysis of multi-storey buildings. The method was based on a linear solution that does not require repetition approach, and could be used for static or dynamic elastic analysis. It was proposed that the deviation factor amplification, Cd, was expected to inflate the deviations and overturning the moments resulting from the effects of P-Δ in static or dynamic elastic levels of displacement analysis. In order to compare the effect of different amounts of Cd values on analytical response, the impact was included in the P-Δ analysis using Cd factors equal to 0 (P- Δ effect ignored), l (P-Δ forces based on elastic limit deflection levels), and 5.5 (P-Δ forces based on extreme inelastic deflection levels). And it could therefore be taken into consideration impact dwell on P-Δ arising from the levels of displacement is flexible, which may occur during a major earthquake to be rough.

The technique was implemented in a revised version of the computer programmer of ETABS and applied in seismic analysis model construction of 31-storey steel. The analyses showed that the story drift, shear and overturning moment responses at al1 levels of a building for any value of Cd are increased if including the P-Δ effects in static analyses. However, the story drift, shear and overturning moment responses at a given story of the building may be increased or decreased in elastic dynamic analyses. So, including the P-Δ effects in elastic dynamic analyses may not necessarily lead to a more conservative design throughout the building than if P-Δ effects are ignored. P-Δ affects the corresponding magnification levels elastic displacement (Cd 1 = 0) and may be seen as a conservative design for the purposes of consideration of the greater of displacement inelastic levels that occur through powerful earthquake (Noise and Maison ,1983).

Montgomery (1981) presented a study of the impact of the effects of P-Δ to respond buildings that were exposed to earthquake base motions using time-history analysis.

Studied the impact of P-Δ effects on the responses of (1, 5, and 10-storey) shear buildings. Analyses were conducted using the method of Newmark Beta with $\beta = 1/6$. Used five earthquake ground motions, which covered a wide range of severity, site conditions, and periods of strong suggestions. The results indicated that the effects of P-Δ have only a small effect on the response of buildings or respond elastically flexible way that is a little bit of ground seismic motions. However, P-Δ effects should be taken into account in a manner to respond is extremely flexible systems. This study indicated that the P-Δ effects often large buildings where the ratio between the weight and the base shear, WN, greater than or equal to (10), or the maximum rate floors drift relative to the production of the yield relative storey drift is greater than (2). The stability factor approach for estimating the influence of P-Δ effects, as later adopted by NBCC 1995, was described by Montgomery (1980). When the response, elastic and non-elastic slightly, the method gives reasonable results. However, Montgomery indicates that the method should not be used for systems to respond in a non-elastic force. When inelastic behavior becomes significant, the transient displacement response was quite different from that of simplified NBCC pseudo-static provisions leading to different values for stability factors. Priestly et al. (1992) a great deal of research on how to take the effects of P-Δ into account was done. The non-mandatory commentary to NBCC 2005 endorses a method based on the stability approach recommended to account for P-Δ effects. Analysis was performed to assess the impact of the effects of P-Δ on the walls of steel plate shear. As the proposed methods and stability in the recommended conditions of the national program of seismic design to reduce the risk of earthquakes by NEHRP (2000) and the NBCC 2005 commentary was evaluated.

Rathbone (1987) explained second-order effects that need them. Structural engineer takes the issue one step forward to a more general aspect of what the effects of P-Δ were important in the context of the design of high rise building.

Davidson et al. (1991) presented a study that acceptable value should be made for P-Δ effects at the seismic of multi-story structures. These are additional overturning moments applied to the structure resulting from the seismic weight. P support of the structure, working through the side deviations (Δ) that leads directly from the horizontal inertia forces of earthquakes. They were the effects of second degree which increases the displacement. Members procedures and prolong the effective structure of the basic period. For design purposes in most cases, seismic induced P-Δ action could be neglected in structures remain elastic throughout the earthquake.

Thomson et al. (1991) analyzed a series of five different frames of six, twelve, eighteen and twenty-four stories to study P-Δ effects on ductile reinforced concrete structures under strong seismic excitations. All structures were subjected to three digitized ground motion records, in all analyses, the duration of the excitation was taken as 15 S. A time step of 0.01 s is used for the majority of analyses. The analyses were carried out using the computer program RUAUMOKO (Carr, 1996). This performs inelastic time-history analyses of two-dimensional structures. Plastic hinges are assumed to form at the ends of the members. The columns allow for an interaction between the axial force and yield moments. Rigid end-blocks are allowed for in both the columns and the beams, and the inelastic action used a Giberson one component hysteresis model (Sharpe, 1974). Figure 2.5 shows the Giberson one component hysteresis model, which has a possible plastic hinge at one or both ends of the elastic central length of the member.

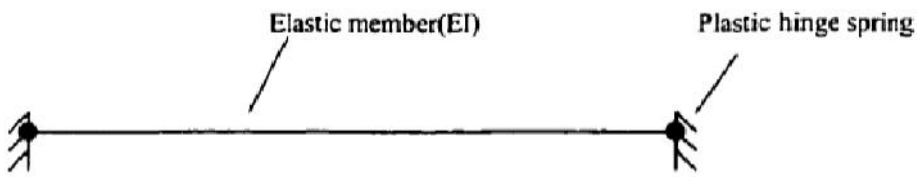


Figure 2.5 Giberson one component hysteresis model (Sharpe, 1974)

The evaluation of P-Δ effects included the calculation of the amplification of both the maximum beam curvature ductility demand, as well as the average curvature ductility amplification over the height of the building. The following conclusions were obtained: P-Δ effects could cause a significant increase in plastic deformations of frames designed to perform in a ductile manner when subjected to strong ground motions. If P-Δ effects are estimated to be excessive, it was considered more practical and effective to strengthen a structure than to stiffen it (Sharpe, 1974).

Fenwick et al. (1992) reviewed the results of inelastic time history analyses done on SDOF structures to assess P-delta effects induced in earthquakes. The conclusions from the analyses with different earthquake ground motions are: (i) P-Δ effects increase with the duration of intense ground shaking; (ii) P-Δ effects are negligible in elastically responding structures; and (iii) reducing the equivalent viscous damping increases the P-Δ effects. Analyses showed that changing the hysteretic response to allow for stiffness degradation and changing the strain hardening ratio have only a small influence on the P-Δ response of ductile SDOF structures. Based on the analyses of SDOF structures under earthquake ground motions with duration of severe shaking in the 15 to 25 second range, a method was proposed to assess the strength increase necessary to prevent the ductility demand to increase when P-Δ effects are included. The required strength increase was determined in terms of the P-Δ amplification factor, B, and a factor K_t, which allows for the influence of the fundamental period and the soi1 type. A series of multi-story walls and frames were analyzed to obtain P-Δ amplification factors. The corresponding equivalent SDOF values were also calculated using the respective wall and frame properties. From these analyses, it was concluded that the method of assessing the strength increase required to counter the additional effect of P-Δ in SDOF structures can be successfully applied to multi-story structures. A set of design steps for calculating the strength increase necessary to counter P-Δ effects in structures were outlined. The distribution of strength increase within the structure was determined by a pin jointed truss mode1 with a deflected profile derived from either the equivalent static approach or from the response spectrum method. This strength distribution was then scaled by the appropriate SDOF B factor, modified by the factor Kt. It was shown that shear wall structures were relatively insensitive to P-Δ effects. The P-Δ amplification factors for walls was only 7.1% for a 24-story building.

Mollick (1997) reported in the P-delta effect through the test on three types of RC frame structure models in one-fourth scale, which represented the lower part of the high rise building subject to seismic force. The selected three test structures were labeled by EF-1, EF-2, and EF-5, the cross sectional properties and overall geometry of columns and beams were shown in Figure 2.6, from the test results and the scope of this study revealed that the P-delta effect should be included in the analysis for the design of high-rise building if the story drift exceed 1/85 rad. The test results also revealed during an expected earthquake excitation in seismic regain that a rigorous analysis should be carried out rather than to use the conventional equations for the prediction of member strength whenever such a high-rise building was to be designed as in Figure 2.7 shown the modeling test structures.

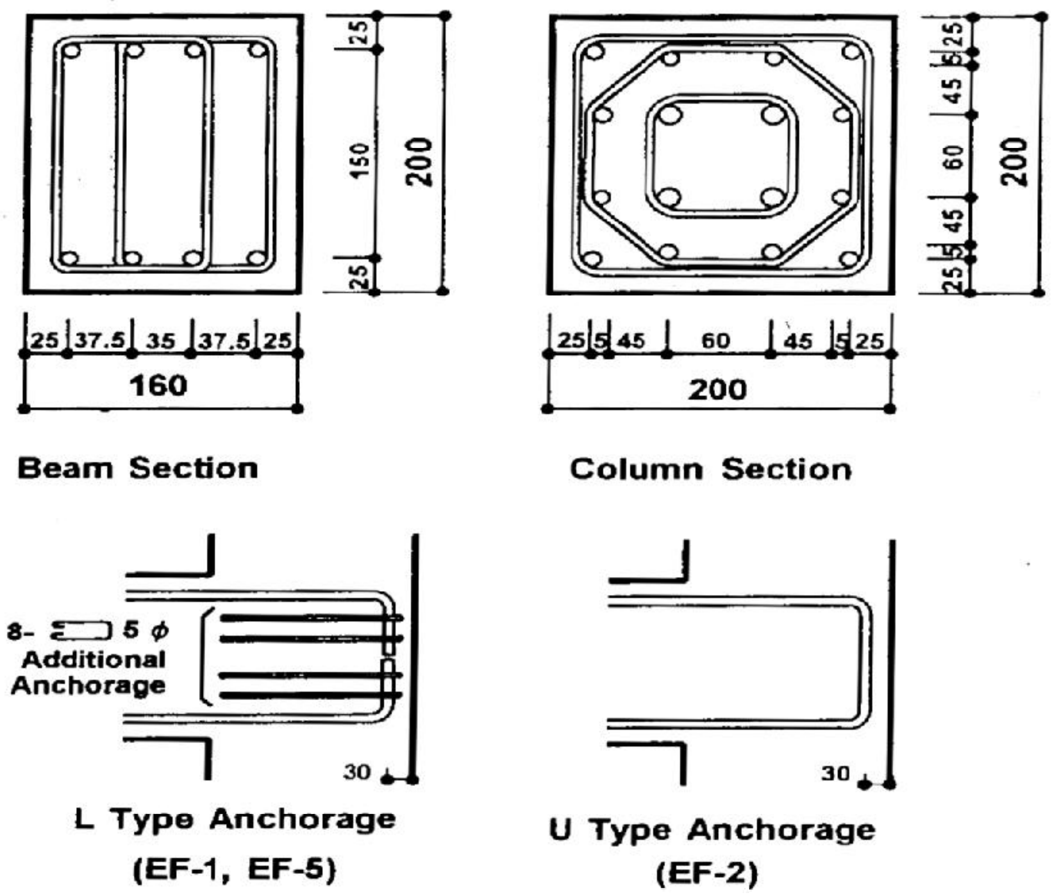


Figure 2.6 Cross section properties of beams, columns and joint adopted (Teaoka et al., 1993)

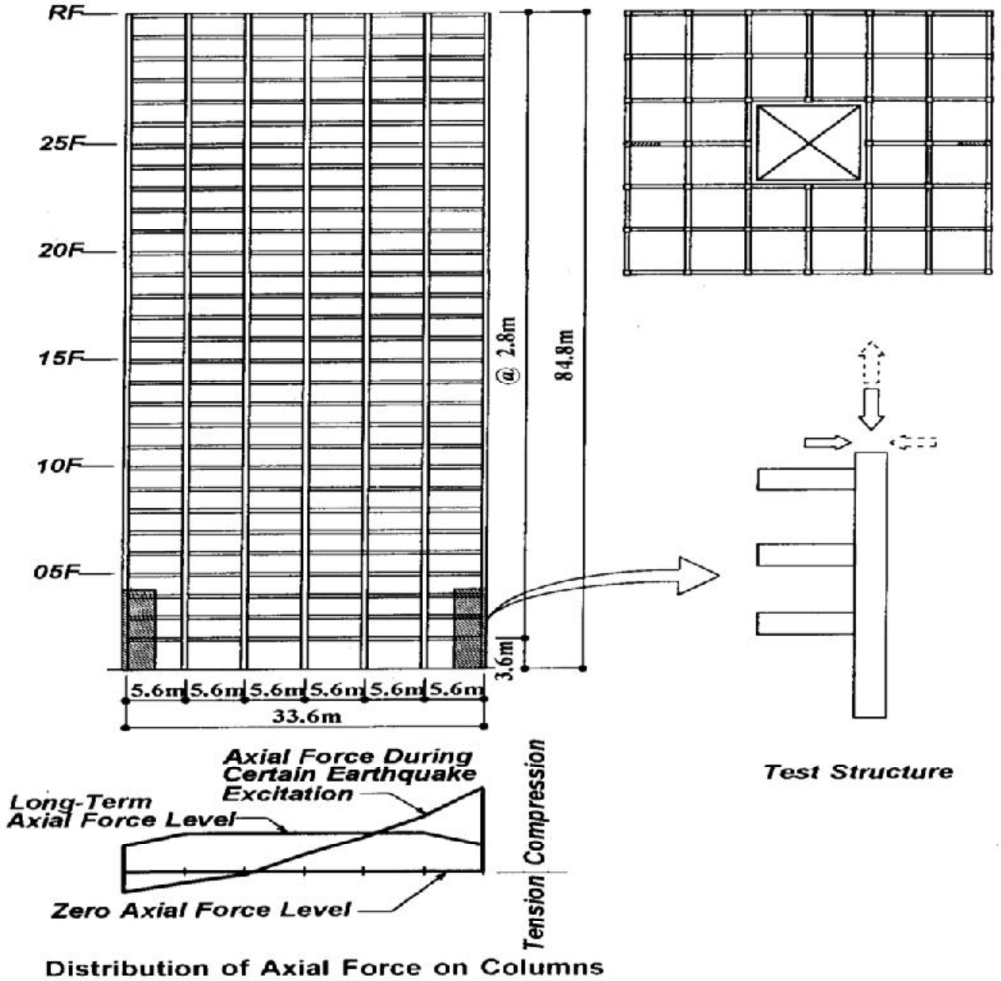


Figure 2.7 Modeling of test structures of a high-rise building (Mollick, 1997)

Cote (1997) studied the use of strength amplification factors to mitigate the P-Δ effects in multi-story reinforced concrete wall structures (1, 3, 5, 10, 15, 25-storey). These wall models included one node per floor, and each node had three degrees-of-freedom corresponding to the horizontal, vertical and rotational displacements, respectively. The mass was assumed to be lumped at the floor levels, and the gravity loads acting at a given floor were lumped at the corresponding node. The bi-linear degrading stiffness hysteresis rule was considered for the inelastic analysis several methods were applied to determine the strength amplification factors for mitigating the P-Δ effects. With the RUAUMOKO computer program, inelastic dynamic analyses with and without P-Δ effects were performed, and the displacement ductility demand was examined. The following results have been obtained: (i) the ductility demand was slightly higher for the bi-linear behavior with stiffness degradation compared to the bi-linear behavior without stiffness degradation, (ii) P-Δ effects had little influence on the displacement ductility demand for the different walls, (iii) Due to P-Δ effects, the increase in the ductility demand under high frequency Eastern North America type of earthquakes is less than that of low frequency Western North America type of earthquakes, and (iv) The value of the ductility demand obtained without P-Δ effects decrease with the number of storeys because the ductility concept used in the thesis is based on the displacement at the top of the structure.

However, Cote (1997) used a simple dynamic model of the walls chat were not designed and detailed as part of a complete building system. The inelastic behavior was only modeled on the base of the walls. To further study P-Δ effects in multi-storey reinforced concrete wall structures, a more realistic wall model, being part of a complete building system, are studied in this thesis. Inelastic hysteresis rules are used to model each storey of the wall. The effects of variation in wall modeling parameters are studied further.

Chen and Wang (1999) when the structure is exposed slender lateral loads such as wind and seismic loads, it is subject to horizontal displacement or influence. When this significant lateral displacement be a reasonable and loads of gravity begin to act with deviation equal to the size of deflection flexibility causing extra moment coup. That is because of the pressure on further than that the structure of the development of second-degree deflection. This is called second-degree influence witnessed message P- Delta effect. If P is the load of gravity, Δ1 is the displacement observed during the first class or the analysis of the flexibility on the side of the forces (F_{wind} or F_{eq}) and h is the height of the story, and the product (P · Δ1) is the moment experienced a coup in addition to (F · h). The impact of the P-delta in Figure 2.8, where Δ2 is the deflection of advanced second-degree view of the effect of P-delta.

It is experienced in all P- delta structure when subjected to axial loads in combination with horizontal displacement. Two different processes observe this secondary effect. The big impact because of deflection structure as a whole (the framework of instability) also called on behalf of P - "BIG" delta (P-Δ) and contributed to the rest of the distortions of the members of the structure (a member of instability), also described as the P "small" delta (P- δ) (Chen and Wang 1999). However, this study was limited to the effect of P Delta illustrated by the lack of structural stability (P-Δ). P-delta magnitude of the effects depends on the size of the axial load (P), stiffness / slenderness of individual elements and the structure as a whole. Therefore, the structures and buildings with the largest number of tall stories usually experience of higher P-delta the effect of than others, and must be designed with adequate consideration for that. P-delta importance of non-linear analysis is constantly increasing, as a new generation of high-rise buildings is getting more and more popular.

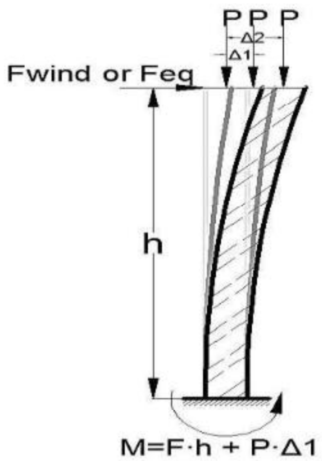


Figure 2.8 P-delta effects on a simple cantilever column (Chen and Wang 1999)

Brown (2002) was explained with great clarity the intended design process for assessing medium sized, orthodox multi-story frames for sway sensitivity in accordance with BS 5950-1:2000.

Dobson (2002) major rules of the current design methods on a linear elastic, or the first linear, approach. This design methods do not consider the development of extra internal forces and displacements due to the effect of P-Δ Chen and complaining research on the seismic response of structures is flexible by Montgomery, Gupta and Tremblay has shown that P-Δ effects are significant on flexible structures and amplify the lateral displacements.

Moghadam and Aziminejad (2004) presented a study on the effects of the P-delta inflexible and non-elastic bands and then evaluated as well as the number of floors and the contribution of lateral load, the degree of contrast and diversity, sensitivity to the characteristics of the ground movement and used four 7, 14, 20 and 30 story according to a model of design procedures and then on the elastic and inelastic fixed and Dynamic behavior with and without looking in P-delta effects are investigated. Each building is 0%, 10%, 20% and 30% of the deviation levels and they concluded that this type of lateral load System resistance plays an important role in the degree of torsion that modify the effects of P-delta. It was also showed that despite the constant analysis and flexible torsion and amplify the effects of P-delta always, but the same is not always true dynamic analyzes. Dynamic analysis of a high level they appear the sensitivity of the movement of the earth results as properties.

Sullivan et al. (2008) studied also on P-delta effects on high-rise RC frame-wall buildings, the case study was the design of a 45-storey reinforced concrete frame-wall structure that was used to highlight the significance of the P-delta limit within the modal response spectrum analysis procedure of the Eurocode 8. It was found that the strength of the structure was dictated by the P-delta limit for seismic actions, despite anticipated storey drifts and ductility demands had relatively low. In addition, a series of non-linear time-history analyses used a suite of the spectrum-compatible real and artificial accelerograms, from the results indicated that P-delta effects did not have a significant influence on displacements or storey drifts of the tall building. In their study also focused on the critical design requirements of the Eurocode 8 identified during the design would be reported and the global response of the structure assessed through non-linear time-history analyses would be presented. The founds prompt a review of the basis of current P-delta limits in codes and considered whether P-delta limits were appropriate for high-rise buildings.

Pettinga and Priestley (2008) presented a method for explicitly including P-Δ in the design process as part of the development of Direct Displacement-Based Design. The researchers were discussed the differences in sensitivity to P-Δ of both Elastic-Plastic (approximating steel response) and stiffness degrading (reinforced concrete) hysteresis, from which a proposed multiplicative factor was derived to account for the enhanced performance of reinforced concrete structures. Which in their study, the researchers designed a 4-storey frame for both reinforced concrete and steel response was tested using non-linear response history analysis with a suite of seven spectrums-compatible (massaged) real records. From the results, it was found that the proposed accounting for P-Δ satisfactorily reduced the storey drift amplifications, such that the design performance targets were maintained at the original level when second-order, effects were not included in the analyses. Design recommendations were also presented based on simplifications to the derived equation through the assumption of reasonable values for typical new structures.

Black (2011) in a first-order analysis, equilibrium and kinematic relationships are based on the unreformed geometry of the structure. Solutions of these analyses are typically simple and straight forward. However, when lateral loads are applied to the structure, it is often assumed a configuration, which deviates quite noticeably from its unreformed configuration, the standard elastic design procedures can prove inadequate if the additional destabilizing moments are not taken into account.

Bellari et al. (2014) studied the P-delta effects on displacement based assessment of RC hinged frames. A parametric study was carried out to investigate the effectiveness of equivalent viscous damping (EVD) accounting for P-Δ effects. Their study was based on the evaluation of the dynamic response of a series of equivalent nonlinear single degree of freedom SDOF systems changing the post-yield stiffness ratio, while keeping constant the displacement ductility μ_{Δ} and the effective period T_{eff}. According to the parametric analysis results, new formulations are proposed in order to include P-Δ effects in the common displacement based assessment procedure. The researchers considered a nonlinear behavior of the SDOF system of Figure 2.9 that characterized by the development of a flexural plastic hinge at the base, it was observed that the system rotation/displacement associated with a selected limit state was the same with or without P-Δ effects. In addition, the researchers obtained also that the moment-rotation relationship was not affected by P-Δ, only the force-displacement loops change in shape due to P-Δ but maintain the same hysteretic energy. For a given lateral deflection (Δ_u), the equilibrium of the system is reached with a lower base shear as shown in Figure 2.10 b. From their study results obtained P-Δ effects, reduced the lateral load associated with a selected deflected shape in the substitute structure, capacity curve leading to a decrease of the effective stiffness and consequently to an increase of effective period. In addition the available formulations of equivalent viscous damping (EVD) do not account for negative.

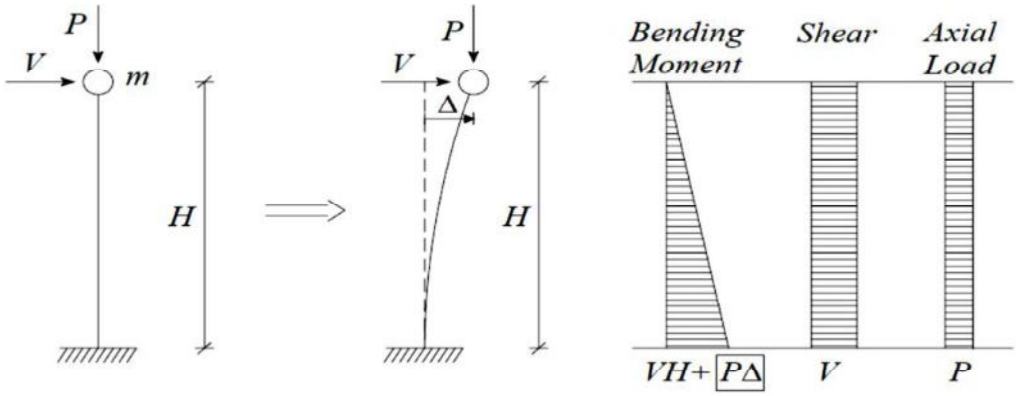
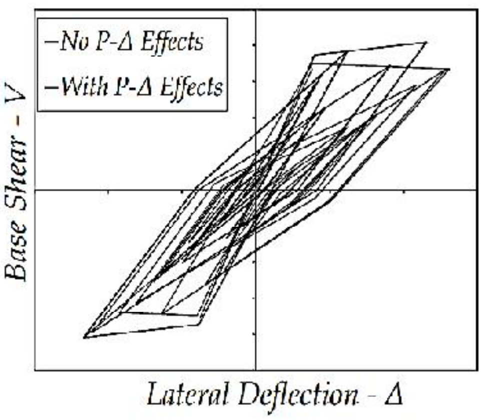
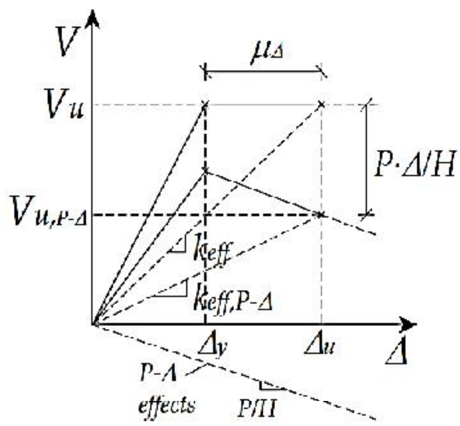


Figure 2.9 Increase of bending moment demand due to P-Δ effects (Bellari et al., 2014)





(a) (b)

Figure 2.10 a) Lateral displacement increase due to P-Δ effects, and b) V-Δ curve modification due to P-Δ effects (Bellari et al., 2014)

Konapure and Dhanshetti (2015) studied the effect of P-delta action on multi-storey buildings, The P-delta analysis was recommended by several design codes such as ACI-318, LRFD, etc. Instead of the moment magnification approach to calculate more pragmatic forces and moments. Seismic analysis of a multi-storey RC building was analyzed by using STAAD structural analysis software. The building models with different storey had analyzed to investigate the maximum response in buildings in terms of displacements, storey drifts, column moment, beam moment, column shear and beam shear. The building model had analyzed from 5 to 27 storeys with 2-storey interval as shown in Figure 2.11. The maximum response in building models occurring at a certain height of floor levels had been studied. The lateral load for the selected frame had carried out as per IS-1893 (Part-I) 2002. The analysis had carried out for the case without P-delta effect to locate the maximum responses, and then it had analyzed for P-Δ (structure deformation) effect with a number of iterations. Again, analysis for P-Δ-δ (with structure and member deformation) effect with a number of iterations for same model had been carried out as shown in Figure 2.12. The maximum response values were compared to notify the P-delta effected. As iterative P-delta method had used numbers of iterations had carried out for each building model with P-Δ and P-Δ-δ until convergence occurs. It was found that convergence of results occurred for a third iteration hence comparison had done for third iteration only.

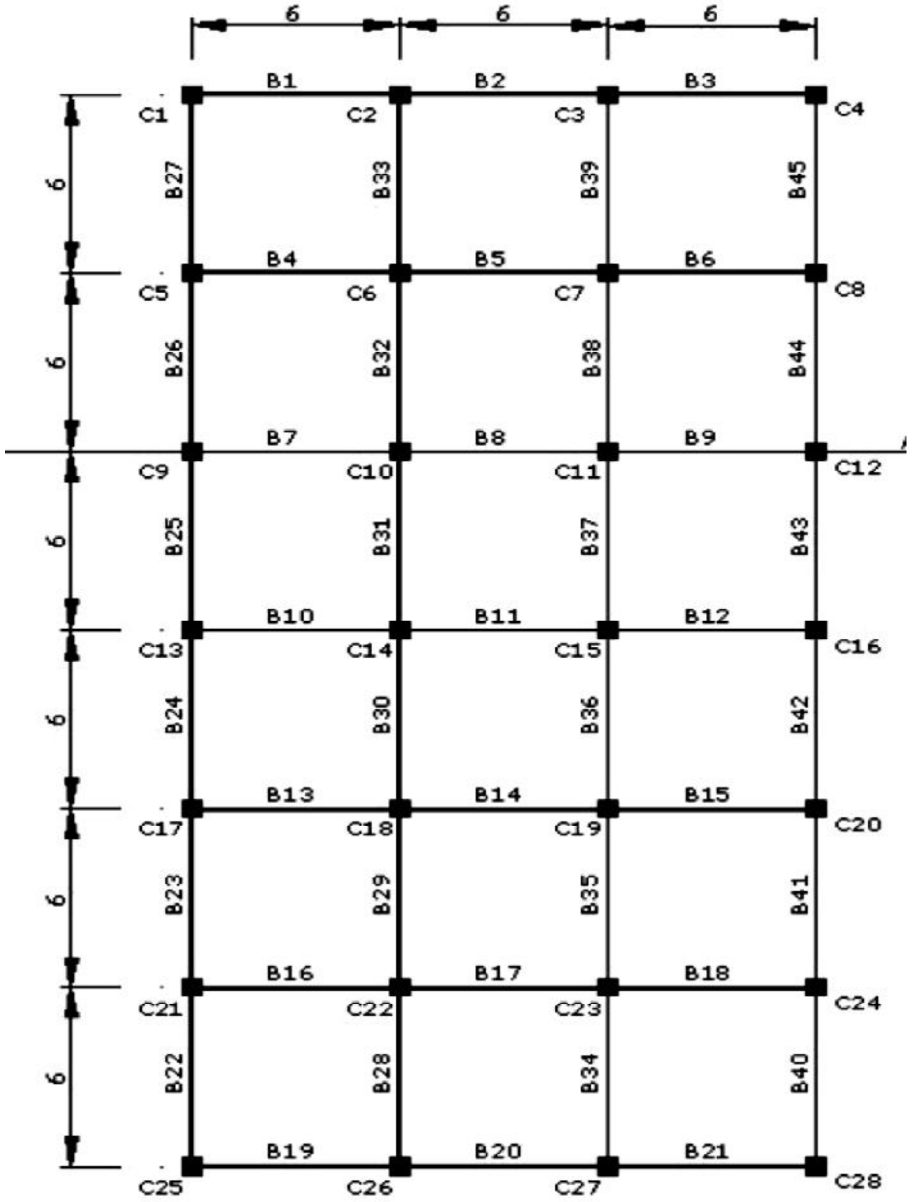
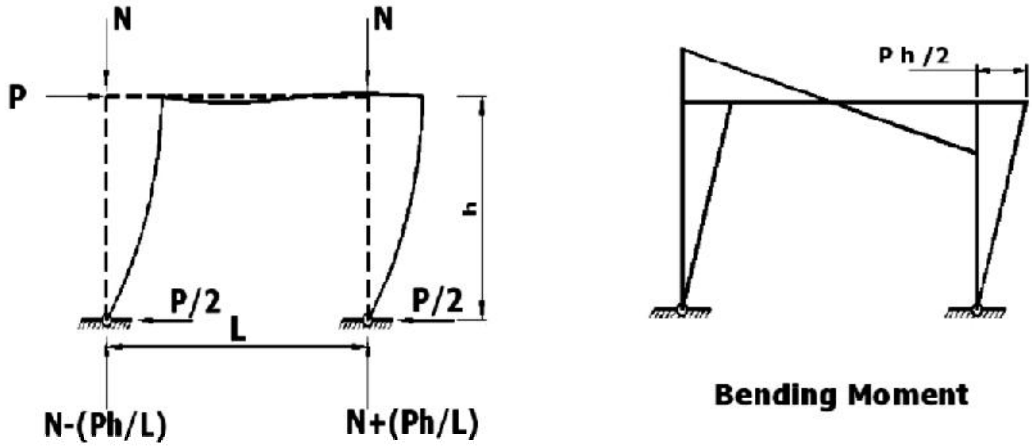
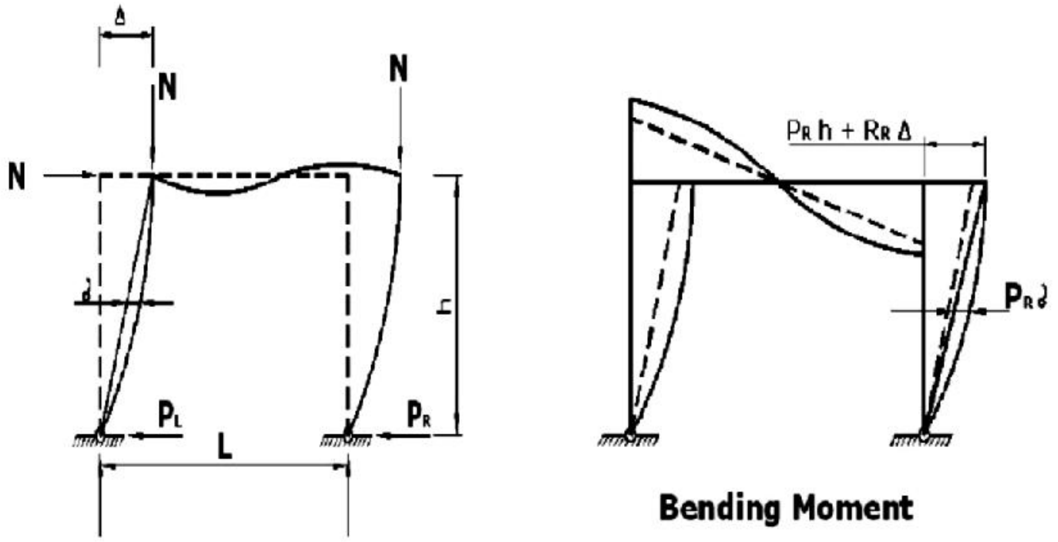


Figure 2.11 Study frame plan by Konapure and Dhanshetti (2015)



(a) First order analysis (b)



(c) Second order analysis (d)

Figure 2.12 Simple portal frame (Konapure and Dhanshetti, 2015)

CHAPTER 3

METHODOLOGY

3.1 General

The design of high-rise buildings essentially involves a conceptual design, approximate analysis, preliminary design and optimization in order to carry the vertical service load and lateral loads safely. The design parameters are the strength, serviceability, stability and serviceability. The strength is satisfied by limit stresses, while serviceability is satisfied by drift limits (Jayachandran et al., 2009).

3.2 Description of case study reinforced concrete structures

In this study, five types of reinforced concrete (RC) multistory structures of 10, 15, 20, 25, 30 stories had been analyzed. SAP2000 v14 (CSI, 2011), which is a general purpose structural analysis programme, was used to conduct analytical models of multistory structures. All buildings have the same plan of 20m×20m; consisting 4 bays in each X and Y-directions. The height of each story was 3.0m identical through the height of the structure. The frame with respect to two perpendicular axes was almost symmetrical in plan, thus two dimensional analysis of the building was conducted.

The material properties were assumed as shown in Table 3.1. For beams, a typical section of 0.5×0.3 m was used for all models. Also for slabs one thickness of 0.15m was used for all models. As gravity loading additional dead load of 1.5 kN/m^2 and live load of 2.0 kN/m^2 were taken as uniformly distributed loads on the slabs. The columns have varying cross-sections as shown in Table 3.2. The column foundations were considered as fixed in all cases. Typical floor and elevation view of the case study (RC) buildings are given in Figures 3.1 and 3.2.

All materials have nonlinear behavior, sideway deformation is permitted, all sections have lumped nonlinear behavior.

Table 3.1 The material properties that used in this study for all models

Material properties*	Concrete	Steel
Modulus of Elasticity Es (Mpa)	25818	205000
Unit Weight (kN/m^3)	23.56	76.82
f_y (MPa)	-	420
fc' (MPa)	30	-
• These properties used for all materials in five types of frames		

Table 3.2 Detail of column sections for five types of frames corresponding to storeys

Number of Stories	Column cross section with respect to storeys			
10-storey	1-3 storeys	4-10 storeys	-	-
	80×80 (cm)	70×70 (cm)	-	-
15-storey	1-3 storeys	4-7 storeys	8-15 storeys	-
	80×80 (cm)	70×70(cm)	60×60(cm)	-
20-storey	1-3 storeys	4-7 storeys	8-13 storeys	14-20 storeys
	80×80(cm)	70×70 (cm)	60×60 (cm)	50×50 (cm)
25-storey	1-3 storeys	4-7 storeys	8-13 storeys	14-25 storeys
	80×80 (cm)	70×70(cm)	60×60 (cm)	50×50 (cm)
30-storey	1-3 storeys	4-7 storeys	8-20 storeys	21-30 storeys
	80×80(cm)	70×70 (cm)	60×60 (cm)	50×50 (cm)

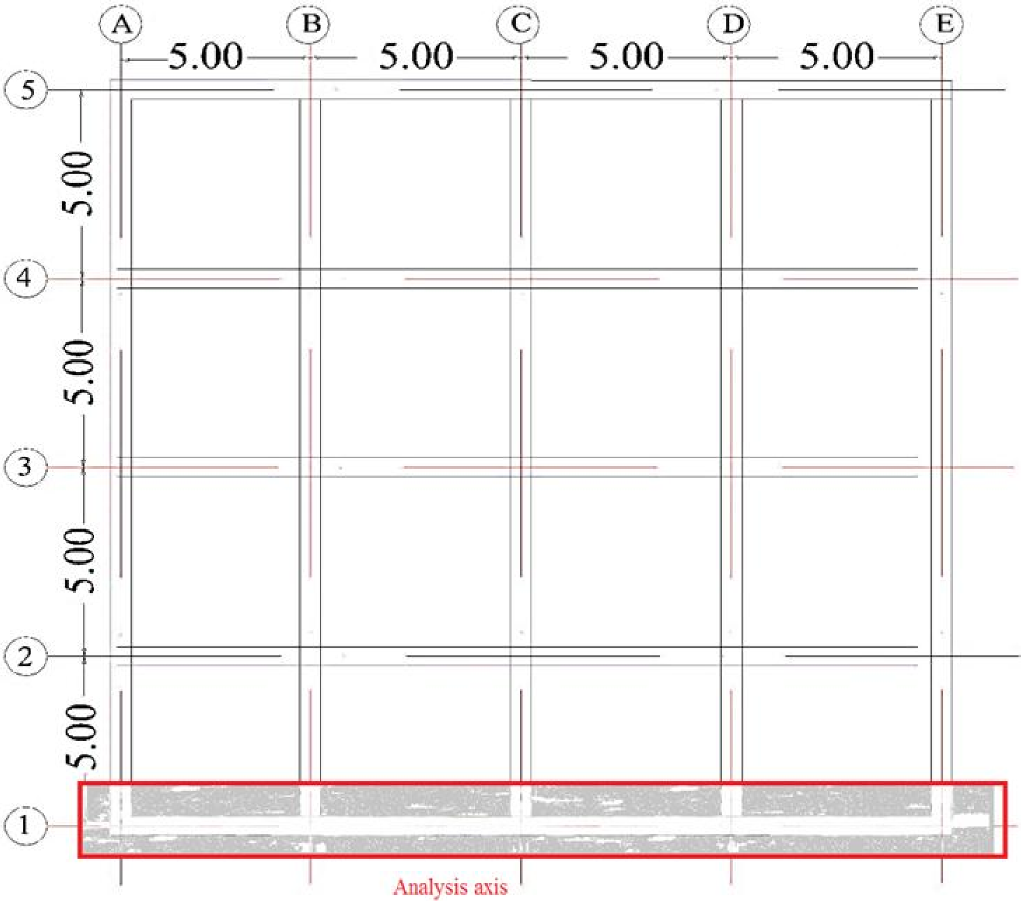
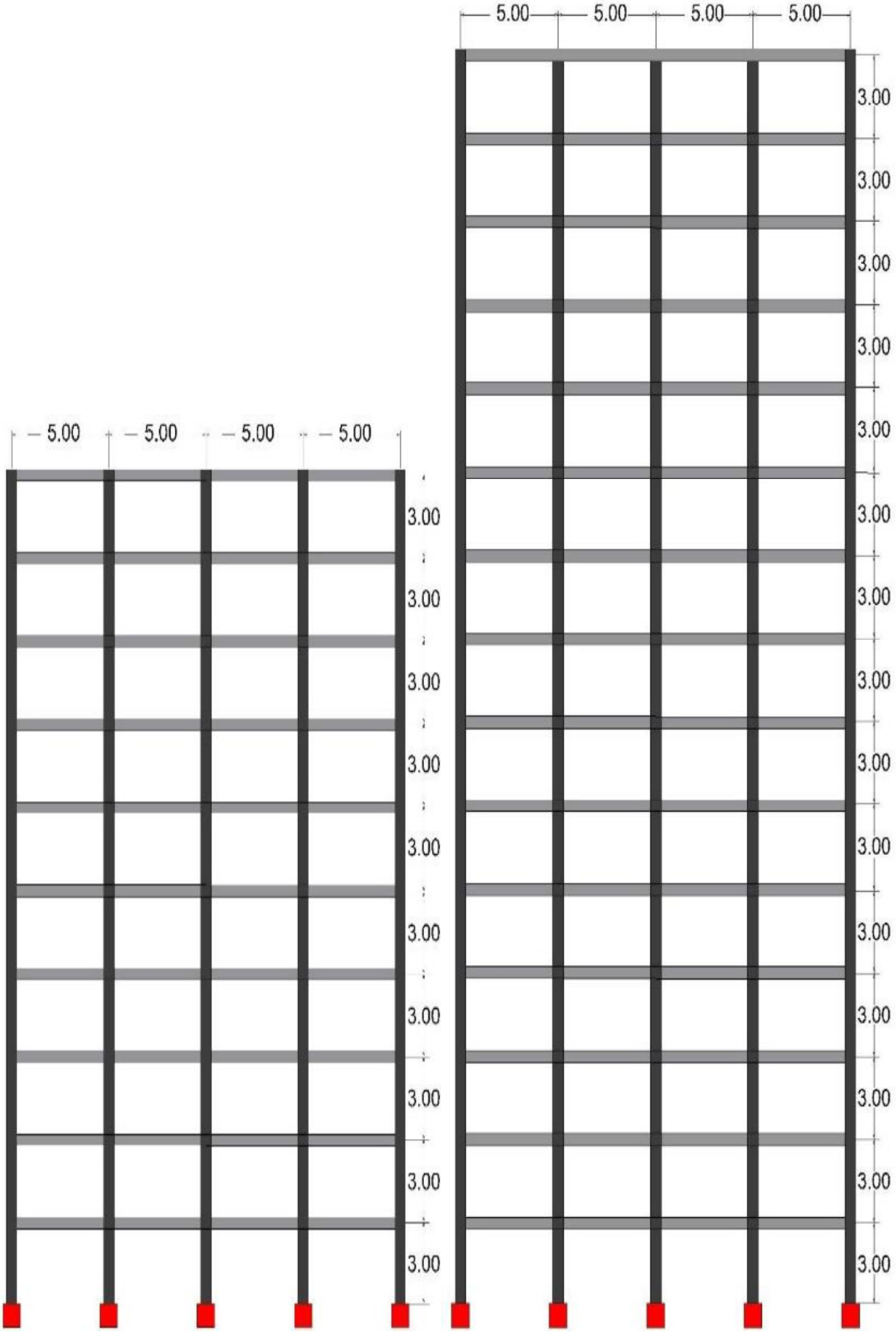


Figure 3.1 Sample plan view of the reinforced concrete buildings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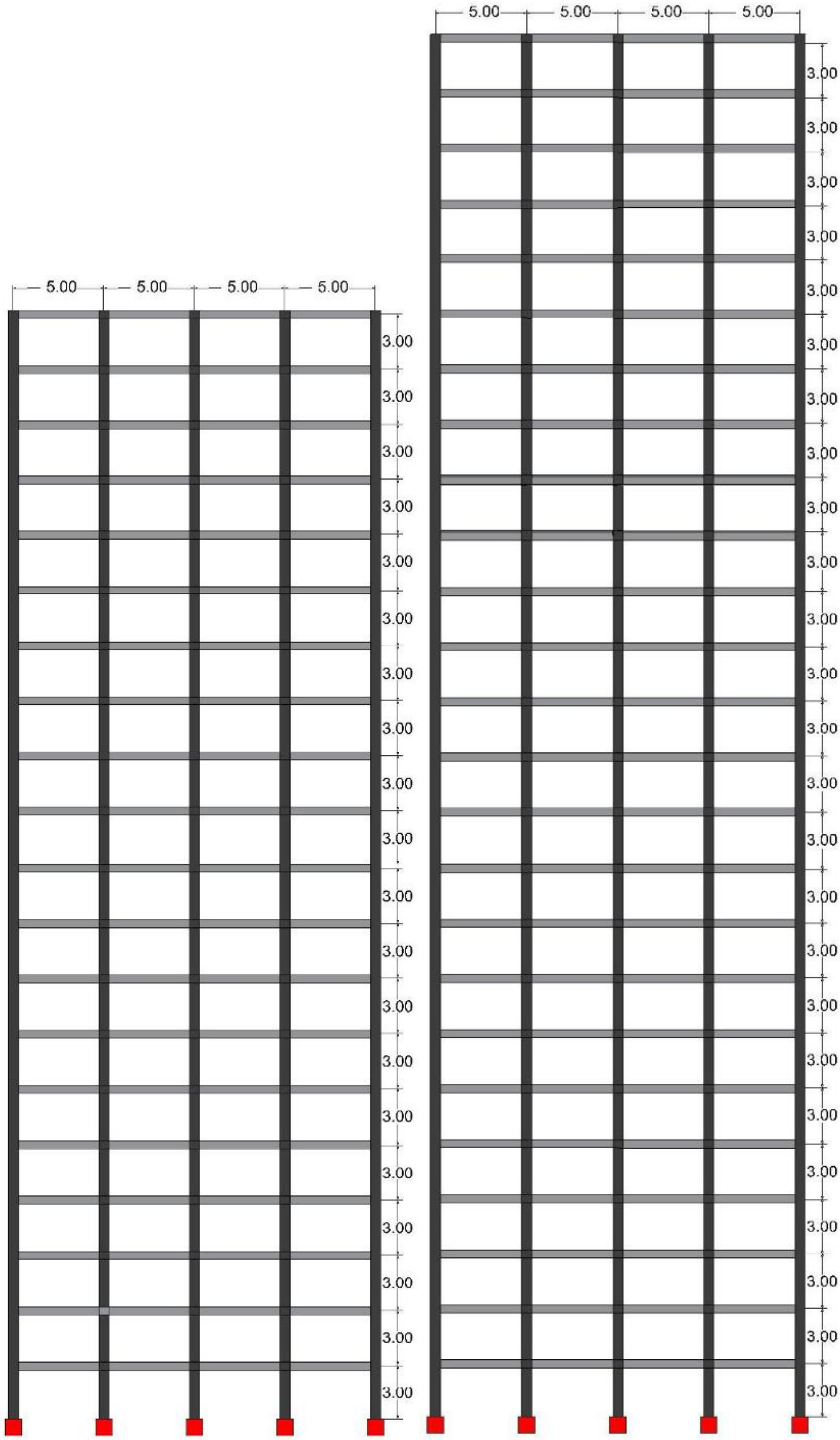
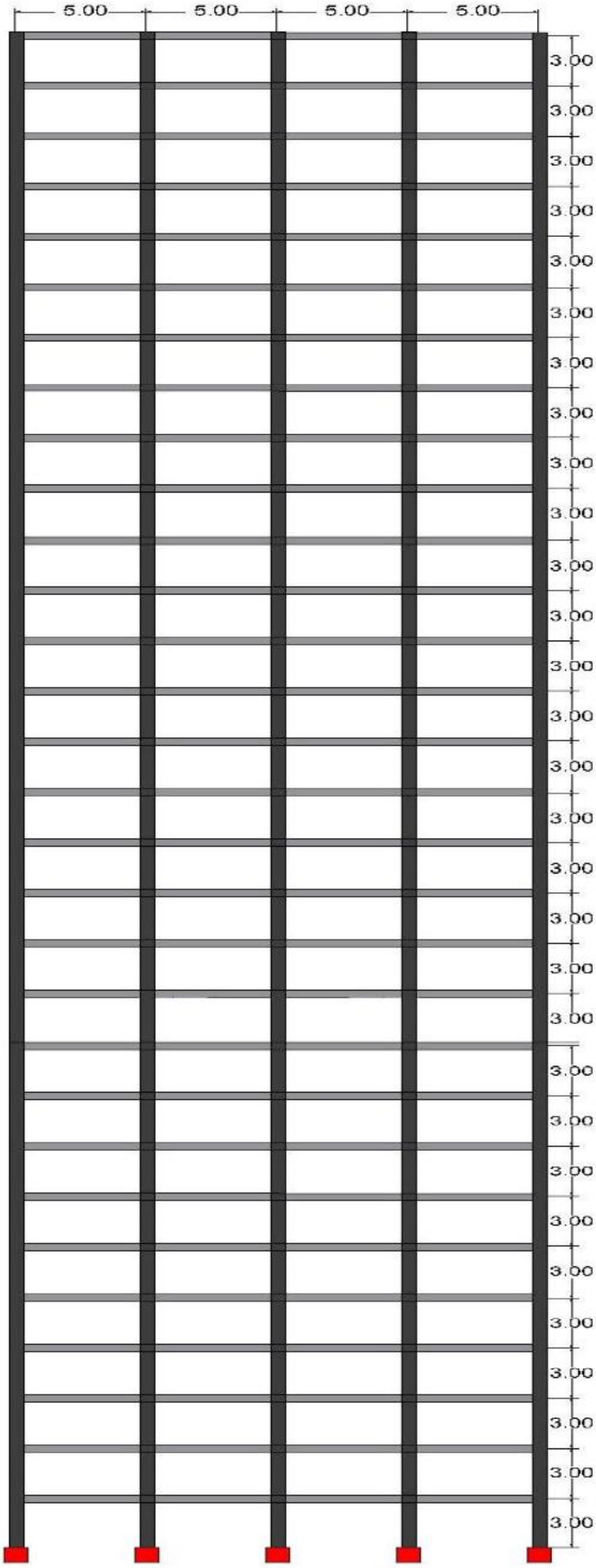
a) b)



c) d)



e)

Figure 3.2 Elevation views of a) 10-storey, b) 15-storey, c) 20-storey, d) 25-storey, and e) 30-storey RC frames

3.3 Analytical model of RC structures with and without the effect of P-delta

The analytical model of the frames, including nonlinear properties of structural members was obtained by using SAP 2000 nonlinear vs 14.0 structural analysis program (CSI.2011). In this study, five reinforced concrete (RC) frames with different heights were investigated, in order to compare the seismic response of the original structures with and without P-delta effect. In the analytical models, it was assumed that all materials have nonlinear behavior, side sway deformation is permitted, all frame members have lumped plasticity.

3.4 P-delta effect in design

The structural behaviors of high-rise buildings under static and dynamic loadings are different. It can be said that the main load which affect P-Δ analysis is the lateral loads, especially the dynamic loads that interfaces with the gravity loads. Stability of a structure due to the shifting of resultant gravity loads from the original center of gravity to the ground level allows the center of gravity to be out from the base area of the structure (Fenwick et al., 1992).

To compensate for P-delta effects in the design two different approaches in design may be used; such as

(i) The stiffness of the structure may be increased. This reduces the deflection and also P-delta induced actions. However, frequently it is not practical or economic to increase the stiffness sufficiently to reduce the actions to the level that they can be ignored in design. Increasing the stiffness generally has the secondary effect of increasing the seismic design action, as the period of the structure is reduced.

(ii) The strength of the structure may be increased. Increasing the strength reduces the maximum displacement. This is contrary to the equal displacement concept. An unrealistic case where P-delta actions are excluding from the analyses is applied (Fenwick et al., 1992).

The treatment of P-Δ effects in (DDBD) Direct Displacement Based Design is comparatively straightforward as shown in Figure 3.3. Unlike conditions in force-based design, the design displacement is known at the start of the design process, and hence the P-Δ moment is known before the required strength is determined. DDBD is based on the effective stiffness at maximum design displacement. When P-Δ moments are significant, it is the stiffness corresponding to the degraded strength and the design displacement that must match the required stiffness. Based on these principles mentioned, the required residual strength, and the initial strength, corresponding to zero displacement, is defined as:

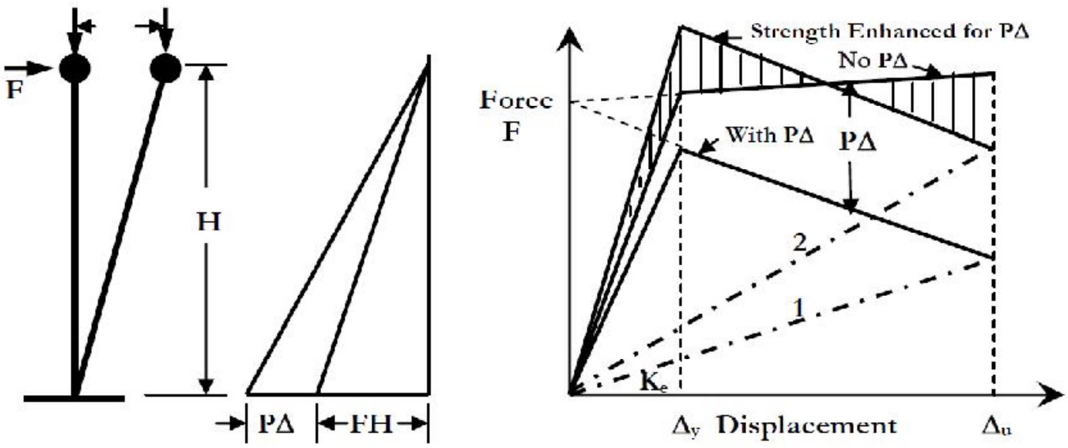
$$F + V_{Base} = K_e \Delta_d \tag{3.1}$$

$$F = K_e + C.\frac{P\Delta_d}{H} \tag{3.2}$$

In addition, the required base-moment capacity can be calculated as:

$$M_B = K_e \Delta_d H + C.P\Delta_d \tag{3.3}$$

Note that it is more consistent to define the P-Δ effect in terms of the base moment, than the equivalent lateral force. Examination of the hysteretic loops indicates that more energy will be absorbed in a design considering displacement and degraded strength, than in a design not considering P-Δ design. It is also observed that steel structures are likely to be more critically affected than will concrete structures. Consideration of these points leads to specifying C =1.0 for steel structures and C =0.5 for concrete structures. Recent time-history analyses (Pettinger and Priestley, 2007) have provided confirmation of these recommendations (Asimakopoulos et. al., 2007).



(a) Structure (b) Moment (c) Force-Displacement Response

Figure 3.3 P-Δ effects on design moments (Asimakopoulos et al., 2007)

3.5 Nonlinear behavior of structural elements

The nonlinear behavior of a building structure depends on the nonlinear responses of the elements that are used in the lateral force resisting system. Hence, before applying any nonlinear analysis method on a building structure, the nonlinear behavior of such elements must be clearly specified and assessed.

In FEMA 356 (FEMA 356, 2000), the generalized load deformation relation of a structural member while exhibiting nonlinear behavior is shown in Figure 3.4. After the member yields (when applied load/yield load proportion (Q/Qy) is equal to 1), the consequent strain hardening gives the strain hardening in the load-deformation relation as the member deforms toward the anticipated strength. The horizontal axis of this diagram may either declare curvature or strain.

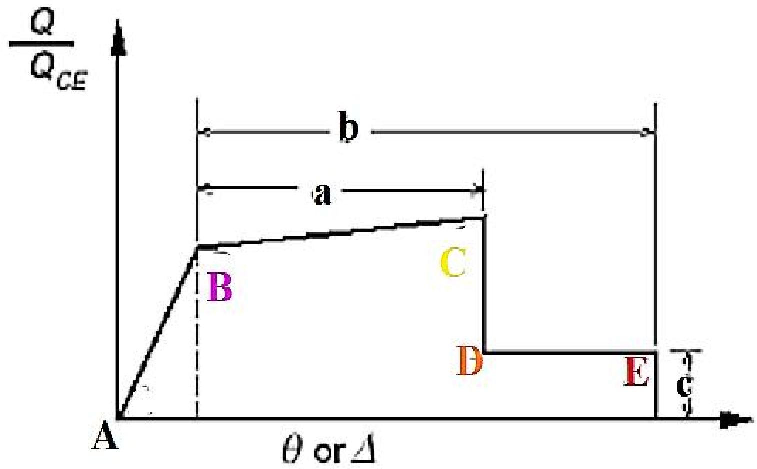


Figure 3.4 The generalized load deformation relation while exhibiting nonlinear behavior of a structural member (FEMA 356, 2000)

Point A corresponds to unloaded condition and point B represents yielding of the element. The ordinate at C corresponds to nominal strength and abscissa at C coincides to the deformation at which significant strength degradation starts. The drop from C to D represents the initial failure of the element and resistance to lateral loads beyond point C is usually unreliable. The residual resistance from D to E allows the frame elements to sustain gravity loads. Beyond point E, the maximum deformation capacity, gravity load can no longer be sustained (FEMA 356, 2000). ATC-40 and FEMA-356 codes also define the acceptance criteria depending on the plastic hinge rotations by considering various performance levels. In Figure 3.5, the acceptance criteria on a force versus deformation diagram are given. In this diagram, the points marked as IO, LS and CP represent immediate occupancy, life safety and collapse prevention performance levels, respectively.

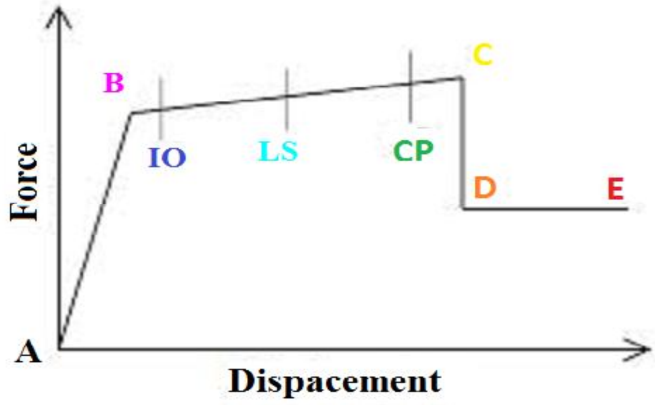


Figure 3.5 Acceptance criteria on a force versus deformation diagram (FEMA 356, 2000)

Hinges can be assigned at any number of locations (potential yielding points) along the span of the frame element as well as element ends. Uncoupled moment (M2 and M3), torsion (T), axial force (P) and shear (V2 and V3) force- displacement relations can be defined. As the column axial load changes under lateral loading, there is also a coupled P-M2-M3 (PMM) hinge, which yields based on the interaction of axial force and bending moments at the hinge location. In addition, more than one type of hinge can be assigned at the same location of a frame element.

There are three kinds of plastic hinge properties in SAP2000. They are default hinge properties, user-defined hinge properties and generated hinge properties. Solely default hinge properties and user-defined hinge properties can be allocated to frame elements. Whenever these hinges properties (default and user-defined) are allocated to a frame element, the program automatically produces a new generated hinge property for every single hinge (CSI, 2011). Default hinge properties could not be adjusted, and they are section dependent. Whenever default hinge properties are used, the program unifies its built-in default criteria with the clarified section properties for every element to propagate the eventual hinge properties. The built-in default hinge properties for steel, and reinforced concrete members are based on ATC-40 (ATC40, 1996) and FEMA-273 (FEMA-273, 1997) norms. Thus, generated hinge properties can be dependent on default properties, or they can be fully user- defined.

In this study, axial force- moment (PMM) hinges according to FEMA356 (FEMA356, 2000) was defined for determining the nonlinear behavior of columns such hinges requires the axial force vs moment interaction diagram to be calculated. When an axial force corresponding moment value of a loading was formed outside the plotted interaction diagram, this column exhibited nonlinear behavior. Moment (M3) hinge according to FEMA 356 (FEMA-356, 2000) was introduced for plastic hinges of the beam elements.

3.6 Time history analysis for case study RC buildings with P-delta effect

There are many different approaches to solving for P-delta effects. In static analyses, the increase in secondary moment can be taken into account as the product of relative inter-storey displacement and the vertical force. In dynamic time history analyses the effect of changing coordinates must be taken into account at every step of time history analysis. For static analyses, the use of a drift limit and the stability index at a certain level of loading can enable the effect of P-delta to be dealt with in a practical and simple way.

The complexity of P-delta effects in dynamic time-history analyses arise because of the characteristics and the intensities of different earthquakes and structural properties of the materials.

In this study, 10, 15, 20, 25 and 30 storey RC moment resisting frames were selected to carry out analysis with and without considering the P-delta effect. The behavior of the frames had been investigated by inelastic time history analysis using earthquake acceleration records of 1979 Imperial Valley, 1987 Superstition-Hills, and 1992 Landers earthquakes. Form the results of the analysis, the influence of the P-delta effect on the response of the structures such as displacement vs. time, displacement vs. storey level, drift ratio vs. storey level and the hysteretic curves were investigated.

3.7 Ground motions accelerations

For the nonlinear time history analysis, one level of seismic hazard for the design code spectrum were considered such as 10% probability of exceedance in 50-year period. The elastic design spectrum was formed according to Turkish Earthquake Code

(TEC,2007) based on third seismic zone and local site class Z4.The comparison between the design code spectrum and elastic spectra of the scaled natural ground accelerations is given in Figure 3.6.The spectrum compatible earthquake acceleration record was selected from the records of Imperial Valley (1979) earthquake ,Landers (1992) earthquake and Superstition Hills(1987) earthquake. Their properties are given in Table 3.4.

Table 3.4 Summary of ground motions used for nonlinear dynamic analysis compatible with design spectrum for 10% exceedance probability in fifty years

Earthquake Record	Year	Magnitude (Mw)	Mechanism	Rjb (km)	Rump (km)	Vs30 (m/s)	PGA (g)	PGV (cm/s)	PGD (cm)
Imperial Valley	1979	6.53	Strike-slip	7.3	7.3	192.1	0.1797	54.5	38.4
Landers	1992	7.28	Strike-Slip	23.6	23.6	353.6	0.22	53.1	45.3
Superstition Hills	1987	6.54	Strike-Slip	23.9	23.9	207.5	0.19	28.4	25.2

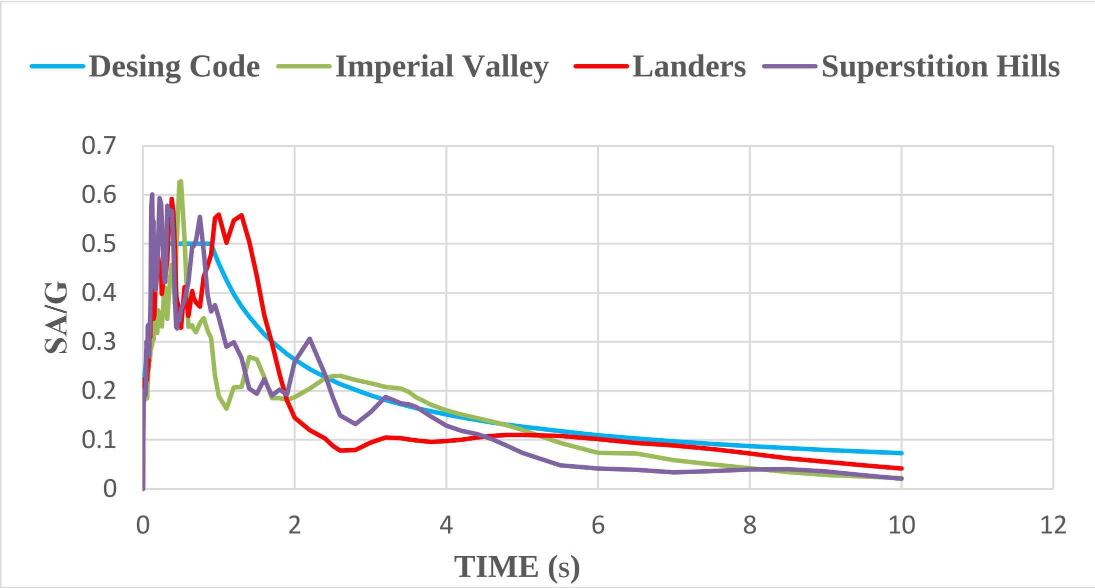


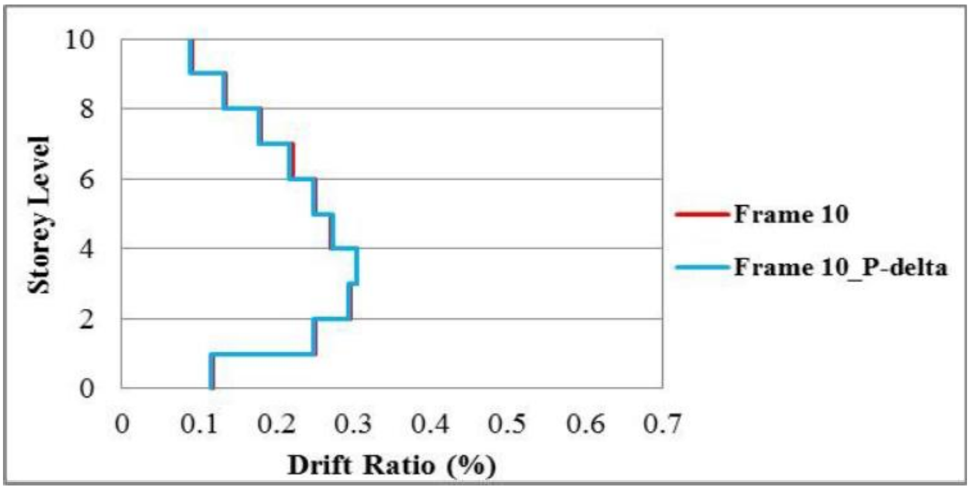
Figure 3.6 Elastic spectral accelerations of the ground motions used in this study

CHAPTER 4

RESULTS AND DISCUSSION

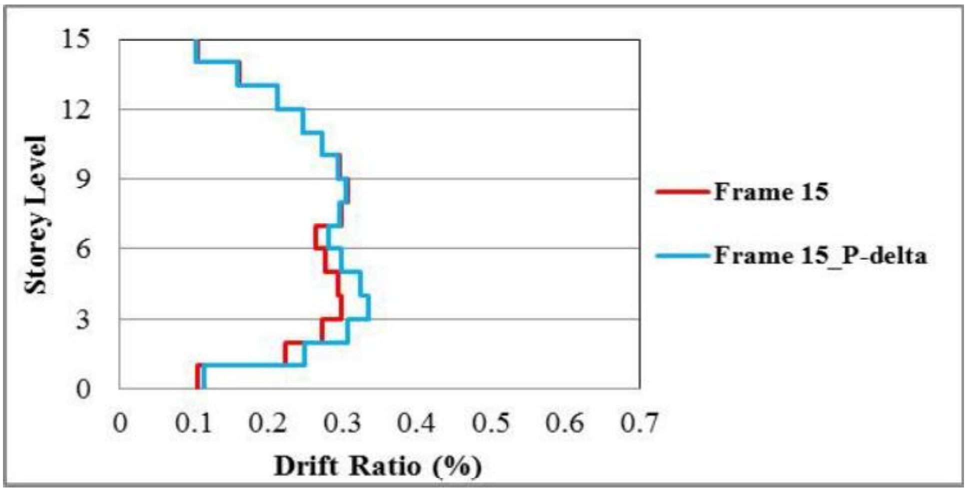
4.1 Distribution of inter-storey drift ratio at different storey levels

Inter-storey drift ratio at different storey levels in different structural system cases is an important indicator of structural behavior in performance based seismic analysis. The plots for the inter-storey drift ratio of the original reinforced concrete (RC) frames and frames with P-delta effects are given in Figures 4.1, 4.2, and 4.3. According to the analysis of the results, with the inclusion of P-delta effect into systems inter-storey drift ratio increased relatively under the Imperial Valley earthquake ground motion. For instance, as seen in Figure 4.1, for the original five structural system cases under the Imperial Valley earthquake, the maximum inter-storey drift ratio of the existing original frames was achieved as about 0.30%, 0.31%, 0.44%, 0.64%, and 0.53% of 10, 15, 20, 25, 30-storey frame buildings, respectively. While the maximum inter-storey drift ratio of the frames with P-delta effect was obtained as about 0.30%, 0.34%, 0.50%, 0.65%, and 0.59%, respectively. In addition, the maximum inter-storey drift ratio of the original 30-storey frame under Superstition Hills earthquake was achieved as about 1.0%, while 30-storey frame with P-delta effect as about 1.4%. Nevertheless, the original 30-storey frame under Landers earthquake was achieved as about 1.8%, while with P-delta effect as about 2.6%. In the mentioned results, it could be observed that the P-delta effect would considerable increase the drifts in the frames as compared with the original frames. Moreover, the storey drift demands seemed to be significantly higher with increasing the storey level of the structure. In general, the inter-storey drift demands over height in frames with and without P-delta effect were evaluated that P-delta effects (active only under the large-displacement analysis regime) increased drifts by around 10% up the height of the building. The P-delta effect had high impact on the drifts, for buildings of 20 to 30 storey levels. In contrast, this effect was decreasing in structure of 10 to 15 storey levels.



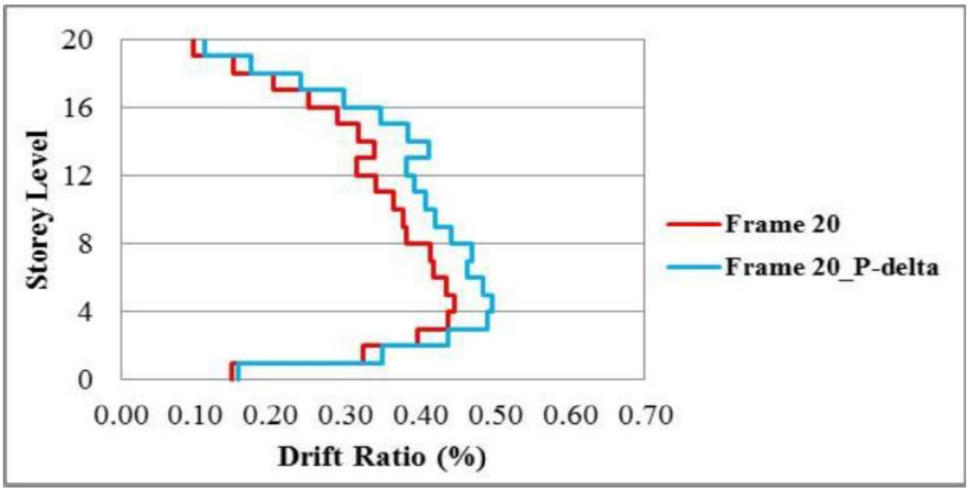
Storey Level
Drift Ratio (%)
Frame 10
Frame 10_P-delta

(a)



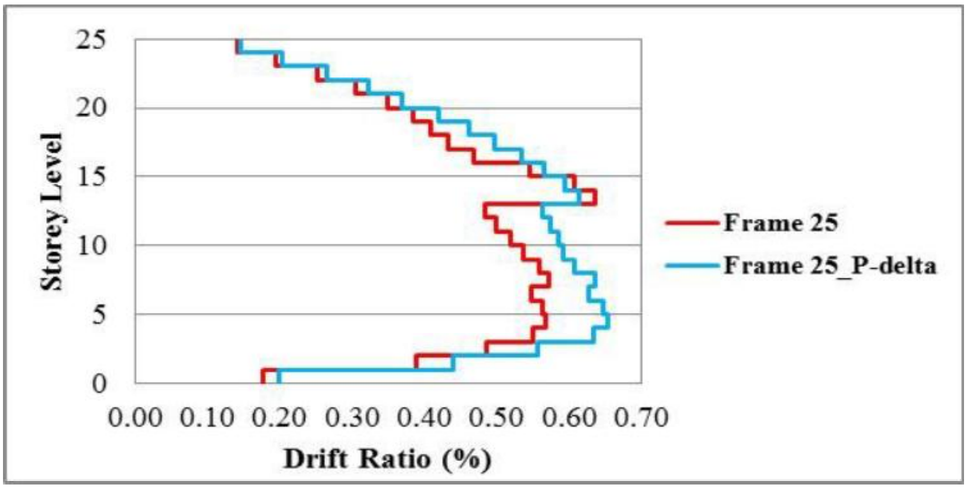
Storey Level
Drift Ratio (%)
Frame 15
Frame 15_P-delta

(b)

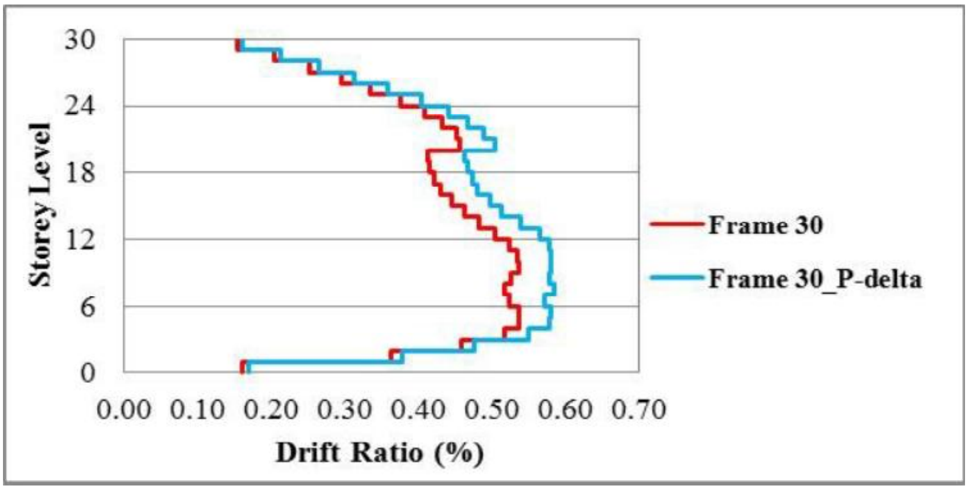


Storey Level
Drift Ratio (%)
Frame 20
Frame 20_P-delta

(c)



(d)



(e)

Figure 4.1 Inter-storey drift ratio of (a) 10-storey, (b) 15-storey, (c) 20-storey, (d) 25-storey and (e) 30-storey frames under Imperial Valley earthquake

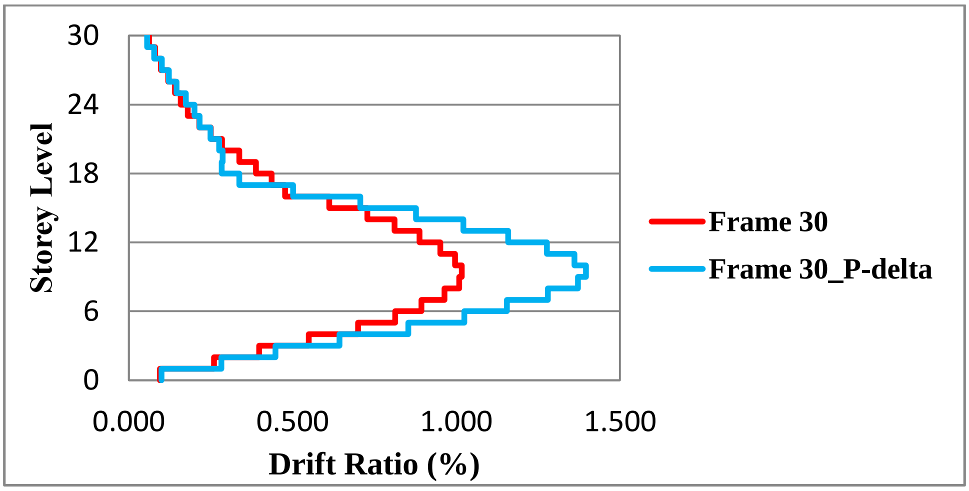


Figure 4.2 Inter-storey drift ratio of 30-storey frame with and without P-delta effect under Superstition Hills earthquake

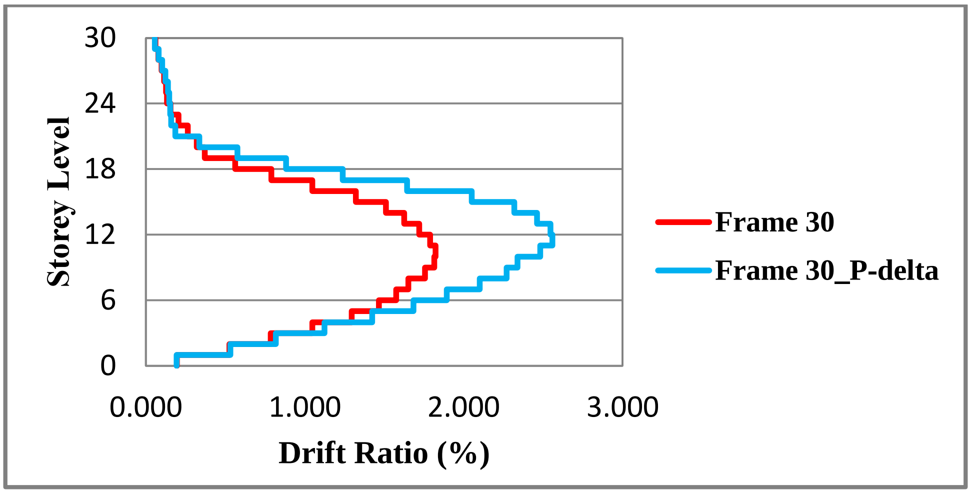
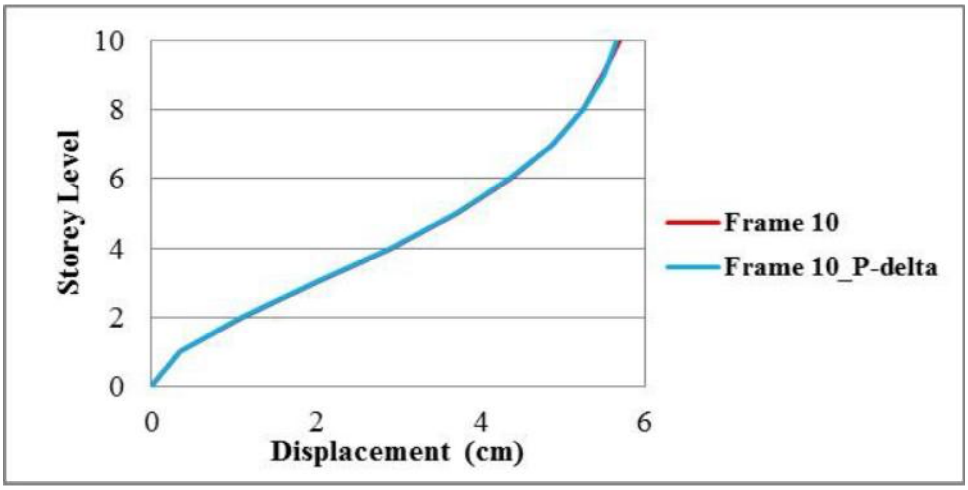


Figure 4.3 Inter-storey drift ratio of 30-storey frame with and without P-delta effect under Landers earthquake

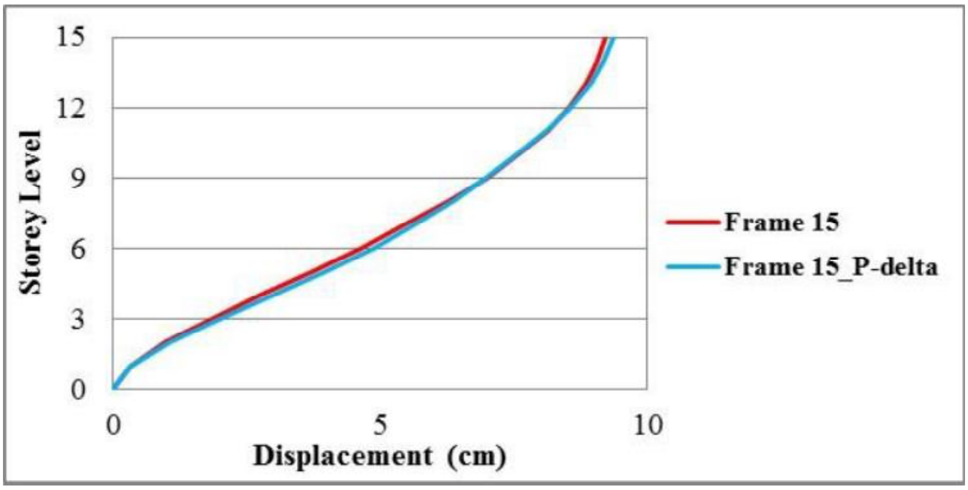
4.2 Variation of displacement time history in different storey level

To investigate the dynamic behavior of the case study structure, inelastic time-history analyses were carried out using the direct integration method. Small and large displacement analyses were conducted to consider the response of the structure under the influence of P-delta effects when subject to real earthquake records. The displacement variations for three types of earthquake Imperial Valley, Superstition Hills, and Landers accelerations were used in different structural system cases as 10, 15, 20, 25, and 30-storey structures are shown in Figures 4.4, 4.5, and 4.6. It was noted that the maximum displacements for frames with P-delta effects continued to increase around 37.2 cm for the 30-storey frame as compared with the maximum displacement of 10-storey frame which is 5.65 cm, whereas the displacement of the original 30-storey frame at around 33.4 cm as a value between 10-storey and 30-storey frames. Also the frames under Superstition Hills earthquake, the displacement was around 39.4 cm of original 30-storey frame and with P-delta was 44.2 cm. However, about Landers earthquake of original 30-storey frame the displacement at around 89.7 cm and for frames without P-delta effect as a value of around 71.0 cm. In the previous result, it was clearly seen that the maximum displacement increased where the number of storey level increased for high-rise building under the consideration of P-delta effect, so the P-delta effect should be taken into account for the analysis of the high-rise building.



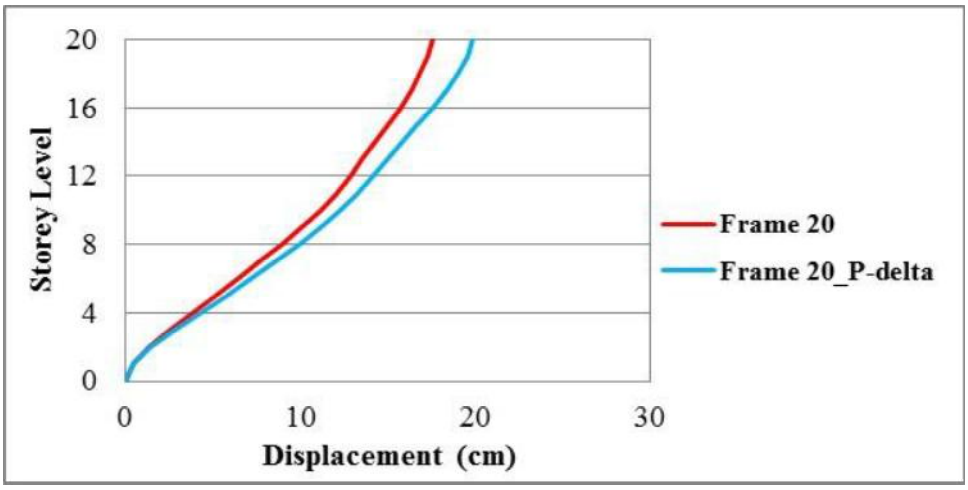
10
8
6
4
2
0
0
2
4
6
Storey Level
Displacement (cm)
Frame 10
Frame 10_P-delta

(a)



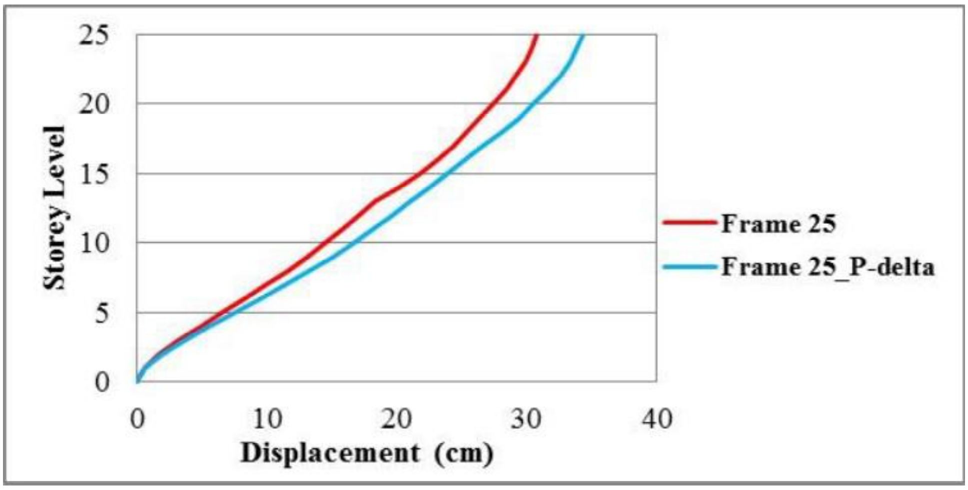
15
12
9
6
3
0
0
5
10
Storey Level
Displacement (cm)
Frame 15
Frame 15_P-delta

(b)

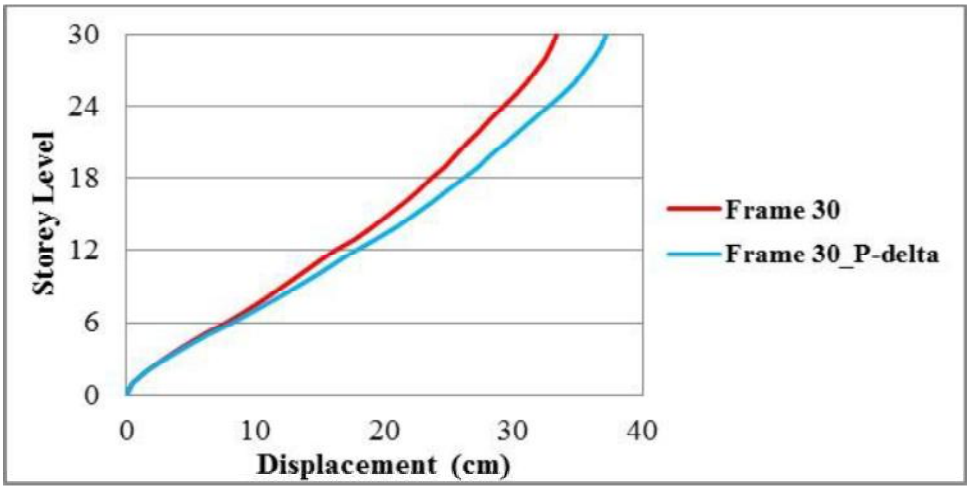


20
16
12
8
4
0
0
10
20
30
Storey Level
Displacement (cm)
Frame 20
Frame 20_P-delta

(c)



(d)



(e)

Figure 4.4 Distribution of storey displacement in (a) 10-storey, (b) 15-storey, (c) 20-storey (d) 25-storey and (e) 30-storey frames under Imperial Valley earthquake

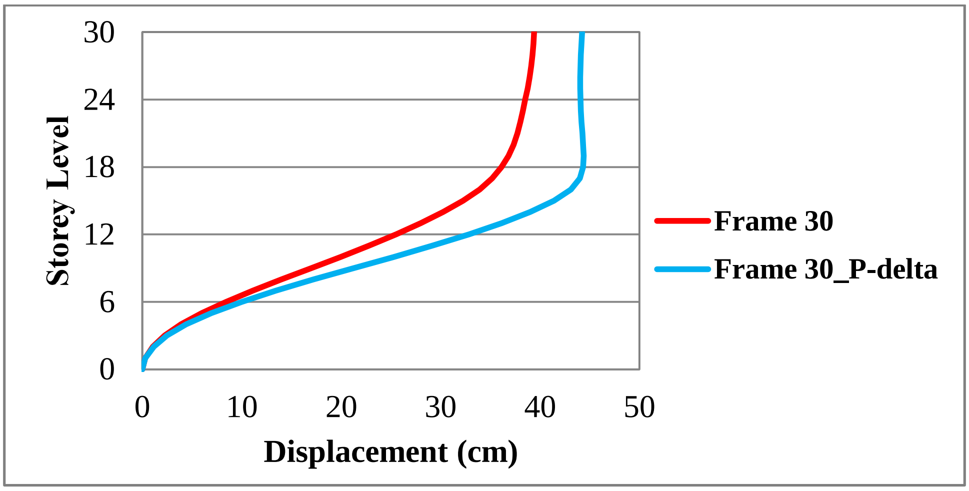


Figure 4.5 Distribution of storey displacement in 30-storey frame with and without P-delta effect under Superstition Hills ground acceleration

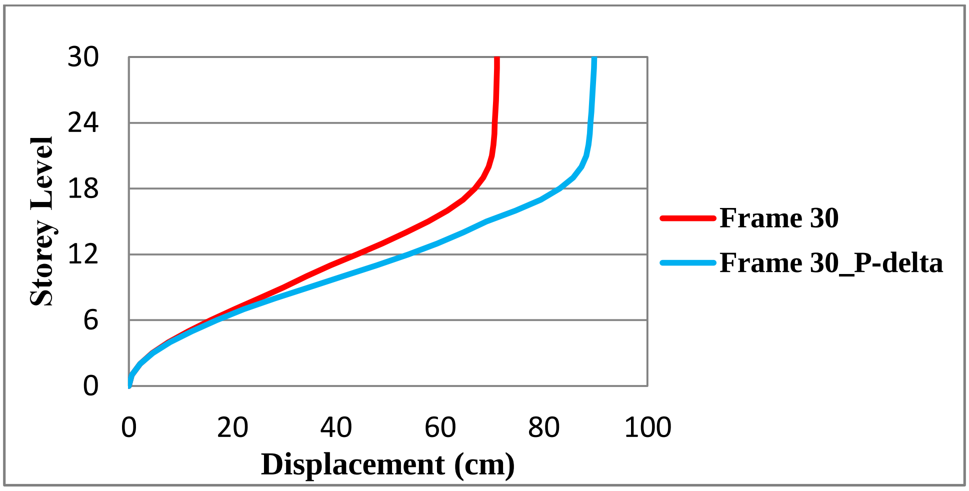
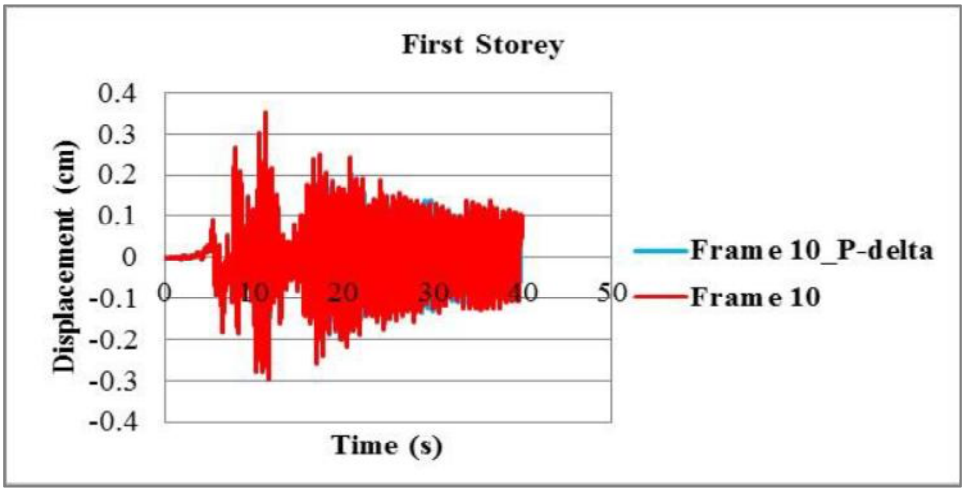


Figure 4.6 Distribution of storey displacement in 30-storey frame with and without P-delta effect under Landers ground acceleration

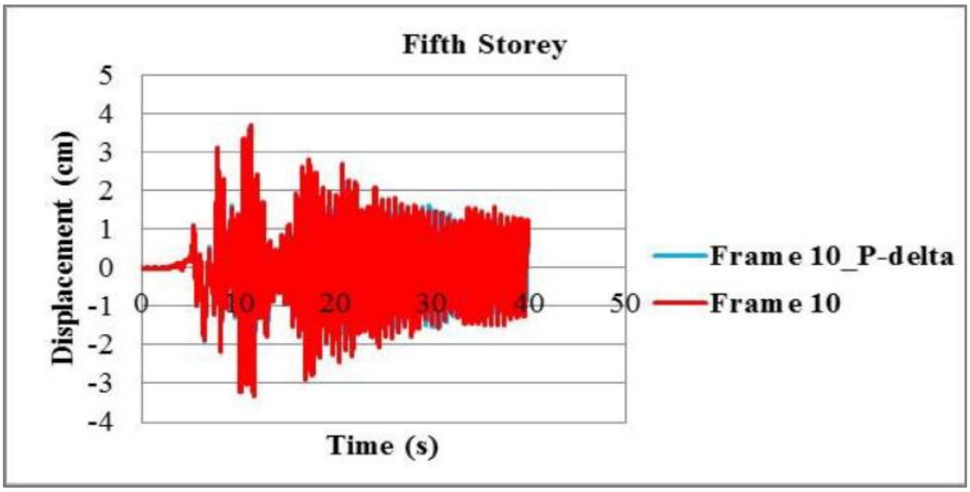
4.3 Variation of storey displacement with time

While checking the amount of displacement in different storey height, it was found that the displacement of the frames under P-delta effect increased with the increase of the storey number. The maximum displacement was obviously appeared in the top storey of the buildings with the effect of earthquake used. In this study, the displacement versus time for all structural systems under the seismic loading, which have been calculated using the SAP2000 program and given in Figures 4.7, 4.8, 4.9, 4.10, 4.11, 4.12, and 4.13.

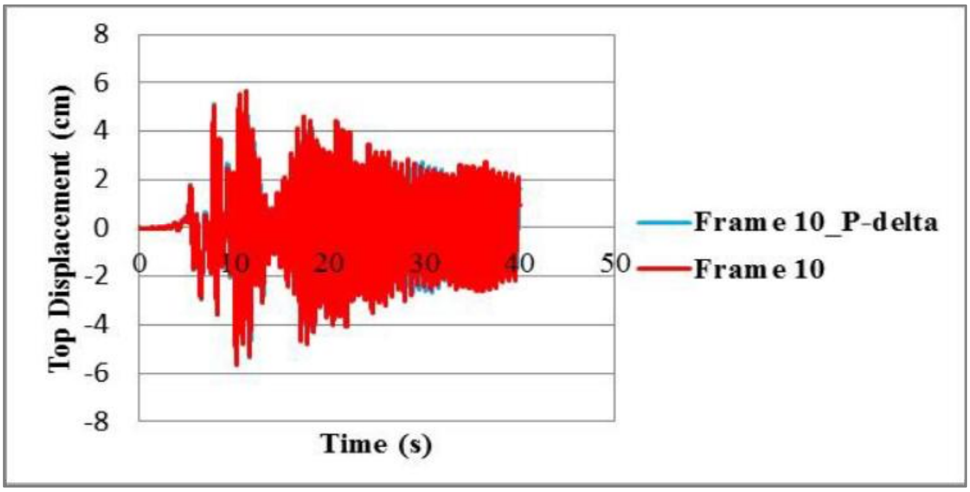
In the forementioned figures, it was observed that all structures with and without P-delta if exposed to earthquake, it would be dramatically affected. Furthermore, the frame type and the number of storey were also effective on the time history of the storey displacement; it meant that especially for 25 and 30 storey frame models,more differences were observed in the maximum displacement of the frame with and without P-delta.For example, the maximum value for three types of ground acceleration were in the twenty-fifth storey, it was about 25.2 cm, 35.0 cm, and 75.0 cm for the orginal frames, respectively, and about 35.9 cm, 41.5 cm, 78.0 cm for the frames with P-delta effect in the time of 7.85 s, 25 s and 20 s, respectively. To establish the object, also the top displacement was studied and it was found that the structure analyzed under P-delta effect caused much more displacement at the top under seismic excitations of Imperial Valley. From the results of the time history analyses of the five original frame in top storey of 10, 15, 20, 25, and 30-storey frames, that was on average 5.9%, 9.5%, 16.3%, 30.5%, and 32.2%, respectively. Nevertheless, the 30-story frame under Superstition Hills and Landers ground accelerations ,it was on average 35.5% and 79.0%, respectively. However, after the P-delta analysis of five cases as mentioned above, from the results of time history analysis under the Imperial Valley ground motion, it was obtained that the top displacement increased about 5.9%, 10.0%, 20.0%, 32.5%, and 37.5%, respectively. Also for Superstition Hills and Landers ground accelerations the top displacement increase of about 41.5% and 95.1%, respectively. It was also noted that the variation was not linear due to P-delta effect. According to the results obtained from the nonlinear time history analysis, all types of RC frame systems with P-delta effect showed significantly increase in the roof displacement of the existing frame under earthquake with the duration of time. So by increasing the storey of the building, which the top displacement would increase with the duration of time.



(a)

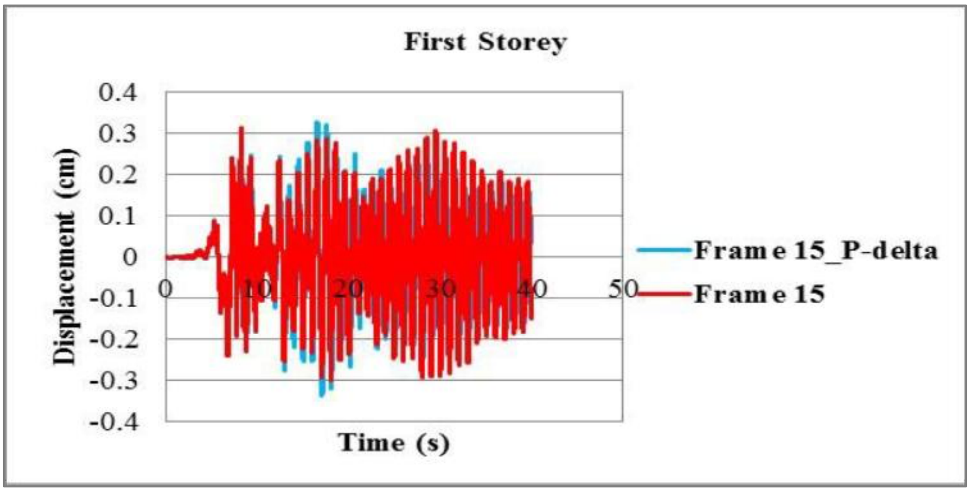


(b)

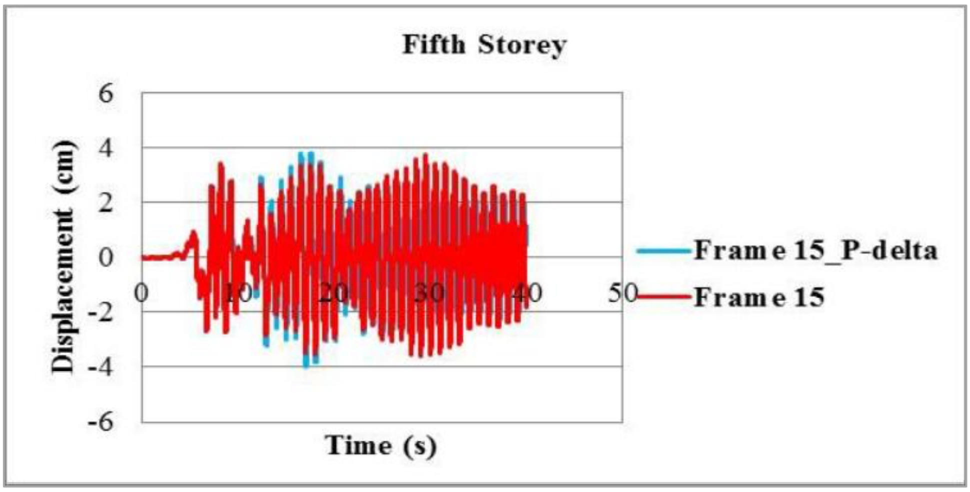


(c)

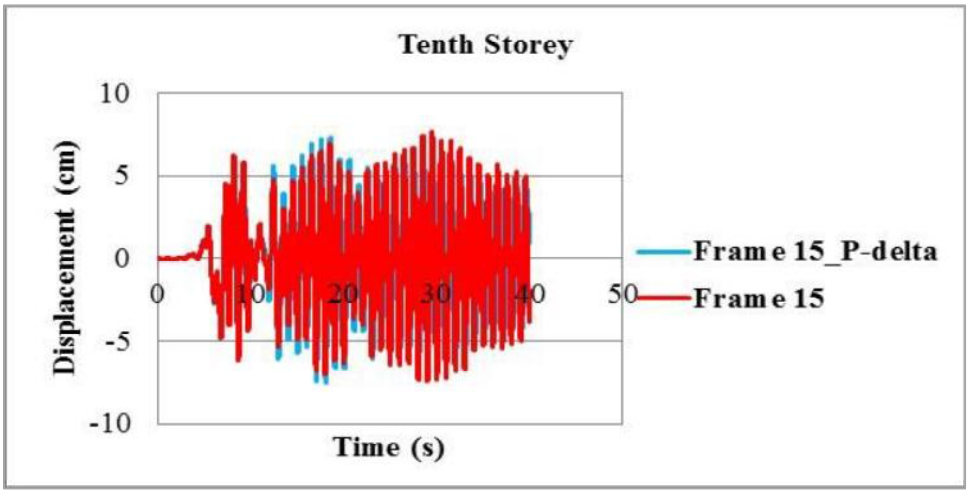
Figure 4.7 Variation in storey displacement with time for 10-storey frame under Imperial Valley earthquake: (a) first storey, (b) fifth storey, and (c) top storey



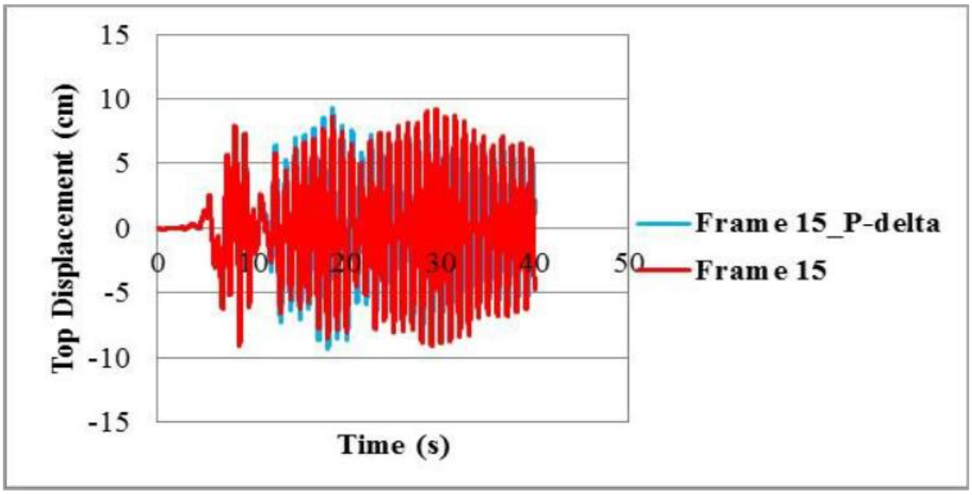
(a)



(b)

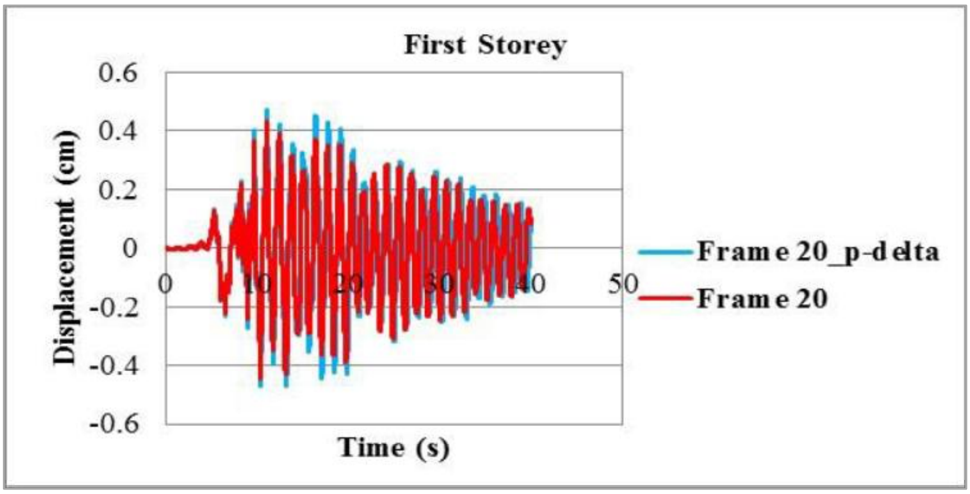


(c)

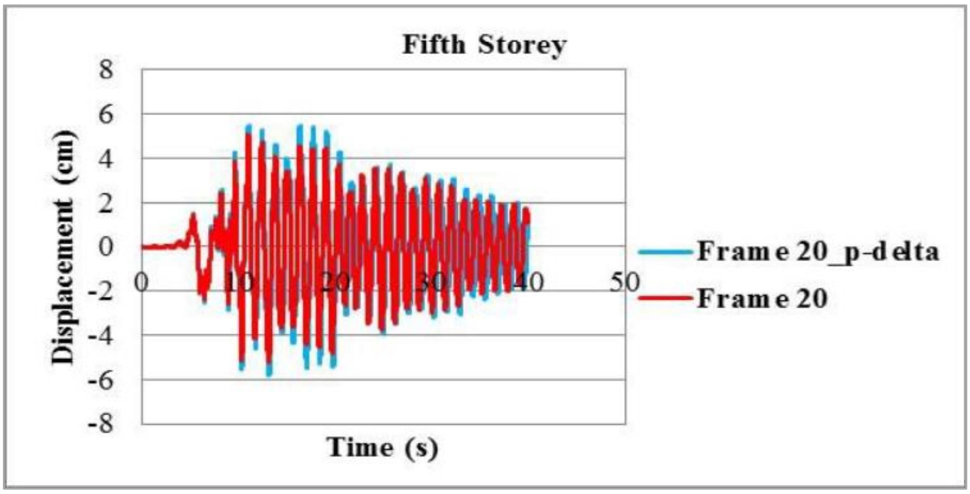


(d)

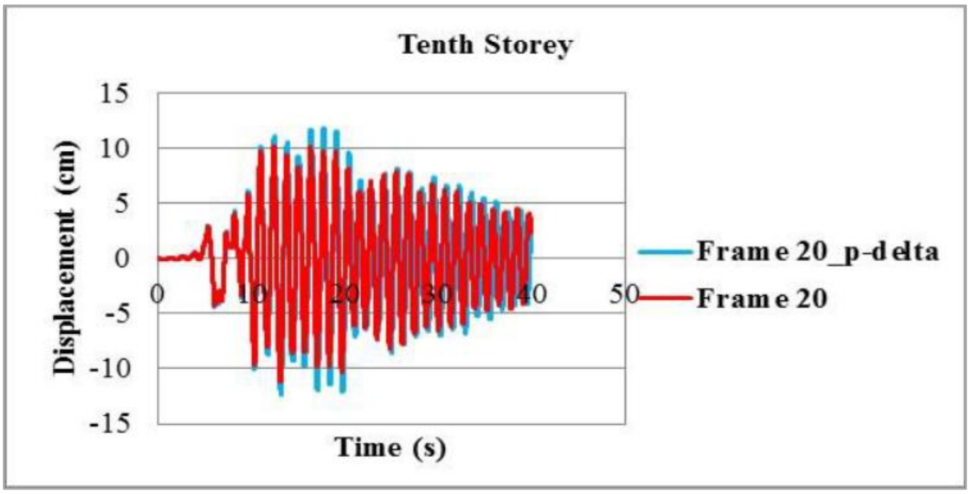
Figure 4.8 Variation in storey displacement with time for 15-storey frame under Imperial Valley earthquake: (a) first storey, (b) fifth storey, (c) tenth storey, and (d) top storey


First Storey
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
0
10
20
30
40
50
Displacement (cm)
Time (s)
Frame 20_p-delta
Frame 20

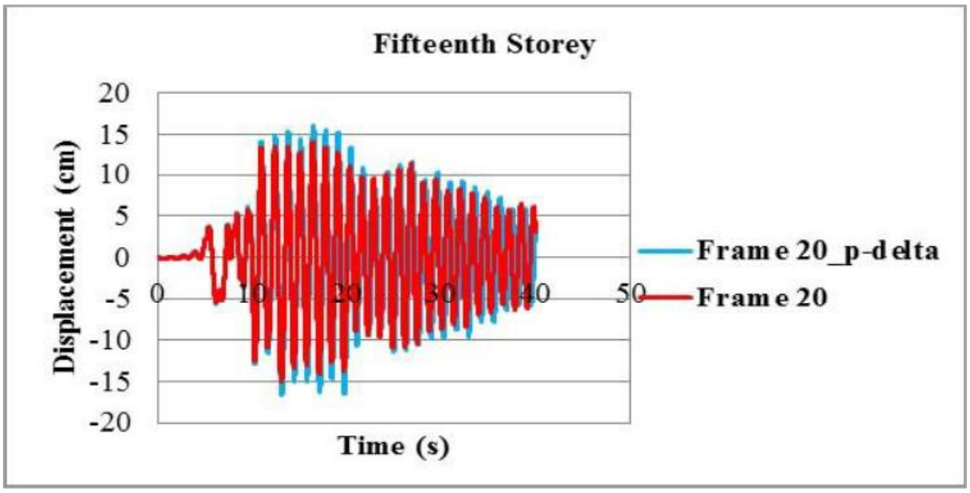
(a)


Fifth Storey
8
6
4
2
0
-2
-4
-6
-8
0
10
20
30
40
50
Displacement (cm)
Time (s)
Frame 20_p-delta
Frame 20

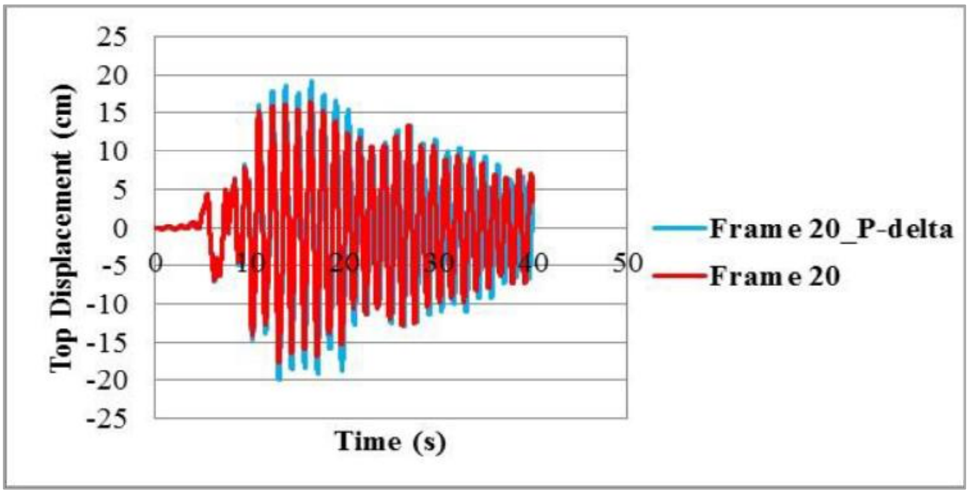
(b)


Tenth Storey
15
10
5
0
-5
-10
-15
0
10
20
30
40
50
Displacement (cm)
Time (s)
Frame 20_p-delta
Frame 20

(c)

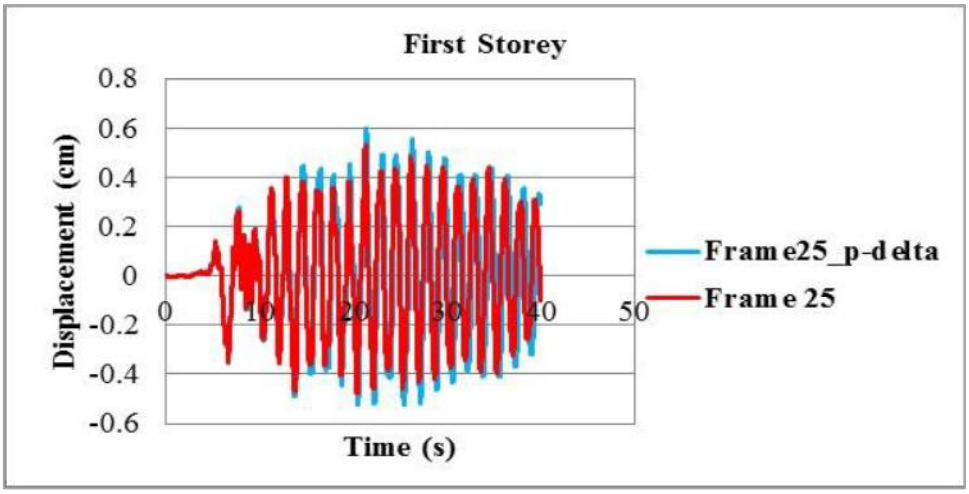


(d)

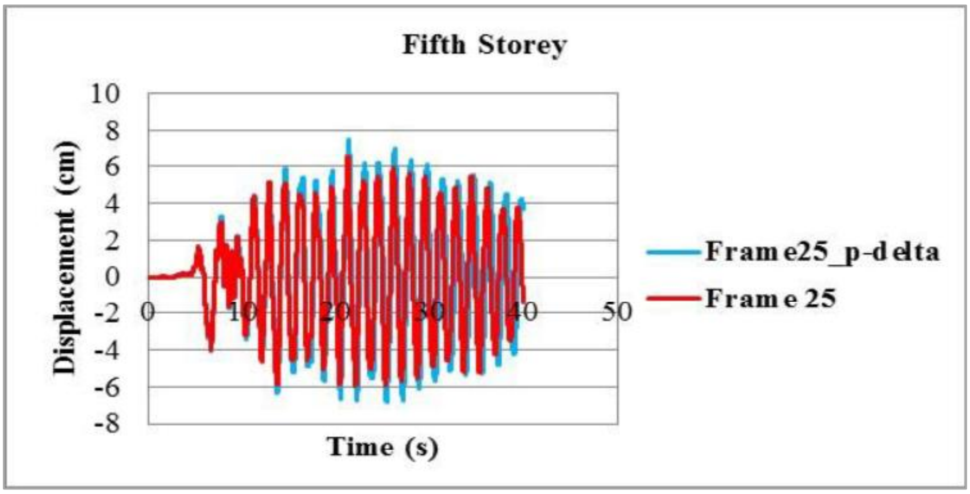


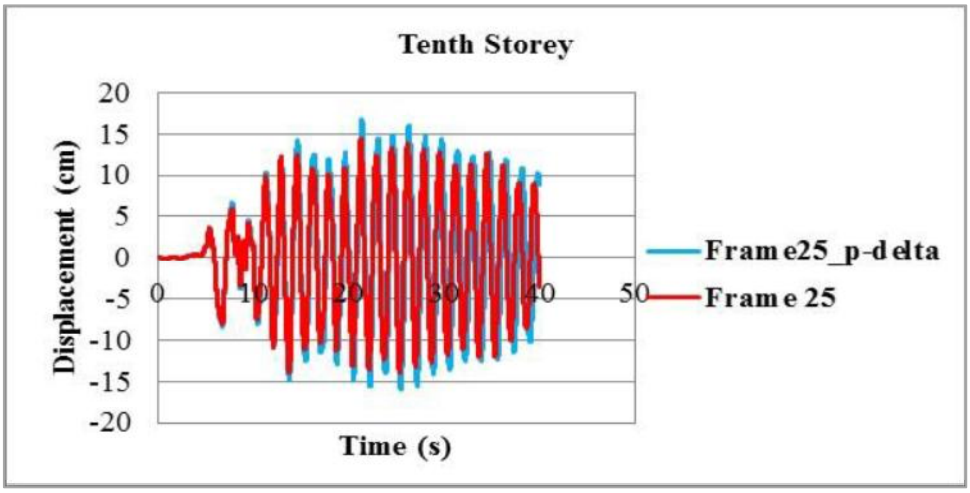
(e)

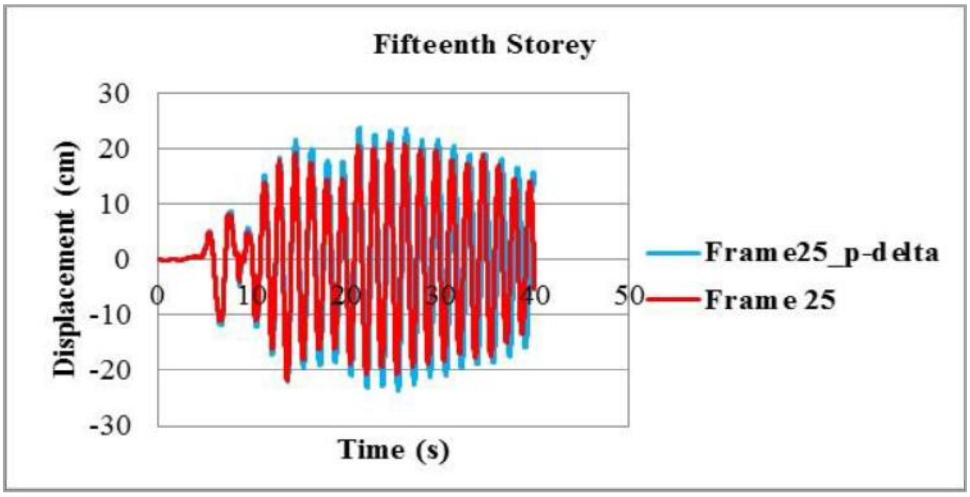
Figure 4.9 Variation in storey displacement with time for 20-storey frame under Imperial Valley earthquake: (a) first storey, (b) fifth storey, (c) tenth storey, (d) fifteenth storey, and (e) top storey

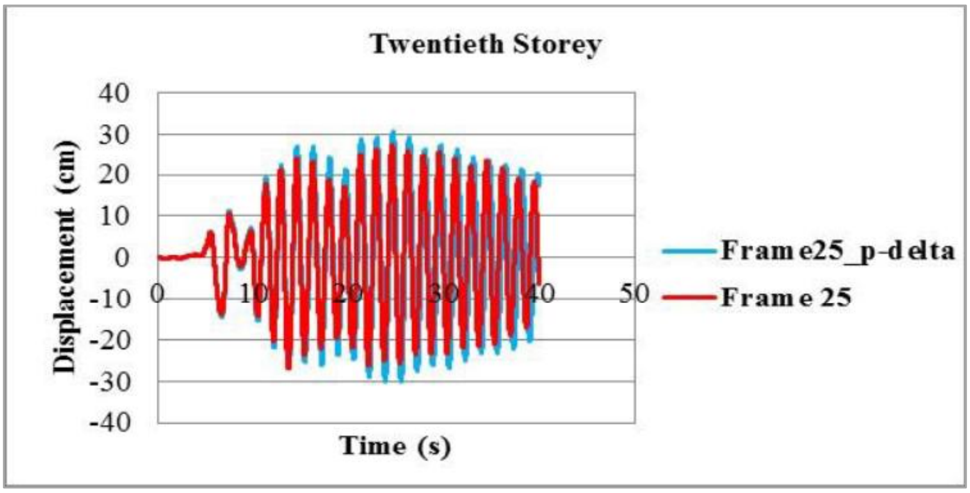


(a)

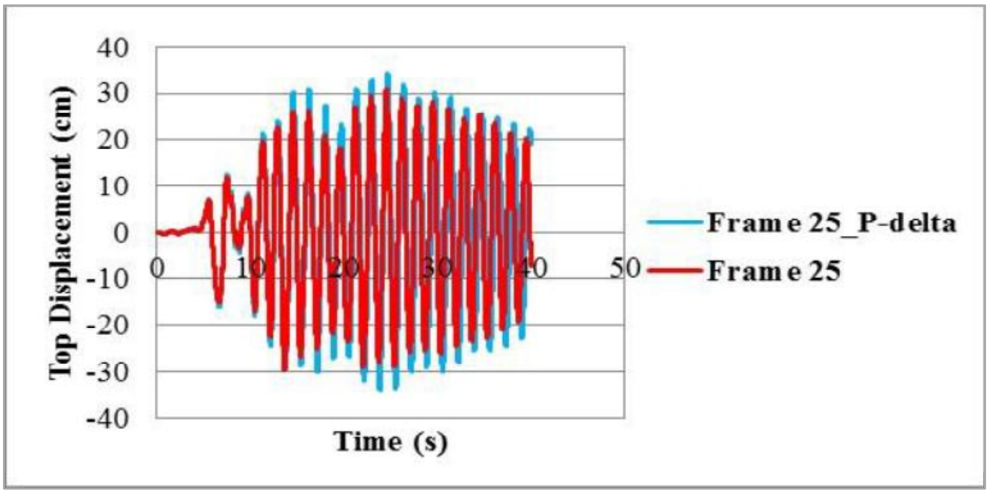

Fifth Storey
Displacement (cm)
Time (s)
10
8
6
4
2
0
-2
-4
-6
-8
0
10
20
30
40
50
Frame25_p-delta
Frame 25
(b)


Tenth Storey
Displacement (cm)
Time (s)
20
15
10
5
0
-5
-10
-15
-20
0
10
20
30
40
50
Frame25_p-delta
Frame 25
(c)


Fifteenth Storey
Displacement (cm)
Time (s)
30
20
10
0
-10
-20
-30
0
10
20
30
40
50
Frame25_p-delta
Frame 25
(d)

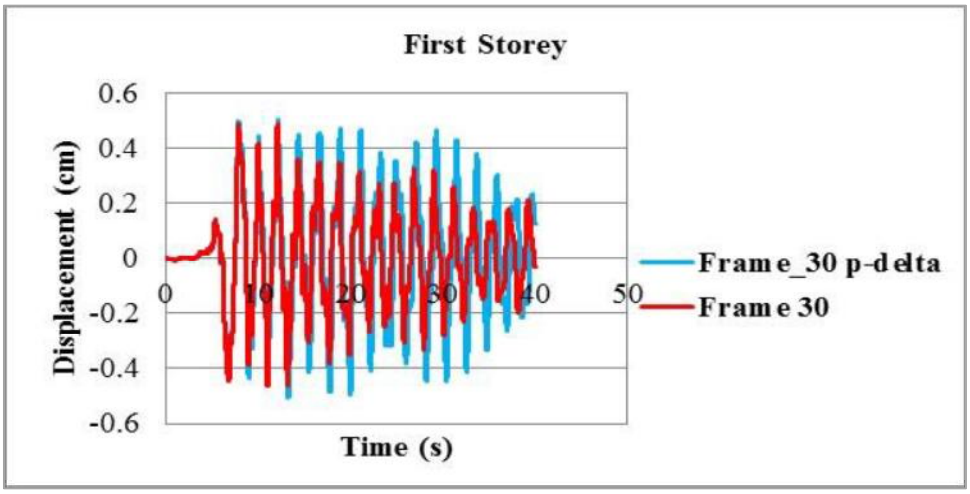


(e)

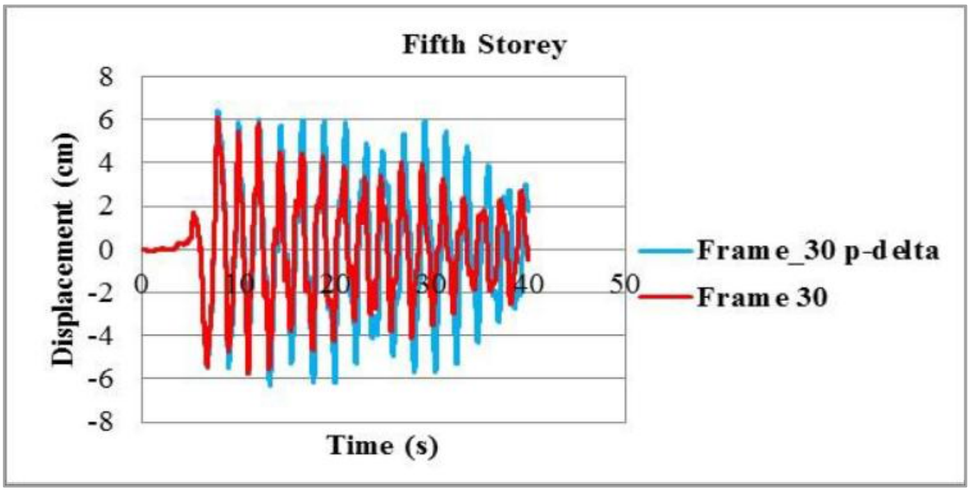


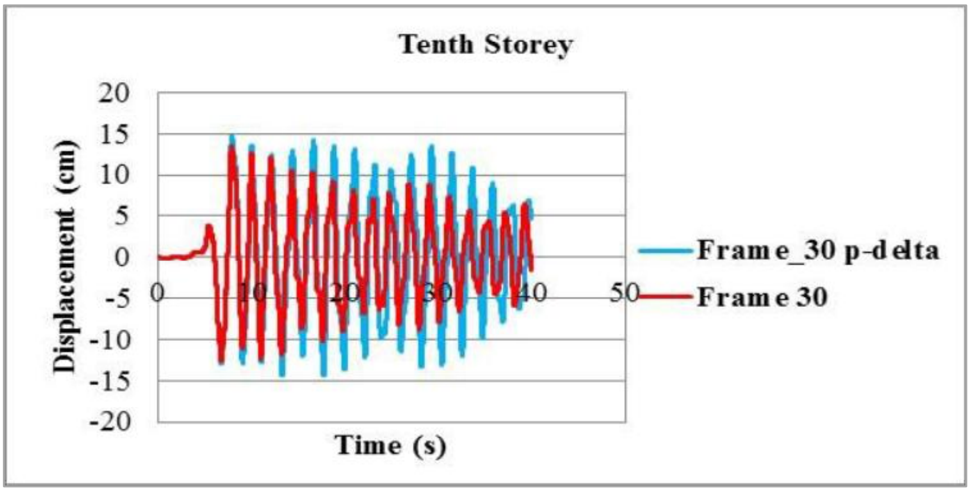
(f)

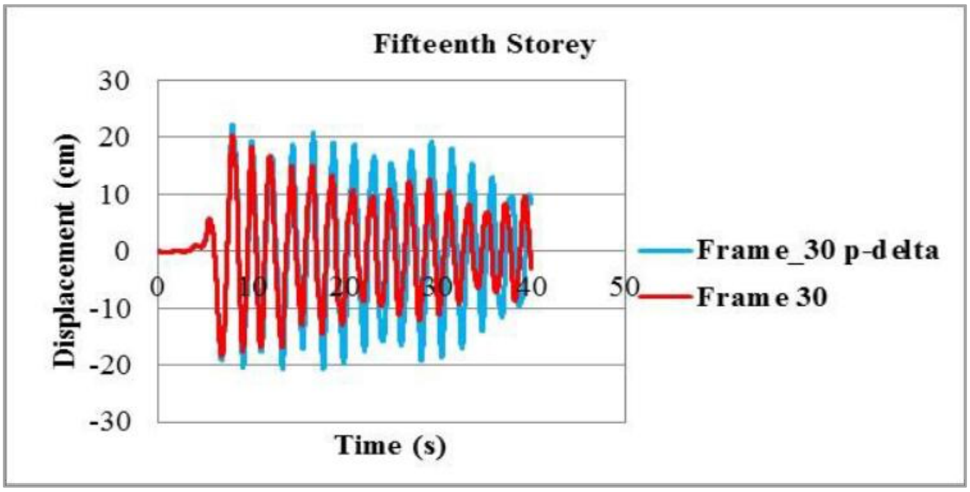
Figure 4.10 Variation in storey displacement with time for 25-storey frame under Imperial Valley earthquake: (a) first storey, (b) fifth storey, (c) tenth storey, (d) fifteenth storey, (e) twentieth storey, and (f) top storey

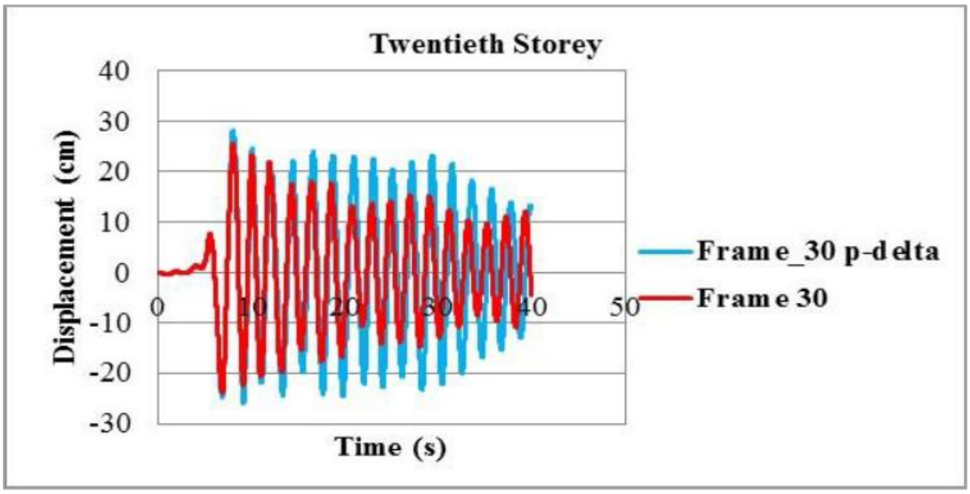


(a)

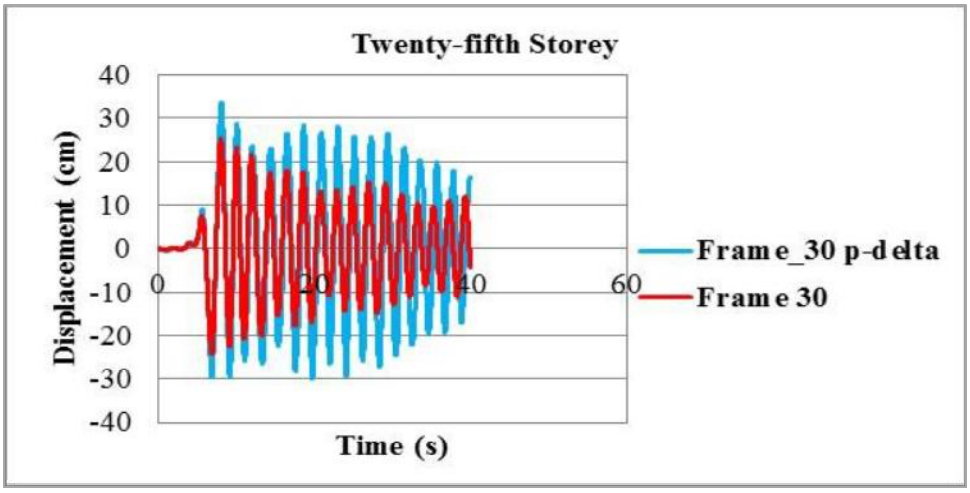

Fifth Storey
Displacement (cm)
Time (s)
Frame_30 p-delta
Frame 30
(b)


Tenth Storey
Displacement (cm)
Time (s)
Frame_30 p-delta
Frame 30
(c)

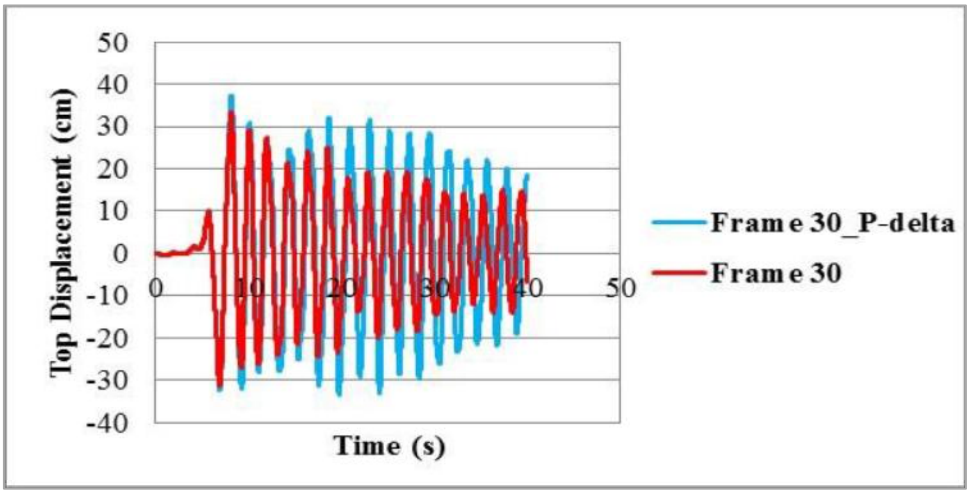

Fifteenth Storey
Displacement (cm)
Time (s)
Frame_30 p-delta
Frame 30
(d)



(e)

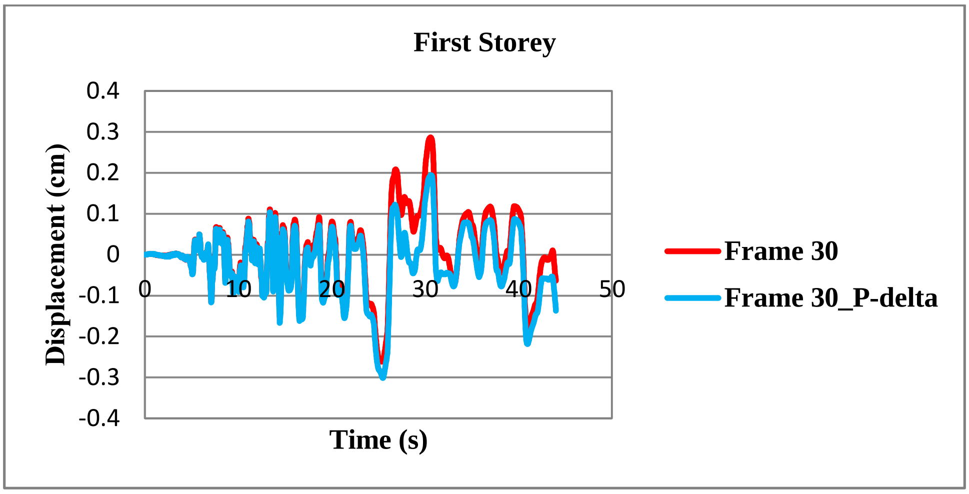


(f)

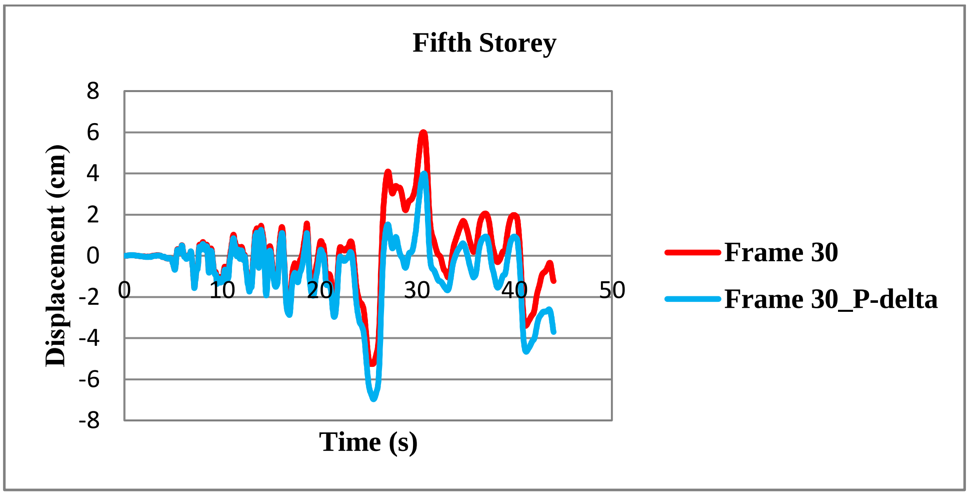


(g)

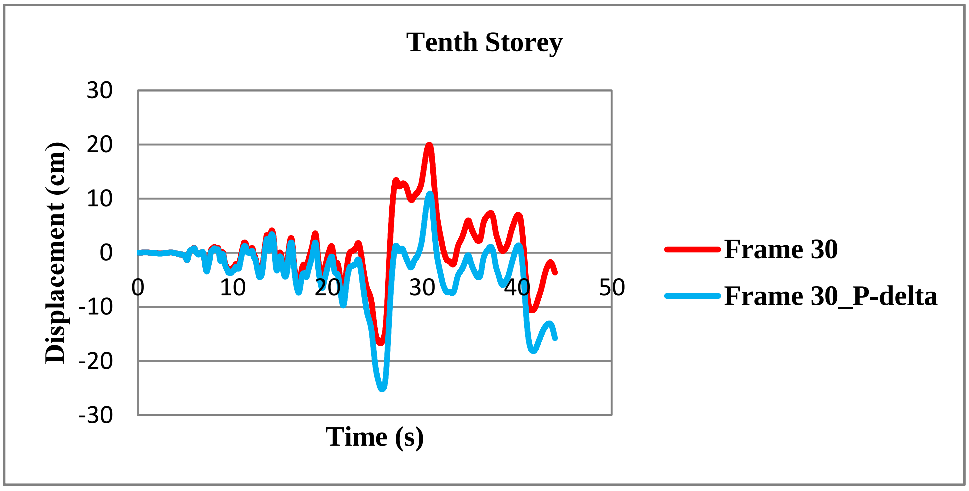
Figure 4.11 Variation in storey displacement with time for 30-storey frame under Imperial Valley earthquake: (a) first storey, (b) fifth storey, (c) tenth storey, (d) fifteenth storey, (e) twentieth story, (f) twenty-fifth storey, and (g) top storey


First Storey
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
Displacement (cm)
0
10
20
30
40
50
Time (s)
Frame 30
Frame 30_P-delta

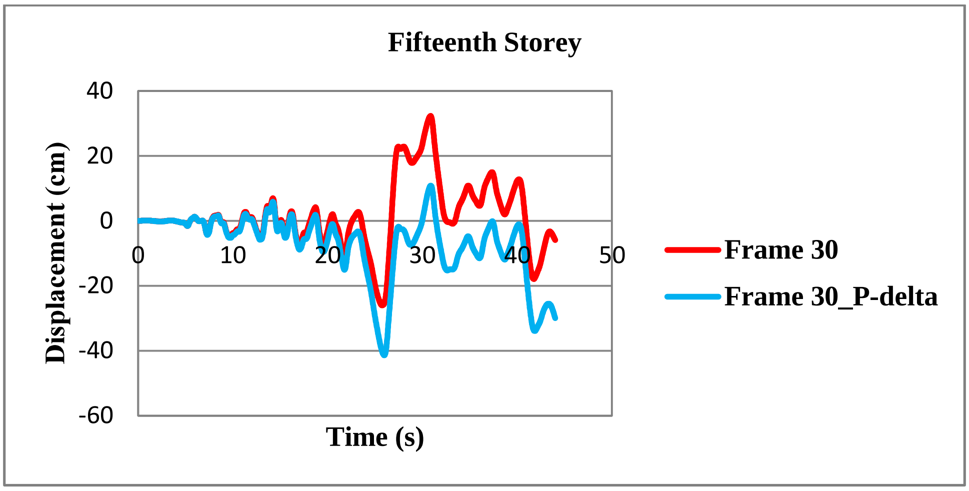
(a)


Fifth Storey
8
6
4
2
0
-2
-4
-6
-8
Displacement (cm)
0
10
20
30
40
50
Time (s)
Frame 30
Frame 30_P-delta

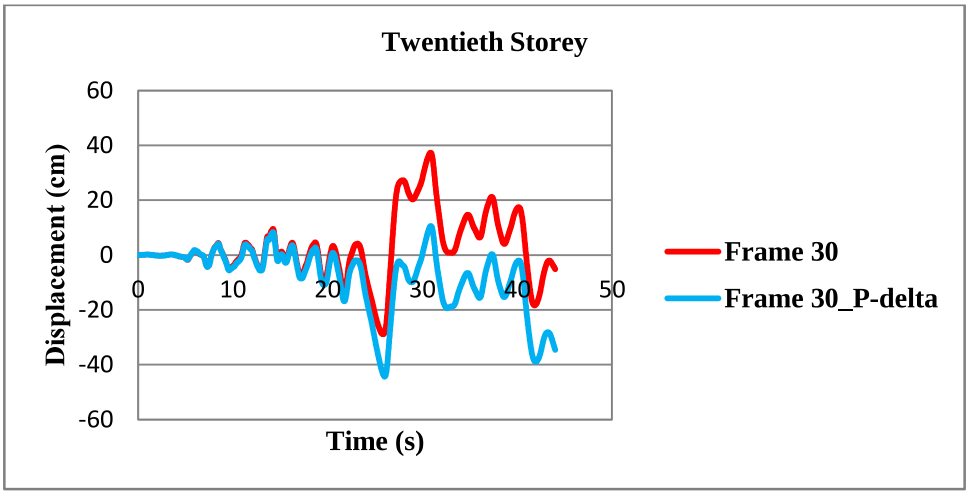
(b)


Tenth Storey
30
20
10
0
-10
-20
-30
Displacement (cm)
0
10
20
30
40
50
Time (s)
Frame 30
Frame 30_P-delta

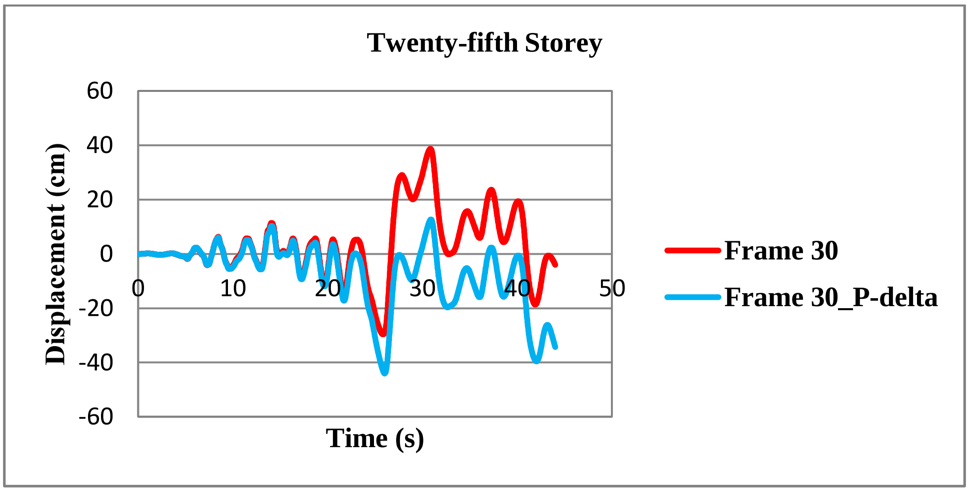
(c)


Fifteenth Storey
40
20
0
-20
-40
-60
Displacement (cm)
0
10
20
30
40
50
Time (s)
Frame 30
Frame 30_P-delta

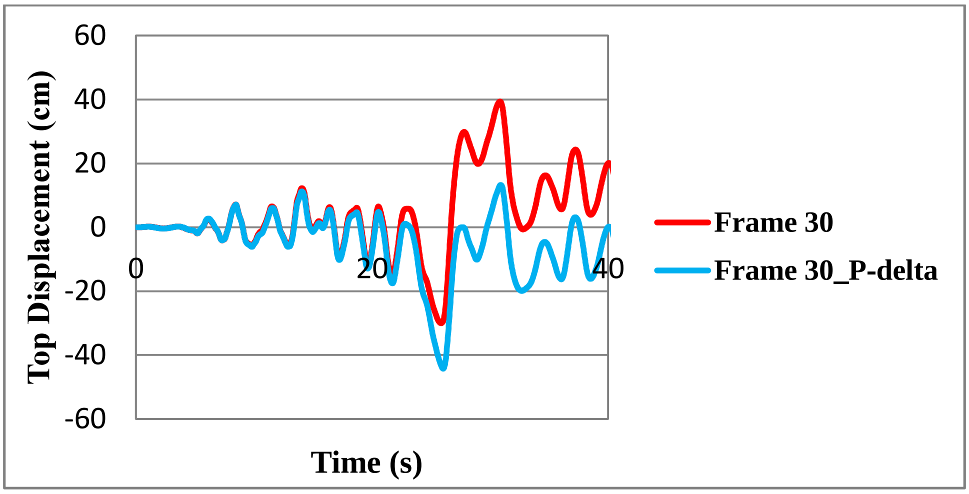
(d)


Twentieth Storey
60
40
20
0
-20
-40
-60
Displacement (cm)
0
10
20
30
40
50
Time (s)
Frame 30
Frame 30_P-delta

(e)

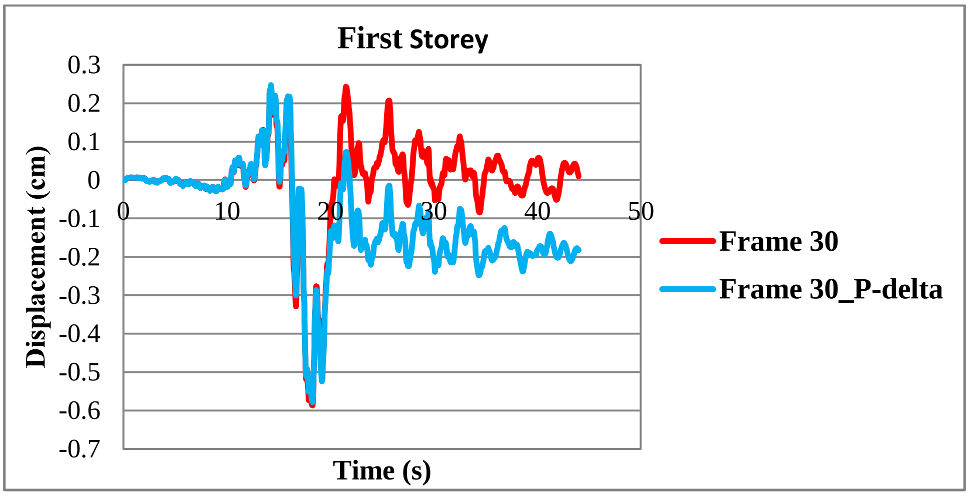

Twenty-fifth Storey
60
40
20
0
-20
-40
-60
Displacement (cm)
0
10
20
30
40
50
Time (s)
Frame 30
Frame 30_P-delta

(f)

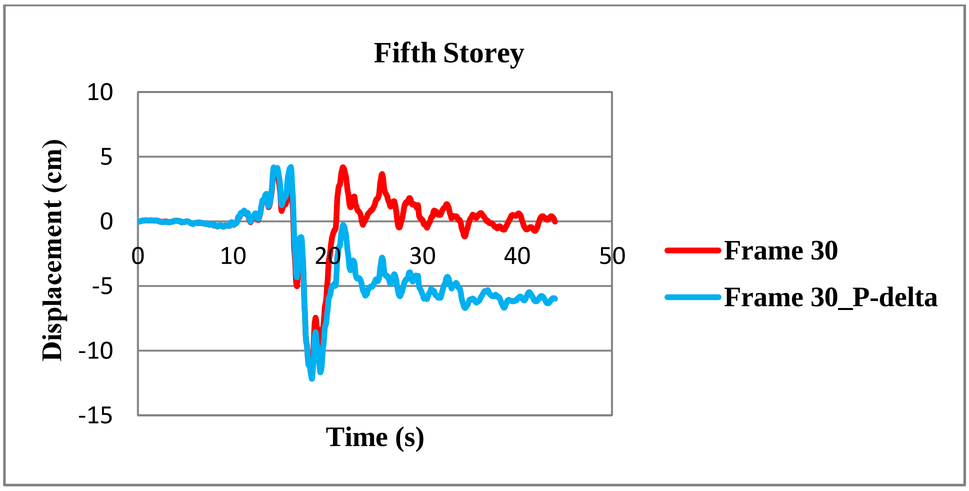


(g)

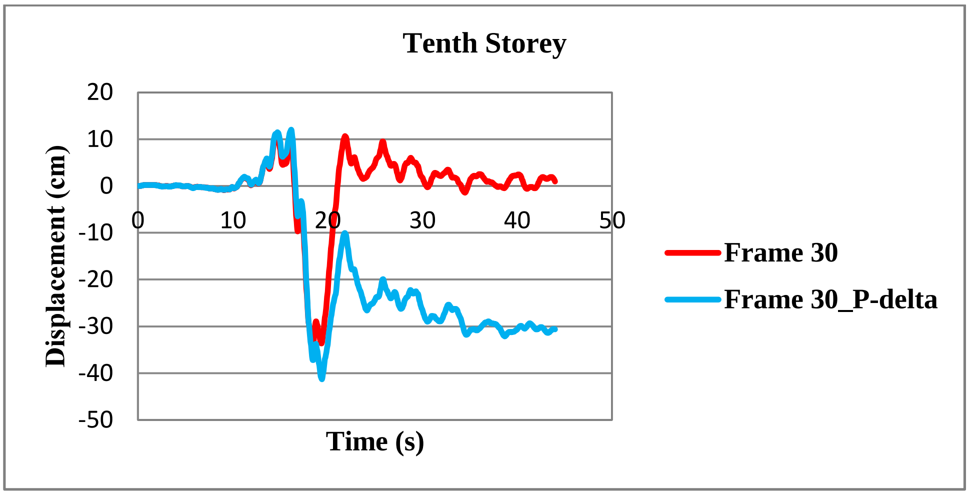
Figure 4.12 Variation in storey displacement with time for 30 storey frame under Superstition Hills earthquake : (a) first storey, (b) fifth storey, (c) tenth storey, (d) fifteenth storey, (e) twentieth story, (f) twenty-fifth storey, and (g) top displacement



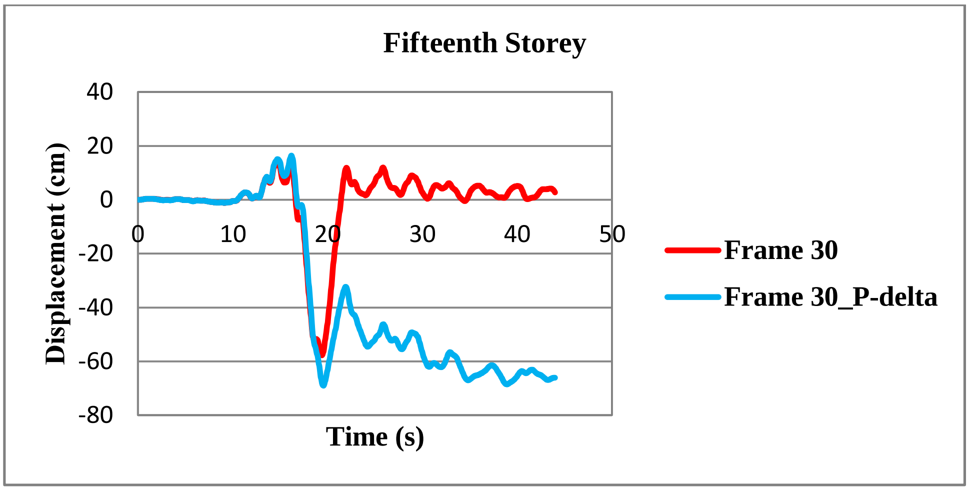
(a)



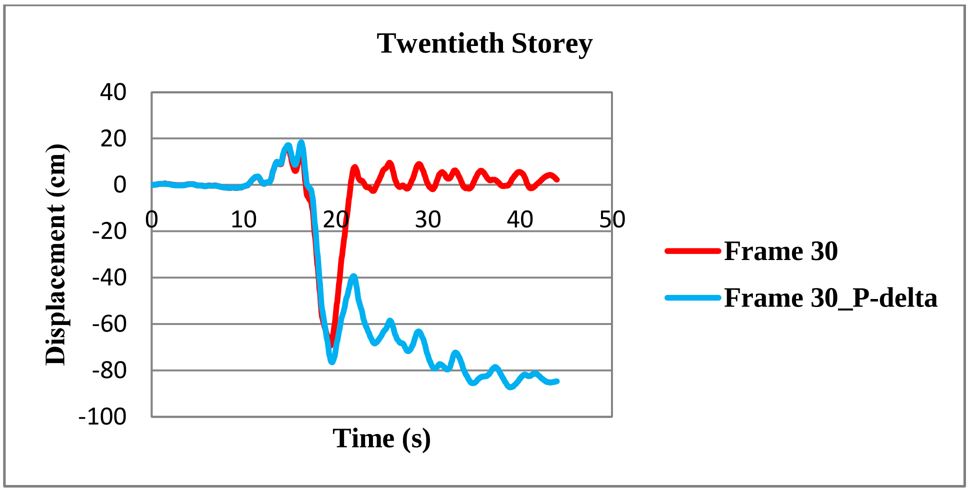
(b)


Tenth Storey
20
10
0
-10
-20
-30
-40
-50
0
10
20
30
40
50
Displacement (cm)
Time (s)
Frame 30
Frame 30_P-delta

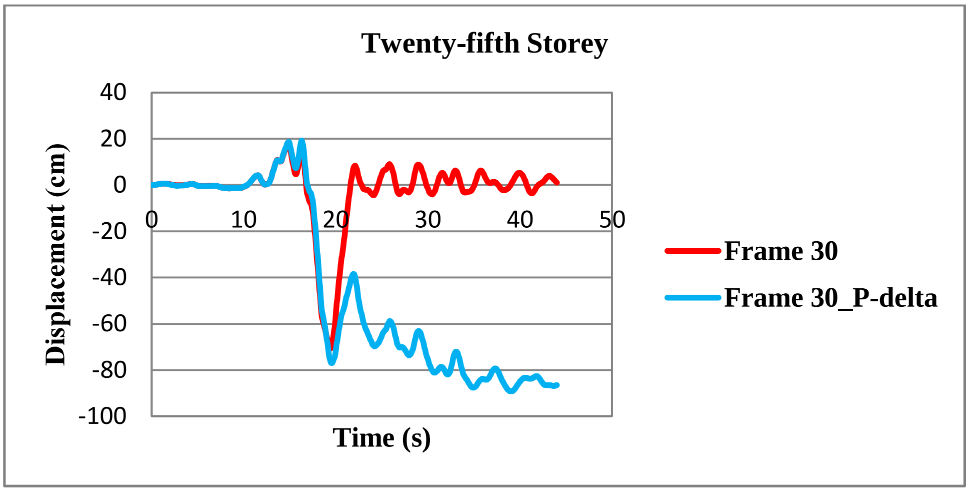
(c)


Fifteenth Storey
40
20
0
-20
-40
-60
-80
0
10
20
30
40
50
Displacement (cm)
Time (s)
Frame 30
Frame 30_P-delta

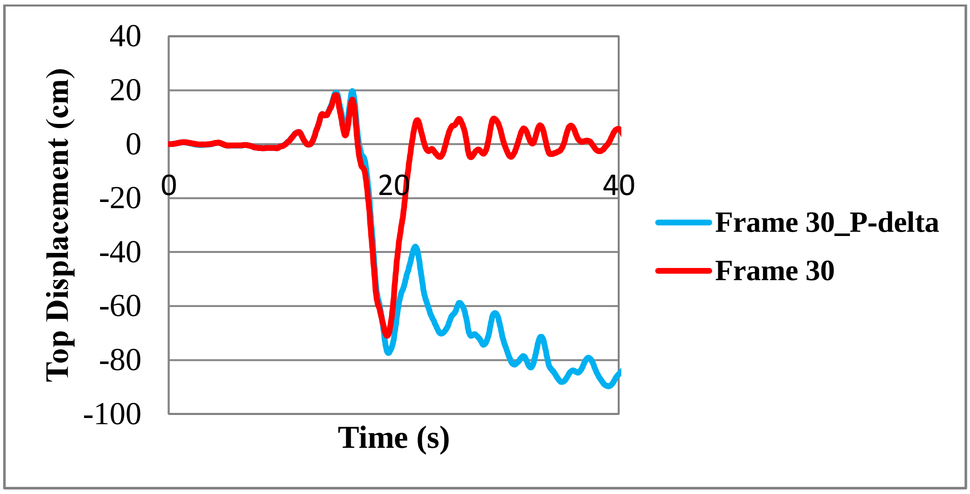
(d)


Twentieth Storey
40
20
0
-20
-40
-60
-80
-100
0
10
20
30
40
50
Displacement (cm)
Time (s)
Frame 30
Frame 30_P-delta

(e)



(f)

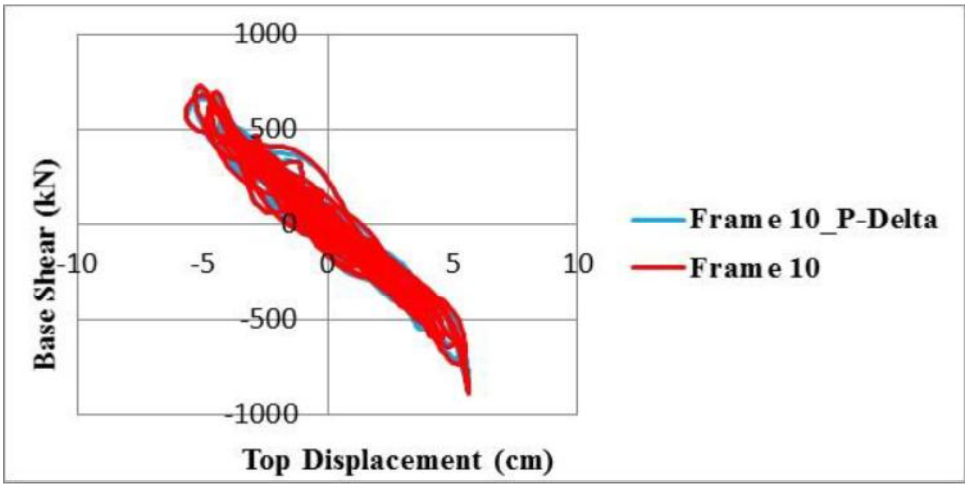


(g)

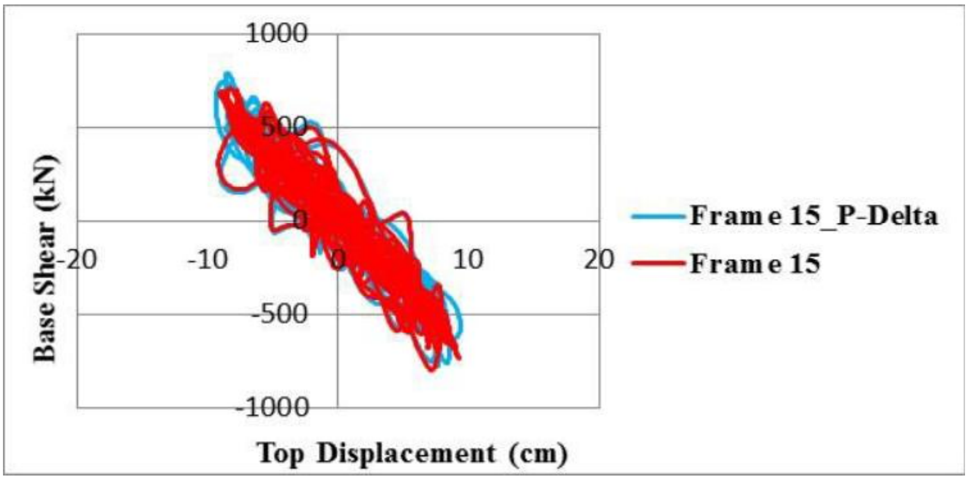
Figure 4.13 Variation in storey displacement with time for 30 storey frame under Landers earthquake : (a) first storey, (b) fifth storey, (c) tenth storey, (d) fifteenth storey, (e) twentieth story, (f) twenty-fifth storey, and (g) top displacement

4.4 Variation in the hysteretic curve (base shear vs. top displacement) in time history analysis

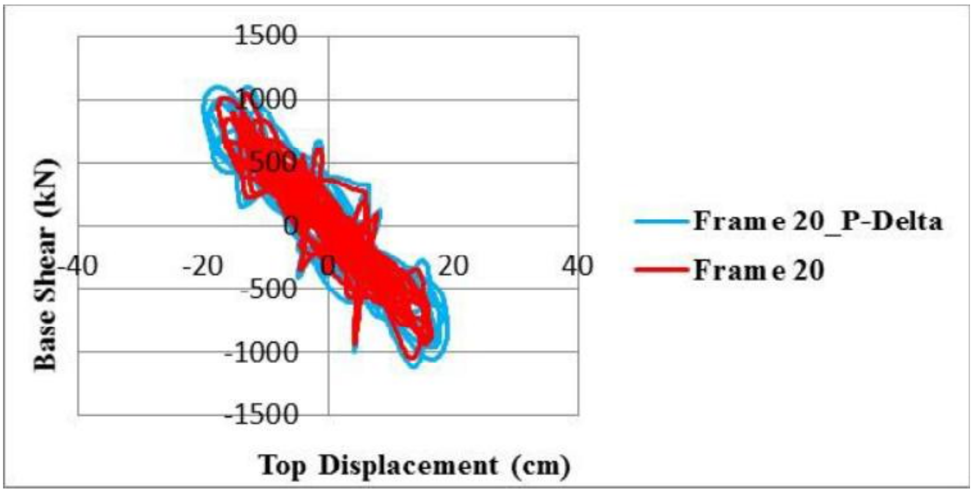
Another relationship type that indicates material nonlinearity is the hysteretic cycle**.** When the base shear and displacement relationship is developed for a component or system subjected to cyclic loading, hysteretic loops are produced. Hysteresis is useful for characterizing dynamic response under application of a time-history record. Depending on structural geometry and materials, a hysteretic cycle may follow one of many different possible patterns. Four possible hysteretic-behavior types are illustrated as non-degrading, stiffness degrading, pinched, and buckling. This method can be achieved with a high degree of accuracy for all the simulations. According to Figure 4.14, for five types of frames having 10, 15, 20, 25, and 30-storey, it was clear that consideration of P-delta effect under Imperial Valley earthquake had no direct effect in the ductility levels associated with the hysteretic loops. However, as expected, the shear demand of the structure was not significantly increased when considering P-delta effects in the analyses. On the other hand, the displacement of the structure had a tendency to increase, especially for higher storey frame structure. Also in Figures 4.15 and 4.16 under Superstition Hills and Landers earthquake, respectively, similar effect was observed for 30-storey frame after analyzing the P-delta effect.



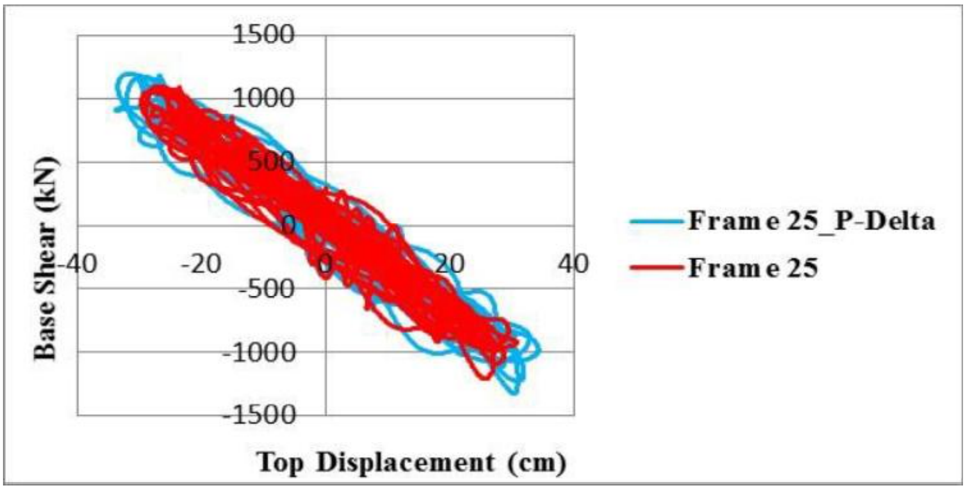
(a)



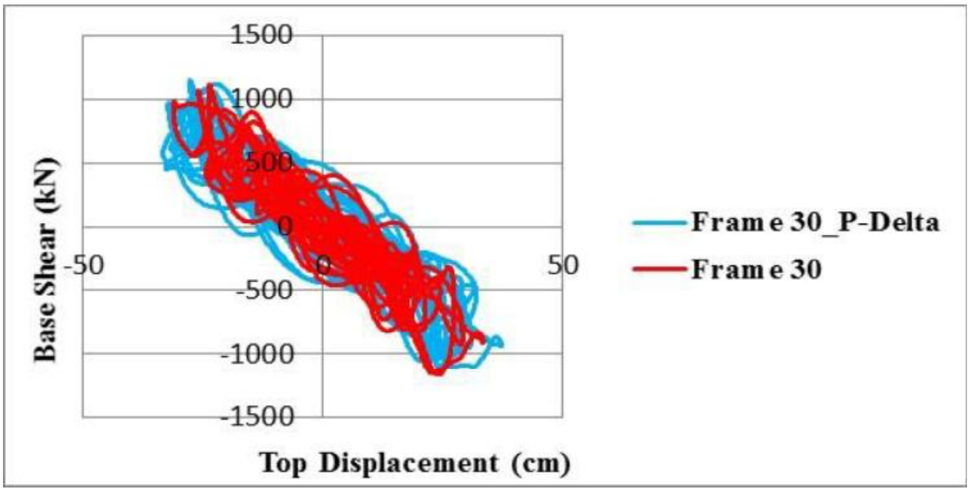
(b)



(c)



(d)



(e)

Figure 4.14 Hysteretic curve distribution of each type of frames and that with P-delta effect in (a) 10-storey, (b) 15-storey, (c) 20-storey, (d) 25-storey, and (e) 30-storey under Imperial Valley earthquake

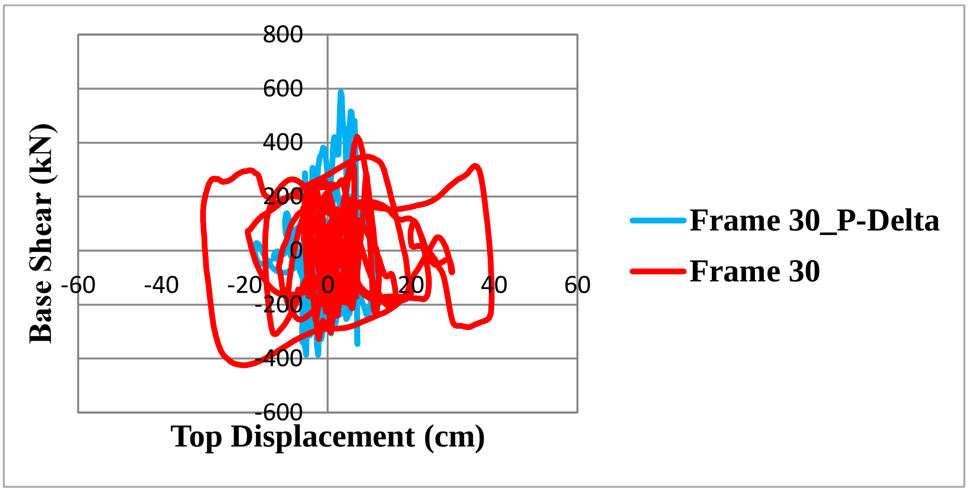


Figure 4.15 Hysteretic curve distribution of 30-storey frame with and without P-delta effect under Superstition Hills earthquake

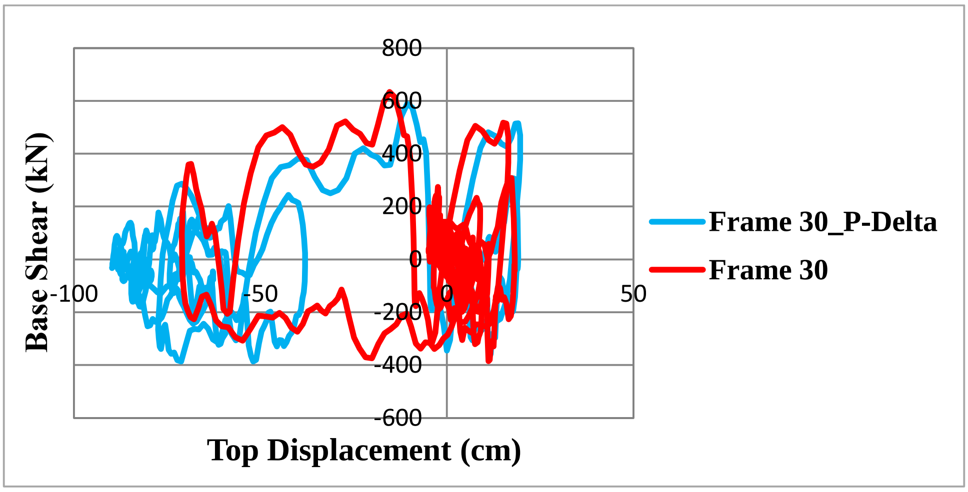


Figure 4.16 Hysteretic curve distribution of 30-storey frame with and without P-delta effect under Landers earthquake

CHAPTER 5

CONCLUSIONS

In this study, nonlinear time history analysis had been applied for five different case study structures. Those are 10, 15, 20, 25, and 30-storey structures having similar plan. As a seismic hazard level, 10% probability of exceedance in 50-year period was taken into consideration. Three types of ground motions such as Imperial Valley (1979), Superstition Hills (1987), and Landers (1992) earthquake accelerations were utilized. Comparing the results from dynamic analysis of this study, the following conclusions can be reached:

- It was observed that with the inclusion of P-delta effect in the analysis, the maximum storey displacement increased with increasing the number of storey of the frame. From the results, it was found that the maximum displacement for 30-storey frame with P-delta effect was 1.12, 1.13 and 1.22 times of that without P-delta effect under Imperial Valley, Superstition Hills and Landers earthquake accelerations, respectively. It was noted that the displacement under Landers earthquake was significantly affected with the consideration of P-delta effect.
- In the analysis of inter-storey drift ratio, the results showed marked increment in the drift ratio, especially in the top storey of frames with P-delta effect compared with original frames. In this study, the maximum inter-storey drift ratio of 30-storey frame with P-delta effect was achieved as about 0.59%, 1.4%, and 2.6% under Imperial Valley, Superstition Hills and Landers earthquakes. Nevertheless, the original 30-storey frame (without P-delta effect) under mentioned earthquakes was achieved as about 0.53%, 1.0% and 1.8%, respectively. It implied that the drift ratio depended mainly on the storey height, number of storeys, properties of earthquake acceleration.

- The frames with P-delta effect had higher roof displacement compared to those without P-delta.
- The analysis of the results also indicated that the displacement time history curves at top storey level were significantly influenced with P-delta in comparison to the other storeys.
- It was noted that the response of the case study structures with P-delta were affected with the properties of the ground motions used.

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