ABSTRACT

SUAREZ, VICTOR R. Evaluation of Displacement-Based Seismic Design of Reinforced Concrete Building Frames. (Under the direction of Dr. Mervyn Kowalsky).

In recent years, seismic design has been shifting towards Displacement-Based methods. Among the several methodologies that have been developed, the Direct Displacement-Based Design Method (DDBD) has been shown to be effective for performance-based seismic design of building structures and bridges.

The main objective of this dissertation is to close the gap between existing research on Direct Displacement-Based Design (DDBD) and its implementation for the design of conventional reinforced concrete (RC) frame structures.

The behavior of multi-story reinforced concrete (RC) frames structures has been studied under different intensity levels of real earthquake records to understand how these intensity levels affect the patterns of displacement and the lateral load distribution along the height of the building. In the present study, RC frames were designed using current US Codes and analyzed using nonlinear time history analysis (NLTHA) to evaluate whether the postulated performance criteria are met and to evaluate the displacement patterns and lateral load distribution pattern. This research presents a new methodology, or proposed modifications to the DDBD methodology, to account for the results obtained using NLTHA. The modifications include revised displacement patterns, lateral load distributions patterns, and a recommendation for the use of DDBD considering P-Δ effects. The research includes an
evaluation of the proposed methodology when irregular frames and P-Δ effects are considered for tall structures.

The results of the research show the effectiveness of the proposed displacement and the lateral load distribution patterns when compared to NLTHA. One of the advantages of the proposed displacement pattern is that it gives results that improve the NLTHA for all levels of earthquake intensity, especially in zones of low earthquake activity, where the existing DDBD procedure does not clearly present a solution. Another advantage of the proposed method is that it simplifies the use of DDBD through the use of one design displacement response spectrum—the displacement values obtained with the proposed method consider the displacement pattern shape and the earthquake intensity level in one equation. The proposed DDBD methodology is simpler in its use, can account for structural irregularities and P-Δ effects, and provides displacement patterns and lateral load distributions that closely match the NLTHA results obtained for RC frame structures.
Evaluation of Displacement-Based Seismic Design of Reinforced Concrete Building Frames

by
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DEDICATION

I dedicate this dissertation to my late father who did not get to see me finish my doctoral work, but wherever you are this is for you.

I also dedicate this work to my mother, (Mama Edith), my sisters and my six children Vanessa, Marielle, Nilsa Amelia, Victor Enmanuel, Victor Rafael and Nicole Victoria. Also to my good friend Ing. Orlando Franco. This dedication also goes to all of those people who were there for me during the time of work, thanks to all.
**BIOGRAPHY**

Victor Suarez was born in Santiago, Dominican Republic in January 15, 1956. In 1978, he received his BS in Civil Engineering at Pontificia Universidad Catolica Madre y Maestra in Santiago. In 1980, he received his master's degree in engineering from the University of California at Berkeley (UCB). From 1980-1991, Mr. Suárez was director of the engineering and real estate Department of J. Armando Bermudez CxA. At the same time 1980-1984, 1994-2004 he worked as a professor at the Pontificia Universidad Católica Madre y Maestra, teaching analysis and design of reinforced concrete structures and bridges. In 1997, he worked as Executive Director of the construction of the Aeropuerto Internacional del Cibao, responsible for its technical and financial implementation, design, construction and administration. Since 2002, he has been General Manager of the Aeropuerto Internacional del Cibao. In 2007, he received another master's degree in engineering from North Carolina State University (NCSU). His research interest includes but not limited to the seismic analysis and design of buildings, rehabilitation and retrofit of structures and forensic engineering. In 2008, he founded the “Instituto Dominicano de Ingeniería Superior y Desastres Naturales, "Vitelmo Bertero"”, Inc. In 2014 he obtained his doctoral degree (PhD) at North Carolina State University (NCSU).
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CHAPTER 1

Introduction and Research Motivation

1.1. Motivation for the study

Major earthquakes have recently resulted in a large loss of life and large property losses, examples of these earthquakes are Haiti January 12, 2010, Chile February 27, 2010, Christchurch New Zealand February, 22 and June 13, 2011, and Japan March 11, 2011 (EERI Special Earthquake Report, 2010, 2011). Typically, extent of the damage and destruction caused by earthquakes results from a combination of many factors, a) the severity of the ground shaking, b) the soil conditions at the site, c) the season and time of day when an earthquake occurs, d) the size and socioeconomic conditions of the affected population, and e) the size, type and seismic vulnerability of structures in the built environment.

While the threat of experiencing severe earthquake ground motions today is approximately the same as it was 100 years ago, the potential for a major earthquake catastrophe has grown alarmingly over this period of time as a result of the increase in population and urbanization in seismically active regions. Existing buildings vary from new buildings designed and built according to modern seismic design codes to very old buildings erected before the advent of earthquake regulations.

The past and current earthquake design requirements are a “force based design”, in which lateral forces are applied to the structure, and the structure is designed to resist these forces. The first U.S. earthquake design requirements appeared in the 1927 edition of the UBC (Uniform Building Code, 1927). However, these provisions were never put into effect in any city, in the U.S., and it was not until after the 1933 Long Beach earthquake that seismic design
requirements were enforced for the first time in the U.S. This means that most buildings built prior to 1933 were not designed to resist earthquake forces (Long Beach Earthquake, 1933).

Seismic design provisions in the United States have evolved from very simplistic recommendations to more reliable requirements which explicitly account for the seismicity of the site, local site conditions, the dynamic characteristics of the building, as well as the type of the structural system as shown in ASCE-7-10 (ASCE/SEI 7-10, 2010). Modifications to seismic requirements have been so rapid that many buildings that are only 20 years old, despite having been designed according to codes which included seismic provisions at the time of design lack modern ductile seismic detailing and in some cases may be potentially hazardous. The number becomes much greater if other types of potentially hazardous buildings (such as nonductile reinforced concrete frame buildings, or precast tilt-up buildings) are included.

In other earthquake prone countries of the world such as Chile, China, Afghanistan, Peru and the Dominican Republic the situation is similar, if not worse. In the case of the Dominican Republic, a major earthquake occurs approximately every 100 years. More than 60 years have passed since the occurrence of the last “big one”. It was not until 1946 that earthquake forces were taken into consideration for seismic design, when General Rafael Trujillo Molina, dictator of the country for 30 years, enforced the use of a building requirement that was appropriate for wind force design, although it had some effect in the seismic resistance of low rise building to earthquake forces. The first seismically oriented design code “Recomendaciones Provisionales para el Análisis Sísmico de Estructuras (DGRS, R079,1979 was proposed and adopted in December 1979 and a reasonable revision of the 1979 code “Reglamento para el Análisis y Diseño Sísmico de Estructuras” (DGRS, R001,2011) was
finished and published in 2011. In 2003, a moderate sized earthquake occurred in the central north part of the Dominican Republic, and damage to important structures such as school buildings was apparent; evidence that much work has to be done to improve the vulnerability of buildings in the Dominican Republic.

New construction in the U.S. adds less than 2 percent to the total building stock each year, so it will be a long time before the inventory of existing buildings reflects even the current knowledge of earthquake engineering. Therefore, one of the most effective ways of minimizing potential earthquake-related losses is to implement easy ways to conduct analysis and design and to upgrade structures identified as hazardous.

Ideally, seismic resistant structures are designed with a simple configuration, such that their behavior can be easily modeled and analyzed, with the objective that energy dissipation takes place in well-defined parts of the structure. In the seismic design of buildings, columns must be designed and detailed carefully to assure ductile behavior and displacement capacity which is in excess of seismic displacement demand.

Earthquake design at present is performed using reduced forces. This means that the real earthquake forces are higher than the building yielding forces. The designer relies on the fact that the displacement capacity of the building is higher than the displacement demand imposed by the earthquake and that the structure can accommodate these displacement demands without collapse. This implies that providing displacement capacity is more important than providing force capacity since structures are typically designed to yield.

Several Displacement-Based Design (DBD) methodologies have been proposed, among them, the Direct Displacement-Based Design Method (DDBD) (Priestley, 1993;}
Kowalsky, 1995; Calvi and Kingsley, 1997; Kowalsky 2002; Dwairi, 2005; Ortiz, 2006; Vinicio Suarez and Kowalsky, 2007) has proven to be effective for performance-based seismic design of bridges, buildings and other types of structures when compared with results of NLTHA.

Based on this experience, this work is a continuation of the development of the Direct Displacement Based Design (DDBD) process and tries to improve the method and its application to concrete buildings to make this tool easier to use by earthquake engineers.

1.2. Objectives

There are 3 main objectives: 1) to study the behavior of buildings designed using the current Direct Displacement-Based Design (DDBD) methodology; 2) to analyze the results obtained from NLTHA for different levels of earthquake intensity as it relates to the displacement patterns, interstory drifts, and force distribution patterns; and 3) to propose changes to the currently used displacement and force distribution patterns to obtain a method that will yield drifts and force distribution patterns that reflect those observed in the NLTHA. In particular this study addresses changes to the current method to account for the following issues:

a. Determine the impact of the level of intensity of the earthquake on the displacement and force distribution patterns for implementation in the Direct Displacement-Based Design.

b. Review the impact of the level of intensity of an earthquake on the displacement and force distribution patterns on the design of irregular reinforced concrete frame structures.
c. Review the impact of the level of intensity of an earthquake on the displacement and force distribution patterns when P-Δ effect is considered for tall buildings.

1.3. Scope

To achieve the above-stated objectives first it had to be understood what the state of the practice was and the research performed to date regarding design methodologies and their consideration of levels of earthquake intensity and P-Δ effects. To accomplish this objective the following tasks were undertaken:

a. A bibliography search of the past studies on design of reinforced concrete buildings, with emphasis on those designed with the latest building code IBC 2006.

b. Review and understand of the Force Based Design (FBD) from ASCE-7-05 and IBC 2006.

c. Review DDBD procedures proposed by Priestley, Kowalsky and Calvi, with the objective of implementing this procedure for reinforced concrete frame structures.

To evaluate the current DDBD procedure and to formulate the new proposed design procedure these tasks were undertaken:

d. Perform nonlinear time history analysis (NLTHA) to understand the behavior of regular and irregular frames to a selection of earthquakes with different levels of excitation along with analysis and interpretation of the results.

e. Perform analyzes and interpret the results of NLTHA considering the P-Δ effect for buildings and to evaluate the effect of this P-Δ on the displacement and lateral load distributions patterns especially for tall buildings.
f. Implement new proposed methodology, or proposed modifications to the DDBD methodology to account for the results obtained using NLTHA and P-Δ effect. This task includes formulation of modified displacement and lateral load distributions patterns, and a recommendation of how to use these new patterns on design performed with DDBD.

1.4. Organization of this Dissertation

Chapter 1 presents the motivation, objectives and scope and organization of the current study.

Chapter 2 presents the results of a literature search of the methodologies available for the seismic design of existing buildings, review of the fundamentals of Direct Displacement-Based Design (DDBD), and review of past studies of the DDBD method including the P-Δ effect.

Chapter 3 presents the impact of the level of intensity of an earthquake on the behavior of regular reinforced concrete frame structures using NLTHA of buildings designed using the current DDBD method, it includes analysis and discussion of the existing displacement patterns and the existing lateral load distributions presented in the current DDBD method when compared to the NLTHA results.

Chapter 4 presents parametric studies to evaluate the influence of several parameters on the drift profiles.

Chapter 5 presents the new proposed displacement pattern and the new proposed lateral load distribution along the height of the structure.
Chapter 6 presents the results of seismic evaluations using NLTHA on the designs made using the newly proposed displacement and lateral load distributions patterns in regular and irregular reinforced concrete frame structures. This chapter also includes analysis and interpretation of the NLTHA results.

Chapter 7 presents the results of seismic evaluations of tall buildings using NLTHA considering P-Δ effects on the designs made using the newly proposed displacement and lateral load distributions patterns and the new proposed equation for the consideration of P-Δ.

Chapter 8 presents conclusions and recommendations for the use of Direct Displacement-Based Design (DDBD) method in reinforced concrete frame structures based on the results presented in Chapters 3, 4, 5, 6 and 7.
CHAPTER 2

Review of Methodologies and Concepts

2.1. Introduction

A literature review of the history of design using displacement starts in 1960 with Muto et al (1960) and Veletsos and Newmark (1960) when it was postulated that the maximum inelastic displacement of a single degree of freedom yielding structure was similar to the elastic displacement of a system having the same initial period and damping and that the maximum displacement could be used as a design parameter.

In 1974 Gulkan and Sozen (1974) defined the secant stiffness method to analyze the behavior of inelastic structures using secant stiffness and increased damping to find the inelastic displacement response and to compare this displacement with the displacement capacity of the structure.

In 1976 Shibata and Sozen (1976) presented a method to analyze an equivalent linear system called the substitute structure that represented the nonlinear system with a secant stiffness, in this method the substitute stiffness is based on the secant stiffness at the displacement capacity of the structure, using modal analysis the properties of the multi-degree of freedom (MDOF) structure can be transformed to a SDOF structure and analyzed and the maximum displacement demand can be obtained and compared with the displacement capacity of the system.

In 1986, Freeman in the Triservice manual TM-809-10-1 (1986) presented a procedure that in 1990 was modified and called the capacity displacement spectrum method in which the response spectra is plotted in the format of acceleration-displacement representing the demand
of the earthquake. The capacity of the structure is found with a pushover analysis, and it is converted to a SDOF system using modal analysis and plotted as a force displacement graph. This graph (capacity) is then compared in the same plot to the acceleration-displacement spectra (demand). Using this method the displacement demand can be obtained at the intersection of the demand and capacity curves and compared with the displacement capacity of the structure.

In 1991 Qi and Moehle (1991) presented a method to estimate the inelastic displacement response of a SDOF system for two period ranges, they provide a formula to calculate the inelastic displacement when the period of the structure is less than a certain Tg, for periods larger than Tg the inelastic displacement is the same as the elastic displacement as concluded by previous authors. Later in the report the authors provide a method to convert the properties of a Multi Degree Of Freedom (MDOF) system to a SDOF system using equivalent mass, equivalent damping and equivalent stiffness.

Later Priestley, Kowalski, and MacRae (1995) present a very comprehensive method for displacement based design for SDOF. An extension of this method by Priestley and Kowalsky for MDOF systems was developed during the 1990’s (Priestley ,1992, Priestley and Calvi ,1997, Priestley and Kowalsky, 2000), and presented in the year 2000 as “Direct Displacement Based Design” (DDBD) (Priestley and Kowalsky, 2000) which will be presented in detail later in this chapter.

2.2. Review of Force Based Design (FBD)

The FBD is the procedure used by current codes in seismic design. In this procedure, the earthquake excitation is an elastic response spectrum computed for 5% damping for most
types of building. The response spectrum is divided by a reduction factor R depending on the type of structure. Modal analysis is used to compute the main periods of the structure and their contribution to seismic response. The member forces obtained from this analysis or from a simplified procedure called the Equivalent Lateral Load procedure (see appendix A) are the seismic demands used for the design and the designer shall provide member capacities greater than the demands. Since the members are designed for lower forces than the earthquake demand, the members will yield. Special detailing shall be provided to ensure that the members can accommodate the displacement demands imposed by the earthquake. A check of the deflections is performed against allowable limits. A more detailed presentation of the FBD is presented below:

a) A preliminary design based on structure geometry, gravity loads and experience is used to define the member sizes such as column heights, inertia masses and design spectrum, this will be the first estimate of the member sizes.

b) Estimate member stiffness based on the size estimates obtained in step one. Reduced stiffness may be used for some elements to account for member softening.

c) Modal analysis is performed to obtain significant periods and participations factors to apply response spectrum method.

d) Using the response spectrum method, participating accelerations will be obtained as shown in Figure 2.1 a)

e) Elastic base shear is determined, and this shear is reduced by a factor R that depends on the type of structure as shown in Figure 2.1 b).
Figure 2.1 Force Base Design a) 5% Acceleration Spectrum b) Equal displacement approximation (Veletsos and Newmark, (1960))

f) Compute inertial forces based on the base shear obtained above or use the Equivalent Lateral Load procedure (see appendix A). Analyze the structure using this inertial forces to obtain the member forces.

g) Design the members to ensure that the axial and bending capacities are greater than the demands. Capacity design principles are used making sure that yielding occurs in the girders and not in the columns (strong column-weak girder principle) and also for shear design of the elements.

h) Comply with allowable code displacement limits, if the displacements are larger than the allowable, the structure shall be modified and reanalyzed until compliance. Appendix A shows a detailed presentation of the Force Based Design Method using the Equivalent Lateral Force procedure.
2.3. Review of DDBD

Displacement Based Design (DBD) has been used extensively lately, as it addresses several shortcomings of the conventional Force Based Design (FBD) procedure; it has been used as a tool for performance-based seismic engineering. The main difference between DBD and FBD is that the first uses displacement as a measure of seismic demand and as an indicator of damage in the structure. DBD uses the fact that displacement correlates better with damage than force levels. DBD solves serious problems of FBD such as ignoring the proportionality between strength and stiffness and the generalization of ductility capacity through the use of force reduction factors (Priestley et al, 2007).

Several Displacement Based Design (DBD) methodologies have been developed among which is the Direct Displacement Based Design Method (DDBD) (Priestley, 1993).

DDBD and the other procedures show similarities such as:

• The use of displacement as a measure of damage and seismic demand.
• They require the computation of specific values of ductility for each structure. As opposed to the FBD approach in which ductility capacity is specified using force reduction factors.
• Capacity Design principles such as strong columns-weak girders and shear design using the moment capacity of the elements are used to assure damage will only occur in predefined locations.

Although they are very similar, DDBD and the other DBD methods differ as follows:
• DDBD goes directly from target displacement to required strength, while the other methods require an iterative process in which strength is assumed and then displacement demand is checked against displacement capacity.

• DDBD uses equivalent linearization, while the other methods use displacement modification, which leads to the use of the equal displacement approximation (Newmark and Hall, 1982) for systems in a constant velocity region of the design spectra.

2.4. Direct Displacement Based Design (DDBD)

The Direct Displacement Based Design Method can be used for performance-based design of bridges, buildings and other types of structures (Priestley et al, 2007). The direct displacement-based design (DDBD) method uses the secant stiffness and the equivalent damping approach to define the structure as an equivalent linear single-degree-of-freedom (SDOF). The DDBD method designs the structure to achieve a selected performance limit state under the earthquake excitation. The procedure uses capacity design principles to ensure that plastic hinges form where intended, (strong column-weak girders) and to avoid any non-ductile behavior such as shear failures.

For multi-degree-of-freedom (MDOF) structures, the DDBD converts the structure into an equivalent SDOF system with an equivalent mass \( (M_{eq}) \) and equivalent force \( (F_{eq}) \) based on equal work done by both systems, (See Figure 2.2). The substitute structure approach developed by Sozen and Shibata (1976) is used for the conversion. The SDOF hysteretic behavior is formulated using the secant stiffness at peak response \( (K_{eff}) \) and an equivalent
damping proportional to equate the energy dissipated, as shown in Figure 2.2b. Equivalent
damping is a simplified way to account for the effect of dissipating energy on the nonlinear
response of the structure. Damping is determined as a function of displacement ductility of the
structure; as shown in Figure 2.3a for different types of structural members.

The design response spectrum is reduced using a damping reduction factor. The
displacements corresponding to, $\xi$, damping ($S_{d, \xi\%}$) can be obtained from the displacements
for 5% damping ($S_{d, 5\%}$) using the EuroCode 8 (1988) formulations shown in Equations 2.4-1
for far field records and Equation 2.4-2 for near field records. Using the target displacement
($\Delta_d$) obtained from the desired performance of the structure and reduced response spectrum,
the equivalent SDOF structure effective period ($T_{eff}$), can be obtained as shown in Figure 2.3b,
with this effective period the effective stiffness ($K_{eff}$) and the design base shear ($V_B$) is
computed as shown in Equation 2.4-12 and Equation 2.4-13, respectively. The design base
shear is distributed along the height of the structure in accordance with Equation 2.4-14 and to
the structural elements in accordance with an elastic analysis. The structure is designed to
ensure that the axial and bending capacities are larger than the demands and using capacity
design principles as explained above.
Figure 2.2 Equivalent SDOF Structure Characterization (Shibata and Sozen, 1976)

\[
S_d = \frac{S_{d5\%}}{\sqrt{2+\frac{7}{2+\xi}}} \\
S_d = \frac{4S_{d5\%}}{2+\xi}
\]

Equation 2.4-1

Equation 2.4-2

Figure 2.3 Obtaining Equivalent SDOF Structure Effective Period (Priestley, Calvi and Kowalsky, 2007)
Advantages of the Direct Displacement Base Design over Force Base Design include:

- It is performance based (limit strains and drifts)
- It is performance driven, not analysis driven
- It covers a wide range of structural system types such as Concrete, Steel, Masonry, Timber, and Precast
- It is a complete re-evaluation of capacity-design method

Problems with Force Base Design include:

- Structural damage (performance level) is related to strain
- Non-structural damage is generally related to drift
- Strain and drift can be integrated to give displacement
- Performance levels can thus be related to displacement
- There is no simple relationship between performance level and strength
- Interdependency of strength and stiffness
- Period calculation related to elastic stiffness
- Ductility capacity and force-reduction factors are empirical relationships
- No explicit ductility of structural systems
- Buildings with unequal wall lengths use the same force reduction factor not considering the different behavior of each element
- Relationship between elastic and inelastic displacement demand is modified with empirical factors

2.5. Overview of DDBD for MDOF systems (Priestley, Calvi and Kowalsky, 2007))

For multi degree of freedoms (MDOF) buildings the DDBD converts the system to an equivalent single degree of freedom (SDOF) using a procedure based on fundamental inelastic mode as shown in the following steps:

1. Determine displaced shape, and characteristic displacement
\[
\Delta_i = \delta_i \cdot \left(\frac{\Delta_{ic}}{\delta_i}\right) \\
\delta_i = H_i / H_n \text{ for } N \leq 4 \\
\delta_i = \frac{4}{3} \left(\frac{H_i}{H_n}\right) \left(1 - \frac{H_i}{4H_n}\right) \text{ for } N > 4
\]

Equation 2.4-3
Equation 2.4-4
Equation 2.4-5

Where:
\( \Delta_i \) = Design Story displacement
\( \delta_i \) = Normalized inelastic mode shape
\( N \) = Number of stories of the building
\( H_i \) = Height at story \( i \)
\( H_n \) = Total height of the building

2 Determine equivalent SDOF Design Displacement
\[
\Delta_d = \sum_{i=1}^{n} \left( m_i \Delta_i^2 \right) / \sum_{i=1}^{n} \left( m_i \Delta_i \right)
\]

Equation 2.4-6

3 Determine Effective Mass (Equivalent Mass)
\[
m_e = \sum_{i=1}^{n} m_i \Delta_i / \Delta_d \]

Equation 2.4-7

4 Determine Effective Height
\[
H_e = \sum_{i=1}^{n} m_i \Delta_i H_i / \sum_{i=1}^{n} m_i \Delta_i
\]

Equation 2.4-8

5 Determine Displacement ductility
\[ \mu = \frac{\Delta_d}{\Delta_y} \]  

Equation 2.4-9

Where

\[ \Delta_y = \theta_y H_e \]  

Equation 2.4-10

\[ \theta_y = 0.5 \varepsilon_y \frac{L}{h_b} \]  

for concrete frames

Equation 2.4-11

6 Determination of Equivalent Viscous Damping

The equivalent damping, \( \xi_{eq} \), for different types of structural elements such as RC beams, unbonded-posttensioned walls, steel members, drilled shafts and piles and isolation/dissipation devices can be obtained using studies performed by Dwari (2005), Blandon (2004) and Suarez (2007). Table 2.4.1 show some of these models where \( \xi_{eq} \) is the viscous damping in the elastic range; \( r \) is the ratio between second and first slopes in a bilinear force-deformation response; \( \mu \) is the ductility demand, \( T_{eff} \) is the effective period that corresponds to secant stiffness and \( a,b,c,d \), and \( A,B,C,D,G,H,I,J \) are coefficients that depend on the type of hysteresis model used.

The linearization of the response of inelastic SDOF using a combination of period and equivalent damping can be performed using the models in Table 2.4.1. ATC-40 (1996) utilizes a model in which the area of the hysteresis loop is used to compute the equivalent damping, as first proposed by Jacobson (1930). In the work done by Dwairi (2005), it has been shown that when utilizing effective period based on secant stiffness to maximum response, the area approach overestimates the equivalent damping. Dwairi (2005), obtains hysteretic damping by
determining the value of damping to be combined with secant stiffness such that the resulting response matches accurately the results of NLTHA. Blandon and Priestley (2004) and Vinicio Suarez and Kowalsky (2007) have used a similar approach to obtain models that can be used with DDBD.

7  Determine Effective Period for Equivalent SDOF Structure
    Use the displacement spectra in (Figure 2.3b) to find the Effective Period (T\text{eff})

8  Determine the Effective Stiffness of the Equivalent SDOF Structure

\[ K_e = 4\pi^2 \frac{m_e}{T_e^2} \quad \text{Equation 2.4-12} \]

9  Determine the Design Base Shear Force

\[ F = V_e = K_e \Delta_e \quad \text{Equation 2.4-13} \]

10 Distribute the Base shear to the mass locations

\[ F_i = V_i \frac{\Delta_e}{\sum m_i \Delta_i} \quad \text{Equation 2.4-14} \]

11 Analyze the structure to determine moments at plastic hinges

Figure 2.4 below shows the flowchart for the general design procedure using DDBD and Figure 2.5 shows the flowchart for DDBD for concrete buildings.
Table 2-4-1 Models for Equivalent Linearization (From Suarez Vinicio, 2007)

<table>
<thead>
<tr>
<th>Source</th>
<th>Model</th>
<th>Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Blandon, 2004</td>
<td>[ \begin{align*} \xi_{\text{eff}} &amp;= \frac{a}{\pi} \left( 1 - \frac{1}{u^b} \right) \left( 1 + \frac{1}{(T_{\text{eff}} + c)^d} \right) \frac{1}{N} \ T_{\text{eff}} &amp;= T_0 \sqrt{\frac{\mu}{1 + \alpha \mu - \alpha}} \end{align*} ]</td>
<td>RC Columns</td>
</tr>
<tr>
<td>Dwairi, 2005</td>
<td>[ \begin{align*} \xi_{\text{eff}} &amp;= \xi_v + C_{ST} \left( \frac{\mu - 1}{\mu \pi} \right) % \ C_{ST} &amp;= 50 + 40(1 - T_{\text{eff}}), \ T_{\text{eff}} &lt; 1 \ C_{ST} &amp;= 50, \ T_{\text{eff}} &gt; 1 \ T_{\text{eff}} &amp;= T_0 \sqrt{\frac{\mu}{1 + \alpha \mu - \alpha}} \end{align*} ]</td>
<td>RC Columns</td>
</tr>
<tr>
<td>FEMA-440</td>
<td>[ \begin{align*} \xi_{\text{eff}} &amp;= A(\mu - 1)^2 + B(\mu - 1)^3 + \xi_v \ T_{\text{eff}} &amp;= \left[ G(\mu - 1)^2 + B(\mu - 1)^3 + 1 \right] T_0 \ \text{If } 1 &lt; \mu &lt; 4 : \ \xi_{\text{eff}} &amp;= C + D(\mu - 1) + \xi_v \ T_{\text{eff}} &amp;= \left[ I + J(\mu - 1) + 1 \right] T_0 \ \text{If } 4 \leq \mu \leq 6.5: \end{align*} ]</td>
<td>Stiffness degrading systems</td>
</tr>
<tr>
<td>ATC-40</td>
<td>[ \begin{align*} \xi_{\text{eff}} &amp;= 0.05 + \frac{2}{\pi} \frac{(\mu - 1)(1 - \alpha)}{\mu(1 + r \mu - \alpha)} \ T_{\text{eff}} &amp;= T_0 \sqrt{\frac{\mu}{1 + r \mu - \alpha}} \end{align*} ]</td>
<td>Bilinear Response</td>
</tr>
<tr>
<td>Vinicio Suarez, 2007</td>
<td>( \xi_{\text{eff}} = \xi \mu^{-0.376} + 13.7 + 10.9 \frac{\mu - 1}{\pi \mu} )</td>
<td>Drilled Shafts in soft clay</td>
</tr>
<tr>
<td>----------------------</td>
<td>-----------------------------------------------</td>
<td>-----------------------------</td>
</tr>
<tr>
<td></td>
<td>( T_{\text{eff}} = T_{o} \sqrt{\frac{\mu}{1 + \alpha \mu - \alpha}} )</td>
<td></td>
</tr>
</tbody>
</table>
Figure 2.4 Structural Design DDBD Modified after FEMA 451 (FEMA 451, 2006)

*Sec. 3.3.4 of the 2003 Provisions defines reduced spectral ordinates for periods greater than T1.

\[ \xi = \text{Damping Coefficient Ratio} \]

Near Field Source

Use Eq’n

\[ S_d = S_{25}\left( \frac{7}{2 + \xi} \right) \]

Far

Use Eq’n

\[ S_d = S_{25}\left( \frac{7}{2 + \xi} \right) \]

Go to Chart 2.4 for structural requirements.
Figure 2.5 DDBD Procedure for Concrete buildings
2.6. Review of literature of the effects of P-Δ on DDBD

Several authors such as Andrews (1977), Paulay (1978), Moss and Carr (1980), Montgomery (1981), have studied the problem of when P-Δ effect can be ignored.

Paulay (1978) and later Priestley (2003) recommended and increase in the strength of reinforced concrete structures to take into account the P-Δ effect and suggested value for which P-Δ effect could be ignored.

Neus et al. (1993) recommends factoring the geometric stiffness matrix to take into account the P-Δ effect, Bernal (1987) investigated this approach and found it unconservative.

MacRae et al (1990) used the substitute structure method considering that the ratio of the secant stiffness to peak response could be included using the ratios of response of structures with and without P-Δ effect.

Paulay and Priestley (1992) studied the P-Δ effect considering energy dissipation levels and limits to the application of P-Δ effect.

In general three main procedures have been used to account for the P-Δ effect:

- **MacRae's Method (1994):** Hysteresis Center Curve Concept, MacRae in this study, investigates the effect of hysteresis curves on P-Δ effect.

A more detailed literature review of the inclusion of the P-Δ effect in DDBD is presented by Pettinga and Priestley (2007).

2.7. Result of literature review

From the literature review performed above on DDBD without P-Δ effect, it can be concluded that none of the authors have studied the effect of the intensity of the earthquake on the deflected shape pattern of the structures. Since the deflected shape pattern is one of the
main parameter used on DDBD, it is very important to study what is the effect of different levels of intensities when considering different shape patterns on the design of the structure.

The current approach considers that the deflected shape is constant at all levels of intensity; this study will investigate what is the effect of different levels of intensity on the displacement shape pattern.

Review of the literature on P-Δ effect with regard to DDBD, also shows that the different levels of earthquake intensities was not considered. Since the deformed shape pattern is important when considering P-Δ effect, this study will also investigate the effect of levels of intensity on the deflected shapes with P-Δ.
CHAPTER 3

Modeling and Analysis of Reinforced Concrete Building Frames

3.1 Introduction

An initial compilation of earthquake damage surveys (Coburn 1986) indicated that significant variations existed between the interpretation and classification of intensity levels from one survey group to another, but that damage state distributions in particular building types and the relative damage levels between two building types were consistent. Since the damage to structures is related to the intensity of the earthquake, it is reasonably understood that building behavior and displacement pattern profiles will be related to the intensity of the ground motion.

With the introduction of performance-based earthquake engineering (PBE) in the 1990’s, the need for a comprehensive and simple seismic design approach was needed. This procedure should allow the designer to control the structure displacement profile, and then control damage for different performance limit states and earthquake intensities. One of those approaches is the DDBD method. In the DDBD method, a structure is designed such that a predefined displacement limit is achieved when the structure is subjected to a predefined earthquake that is consistent with that assumed for the design. The DDBD procedure for MDOF structures starts with selecting a displacement profile pattern to obtain a target displacement that corresponds to the desired level of damage. An equivalent linear SDOF structure is then characterized by the secant stiffness to maximum response and equivalent viscous damping. The required effective period of the equivalent structure is then determined using the elastic design spectra reduced based upon the equivalent damping value.
Given the expectation that damage will occur in moderate to large earthquakes, it is logical that the design methodology that will be employed should (1) directly address the issue of inelastic response, and (2) provide a method for controlling the amount of damage which occurs. However, current design approaches, which are predominantly force-based in nature, cannot reliably meet these needs mainly because force magnitudes are not good indicators of potential damage.

It is clearly understood that for “normal” loads, such as dead and live loads, design strength capacity is higher than design load demand (no damage under design loads). For seismic actions, design strength capacity is lower than elastic design forces (damage is expected under the design earthquake). Once the structure yields, damage will depend on the deformation produced by the lateral movement, so the most important parameter for design should be drift control or displacement control. Since performance levels are usually described in terms of displacement, it is better to have a design method that considers displacements rather than forces in the beginning of the design. If the nature of the damage is specified for a given seismic event, it is necessary to specify deformations for which the structure should be designed. Deformation quantities such as material strains are much more reliable indicators of performance than forces. Strains can be correlated to inelastic displacements, which can then be used in the DDBD method. The mechanisms by which a complex structure deforms inelastically must first be understood before inelastic displacement can be specified. As a result, as stated before, the primary objective of this research is to develop improved new tools, specifically methods for establishing inelastic displacement patterns for multi-degree of
freedom structures that are related to the intensity of the earthquake that can then be applied in a direct displacement-based seismic design approach.

The research in this dissertation aims to: (1) Identify the types of displacement patterns typically encountered in building design, as a function of the intensity of the ground motion, (2) identify when such patterns are likely to occur, and (3) apply the results to DDBD while demonstrating its application and providing a suite of analysis results for verification. This will simplify the process used in an iterative approach. The new postulated method will use a unique displacement response spectrum for all cases in which the structures displacement varies as a function of the severity of the ground motion.

3.2 Earthquake Records Definition

Medina and Krawinkler (2003) selected 80 recorded ground motions from Californian earthquakes of moment magnitude between 5.8 and 6.9 and closest distance to the fault rupture between 13 km and 60 km. These records were classified into four magnitude-distance categories for the purpose of performing statistical evaluation and regression analysis. These categories are shown in Table 3.1.

Twenty near field records of those 80 recorded ground motions used by Medina and Krawinkler (2003) were selected for this research, they correspond to the LMSR (Large Magnitude Short Distance) used by them in Table 3.1. These records were selected based on the recommendations of Dr. Eduardo Miranda since he was very involved with the record selection of Medina and Krawinkler (2003) research. The records are heavily biased towards
Loma Prieta and Northridge records, but they are a reasonable representation of records with large magnitude and a short distance from the fault. These records are shown in Table 3.2.

Table 3.3 shows 5 far field records LMLR (Large Magnitude Long Distance) used to evaluate the influence of far field records on the displacement shape profile of the structure. These ground motions were recorded on NEHRP site class D (Medina and Krawinkler, 2003). The records were obtained from the Pacific Earthquake Engineering Research Center (PEER) Ground Motion Database (http://peer.berkeley.edu/smcat/).

<table>
<thead>
<tr>
<th>Table 3-1 Earthquake Classifications from (Medina and Krawinkler, 2003)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>LMSR (Large Magnitude Short Distance)</strong></td>
</tr>
<tr>
<td>(20 Records)</td>
</tr>
<tr>
<td>(6.5 ≤ Mw ≤ 7.0) (13 km. ≤ R ≤ 30 km.)</td>
</tr>
<tr>
<td><strong>LMLR (Large Magnitude Long Distance)</strong></td>
</tr>
<tr>
<td>(20 Records)</td>
</tr>
<tr>
<td>(6.5 ≤ Mw ≤ 7.0) (30 km. ≤ R ≤ 70 km.)</td>
</tr>
<tr>
<td><strong>SMSR (Small Magnitude Short Distance)</strong></td>
</tr>
<tr>
<td>(20 Records)</td>
</tr>
<tr>
<td>(5.5 ≤ Mw ≤ 6.5) (13 km. ≤ R ≤ 30 km.)</td>
</tr>
<tr>
<td><strong>SMLR (Large Magnitude Short Distance)</strong></td>
</tr>
<tr>
<td>(20 Records)</td>
</tr>
<tr>
<td>(5.5 ≤ Mw ≤ 6.5) (30 km. ≤ R ≤ 70 km.)</td>
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<td>EQ ID</td>
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<td>EQ01</td>
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<td>EQ02</td>
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<td>EQ03</td>
</tr>
<tr>
<td>EQ04</td>
</tr>
<tr>
<td>EQ05</td>
</tr>
<tr>
<td>EQ06</td>
</tr>
<tr>
<td>EQ07</td>
</tr>
<tr>
<td>EQ08</td>
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<tr>
<td>EQ09</td>
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<tr>
<td>EQ10</td>
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<td>EQ11</td>
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<td>EQ12</td>
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<td>EQ13</td>
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<td>EQ14</td>
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<td>EQ15</td>
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<td>EQ16</td>
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<td>EQ17</td>
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<tr>
<td>EQ18</td>
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<tr>
<td>EQ19</td>
</tr>
<tr>
<td>EQ20</td>
</tr>
<tr>
<td>EQ ID</td>
</tr>
<tr>
<td>-------</td>
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<td>EQ21</td>
</tr>
<tr>
<td>EQ22</td>
</tr>
<tr>
<td>EQ23</td>
</tr>
<tr>
<td>EQ24</td>
</tr>
<tr>
<td>EQ25</td>
</tr>
</tbody>
</table>
3.3 Plots of acceleration, velocity and displacement time history

Figures 3.1 and 3.2 show examples of 2 records, a) the Loma Prieta Agnes Hospital a LMSR record referred in table 3.2 as EQ1 and b) El Centro Array #9 a LMLR referred in Table 3.3 as EQ21.

Figure 3.1 Example of time history of Loma Prieta Agnews ST Hospital a near field record
Figure 3.2 Example of time history of El Centro array #9 a far field record
3.4 Earthquakes scaling

The objective of scaling the time history records representing the earthquakes was to match their acceleration response spectrum to the design acceleration response spectrum used for the structures under study. These matched records were considered as records at 100% intensity for the purpose of this research. Two procedures were used to scale the earthquakes:

- Scaling at the periods of interest, from 0.5 $T_e$ to 1.5 $T_e$, where $T_e$ is the effective period of each structure under study.
- And modify the earthquake time histories to match the design response spectrum over the entire period range of the spectrum, using the spectrum matching wavelet procedure by Montejo (2004).

3.4.1 Scaling at period of interest

The scaling of the earthquake records at the period of interest was accomplished as follows:

- For each earthquake compute the area under the acceleration response spectrum curve for the periods of interest and divide by $T_e$, this ratio would be called factor F1. This procedure will yield 20 F1 factors for each $T_e$. See Figure 3-3 for an example.
- Compute the area under the design spectrum curve for the periods of interest and divide by 2 $T_e$, this ratio would be called factor F2. This procedure will yield one F2 factor for each $T_e$. See Figure 3-4 for an example.
- All of the 20 F1 factors are then divided by a factor F2 for the particular T_e. This will yield 20 F2/F1 scale factors.
- Each of the 20 time histories will be scaled by the corresponding scale factor.
- The procedure above will be repeated for each T_e, where T_e depends on the type of structure and number of stories.

Figure 3.3 Area under the acceleration response spectrum for determining factor F1
Figure 3.4  Area under the acceleration response spectrum for determining factor F2

Table 3.4 shows the scaled factors for the twenty LMRS records using the procedure scaling at the period of interest.

<table>
<thead>
<tr>
<th>Record Name</th>
<th>File Name</th>
<th>Scale Factor 4</th>
<th>Scale Factor 8</th>
<th>Scale Factor 12</th>
<th>Scale Factor 20</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Loma Prieta Agnews ST Hospital</td>
<td>LPSTH.eqp</td>
<td>3.05</td>
<td>2.60</td>
<td>2.25</td>
<td>2.38</td>
</tr>
<tr>
<td>2. Loma Prieta Capitola.</td>
<td>LPC.eqp</td>
<td>2.52</td>
<td>4.10</td>
<td>5.81</td>
<td>8.72</td>
</tr>
<tr>
<td>3. Loma Prieta Gilroy #3</td>
<td>LPG3.eqp</td>
<td>1.16</td>
<td>1.34</td>
<td>1.99</td>
<td>3.83</td>
</tr>
<tr>
<td>4. Loma Prieta Gilroy #4</td>
<td>LPG4.eqp</td>
<td>1.63</td>
<td>2.49</td>
<td>3.60</td>
<td>5.97</td>
</tr>
<tr>
<td>5. Loma Prieta Gilroy #7</td>
<td>LPG7.eqp</td>
<td>7.92</td>
<td>9.07</td>
<td>11.73</td>
<td>17.31</td>
</tr>
<tr>
<td>6. Loma Prieta Hollister City Hall</td>
<td>LPHCH.eqp</td>
<td>1.62</td>
<td>1.56</td>
<td>1.48</td>
<td>1.60</td>
</tr>
<tr>
<td>7. Loma Prieta Hollister Differ Array</td>
<td>LPHDA.eqp</td>
<td>1.99</td>
<td>1.78</td>
<td>1.78</td>
<td>2.25</td>
</tr>
<tr>
<td></td>
<td>Location</td>
<td>Record</td>
<td>Factor 1</td>
<td>Factor 2</td>
<td>Factor 3</td>
</tr>
<tr>
<td>---</td>
<td>---------------------------------</td>
<td>--------</td>
<td>----------</td>
<td>----------</td>
<td>----------</td>
</tr>
<tr>
<td>8</td>
<td>Loma Prieta Sunnyvale Colton Ave.</td>
<td>LPSCA.eqp</td>
<td>1.34</td>
<td>1.06</td>
<td>1.05</td>
</tr>
<tr>
<td>9</td>
<td>Northridge Canoga Parkt</td>
<td>NCP.eqp</td>
<td>0.94</td>
<td>1.10</td>
<td>1.80</td>
</tr>
<tr>
<td>10</td>
<td>Northridge 17645 Saticoy St</td>
<td>NSS.eqp</td>
<td>1.39</td>
<td>1.84</td>
<td>2.30</td>
</tr>
<tr>
<td>11</td>
<td>Northridge Glendale Las Palmas</td>
<td>NGLP.eqp</td>
<td>17.96</td>
<td>19.61</td>
<td>22.67</td>
</tr>
<tr>
<td>12</td>
<td>Northridge La Crescenta New York</td>
<td>NLCNY.eqp</td>
<td>9.71</td>
<td>14.53</td>
<td>17.06</td>
</tr>
<tr>
<td>13</td>
<td>Northridge LA Fletcher Dr</td>
<td>NLAFD.eqp</td>
<td>4.40</td>
<td>6.26</td>
<td>8.06</td>
</tr>
<tr>
<td>14</td>
<td>Northridge LA Hollywood Store FF</td>
<td>NLAHS.eqp</td>
<td>2.62</td>
<td>3.34</td>
<td>4.92</td>
</tr>
<tr>
<td>15</td>
<td>Northridge LA N Faring</td>
<td>NLANF.eqp</td>
<td>5.96</td>
<td>7.48</td>
<td>10.14</td>
</tr>
<tr>
<td>16</td>
<td>San Fernando LA Hollywood</td>
<td>SFLAH.eqp</td>
<td>4.30</td>
<td>3.78</td>
<td>4.10</td>
</tr>
<tr>
<td>17</td>
<td>Superstition Hill El Centro Imp</td>
<td>SHECI.eqp</td>
<td>1.53</td>
<td>1.85</td>
<td>2.18</td>
</tr>
<tr>
<td>18</td>
<td>Superstition Hill Plaster Cityt</td>
<td>SHPC.eqp</td>
<td>2.55</td>
<td>2.89</td>
<td>3.26</td>
</tr>
<tr>
<td>19</td>
<td>Superstition Hill Brawley</td>
<td>SHB.eqp</td>
<td>5.49</td>
<td>6.15</td>
<td>6.27</td>
</tr>
<tr>
<td>20</td>
<td>Superstition Hill Westmorland Fire Station</td>
<td>SHWFS.eqp</td>
<td>2.50</td>
<td>2.39</td>
<td>2.13</td>
</tr>
</tbody>
</table>

Figures 3.5 a) and b) show the scaled 20 records in gray compared to the design acceleration response spectrum in black for the 4 story frame and the 20 story frame described below in Section 3.6 respectively.
Figure 3.5 Example of Scaling of the 20 records a) 4 story frame b) 20 story frame
3.4.2 Matching over the entire spectrum

A very common method to scale time history records is to modify the Fourier amplitude coefficients to match the design acceleration response spectrum. The 20 records were modified using the program Artifquakelet by Montejo (2004). NLTHA was performed for the 100% intensity for the 8 story building, described below in Section 3.6, to observe the effect of this type of matching on the drift profile. Figure 3.6a) shows a good match of the acceleration spectrum and Figure 3.6b) a bad match for the displacement spectrum. Results of the NLTHA using this approach are discussed in Chapter 4.

![Figure 3.6 Example of matching over the entire spectrum a) ARS](image)

a)
3.5 Definition of intensity levels

Since one of the main objectives of this research is to investigate the effect of earthquake intensity levels on the displacement patterns that will be used for design, several intensity levels are defined. For the purpose of this research seven levels of intensity, 25%, 50%, 75%, 100%, 125%, 150% and 200% were selected. After scaling the earthquake records using the scaling procedure defined as scaling at period of interest shown above, these earthquakes will be called the 100% intensity records. The records will then be multiplied by 0.25, 0.5, 0.75, 1.25, 1.5 and 2.0 respectively to modify the 100% intensity ground motion records selected in Section 3.2
3.6 Buildings definition

Four regular two dimensional frames with 4, 8, 12 and 20 stories were used in this research. Figure 3.7 shows the geometry of the eight story frame and Table 3.5 defines the geometry of all four frames.

![Figure 3.7 Example of 8 Story frame geometry](image)

<table>
<thead>
<tr>
<th>Building name</th>
<th>No. of Stories</th>
<th>Number of spans</th>
<th>1st story height</th>
<th>Typical story height</th>
<th>Span length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building 1</td>
<td>4</td>
<td>3</td>
<td>4.0 m</td>
<td>3.0 m</td>
<td>8.0 m</td>
</tr>
<tr>
<td>Building 2</td>
<td>8</td>
<td>3</td>
<td>4.0 m</td>
<td>3.0 m</td>
<td>8.0 m</td>
</tr>
<tr>
<td>Building 3</td>
<td>12</td>
<td>3</td>
<td>4.0 m</td>
<td>3.0 m</td>
<td>8.0 m</td>
</tr>
<tr>
<td>Building 4</td>
<td>20</td>
<td>3</td>
<td>4.0 m</td>
<td>3.0 m</td>
<td>8.0 m</td>
</tr>
</tbody>
</table>

Table 3-5 Definition of building geometry

L_b = 8.00 m (Bay length)

h_b = 0.57 m (Beam depth)

ε_y = 0.002 (Steel Strain)

Mass

m_i = 6605.50 kg each story

m_t = 6116.21 kg top story
3.7 Design using DDBD and SAP2000

The design of the 4 buildings was made using the forces obtained from the DDBD method and the program SAP2000 from CSI (SAP2000 CSI, 2014). The design was performed considering first the column fixed at the base and then the columns pinned at the base with an applied moment equal to the base shear divided by the number of columns and multiplied by 0.6 the height of the first floor. Results from this analysis gave similar drift profiles using both approaches. The design is presented in Appendix B.

3.7.1 Results of design-Beam and column sizes

Table 3.6 presents the results of the design in terms of column and beam sizes and their respective steel ratios. For the beam steel ratios, the numbers presented represent the percent of steel at left, center and right section on the beam. The results for steel ratios are reasonable according to ACI 318-05. It can be observed that the value of 3.4% is on the higher range but it still complies with the 4% limit given by ACI.

<table>
<thead>
<tr>
<th>Building name</th>
<th>Columns size</th>
<th>Column steel ratio</th>
<th>Beam size</th>
<th>Beam steel ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building 1</td>
<td>400x400 mm</td>
<td>2.43%</td>
<td>300x570 mm</td>
<td>1.85, 0.95, 1.85%</td>
</tr>
<tr>
<td>Building 2</td>
<td>500x500 mm</td>
<td>3.40%</td>
<td>300x570 mm</td>
<td>1.80, 0.65, 1.80%</td>
</tr>
<tr>
<td>Building 3</td>
<td>600x600 mm</td>
<td>1.70%</td>
<td>300x570 mm</td>
<td>1.75, 0.65, 1.75%</td>
</tr>
<tr>
<td>Building 4</td>
<td>750x750 mm</td>
<td>1.30%</td>
<td>300x570 mm</td>
<td>1.80, 0.65, 1.80%</td>
</tr>
<tr>
<td>floors 1 to 10</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Building 4</td>
<td>600x600 mm</td>
<td>1.00%</td>
<td>300x570 mm</td>
<td>1.55, 0.60, 1.55%</td>
</tr>
<tr>
<td>floors 11 to 20</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
3.8 Nonlinear Time History analysis of frames using the existing design methodology

Time-history analyses were conducted using the 20 scaled ground motion records. A time increment of 0.005 second and damping ratio of 5% was used in the analyses. The analyses were conducted using the Ruaumoko2D computer program (Ruaumoko, 2001).

As explained above, seven intensities of the earthquake records have been considered (25%, 50%, 75%, 100%, 125%, 150% and 200%), each ground motion record has been scaled to a different factor for each frame and each earthquake. NLTHA was performed for each frame in total, 20 Records, 7 Intensities, and 4 different Frames, with and without P-Δ a total of 1120 NLTHA analyses.

The NLTHAs were carried out on the four reinforced frames to obtain information regarding the base shear, story shear, the displacements profiles and interstory drift of the structure. Appendix C presents a detailed description of how the structures were modeled using Ruaumoko to carry out the NLTHA. The main assumptions made to model the structure are:

- The stress-strain relationship for confined and unconfined concrete in the elements by Mander et al. (1988) was used to obtain beam and column properties.
- The steel reinforcement behavior was modeled using King Program (1986).
- The hysteretic behavior of concrete sections (Beams and Columns) was modelled using a modified Takeda (Takeda, Sozen, Nielsen., 1970) model. The strength and stiffness
of the beams were estimated without taking into account the contribution of the slab. The strength was computed without the use of strength reduction factor $\phi$.

- The CUMBIA (Montejo and Kowalsky, 2007) program was used to calculate the moment curvature of concrete sections, as well as the P-M interaction of concrete columns.

- Beams: The moment-curvature relationships for the beams were computed using the material properties described above and using the assumptions that plane sections remain plane after flexural deformation and that there exists complete compatibility of strains between steel and concrete. Beams and columns were modeled using Giberson’s beam and column models (Ruaumoko, 2001).

- Joint flexibility was not considered, the effect of the beam-column joint geometry was not considered in the analysis. The member lengths were considered to the intersection of the axes of the members. The inelastic deformation was concentrated in one point at the end of each member.

- Columns: The moment of inertia, to model the columns was computed using the same procedure as for the beams but considering a high axial compression force for the lower story columns and a lower axial compression force for, the higher columns. Inertias in the columns range from $0.50I_g$ to $0.55I_g$ in the analysis made.

### 3.9 Results from the analysis (NLTHA)

Displacements and drift profiles were calculated for the 20 ground motion records for each frame and the 7 intensity levels with and without P-\(\Delta\). The maximum drift profile was
obtained at the maximum of the absolute value of positive and negative drift. The maximum displacement profile was defined as the displacement profile at the time step where the maximum drift occurs in the time history. Results of displacement and drift profiles were averaged over the 20 records.

The maximum shear profiles were calculated using the same procedure used for the calculation of the drift profiles. The following section presents discussion and interpretation of the displacements, drifts and interstory shear profiles obtained from the analyses performed. It also presents a discussion of the implications of these results regarding the adequacy of DDBD.

The following section presents a discussion and interpretation of the results regarding displacement, drift and interstory shear profiles and the implications in evaluating the DDBD.

3.9.1 Displacement and drift Profiles

This section shows the results from the NLTH analysis for the four frames under study of the 20 earthquakes ground motion and the different level of intensity of those ground motion records.

Four story frame. Results for the four story frame are presented below starting with the plots of the results followed by a discussion and interpretation.

3.9.2 Results without considering the P-Δ effect four story frame

Figures 3.8 and 3.9 show the results of the NLTHA without considering P-Δ effects
Figure 3.8 Displacement profile 4 story frame a) Displacement profile at 100% intensity of the 20 records and b) Average of the 20 records at each intensity.
Figure 3.9 a) Normalized displacement profile 4 story frame b) Drift profile

Figure 3.8a) shows deformations of the four story frame at 100% intensity level for the 20 earthquakes considered. The solid line at the positive side of the plot shows the average for
the positive values of deformation, similarly the solid line at the negative side of the plots shows the average for the negative value. The dashed lines are the positive average plus one standard deviation and the negative average minus one standard deviation. Figure 3.8 shows that the average displacement pattern is linear. The results for the displacement patterns for the other intensity levels also showed that the displacement patterns are linear as it is shown in Figure 3-8b).

Figure 3-8a) not only shows that the displacement pattern for the average is linear but that the individual displacement patterns for most of the earthquakes are linear with a few exceptions. Similarly plots of the displacement patterns for different earthquakes for other levels of intensity not shown here were also linear.

Figure 3-8b) shows the maximum absolute value of average deformation pattern for the 20 earthquakes considered for each intensity level, it also shows the deformation pattern used in the existing methodology for DDBD as proposed by Priestley, Calvi and Kowalsky (2007). The plot shows that as the intensity of the earthquake increases the deformations increase as they should. This plot also shows that a linear or close to a linear deformation pattern occurs when different levels of intensity of the earthquake are applied. This suggest that for buildings under or equal to four stories the level of intensity is not a variable in the assumption for the design deformed shape and that the deformation pattern is approximately linear for all levels of intensity, validating the deformation pattern proposed by Priestley, Calvi and Kowalsky (2007). The existing deformation pattern proposed by Priestley et al. is close to the average of
the 200% intensity level deformation which may indicate that the seismic design for the four story frame is a conservative design.

Figure 3-9a) shows the average of the 20 earthquakes displacement patterns normalized to the roof displacement deformation for all intensity levels, this plot clearly shows a nearly linear deformation shape for the four story frame. This indicates that a linear deformation shape as proposed by Pristley, Calvi and Kowalsky (2007) is adequate for frames up to four stories.

Figure 3-9b) shows average drift profiles of the 20 earthquakes for the different intensity levels of the earthquakes, it shows a small variation of the drift profile at each story, variations between floors are 12% on average for the 25% intensity level, and 10% for the 200% intensity level. The deviation from the design drift for the 200% intensity is: a) 3.4%, on the first floor, b) 5.6% on the 2nd floor; c) 3.4% on the 3rd floor and d) 13% on the 4th floor. It shows that the existing design drift is close to the 200% intensity level, from this comparison one may conclude that the design using the existing method yields a conservative design. An explanation of why this happens is as follows:

a) The results obtained in figures 3.8 and 3.9 are from a frame designed considering gravity loads and seismic load combined as required by existing building codes.

b) An additional study was to design the frame considering only the strength required using the results from the moments obtained using DDBD method without the addition of the dead and live load moments and using effective stiffness for the beams and columns obtained
using $I_{ef} = \frac{M_y}{E\phi_y}$, where $M_y$ and $\phi_y$ where obtained from the moment curvature diagrams with zero axial load. NLTHA was performed on this design.

The objective of this NLTHA was to evaluate what were displacements, the drifts and the normalized displacements obtained when only the earthquake moment demand. The results of this NLTHA's were used to verify the statement that the dead and live loads added strength and stiffness to the buildings and therefore the drifts obtained using the larger moment capacities were smaller than the ones using only the earthquake forces as it was obtained in the analysis as shown in Figure 3.9.

To perform this evaluation the 4 and 8 story building were designed using the existing DDBD method considering the earthquake forces only, the design was performed as follows:

1. The inertia forces were calculated using the 100% displacement response spectrum level.

2. The EI of the beams and columns were obtained using $I_{ef} = \frac{M_y}{E\phi_y}$.

3. The resulting moment diagrams were obtained using the program SAP2000 and the level of forces and inertias for the beams and columns of item 1 and 2 above.

4. Capacities for the beam and columns were designed (assigned) as shown in Figure 3.10 and 3.11 for the 4 and 8 story buildings. The four story building was divided in 2 tiers for design and the 8 story building was divided in 3 tiers as explained below.
a. For the 4 story building the column capacities at the 1st and 2nd floor were the same and the beam capacities for the 1st and 2nd floor were also the same. Similarly for floors 3 and 4 a smaller moment capacity was assigned for the columns and beams.

b. For the 8 story building the column capacities at the 1st, 2nd and 3rd floor were the same and the beam capacities for the 1st, 2nd and 3rd floor were the same. Similarly the 4th 5th and 6th moment capacities were the same for beams and columns (smaller). The same was done 7th and 8th (still smaller moment capacities).

5. The results obtained were the following:

a. Displacements for earthquake intensities of 25%, 50% 75%, 100%, 125% 150% and 200% using NLTHA and only earthquake loads in the nonlinear runs. Displacements using earthquake all levels of intensities and dead and live loads included in the NLTHA.

b. Drifts for earthquake intensities of 25%, 50% 75%, 100%, 125% 150% and 200% using NLTHA and only earthquake loads in the nonlinear runs. Drifts using earthquake all levels of intensities and dead and live loads included in the NLTHA.

Normalized displacements for earthquake intensities of 25%, 50% 75%, 100%, 125% 150% and 200% using NLTHA and only earthquake loads in the nonlinear runs. Normalized
displacements using earthquake all levels of intensities and dead and live loads included in the NLTHA.

Figure 3.10 Four story building-Moment diagram from SAP2000 run (in red) and assigned capacities (in green)

Figure 3.11 Eight story building-Moment diagram from SAP2000 run (in red) and assigned capacities (in green)
Figure 3.12 presents the results of the displacements for the run without gravity loads for all levels of earthquake, the displacements for the 100% obtained are smaller than the design displacement for the upper floors and the displacements become nonlinear as the intensity increases.

**Figure 3.12 Displacements of NLTHA without gravity loads**

Figure 3.13 presents the results of the run for all levels of earthquake including the gravity loads in the analysis but not in the design. The displacements are still smaller than the design displacements but not as small as when the gravity loads are not used in the NLTHA run.
Figure 3.14 presents the results of the drift for the run without gravity loads for all levels of earthquake, the displacements for the 100% obtained are smaller than the design drift for the upper floors.
Figure 3.15 presents the drift results of the run for all levels of earthquake including the gravity loads. The drift at the 1st floor obtained is almost the design drift and the drift at the upper floors are still smaller than the design displacements but not as small as when the gravity loads are not used in the NLTHA run.
Figure 3.16 presents the results of the normalized displacement for the run without gravity loads for all levels of earthquake. The normalized displacement shape is different than the design normalized displacement for all levels of earthquakes. The resulting curves are all nonlinear.
Figure 3.16 Normalized displacements without gravity loads in the analysis

Figure 3.17 presents the normalized displacements results of the run for all levels of earthquake including the gravity loads. The normalized displacement shape is different than the design displacement for all levels of earthquakes but for the 25 to 75%, the results for these 2 levels of earthquake are very similar to the design shape.
Figure 3.17 Normalized displacements including gravity loads in the analysis

Conclusion: The drifts obtained from the NLTHA runs are very close to the design drift for the first floor of the 4 story frame when gravity loads are included and smaller when the gravity loads are not considered, for all other floors the resulted drift is smaller than the design drift for both cases. This is because the inclusion of an increased strength in the above level due to higher moment capacity used in the design of the frame. When the gravity loads are included in the analysis then the drifts are larger but the cracking and distress are also larger.

When considering actual code provisions in the design the resulted drift are as shown above on Figure 3.9b).

A further study was performed: Capacities for the beam and columns were designed (assigned) as shown in Figures 3.18a) and 3.18b) for the 4 and 8 story buildings considering only moments from the existing DDBD procedure, no gravity loads were included in the analysis and design. The objective of this NLTHA was to evaluate what were displacements
and the drifts obtained when only the earthquake moment demand were considered at all levels of the frames. The results of these NLTHAs were used to verify the statement that additional strength provided at upper level has an influence on the displacement shape of the analyzed frames.

Figure 3.18 a) Four story building-Moment diagram from SAP2000 run and assigned capacities in red, b) Eight story building-Moment diagram from SAP2000 run and assigned capacities in red.
Results from the analysis:

Figure 3.19 presents the results of the displacements for the run without gravity loads for 100% intensity level of earthquake. The displacements for the 100% obtained are about the same as the design displacement.

![Figure 3.19 Displacement shape for the four story frame considering only earthquake forces and moment capacities design from those forces.](image)

Figure 3.20 presents the drift results of the run for 100% intensity level of earthquake. The drift at the 1st and 3rd floor are smaller than the design drift and the drift at the 2nd and 4th floors are larger than the design drift, in average of the four floors the drift is about the same as the design drift. That is the reason why the displacement shape is linear for the four story frame.
Figure 3.20 Drift profile for the four story frame considering only earthquake forces and moment capacities design from those forces.

3.9.3 Results considering the P-Δ effect four story frame

Figures 3.21 and 3.22 show the results of the NLTHA considering P-Δ effects
Figure 3.21 Displacement profile 4 story frame including P-Delta. a) Displacement profile at 100% intensity of the 20 records and b) Average of the 20 records at each intensity
Figure 3.22 a) Displacement Normalized Shape and b) drift profile including P-Delta

Figures 3.23a), 3.23b), 3.24a) and 3.24b) with the inclusion of the P-Delta effect are similar to Figures 3.8a), 3.8b), 3.9a) and 3.9b). These figures show the same behavior as in the
figures without the P-Δ effect. In this case, in Figure 3.23a), for the 100% intensity some of the individual patterns are nonlinear; however, the average displacement patterns continue to be linear with a small deviation at the intensity level 200%, but is still very close to a linear pattern on average for all intensity levels. A comparison of these 4 figures and the similarity with the four figures without P-Δ indicate that P-Δ is not controlling the design and is having little influence on the behavior of the four story frame.

Eight story frame. Results for the eight story frame are presented below starting with the plots of the results followed by a discussion and interpretation.

3.9.4 Results without considering the effect of P-Δ eight story frame

Figures 3.23 and 3.24 show the results of the NLTHA without considering P-Δ effect.

![Displacement profile 8 story frame a) for the 20 records at 100% Intensity](image)

a) Figure 3.23 Displacement profile 8 story frame a) for the 20 records at 100% Intensity
Figure 3.23 Displacement profile 8 story frame a) for the 20 records at 100% Intensity, b) average of 20 records for each intensity

Figure 3.24 a) Displacement Normalized Shape
Figure 3.24 a) Displacement Normalized Shape and b) drift profile

Figure 3.23a) shows deformations of the eight story frame at 100% intensity level for the 20 earthquakes considered. The black solid line at the positive side of the plot shows the average for the positive values of deformation, similarly the black solid line at the negative side of the plots shows the average for the negative value. The dashed lines are the positive average plus one standard deviation and the negative average minus one standard deviation. This figure shows that the average displacement pattern is nonlinear. The results for the displacement patterns for the other intensity levels also show that the displacement patterns are nonlinear as it is shown in Figure 3.23b).

Figure 3.23a) not only shows that the displacement pattern for the average is nonlinear but that the individual displacement patterns for all earthquakes for 100% intensity are
nonlinear. Similarly plots of the displacement patterns for individual earthquakes for other levels of intensity not shown here were also nonlinear.

Figure 3.23b) shows the maximum absolute value of average deformation pattern for the 20 earthquakes considered for each intensity level, it also shows the deformation pattern used in the existing methodology for DDBD as proposed by Priestley, Calvi and Kowalsky (2007). The plot shows that as the intensity of the earthquake increases the deformations increase as it should be. This plot also shows that the nonlinearity increases as the level of earthquake increases. This suggests that, for eight story buildings, the level of intensity is a variable in the assumption of the design deformed shape and that the deformation pattern is nonlinear for all levels of intensity. The existing deformation pattern proposed by Priestley et al. is close to the average of the 150% intensity level deformation which may indicate that the seismic design for the eight story frame is also a conservative design, not as much as for the four story building but still conservative. This validates the shape of the nonlinear deformation pattern proposed by Priestley, Calvi and Kowalsky (2007), though; the displacement does not correspond to the 100% intensity level.

Figure 3.24a) shows the average of the 20 earthquakes displacement patterns normalized to the roof displacement deformation shapes for all intensity levels, this plot clearly shows a nonlinear deformation shape for the eight story frame.

Figure 3.24b) shows average drift profiles of the 20 earthquakes for the different intensity levels of the earthquakes, it shows: a) the maximum story drift does not occur at the first floor, b) for the 25% the maximum drift occurs at the 4th level with nearly constant value from level...
2 to 5; the drift decreases from level 6 thru 8, c) as the intensity increases from 25% to 200% the maximum drift shifts from the 5th floor to the 2nd floor, d) for intensities 75% to 125% the drift is nearly constant for level 2 to 4, d) the drift decreases from level 5 thru 8, e) for intensity 150% the drift is maximum at level 2 and decreases from level 3 to 8, the drift at level 2 corresponds to the design drift, again showing that the design is conservative, but not as much as for the 4 story building.

The eight story frame was also studied using the existing DDBD method considering the earthquake forces only and the design considerations as explained above for the four story frame.

Figure 3.25 presents the results of the displacements for the run without gravity loads for all levels of earthquake, displacement for 100% intensity is very similar than the design displacement.
Figure 3.25 Displacements without gravity loads in the analysis

Figure 3.26 presents the results of the displacements for the run with gravity loads for all levels of earthquake, for the 100% intensity the displacements are smaller than the design displacement, when gravity loads are included the design shape is very close to 125% but for the upper stories 6 to 8.
Figure 3.26 Displacements including gravity load in the analysis

Figure 3.27 presents the results of the drift for the run without gravity loads for all levels of earthquake, the drift for the 100% obtained are the same as the design drift for the lower two floor and smaller than the design drift for the upper floors.
Figure 3.27 Drift without gravity loads in the analysis

Figure 3.28 presents the drift results of the run for all levels of earthquake including the gravity loads. The drift at the top floors is almost the design drift for 100% intensity and the drift at the lower floors are smaller than the design displacements. This occurs because the dead loads are more important in the upper floors.
Figure 3.28 Drift with gravity loads in the analysis

Figure 3.29 presents the results of the normalized displacement for the run without gravity loads for all levels of earthquakes, it shows that the displacement shape is a function of the intensity with results smaller than the design shape for lower level of intensities and larger values as the intensity increases.
Figure 3.29 Normalized displacements without gravity loads in the analysis

Figure 3.30 presents the normalized displacements results of the run for all levels of earthquake including the gravity loads. The normalized displacement shape are smaller than the design shape for all level of intensities of the earthquakes used.
An additional run was performed for the eight story frame using the capacity moments as shown on Figure 18b) the results from the NLTHA are shown below:

Figure 3.30 presents the results of the displacements for the run for the 100% intensity level of earthquake, the displacements are very close to the design displacement.
Figure 3.31 Displacement shape for the eight story frame not including gravity loads in the design and analysis.

Figure 3.31 presents the drift results of the run for 100% intensity level of earthquake not including the gravity loads in the analysis and design. The drift at the top floors is almost the design drift for 100% intensity and the drift at the lower floors are smaller than the design displacements.
Conclusion: For the eight story frame the drift for the first and second floors are close to the design drift when gravity loads are not included in the analysis. When gravity loads are included in the analysis the resulted drift are smaller for the lower floors than the design drift and larger for the upper floors. When the moment capacity used for design is the same as those obtained from the DDBD design forces the drift is smaller than the design drift for the lower floors, higher at the 6th floor and about the design drift for the two upper floors. This is because gravity loads are having more influence on the upper level than the design earthquake forces considered in the design.

3.9.5 Results considering the of P-Δ effect eight story frame

Figures 3.33 and 3.34 show the results of the NLTHA considering the effects of P-Δ, the bold gray lines in Figure 3.33 are the displacement shape for the 20 records, 100% intensity
and the black solid line are the average for the positive and negative displacement for the 20 records.

**Figure 3.33 Displacement profile 8 story frame including P-Delta**

a) for the 20 records at 100% Intensity, b) average of 20 records for each intensity
Figures 3.33a), 3.33b), 3.34a) and 3.34b) with the inclusion of the P-Delta effect are similar to Figures 3.23a), 3.23b), 3.24a) and 3.24b), these figures show the same behavior as in the figures without the P-Δ effect. The comparison of these 4 figures and the similarity with
the four figures without P-\(\Delta\) indicates that P-\(\Delta\) is not controlling the design and is having little influence on the behavior of the eight story frame.

**Twelve story frame.** Results for the twelve story frame are presented below starting with the plots of the results followed by a discussion and interpretation.

### 3.9.6 Results without considering the effect of P-\(\Delta\) of the twelve story frame

Figures 3.35 and 3.36 show the results of the NLTHA without the effects of P-\(\Delta\), the bold gray lines in Figure 3.35 are the displacement shape for the 20 records, 100% intensity and the black solid line are the average for the positive and negative displacement for the 20 records.

**Figure 3.35 Displacement profile 12 story frame a) for the 20 records at 100% Intensity**
b)

Figure 3.35 Displacement profile 12 story frame a) for the 20 records at 100% Intensity, b) average of 20 records for each intensity

a)

Figure 3.36 a) Displacement Normalized Shape
Description of the content of the Figures 3.35a) and b) and Figures 3.36 a) and b) is similar as the description of 3.23a) and b) and 3.24a) and b). Analysis of the data for Figure 3.35a) and b) is similar to the one for Figure 3.23a) and b) with the exception that the existing deformation pattern proposed by Priestley et al. is close to the average of the 125% intensity level deformation which may indicate that the seismic design for the twelve story frame is also a conservative design, not as much as for the four and eight story buildings but still conservative. The shape of the deformation patterns from NLTHA are more nonlinear than the one proposed by Priestley, Calvi and Kowalsky (2007).

Figure 3.36b) shows average drift profiles of the 20 earthquakes for the different intensity levels of the earthquakes, it shows: a) the maximum story drift does not occur at the first floor, b) for the 25% the maximum drift occurs at the 3rd level with nearly constant value from level
2 to 9, the drift decreases from level 10 thru 12, c) as the intensity increases from 25% to 150% the maximum drift still occurs at the 3rd floor; d) the drift decreases from level 3 thru 12, e) for intensity 200% the drift is maximum at level 2 and decreases from level 3 to 12, f) the drift at level 2 for intensity 100% corresponds to the design drift. This validates the design proposed by Priestley, Calvi and Kowalsky (2007).

3.9.7 Results considering the effect of P-Δ twelve story frame

Figures 3.37 and 3.38 show the results of the NLTHA considering the effects of P-Δ, the bold gray lines in Figure 3.37 are the displacement shape for the 20 records, 100% intensity and the black solid line are the average for the positive and negative displacement for the 20 records.
Figure 3.37 Displacement profile 12 story frame including P-Delta for the 20 records at 100% Intensity, b) average of 20 records for each intensity
Figures 3.37a), 3.37b), 3.38a) and 3.38b) with the inclusion of the P-Δ effect are similar to Figures 3.35a), 3.35b), 3.36a) and 3.36b), these figures show similar nonlinear behavior as
in the figures without the P-Δ effect, but more pronounced. Comparison of the maximum drift shown in Figure 3.38b) with Figure 3.36b) shows an increase of 25% for intensity 100% and 14% for intensity 200%. Comparison of Figures 3.37b) with 3.35b) shows an increase of the maximum displacement of only 1%. It should be noted that although the increase of maximum displacement is small, the maximum drift increases significantly indicating that the P-Δ effect starts to be significant for the twelve story building. The drift at level 3 for intensity 100% corresponds to the design drift similar to the analysis without the P-Δ effect.

**Twenty story frame.** Results for the twenty story frame are presented below starting with the plots of the results followed by a discussion and interpretation.

### 3.9.8 Results without considering the effect of P-Δ twenty story frame

Figures 3.39 and 3.40 show the results of the NLTHA without the effects of P-Δ, the bold gray lines in Figure 3.39 are the displacement shape for the 20 records, 100% intensity and the black solid line are the average for the positive and negative displacement for the 20 records.
Figure 3.39 Displacement profile 20 story frame a) for the 20 records at 100% Intensity, 
b) average of 20 records for each intensity
Figure 3.40 a) Displacement Normalized Shape and b) drift profile

Description of the content of Figures 3.39 a) and b) and Figures 3.40 a) and b) is similar as the description of 3.35 a) and b) and 3.36 a) and b). Analysis of the data for Figure 3.39a)
and b) is similar to the one for Figure 3.35a) and b) with the exception that the existing deformation pattern proposed by Priestley et al. is close to the average of the 75% intensity level deformation for level 1 thru 12, there is a transition at levels 12 thru 15 and a match for the 100% intensity level for level 16 thru 18, for levels 19 and 20 the design deformation exceeds the for 100% intensity level. This may indicate that the seismic design for the twenty story frame is an unconservative design for levels 1 thru 14. The shape of the deformation patterns from NLTHA are much more nonlinear than the one proposed by Priestley, Calvi and Kowalsky (2007).

Figure 3.40b) shows average drift profiles of the 20 earthquakes for the different intensity levels of the earthquakes, it shows: a) the maximum story drift does not occur at the first floor, b) for the 25% the maximum drift occurs at the 4th level with nearly constant value from level 2 to 9, the drift decreases from level 10 thru 20, c) as the intensity increases from 25% to 150% the maximum drift still occurs at the 4th floor; d) the drift decreases from level 4 thru 20, e) for intensity 200% the drift is maximum at level 2 and decreases from level 3 to 20, f) the drift at level 3 thru 6 for intensity 100% exceeds by 16% the design drift.

3.9.9  Results considering the effect of P-Δ twenty story frame

Figures 3.41 and 3.42 show the results of the NLTHA with the effects of P-Δ, the bold gray lines in Figure 3.41 are the displacement shape for the 20 records, 100% intensity and the black solid line are the average for the positive and negative displacement for the 20 records.
Figure 3.41 Displacement profile 20 story frame including P-Δ a) for the 20 records at 100% Intensity, b) average of 20 records for each intensity
Figures 3.41a), 3.41b), 3.42a) and 3.42b) with the inclusion of the P-Δ effect are similar to Figures 3.39a), 3.39b), 3.40a) and 3.40b), these figures show similar nonlinear behavior as
in the figures without the P-Δ effect, but more pronounced. Comparison of the maximum drift shown in Figure 3.42b) with Figure 3.40 b) shows an increase of 86% for intensity 100% and 55% for intensity 200%. Comparison of Figures 3.41b) with 3.39b) shows an increase of the maximum displacement of only 1%. It should be noted that although the increase of maximum displacement is small, the maximum drift increases significantly indicating that the P-Δ effect is very significant for the twenty story building. The drift at level 3 for intensity 50% corresponds to the design drift. The drift at level 2 thru 6 for intensity 100% exceeds the design drift by a maximum of 56%. Indicating that the existing design method for DDBD even when performed using P-Δ effect is unconservative for the 20 story building and may have to be modified to take into account this unconservatism. The fact that for taller buildings the total maximum displacement is about the same at the roof but the interstory drifts are higher at the lower level could be explained as follows: For the explanation of the same displacement at the roof by the equal displacement approximation where buildings with high period of vibration have the same displacement when considering linear and nonlinear behavior. The higher drift at the lower lever can be explained considering that the earthquake deformations have to be larger at the lower floors because the effect of the input motions entering the structure will be higher at the lower floors.
Figures 3.43 show the displacement profiles for the 4 frames without P-Δ effect with a comparison with the existing displacement design profile. As presented before, if only an analysis of the displacement profile is studied it can be concluded that the design profile is adequate for design, however, an analysis as presented of the drift profiles yields that the DDBD should be modified.
3.10 Shear distribution

Figures 3.44 show the: a) average shear, b) the Average plus one standard deviation distribution and c) Maximum shear at each story obtained from the NLTHA of the 100% intensity levels considering P-∆ effect.

Figure 3.44 Shear distribution four frames
Figure 3.44 shows that the maximum shear for the average a) only occurs at the first floor for the 4 story building; b) at the 4th floor for the 8 story building, c) at the 3rd floor for the 12 story building and d) at the 2nd floor for the twenty story building. Table 3.7 shows the comparison of the design shears, the maximum of the average shear and the maximum shears for the four buildings. It can be observed that the story shear capacities obtained from the NLTHA are higher that the design shears by a considerable amount for all buildings. This can be justified because the design of the building not only takes into account the design base shear, but it considers capacity design for strong columns weak girder, capacity design for shear column design and increases due to $\phi$ factors, steel and concrete overstrength and the fact that all members do not yield at the same time.

<table>
<thead>
<tr>
<th>No. Story</th>
<th>Weight (kN)</th>
<th>Design shear (kN)</th>
<th>% of weight</th>
<th>Max. of Average story shear (kN) floor</th>
<th>% of weight</th>
<th>Max. story shear (kN) floor</th>
<th>% of weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>2495</td>
<td>232.5</td>
<td>9%</td>
<td>790.4 1st</td>
<td>32%</td>
<td>1061.1 2nd</td>
<td>42%</td>
</tr>
<tr>
<td>8</td>
<td>5037</td>
<td>555.1</td>
<td>11%</td>
<td>1434.7 4th</td>
<td>28%</td>
<td>1729.9 4th</td>
<td>34%</td>
</tr>
<tr>
<td>12</td>
<td>7579</td>
<td>571.7</td>
<td>8%</td>
<td>1725.0 3rd</td>
<td>23%</td>
<td>2025.1 2nd</td>
<td>27%</td>
</tr>
<tr>
<td>20</td>
<td>12662</td>
<td>583.5</td>
<td>5%</td>
<td>2684.1 2nd</td>
<td>21%</td>
<td>3140.1 2nd</td>
<td>25%</td>
</tr>
</tbody>
</table>
3.11 Conclusions and proposed modifications

i. The deformed shape profile changes with changes of intensity level.

ii. The deformed shape is influenced by P-Δ effects especially for buildings higher than 8 stories.

iii. Changes became more important as the number of stories increases.

iv. The design displacement shape profile proposed by Priestley et al. is adequate for the 4 and 8 story buildings but it is not for the 12 and 20 story buildings.

v. P-Δ effect does not have a large impact on the final displacement results for buildings 8 stories and below; it has some effect on the 12 and 20 story buildings. But it has a significant effect on the drift for the 12 and 20 story buildings.

vi. For the 20 story frame and 100% intensity, the roof average displacement increased by 1%, when P-Δ is included but the maximum average drift increased by 25% for the 12 story building and 86% for the 20 story building which indicates that roof displacement profile is not a good indication for evaluating design results when including P-Δ, since drift is a code design parameter.

vii. For the twenty story frame and 200% Intensity, the roof average displacement increased by 1%, when P-Δ is included and the maximum average drift increases by 55%.

viii. The maximum drift observed was not at the 1st floor; it occurs at levels above the 1st, usually at 3rd to 5th floor. One reason could be because there is a plastic deformation concentration at these levels, but also it depends on the support conditions.
ix. It is shown that the maximum drift location is a function of the intensity of the earthquake, but it is always between the second and the fifth floor.

x. Since maximum drift is not occurring at the first level then the maximum shear is not occurring at the first floor as shown on Figure 3.44.

xi. The existing design equation is suitable for designing frames up to 8 stories; above 8 the design equation needs some improvement.

xii. As shown on Figure 3.8b) for 4 story frames, design shape is linear and with constant drift but it matches the 200% intensity level of displacement, in Figure 3.23b), the 8 story frame displacement shape is more parabolic but it matches 150% intensity level of displacement, in Figure 3.35b) for the 12 story building, the displacement shape matches the 125% intensity level and in Figure 3.39b) for the 20 story frame; the displacement shape matches the 75% intensity level.

xiii. For the 4 and the 8 story frame, displacement shape matches the 100% intensity level when they are designed and analyzed considering earthquake forces only as shown in Figures 3.19, and 3.31 but when gravity loads are included in the analysis these loads are controlling the design. Drift profiles in both cases have values higher than the design drift at some floors of the frames analyzed when designed for earthquake forces only. (See Figures 3.20 and 3.32)

xiv. The design displacement shape should be modified to accommodate all levels of earthquake intensities and to have in consideration the number of story for the frame to be analyzed.
CHAPTER 4

Parametric studies

4.1 Introduction to parametric studies

In these parametric studies the question is, what factor may influence the shape of the drift profile?

The shape of the drift profile with the maximum drift occurring at the 2nd to 5th floors and not at the 1st was not expected, and initially it was postulated that it may be the influence of many factors. The objective of this chapter is to study the impact of several parameters on the drift profiles shape obtained using NLTHA. The parameters studied were:

- Influence of far field earthquakes
- Influence of use of spectrum compatible ground motion records
- Influence of axial deformation in the drift profile
- Influence of base support condition
- Influence of considering elastic behavior of columns above the first floor

4.2 Influence of far field earthquake

It was suggested by Professor James Nau to study far field records just to see if they had any influence on the shape of the drift profile and 5 records were suggested at that time.

For this case and for the study of the drift profile 5 records show the trend of the drift profile and the conclusion that far field is not a factor on the shape of the drift profile.
To show the influence of far field records versus near field on the drift profiles NLTHAs were performed for five earthquakes ground motion records with the same four building frames used in chapter 3, the study was performed using the 100% intensity level earthquakes.

Five earthquake records were selected from work done by Medina and Krawinker (2003), those records correspond to Large Magnitude Long Range set used by Medina and Krawinkler.(2003) Table 3-3 shows the earthquakes used in this study.

4.2.1 Results from analysis far field

Figure 4.1 thru 4.4 show the average drift profile of the 5 far field earthquake records compared to the drift profile of the 20 ground motion records for near field.

![Drift profile for 4 story frame](image)

Figure 4.1 Drift profile for 4 story frame
Figure 4.1 shows very little difference on the drift profiles for the 4 story building.

![Graph showing drift profiles for 4 story building](image)

**Figure 4.2 Drift profile for 8 story frame**

Figure 4.2 shows that the shapes of the drift profiles are very similar for floors 6 through 12 for the far field and near field earthquakes. For floors 1 through 6, the drift profile for the near field earthquakes is approximately 30% higher than for the far field earthquakes.
Figure 4.3 shows that the shape of the drift profile is very similar although the near field record have larger drifts for levels one thru six. The near field earthquake drift for story 3 is approximately 43% higher.
Figure 4.4 shows that the shape of the drift profile is very similar although the near field record have moderately larger drifts for levels one thru twelve. The near field earthquake drift for story 2 is approximately 15% higher.

Based on these results, although there are differences in the values of the drift profile for the near field and far field records of the 8, 12 and 20 story frames, the shapes are similarly indicating that the effect of near vs. fair field earthquakes is not significant.

4.3 Influence of use of spectrum compatible ground motion records

A very common method to scale time history records used in the nuclear industry was to modify the Fourier amplitude coefficients to match the design acceleration response spectrum. Recently the matching of the spectra has changed to do the matching at the period of interest.
as indicated in ASCE-41 (ASCE/SEI-41-06). The objective of these NLTHA was to evaluate what was the effect of this process on the drift profiles.

Montejo’s (2003) wavelet computer program was used (Artifquakelet) to match the Acceleration Response Spectrum (ARS) of the twenty records as explained in more detail in Chapter 3; only the eight story frame was analyzed for this particular study, all twenty Large Magnitude Short Range (LMSR) records were used and the drift profiles were averaged for all ground motion records.

4.3.1 Results from analysis

Figure 4.5 presents the results of the study; the black line is the drift profile using the records matched using spectrum compatible, the brown line is the drift profile using the matching at the periods of interest and the red one is the design drift profile using DDBD. The drift profile shape is very similar for the case of spectrum matching and period of interest matching. The drift is greater when using the period of interest matching process. Based on these results the use of spectrum compatible records will not affect the shape of the drift profile, only affects the intensity.
4.4 Influence of axial deformation in the drift profile

In discussions with Professor Eduardo Miranda of Stanford University, regarding the shape of the drift profile it was suggested that the axial deformation might influence the shape of the drift profile, to study this behavior NLTH analysis considering P-Δ effect was performed for the 20 story frame with 5 LMSR ground motion records, not considering axial deformation in the columns. The question is to evaluate if the axial deformation had any major influence in the shape of the drift profile, regarding the fact that the maximum drift did not occur in the first floor. Although the effect of the inclusion of the axial load shows a larger drift for floors 3 through 6 the maximum drift still does not occur on the first floor and the general shape of the drift profiles are similar. As shown in Figure 4.6, the profile shape is not influenced significantly by the axial deformation of the columns of the structure analyzed. Therefore, it
was considered that the analysis performed considering column axial deformation was adequate.

![Figure 4.6 Drift profile 20 story frame not considering axial deformation of the columns.](image)

**Figure 4.6 Drift profile 20 story frame not considering axial deformation of the columns.**

### 4.5 Influence of base support condition

To study if the support condition had any influence on the drift profile, the support condition was changed from a fixed support to pin support at the base of the columns. The analysis was performed for the 20 story frame considering P-Δ effect and with 5 LMSR ground motion records. Figure 4.7 shows that the shape of the drift profile is influenced by the support condition, and it has to be considered in the analysis of this type of structures. The drift increases by approximately 40%.
It is recommended, for future research that different soil structure interaction conditions be analyzed, to understand the shape profile of frames, as the support becomes flexible.

![Figure 4.7 Drift profile for 20 story frame considering pinned base support.](image)

4.6 Influence of considering elastic behavior of columns above the first floor

To understand, if considering all columns having inelastic behavior, has an influence on the drift profile, the 8 story frame was analyzed considering that all column from the second level and above behave elastically and the ones in the first floor behave inelastically. The results from this analysis were compared to the results obtained in Chapter 3 for the 8 story building and 100% intensity level.
4.6.1 Results from analysis considering elastic behavior of columns above first floor.

![Figure 4.8 Drift profile for 8 story frame considering elastic behavior of upper columns.](image)

Figure 4.8 shows that the NLTHA behavior of frames analyzed using columns behaving inelastically in all stories and using columns behaving inelastically in only the first floor is very similar. An explanation for the similarities in the results is that the design of the columns is completed using capacity design method preventing formation of plastic hinges on all columns except at the bottom of the columns at the first floor. This is accomplished by using a design that enforces a strong column-weak girder design. That is the reason why all upper columns behave elastically by design. Therefore, using nonlinear columns in the upper floors will have very little influence on the shape of the drift profile of the 8-story frame under study.
4.7 Summary of parametric studies

- Influence of far field earthquakes, Figures 4.1 thru 4.4 show that the effect of far field does not influence the shape of the drift profile, the level of drifts are less intense but with the same shape.

- Influence of use of spectrum compatible ground motion records, Figure 4.5 also shows that the level of drifts are less intense than those of scaled to the period of interest drifts, but having the same shape.

- Influence of axial deformation in the drift profile, in Figure 4.6 shows that the axial deformation has influence on the intensity of the drift but not on the shape.

- Influence of base support condition, Figure 4.7 shows that the shape is greatly influence by the support condition at the columns base.

- Influence of considering elastic behavior of columns above the first floor, Figure 4.8 shows that there is not influence of this effect on the shape of the drift profile, the reason for these results is that the design is done considering strong-columns-weak girders which prevents yielding of the columns above the first floor.

In conclusion the drift profile shape is not significantly influenced by any of the parameters studied above with the exception of the change of boundary conditions at the base of the columns. Therefore it is very important that the model used for design considers the correct boundary conditions.
CHAPTER 5

New Proposed Design Recommendations

5.1 Introduction

In Chapter 3, it was concluded that the drift profile of the existing method does not adequately represent the results of the NLTHA. The displacement shape is influenced by the level of earthquake intensity and the number of floors of the building. The following is an attempt to consider the level of earthquake intensity and the number of floors of the building in a new proposed equation for the displacement shape and new distribution of lateral loads along the height of the building.

5.2 New displacement profile shape and vertical force distribution

Design equation by Priestley et al. (2007) is adequate for concrete frames up to 8 stories, it is recommended that a new displacement shape profile be used, and this new equation should comply with the effect of the number of stories and the intensity level of the ground motion.

i. The following proposed equation takes in consideration both conditions, intensity level and number of story of reinforced of concrete frames. This equation was developed as follows:

a. Nonlinear regression was developed using the normalized displacement shapes shown in Chapter 3 for the different frames analyzed.
b. The curve fitting was first performed for each frame for all intensities of ground motions, from there the contribution of the intensity to the equation was obtained.

c. The second adjustment to the curve was performed using the normalized displacement shape for constant intensity to consider influence of the number of stories. The number of stories was directly tied to the building height.

$$\delta_i = \left(\frac{h_i}{H_n}\right)^{1-NS_s/165} \text{ for } N > 4$$

Equation 5-1

Where:

i = level i of the structure

n = top level of the structure

N = Number of story of building

H = total height of the structure

h = floor height

Ss = mapped Maximum considered earthquake record (MCER), 5 percent damped spectral response acceleration parameter at short periods as defined in Section 11.4.1 ASCE 7-10 (2010).
Priestley et. al (2007), recommends considering three cases shown below. The new proposed method considers the level of earthquake intensity in the equation and covers these 3 cases.

Case 1: Where Yield displacement and design displacement are smaller than the maximum displacement of the response spectrum.

Case 2: Where yield displacement is smaller than the maximum displacement of the response spectrum and the design displacement is larger than the maximum displacement of the response spectrum.

Case 3: Where yield and design displacements are larger than the maximum displacement of the response spectrum.

For each of the three cases the new proposal a) uses a base design response spectrum for 100% which corresponds to a high seismic region and b) modifies the analysis depending on the location because $S_\text{s}$ is included in equation 5-1

Based displacement response spectrum for design

Figure 5.1 shows the base Displacement Response Spectrum (DRS) that should be used for the DDBD procedure for all cases in conjunction with the proposed equation. This base DRS is calculated using $S_\text{s}=1.65$ and $S_1=0.68$, $F_a=1.0$ and $F_v=1.3$, $T_c$=corner period =8 sec. as per mapped TL values in ASCE 7 Chapter 22 (2005). This DRS should be modified for different values of $F_a$ and $F_v$ according to soil conditions at the site of interest.
a. New vertical force distribution profile shown in Figure 5.2 should be used to better match the shear force distribution shown in Figures 5.3. It was observed that the maximum story shear from the NLTHAs was not occurring at the base. Therefore a new equation that considered this effect was developed. To obtain this distribution a negative force had to be applied at the lower floors of the structure. From the shear distribution shape shown in Figure 3.44, it was observed that the equation shape used by Priestley et al. (2007) was adequate to represent the shear at the upper floors. Since the maximum shear observed was obtained for floor 2 to 5 it was decided that the maximum shear in the equation was on the 3rd floor on average. The negative force that matched the shear distribution was dependent on the number of floors of the frame with a
factor of \( \frac{1}{4} \) to better match the distribution observed. The following is the proposed vertical force distribution for analysis using DDBD. From the third level and above the distribution is equal to the following equation:

\[
F_i = \frac{V_{\text{Base}}(m_i \Delta_i)}{\sum_{i=1}^{n}(m_i \Delta_i)}
\]

Equation 5-2

This is the same as the recommendations Priestley et al. (2007)

b. The distribution for the level below the third floor is equal to the following equation:

\[
F_i = -\frac{N}{4}V_{\text{Base}}(m_i \Delta_i) / \sum_{i=1}^{n}(m_i \Delta_i)
\]

Equation 5-3

Where:

- \( N \) = number of story of the frame or building being analyzed
- \( V_{\text{Base}} \) = Base Shear from DDBD analysis
- \( m_i \) = Mass of story \( i \)
- \( \Delta_i \) = Displacement value at level \( i \)

c. At level 3 use \( F=0.0 \)

d. The unbalance base shear from equations above should be placed at the top floor, and this would account for high mode effect of the structure. This unbalance value is in the order of 7.5% of the base shear for the 20 story frame and of 10% for the eight story frame.
Figure 5.2 Proposed vertical force distribution

Figure 5.3 shows examples of the shear distribution for the 8, 12 and 20 story frames using the new proposed procedure. In Chapter 6 it will be shown to match the drifts obtained using NLTHA distribution.
Figure 5.3 Examples of shear distribution for frames

NLTH analysis with the new proposed displacement profile shape and equations for lateral force distribution will be performed for regular and irregular frames, comparisons and analysis of the results will be discussed in Chapter 6.
CHAPTER 6

DDBD implementation for building with new proposed method

6.1 Introduction

This chapter presents the results of NLTHA of the frames designed using the new proposed displacement profile and distribution of shears along the height of the building. Since the objective of this chapter is to evaluate the new proposed method with respect to the drifts obtained using NLTHA, first it was performed the design using the new proposed method but without using the additional forces recommended by the existing procedure to take into account the P-Δ effect. The NLTHA were performed with and without the consideration of P-Δ effect and the comparison of the results, with the design drifts, were used to evaluate the need for a new method to consider the P-Δ effect which was done in Chapter 7. The NLTHA was performed on regular and irregular buildings as will be described below. The results will be discussed, analyzed and conclusions will be drawn from this study. Then the objective of the study of the different buildings was to evaluate if the design using the proposed method would give acceptable drift limits below the postulated design drift limits when a NLTHA is performed.

6.2 Building geometry definition

Three regular frames 8, 12, 20 stories and nine irregular frames 8, 12 and 20 stories were analyzed using the new displacement profile equation and the new vertical force distribution. The regular frames consist of 3 bays 8.0 m long. The irregular frames consist of three frames
with 2 bays 4.5 m and 7.5 m long, three frames with 3 bays 4.5 m, 7.5 m and 4.5 m long and three frames with 5 bays with the first 2 bays 4.5 m long and the next 3 bays 7.5 m long.

Figures 6.1, 6.2, 6.3 and 6.4 show the geometry, masses and design drift for the 8 story frame. The 12 and 20 story frames were similar but with different number of stories.

\( L_b = \) Beam length

\( h_b = \) Beam height

\( \varepsilon_y = \) Steel yield strain

\( m_i = \) Mass at each floor

\( m_t = \) Mass at the top of the building

---

**Figure 6.1 Example of 8 Story regular frame geometry**

\[ L_b = 8.00 \text{ m (Bay length)} \]

\[ h_b = 0.57 \text{ m (Beam depth)} \]

\[ \varepsilon_y = 0.002 \text{ (Steel Strain)} \]

**Mass**

\[ m_i = 6605.50 \text{ kg each story} \]

\[ m_t = 6116.21 \text{ kg top story} \]
Figure 6.2 Example of 8 Story 3 bay irregular frame geometry

Figure 6.3 Example of 8 Story 2 bay irregular frame geometry
6.3 Building design regular frames

The new proposed method was used to calculate the displacement shape and load distribution along the height.

Table 6-1 shows the base shears of the proposed method and comparison with the existing DDBD method for 100% intensity level earthquake. The comparison shows that, for the existing method, the base shears are very similar for the 8, 12 and 20 story frames with a change of 5% from the 8 story frame to the 20 story frame. With the new proposed process, the base shear is not nearly constant, and changes of 30% between the 8 and 12 story frame and 48% between the 8 and 20 story frames are observed. This observation is more in accord with the intuitive concept that a taller building would have a larger base shear.

Figure 6.4 Example of 8 Story 5 bay irregular frame geometry

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L_b$</td>
<td>varies</td>
<td>(Bay length)</td>
</tr>
<tr>
<td>$h_b$</td>
<td>0.57 m</td>
<td>(Beam depth)</td>
</tr>
<tr>
<td>$\varepsilon_y$</td>
<td>0.002</td>
<td>(Steel Strain)</td>
</tr>
</tbody>
</table>

**Mass**

- $m_i$: 6605.50 kg each story
- $m_t$: 6116.21 kg top story
The structural analysis and design of the frame structures were performed using the design features of the computer program SAP2000 (SAP CSI Computer and Structure Inc., 2012). The design was done using ACI 318 provisions (American Concrete Institute ACI-318-05). Table 6-3 presents the result of this design. Buildings with 4, 8 and 12 stories had the beams and columns with the same geometry and reinforcement for all floors. The 20 story building had one type of uniform reinforcement for levels 1 thru 10 and a different uniform reinforcement for levels 11 thru 20.

In order to verify that the new proposed equation is reasonable and gives increasing base shears with increasing intensity levels, base shears for 50% 100% and 200% were calculated for the twenty story building and compared, Table 6.2 shows this comparison.

<table>
<thead>
<tr>
<th>Table 6-1 Base shear results for 100% intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base shear</td>
</tr>
<tr>
<td>Vb existing eq.</td>
</tr>
<tr>
<td>Vb new eq.</td>
</tr>
</tbody>
</table>
### Table 6-2 Comparison of base shear for design at different intensity levels for the 20 story building

<table>
<thead>
<tr>
<th>Intensity</th>
<th>Base shear</th>
</tr>
</thead>
<tbody>
<tr>
<td>50%</td>
<td>527.6 kN</td>
</tr>
<tr>
<td>100%</td>
<td>873.8 kN</td>
</tr>
<tr>
<td>200%</td>
<td>2554.5 kN</td>
</tr>
</tbody>
</table>

### Table 6-3 Results of design with new proposed method

<table>
<thead>
<tr>
<th>Building name</th>
<th>Columns size</th>
<th>Column steel ratio</th>
<th>Beam size</th>
<th>Beam steel ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building 1 4 story</td>
<td>400x400 mm</td>
<td>2.45%</td>
<td>300x570 mm</td>
<td>1.85, 0.88, 1.85%</td>
</tr>
<tr>
<td>Building 2 8 story</td>
<td>500x500 mm</td>
<td>3.40%</td>
<td>300x570 mm</td>
<td>1.80, 0.65, 1.80%</td>
</tr>
<tr>
<td>Building 3 12 story</td>
<td>650x650 mm</td>
<td>1.75%</td>
<td>300x600 mm</td>
<td>1.75, 0.65, 1.75%</td>
</tr>
<tr>
<td>Building 4 20 story floors 1 to 10</td>
<td>850x850 mm</td>
<td>1.27%</td>
<td>300x700 mm</td>
<td>1.90, 0.70, 1.90%</td>
</tr>
<tr>
<td>Building 4 floors 11 to 20</td>
<td>850x850 mm</td>
<td>1.00%</td>
<td>300x700 mm</td>
<td>1.70, 0.65, 1.70%</td>
</tr>
</tbody>
</table>

Table 6-4 are the design results with the existing DDBD procedure.
Table 6-4 Result of the design presented in chapter 3 (same table as Table 3-5)

<table>
<thead>
<tr>
<th>Building name</th>
<th>Columns size</th>
<th>Column steel ratio</th>
<th>Beam size</th>
<th>Beam steel ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building 1</td>
<td>400x400 mm</td>
<td>2.43%</td>
<td>300x570 mm</td>
<td>1.85, 0.95, 1.85%</td>
</tr>
<tr>
<td>Building 2</td>
<td>500x500 mm</td>
<td>3.40%</td>
<td>300x570 mm</td>
<td>1.80, 0.65, 1.80%</td>
</tr>
<tr>
<td>Building 3</td>
<td>600x600 mm</td>
<td>1.70%</td>
<td>300x570 mm</td>
<td>1.75, 0.65, 1.75%</td>
</tr>
<tr>
<td>Building 4</td>
<td>750x750 mm</td>
<td>1.30%</td>
<td>300x570 mm</td>
<td>1.80, 0.65, 1.80%</td>
</tr>
<tr>
<td>floors 1 to 10</td>
<td>600x600 mm</td>
<td>1.00%</td>
<td>300x570 mm</td>
<td>1.55, 0.60, 1.55%</td>
</tr>
</tbody>
</table>

A comparison of these 2 tables shows that, for building 1 and 2, there is no change in the design. For building 3 column size increases from 600x600 mm to 650x650 mm with a change in steel reinforcement from 1.7% to 1.75%, the beams height increases from 570 mm to 600 mm. Similarly for building 4 it can be observed from Table 6-3 a-n increase of column size, beam size and steel reinforcement, these are significant changes for the 20 story frame.

6.4 Results of NLTHA using the new proposed displacement profile and force distribution for the displacement profiles

NLTHAs were performed using 100% intensity earthquake level for the 20 story frame for the 20 records described in Section 3.3, and these results were compared to the design with existing DDBD procedure for the same intensity.
Figure 6.5 shows the displacement profiles using both the existing process and the new proposed method. The solid lines are the profile shapes using the existing and the proposed method, and the dotted lines are the profile shapes for the average of the 20 records from the NLTHA of the structures. This figure shows that the displacement obtained using the existing equation and the new proposed equation match the results of NLTHA for both cases.

6.5 Results with the new proposed displacement and force distribution profiles for the drift profiles

The four regular frames were designed using the 100% intensity level earthquake with the new proposed method and analyzed using NLTHA with 100% and 200% intensity level of the 20 ground motion records and the results were averaged as presented in Chapter 3. The following figures show the drift profiles using the new displacement profile equation and the
new vertical force distribution compared to the existing approach. In the Figure 6-6 the black solid line shows the average of drift profile of the 20 story frame designed for 100% intensity and subjected to 100% intensity. The blue line correspond to the average of drift profile frame designed for 100% intensity base shear design but subjected to 200 % intensity Level. The figure shows that the black line corresponding to the building designed with 100% intensity and analyzed using earthquakes with 100% intensity is below the design drift. On the other hand, when the buildings is analyzed using an earthquake with 200% intensity level the design drifts are exceeded by about 50% at floors 3 to 11. This validates the new proposed method because with the new method the behavior of the building shows drifts below the design drift and when a larger earthquake (200%) is applied the drifts are larger than the design drifts.

![Drift profile 20 story frame](image)

**Figure 6.6 Drift profile 20 story frame**
Two additional studies were performed; the building was designed using the 200% intensity level with the new proposed equations and the NLTHA was performed using the 200% percent intensity level with and without P-Δ. Figure 6.6 shows average drift profiles: a) frame designed using the new proposed method considering a 100% intensity level and analyzed using 100% intensity level b) frame designed using the new proposed method considering a 100% intensity level and analyzed using 200% intensity level, c) frame designed using the new proposed method considering a 200% intensity level and analyzed using 200% intensity level without P-Δ effect d) frame designed using the new proposed method considering a 200% intensity level and analyzed using 200% intensity level with P-Δ effect. The drift for the 200% design and 200% level of intensity without P-Δ effect is below the design drift indicating that the new proposed process is adequate. However the drift for the 200% design and 200% level of intensity with P-Δ effect exceeds the design drift by approximately 35% indicating that, for the extreme event, the drifts could exceed the design drift.
Figure 6.7 shows the comparison of results between the existing design equations and the new proposed equations for the 20 story. The building was designed using the 100% intensity level earthquake with the new proposed method and analyzed using NLTHA with 100% intensity level with and without P-Δ. The results of the NLTHA using 100% earthquake level indicate that, for a design at 100% intensity level and an analysis at 100% intensity level, the drift is below the design drift validating the new proposed method with and without consideration of P-Δ effect. The other 2 drift profiles correspond to the building designed using the existing method at 100% intensity level and analyzed at 100% intensity level with and without P-Δ. The drift profiles exceed the design drift in both cases; these results were obtained in Chapter 3 and are shown here for comparison.
6.6 Conclusions of the comparison of new proposed method for the regular frames

As observed in Figures 6.6 to 6.8 the NLTHA of the frame designed using the new proposed method yields drifts lower than the design drift which indicates that the new proposed displacement profile equation and the new proposed lateral force distribution result in a better design than the existing one.

6.7 Verification for the 50% and 200% drift profiles and base shear calculation using new equations.

To verify the logic, and adequacy of the new proposed method when different intensity levels are used in the design with regards to base shear level and drift profile, the 20 story building was designed using a 50% and 200% intensity level using the new proposed method. These designs were analyzed with NLTHA using 50% and 200% intensity level earthquakes.
The values of base shear shown below were extracted from Table 6-2 above, the base shears are as follows:

\[ V_{b50\%} = 527.6 \text{ kN} \]

\[ V_{b200\%} = 2554.0 \text{ kN} \]

This comparison shows that the base shear increases when the design is performed for higher levels of intensity. This result follows simple logic, larger earthquake leads to larger forces.

Figure 6.9 shows the calculated drift profiles, the drifts are kept below the design drift for both levels of earthquake, validating the new proposed method.

![Drift profile example](image-url)

**Figure 6.9** Drift profile examples at 50% and 200% intensity
6.8 Building design of irregular frames

The three additional irregular frame configurations were designed considered using 100% intensity levels, the proposed lateral force distribution forces and the new proposed displacement profile without considering P-Δ. The NLTHAs were performed with and without P-Δ. For the calculation of the yield rotation used to calculate the lateral forces using DDBD, it was considered that all beam members have the same yield rotation. The structural analysis and design of the frame structures were performed using the design features of the computer program SAP2000 (SAP Computer and Structure Inc., 2012). The design was done using ACI 318-05 provisions (American Concrete Institute 2005). Details of the sections sizes and reinforcement ratios are presented in Tables 6-5 thru 6-7

<table>
<thead>
<tr>
<th>Building name</th>
<th>Columns size</th>
<th>Column steel ratio</th>
<th>Beam size</th>
<th>Beam steel ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building 8 story</td>
<td>550x550 mm</td>
<td>1.45%</td>
<td>300x600 mm</td>
<td>1.40,0.81,1.40%</td>
</tr>
<tr>
<td>Building 12 story</td>
<td>600x600 mm</td>
<td>1.90%</td>
<td>300x600 mm</td>
<td>1.55,0.81,1.55%</td>
</tr>
<tr>
<td>Building 20 story</td>
<td>750x750 mm</td>
<td>2.52%</td>
<td>300x700 mm</td>
<td>1.60,0.80,1.60%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Building name</th>
<th>Columns size</th>
<th>Column steel ratio</th>
<th>Beam size</th>
<th>Beam steel ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building 8 story</td>
<td>600x600 mm</td>
<td>2.05%</td>
<td>300x600 mm</td>
<td>1.77,0.81,1.77%</td>
</tr>
<tr>
<td>Building 12 story</td>
<td>650x650 mm</td>
<td>1.66%</td>
<td>300x600 mm</td>
<td>1.70,0.81,1.70%</td>
</tr>
<tr>
<td>Building 20 story</td>
<td>850x850 mm</td>
<td>2.35%</td>
<td>300x700 mm</td>
<td>1.75,0.80,1.75%</td>
</tr>
</tbody>
</table>
### Table 6-7 Building 1 irregular five bays

<table>
<thead>
<tr>
<th>Building name</th>
<th>Columns size</th>
<th>Column steel ratio</th>
<th>Beam size</th>
<th>Beam steel ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building 8 story</td>
<td>500x500 mm</td>
<td>2.05%</td>
<td>300x700 mm</td>
<td>0.91, 0.40, 0.91%</td>
</tr>
<tr>
<td>Building 12 story</td>
<td>600x600 mm</td>
<td>1.82%</td>
<td>300x700 mm</td>
<td>1.05, 0.40, 1.05%</td>
</tr>
<tr>
<td>Building 20 story</td>
<td>850x850 mm</td>
<td>2.35%</td>
<td>300x750 mm</td>
<td>1.48, 0.50, 1.48%</td>
</tr>
</tbody>
</table>

### 6.9 Nonlinear Time History analysis of irregular frames

The nine irregular frames were analyzed using NLTHA. Figures 6.10 thru 6.18 show the average drift profiles obtained using the new proposed displacement profile equation and vertical force distribution for 100% intensity level of the 20 ground motion records.

Figures 6.10 thru 6.12 show the average drift profile for the 8 story frame with 3, 2 and 5 bays for the irregular buildings. The results of the NLTHA without P-Δ show the drift profiles are at or below the design drift, with a minor exceedance of 4% for the 3 bay frame. The results of the NLTHA with P-Δ show the drift profiles exceed the design drift with the exception of the 2 bay frame. This indicates that P-Δ should be considered in the design even for the 8 story irregular frames.
Figure 6.10 Drift profile 8 story 3 bays irregular frame

Figure 6.11 Drift profile 8 story 2 bays irregular frame
Figures 6.13 thru 6.15 show the average drift profile 12 story frame with 3, 2, 5 bays irregular building. The results of the NLTHA without P-Δ show the drift profiles are at or below the design drift, with a minor exceedance of 2% for the 2 bay frame. The results of the NLTHA with P-Δ show the drift profiles exceed the design drift. This also indicates that P-Δ should be considered in the design for the 12 story irregular frames.
Figure 6.13 Drift profile 12 story 3 bays irregular frame

Figure 6.14 Drift profile 12 story 2 bays irregular frame
Figures 6.16 thru 6.18 show the average drift profile 20 story frame with 3, 2, 5 bays irregular building. The results of the NLTHA without P-Δ show the drift profiles are at or below the design drift, with a minor exceedance of 7% for the 3 bay frame. The results of the NLTHA with P-Δ show the drift profiles exceed the design drift. This also indicates that P-Δ should be considered in the design for the 20 story irregular frames.
Figure 6.16 Drift profile 20 story 3 bays irregular frame

Figure 6.17 Drift profile 20 story 2 bays irregular frame
6.10 Conclusions of the comparison of new proposed method for the irregular frames

As observed in Figures 6-10 to 6-18 the NLTHA of the frame designed using the new proposed method yields drifts lower the design drift with some minor exceedance of 2% to 7% which indicates that the new proposed displacement profile equation and the new proposed lateral force distribution result in a better design than the existing one. Result of the analyses when using the P-Δ effect show that they exceed the design drift for most of the cases presented.

6.11 Conclusions regular and irregular frames

The new proposed method, yields members that, have a larger section and larger reinforcement ratios. These larger members are the result of larger design forces obtained with the new proposed procedure; this is necessary to keep the resultant drifts below the design drift.
Drift profiles for regular and irregular frames have the same shape, having the maximum drift occurring at locations other than the first story and maximum at the 2 thru 5 levels of the structure.

From the analysis performed of buildings designed without using P-Δ effect, one may conclude that in general a design using the new proposed method for DDBD will give interstory drifts below the postulated design drift and therefore the buildings would behave as designed. However, when considering P-Δ effect in the NLTHA the results indicate that the design drift limit is exceeded for most of the frames considered, and a modification of the existing method and proposed method shall include P-Δ effect. Proposed modification to the DDBD procedure, when P-Δ effect is included, will be considered in Chapter 7.
CHAPTER 7

Evaluation and new proposed method to consider P-Δ effect for DDBD

7.1 Introduction to the problem of consideration of P-Δ Effect

The objective of this chapter is to investigate the effect of P-Δ and propose a new method to include this effect in DDBD. In Chapter 6, it was found that the design using the new proposed method yielded drifts that were higher than the design drift when NLTHA was used taking into account the effect of P-Δ. The analysis of the different buildings was performed to evaluate if the design using the proposed method would give acceptable drift limits below the postulated design drift limits. The earthquake records considered for this study are the same 20 used in Chapter 3 and the intensity level was 100%

7.2 Buildings definition to evaluate P-Δ effect

Since the axial load is higher, the effect of P-Δ is more pronounced for a tall building. The buildings considered were 3 regular and 3 irregular frames with 15, 20 and 30 stories. These six concrete frames which have a stability index ($\theta$) larger than 10%, where $\theta$ is the stability index of the structure calculated with the following equation: $\theta_d = \frac{P\Delta_d}{VH_e}$ where P is equal to the effective mass times gravity, $H_e$ is the effective height of the equivalent structure, $\Delta_d$ is the design displacement, and V is the base shear.

These buildings were designed using the new proposed displacement and force distribution patterns described in Chapter 5 but with the existing method by Priestley et al.
(2007) to consider the effects of P-Δ. Figure 7.1 shows the typical span and story height for the 15 story building, spans and story height for the 20 and 30 story building were the same as for the 15 story shown. Table 7-1 shows the building geometry.

Figure 7.1 a) Regular configuration and b) Irregular configuration
7.3 Design of the buildings

The design of the buildings was performed using the new proposed method with the existing consideration of P-Δ by Priestley et al. (2007) and similarly to the previous chapters using SAP2000 (SAP, CSI, Computer and Structure Inc., 2012) and ACI (American Concrete Institute ACI-318-05) for the concrete design.

Table 7-2 presents the result of the design dimensions and steel ratios for the columns and beams using the proposed methods for displacement and force distribution and the existing method for considering the P-Δ effect.
<table>
<thead>
<tr>
<th>Building name</th>
<th>Columns size</th>
<th>Column steel ratio</th>
<th>Beam size</th>
<th>Beam steel ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building 1</td>
<td>700x700 mm</td>
<td>1.15%</td>
<td>300x700 mm</td>
<td>1.37,0.46,1.37%</td>
</tr>
<tr>
<td>Regular</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15 stories</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Building 2</td>
<td>750x750 mm</td>
<td>1.30%</td>
<td>300x700 mm</td>
<td>1.80,0.65,1.80%</td>
</tr>
<tr>
<td>Regular</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20 stories</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Building 3</td>
<td>1100x1100 mm</td>
<td>1.78%</td>
<td>300x750 mm</td>
<td>1.78,0.71,1.78%</td>
</tr>
<tr>
<td>Regular</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30 stories</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Building 4</td>
<td>700x700 mm</td>
<td>1.54%</td>
<td>300x600 mm</td>
<td>1.62,0.80,1.62%</td>
</tr>
<tr>
<td>Irregular</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15 stories</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Building 5</td>
<td>750x750 mm</td>
<td>3.48%</td>
<td>300x700 mm</td>
<td>1.84,0.64,1.84%</td>
</tr>
<tr>
<td>Irregular</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20 stories</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Building 6</td>
<td>1200x1200 mm</td>
<td>2.46%</td>
<td>400x700 mm</td>
<td>1.53,0.49,1.53%</td>
</tr>
<tr>
<td>Irregular</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30 stories</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The sections and reinforcement ratios for these design are larger than in Chapter 6 for the case of the 20 story frame, due to the consideration of P-Δ.

7.4 NLTH analysis results of building designed using the new proposed method for displacement and force distribution and the existing method to consider the P-Δ effect.

The six frames designed using the new proposed method for displacement and force distribution, and the existing method to consider the P-Δ effect were analyzed using NLTHA
with 100% intensity level earthquakes. The following figures present the results of the NLTH analysis.

Figure 7.2 presents the results of the average of the 20 earthquakes for the 15 story building. The figure shows that the design drift is exceeded for both the regular and irregular frame, the exceedance is approximately by 37% for the irregular frame. This indicates that the existing equations to consider the P-Δ effect needs to be revised. A new equation is proposed later in this chapter, and new design and NLTHA will be performed to test the adequacy of the new proposed method.

![Figure 7.2 Drift profile 15 story frame](image)
Figure 7.3 presents the results of the average of the 20 earthquakes for the 20 story building. The figure shows that the design drift is exceeded for both the regular and irregular frame by approximately 45%. This also indicates that the existing equations to consider the P-Δ effect need to be revised.

![Figure 7.3 Drift profile 20 story frame](image)

Figure 7.4 presents the results of the average of the 20 earthquakes for the 30 story building. The figure shows that the design drift is minimally exceeded for the regular frame. This indicates that the existing equations to consider the P-Δ effect may be adequate for buildings taller than 30 stories.
7.5 Discussion of results

Figures 7.2, and 7.3 show that for the 15 and 20 story frames the drift profile obtained from the NLTHA exceeds at levels two to level five, by approximately 35 to 45% and a new equation may be needed to represent the P-Δ effect. Figure 7.4 shows that, for the 30 story frame, the existing method is suitable for regular and irregular frames when P-Δ is considered.

7.6 New proposed modification to consider the effect of P-Δ

Design Recommendation in DDBD is to increase the design base shear force by a factor C, times the shear force produced by the P-Δ moment. The following equation was proposed by Priestley et al. to consider that increase in shear design. The value of C for concrete buildings recommended was C=0.5.
\[ F = K_d \Delta_d + C \frac{P\Delta_d}{H} \]  

Equation 7-1

In the comparisons of the drift obtained with the NLTHA performed and the design drift in the sections above it is clear that, a new process to consider the effect of P-\(\Delta\) is needed. A new method is proposed to modify the design equation for frames under 30 stories high, for frames 30 stories high and above the existing method but with the new proposed displacement profile and the new lateral load distribution may be used.

Three values of C were used to perform the design, C=0.5, C=0.75, and C=1.0. This was done to evaluate what was the effect of C in the results for the drift profile. Figure 7.5a) shows the results for C=0.5, in this figure the design drift is exceeded by 40% at level 3 indicating that the use of C=0.5 gives results that are unconservative. Figure 7.5b) shows the results for C=0.75, in this figure the design drift is exceeded by 23% at level 3 also indicating that the use of C=0.75 gives results that are unconservative Figure 7.5c) shows the results for C=1.0, in this figure the design drift is not exceeded at any level indicating that the use of C=1 gives good results. Figure 7.4 shows the results for C=0.5 for the 30 story building, in this figure the resulting drifts are below the design. From this analysis, it is recommended to use C=1.0 for concrete frame buildings below 30 stories.
Figure 7.5 a) Drift profile 20 story frame, C=0.5 b) Drift profile 20 story frame, C=0.75
From the analysis of the results of NLTHA it was observed that the strength and effective stiffness of the frames had to be increased to reduce the interstory drift for buildings with less than 30 stories. Figure 7.6 shows the force displacement curves when P-Δ is considered, the designs using existing equation 7-1 when C=0.5 has proven to give drifts higher than the design drift for building with less than 30 stories when NLTHAs were performed. From Figure 7.6 it can be observed that use of C greater than 0.5 has the effect of increasing the strength and increasing the effective stiffness as it is shown for the case presented with C=1

\[ F = K_e \Delta_d + C \frac{P \Delta_d}{H} \]  

Equation 7-1
It is clear from the discussion above that the value of $C=1.0$ results in a design that when analyzed using NLTHAs including P-Δ gives drift profiles below the design drift.

The new proposed method is as follows:

The following equation shall be used with the new proposed P-Δ strength enhancement:

$$ F = K_e \Delta_d + C \frac{P \Delta_d}{H} $$  \hspace{1cm} \text{Equation 7-2}$$

This equation may be also be written as equation 7-3:

$$ F = K_e \Delta_d (1 + C \theta) $$ \hspace{1cm} \text{Equation 7-3}
Where $\theta$ is the stability index of the structure calculated with the following equation:

$$\theta_D = \frac{P\Delta_d}{VH_e}$$  \hspace{1cm} \text{Equation 7-4}

Where $P$ is equal to the effective mass times gravity, $H_e$ is the effective height of the equivalent structure, $\Delta_d$ is the design displacement, and $V$ is the base shear.

$C=1.0$ for building up to 30 stories.

Equation 7-3 becomes equations 7-5 and 7-6:

For building 30 stories and taller the following equation may be used.

$$F = K_e\Delta_d(1+0.5\theta)$$  \hspace{1cm} \text{Equation 7-5}

For building shorter than 30 stories

$$F = K_e\Delta_d(1+\theta)$$  \hspace{1cm} \text{Equation 7-6}

7.7 Design using new proposed modification to consider the effect of $P-\Delta$

The design with the new proposed method for displacement, distribution of forces and new proposed equation for $P-\Delta$ was performed similarly to the previous design. Table 7-3 shows the results from this design. The sections and the steel ratios were increased to reflect the result of the new higher base shear.
Table 7-3 also shows the comparison of the results of design with the new proposed method for displacement, distribution of forces and new proposed equation for P-Δ vs. the existing process for P-Δ. The design using the new proposed process is noted in bold and italics.

<table>
<thead>
<tr>
<th>Building name</th>
<th>Columns size</th>
<th>Column steel ratio</th>
<th>Beam size</th>
<th>Beam steel ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Building 1</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Regular 15 stories</strong></td>
<td>700x700 mm</td>
<td>1.63%</td>
<td>300x700 mm</td>
<td>1.71,0.60,1.71%</td>
</tr>
<tr>
<td>700x700 mm</td>
<td>1.15%</td>
<td>300x700 mm</td>
<td>1.37,0.46,1.37%</td>
<td></td>
</tr>
<tr>
<td><strong>Building 2</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Regular 20 stories</strong></td>
<td>850x850 mm</td>
<td>1.27%</td>
<td>300x700 mm</td>
<td>1.90,0.70,1.90%</td>
</tr>
<tr>
<td>750x750 mm</td>
<td>1.30%</td>
<td>300x700 mm</td>
<td>1.80,0.65,1.80%</td>
<td></td>
</tr>
<tr>
<td><strong>Building 3</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Regular 30 stories</strong></td>
<td>1300x1300 mm</td>
<td>2.42%</td>
<td>400x800 mm</td>
<td>2.22,0.99,2.22%</td>
</tr>
<tr>
<td>1100x1100 mm</td>
<td>1.78%</td>
<td>300x750 mm</td>
<td>1.78,0.71,1.78%</td>
<td></td>
</tr>
<tr>
<td><strong>Building 4</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Irregular</strong></td>
<td>700x700 mm</td>
<td>1.92%</td>
<td>350x600 mm</td>
<td>1.66,0.70,1.66%</td>
</tr>
<tr>
<td>700x700 mm</td>
<td>1.54%</td>
<td>300x600 mm</td>
<td>1.62,0.80,1.62%</td>
<td></td>
</tr>
<tr>
<td><strong>Building 5</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Irregular</strong></td>
<td>750x750 mm</td>
<td>3.93%</td>
<td>350x700 mm</td>
<td>1.68,0.52,1.68%</td>
</tr>
<tr>
<td>750x750 mm</td>
<td>3.48%</td>
<td>300x700 mm</td>
<td>1.84,0.64,1.84%</td>
<td></td>
</tr>
<tr>
<td><strong>Building 6</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Irregular</strong></td>
<td>1300x1300 mm</td>
<td>2.79%</td>
<td>500x800 mm</td>
<td>1.38,0.46,1.38%</td>
</tr>
<tr>
<td>1200x1200 mm</td>
<td>2.46%</td>
<td>400x700 mm</td>
<td>1.53,0.49,1.53%</td>
<td></td>
</tr>
</tbody>
</table>
7.8 Nonlinear Time History analysis with the new proposed method for displacement, distribution of forces and new proposed equation for P-Δ

The six frames designed using the new proposed methods for displacement and force distribution, and the new proposed method to consider the P-Δ effect were analyzed using NLTHA with 100% intensity level earthquakes. The following figures present the results of the NLTH analysis.

Figure 7.7 presents the results of the average of the 20 earthquakes for the 15 story building. The figure shows that the design drift is exceeded marginally for the regular and by 10% for the irregular frame. This indicates that the new proposed equations to consider the P-Δ effect is adequate since the exceedance is very small.

![Figure 7.7 Drift profile 15 story frame modified method.](image)
Figures 7.8 and 7.9 present the results of the average of the 20 earthquakes for the 30 story building. The figures show that the design drift is not exceeded for both figures; design drift is 10% higher than the maximum drift of the regular and irregular 20 story frames and 23% higher than the maximum of the 30 story regular frame. This indicates that the new proposed equations to consider the P-Δ effect is adequate for the 20 and 30 story buildings.

![Figure 7.8 Drift profile 20 story frame modified method.](image-url)
An additional study was performed to consider a 25 story frame:

a) A 25 story regular frame was designed using the proposed displacement shape equation of DDBD for 100% intensity and considering ACI code requirements for the strength of all members.

b) Analysis was performed using SAP200 for determining the analysis forces for design.

c) Values of $C=0.5$ and $C=1.0$ were considered

d) Drift profile was calculated for the averaged of the 20 earthquake records considered for values of $C$ in item c above.

Figure 7.10 shows the results of the drift for the average of the 20 earthquake records, it shows that when $C=0.5$ the resulted drift is larger than the design drift.
for the floors 3 to 6 and smaller for all other floors. When using $C=1$ the drifts are smaller than the design drift for all floors.

![Drift profile 25 story frame considering different values of C for the P-Δ analysis.](image)

**Figure 7.10** Drift profile 25 story frame considering different values of C for the P-Δ analysis.

Conclusion when using $C=1.0$ the resulted drift are smaller than the design drift for the 25 story frame. Therefore $C=1.0$ should be used.

### 7.9 Conclusion of the evaluation of the new method including the new proposed method for P-Δ effect.

The objective of the study of the different buildings was to evaluate, if the design using the proposed method would give acceptable drift limits below the postulated design drift limits, when a NLTHA was performed. From the results of the NLTHA presented in Figure 7.6 for
the regular 15 story building we can observe that at interstory 4 and 5 there is some minor exceedance of the design drift of approximately 4%, this exceedance may be considered acceptable since the drift is very close to the design drift. For the irregular building in Figure 7.6 the results of the NLTHA show that some exceedance occurs in stories 3 and 4, this exceedance is approximately 10%, since most of the interstories have drift that are below the design drift it may possible to justify the exceedance as acceptable since the percentage of exceedance is low.

For the 20 and 30 story building the results of the NLTHA presented in Figures 7.7 and 7.8 show that, for buildings designed using the new proposed procedure, the drifts obtained for regular and irregular buildings are all below the design drift, this demonstrates that if the building is designed using the new proposed method the obtained drift will be acceptable.

From the analysis above one may conclude that in general a design using the new proposed method for DDBD will give interstory drifts below the postulated design drift and therefore the building would behave as intended.

A summary of the new equations to consider the P-Δ effect is presented below:

The proposed method as presented in Section 7.7 is to use equation 7-3:

\[ F = K_e \Delta_e (1 + C\theta) \]

Equation 7-3

Where \( \theta \) is the stability index of the structure calculated with the following equation:
\[ \theta_D = \frac{P\Delta_d}{VH_e} \]  \hspace{1cm} \text{Equation 7-4}

C=1.0 for building up to 30 stories.

Equation 7-3 becomes equations 7-5 and 7-6:

For building of 30 stories the following equation may be used.

\[ F = K_e \Delta_d (1 + 0.5\theta) \]  \hspace{1cm} \text{Equation 7-5}

For building up to 30 stories

\[ F = K_e \Delta_d (1 + \theta) \]  \hspace{1cm} \text{Equation 7-6}
CHAPTER 8

Summary, General Conclusions and Recommendations

8.1 Introduction

The main objective of this work is to evaluate the DDBD with respect to its adequacy when different levels of intensity of the earthquake and the effect of P-Δ are considered. Chapter 1 consists of a brief introduction and the motivation for this study; Chapter 2 consists of a literature survey and the results of this survey. Chapter 3 consists of an evaluation of the existing DDBD method. Chapter 4 consists of several parametric studies in order to investigate the influence of several parameters on the shape of the drift profile. Chapter 5 consists of a presentation of the new proposed method. Chapter 6 consists of an evaluation of the new proposed method using NLTHA. Chapter 7 consists of an evaluation of the existing formulation for the consideration of P-Δ effect and a formulation of a new proposed method to consider the P-Δ effect. Chapter 8 presents a summary and the conclusion of this study.

8.2 Summary of chapter 2

Chapter 2 was a literature review of the DDBD procedure considering and not considering the P-Δ effect, from this review it was found that none of the authors studied the effect of the intensity of the earthquake on the deflected shape pattern of the structures, since the deflected shape pattern is one of the main parameter used on DDBD it was very important to study what is the effect of different levels of intensities on the shape patterns on the design of the structure.
8.3  Summary of chapter 3

Chapter 3 was a review of the existing method of the DDBD, 4 regular frames of 4, 8, 12 and 20 stories were studied for seven intensity level (25%, 50%, 75%, 100%, 125%, 150%, 200%) of earthquakes. A set of 20 earthquake records for near field and large magnitude were considered from past study done by Medina and Krawinkler (2013). The objective of this study was to evaluate the adequacy of the existing method regarding the impact of the intensity level on the deformed shape used for design. It was found a) the displacement pattern is a function of the intensity of the earthquakes and that the maximum drift was not occurring at the first story of the frames b) the maximum shear was not occurring at the first story of the frames c) the displacement shape pattern of the existing method is adequate for the 4 story frame but not for the 8 and higher story frames. d) deformed shape is influenced by the P-Δ effect, e) The P-Δ influences more the drift on the structure than the maximum displacement f) Displacement pattern is linear for the 4 story frame a becomes more nonlinear as the stories increases, g) It was concluded that a new displacement shape pattern should be developed, having in consideration the intensity of the earthquakes and also that a new lateral force distribution should be considered for design.

8.4  Summary of chapter 4

The goal of this chapter was to study the impact of different parameters on the drift profiles obtained using NLTHA. The parameters studied were: a) Influence of far field earthquakes, b) Influence of use of Spectrum compatible ground motion records, c) Influence of axial deformation in the drift profile, d) Influence of base support condition,
e) Influence of considering elastic behavior of columns above the first floor. It was concluded that the drift profile is not significantly influenced by any of the parameters studied with the exception of the change of boundary conditions at the base of the columns.

8.5 Summary of chapter 5

Chapter 5 presents the new proposed process to consider the level of earthquake intensity and the number of floors of the building. A new proposed equation for the displacement shape and new distribution of lateral loads along the height of the building were developed. A summary of the proposed procedure is presented in Section 8.3.

8.6 Summary of chapter 6

Chapter 6 presents the results of NLTHA of the frames designed using the new proposed displacement profile and distribution of shears along the height of the building, the design was performed without using the P-Δ effect. The NLTHA was performed on regular and irregular buildings. For the NLTHA 3 regular frames of 8, 12 and 20 stories and nine irregular frames 8, 12 and 20 stories each consisting of a) 2 bays 4.5 m and 7.5 m long, b) 3 bays 4.5 m, 7.5 m and 4.5m long and c) 5 bays with the first 2 bays 4.5 m long and the next 3 bays 7.5 m long were studied for seven intensity level (25%, 50%, 75%, 100%, 125%, 150%, 200%) of earthquakes. A set of 20 earthquake records was used, and the results were averaged.

The goal of the study of the different buildings was to evaluate if the design using the proposed method would give acceptable drift limits below the postulated design drift limits when an NLTHA was performed.
The new proposed method yields members with a larger section and larger reinforcement ratios. These larger members are the result of larger design forces obtained with the new proposed method; this is necessary to keep the resultant drifts below the design drift.

From the analysis performed of building designed without considering P-Δ effect, the design using the new proposed method for DDBD will give drifts below the postulated design drift and therefore the buildings would behave as designed. However, when considering P-Δ effect in the NLTHA the results indicate that the design drift limit is exceeded for most of the frames considered, and a modification of the existing method and proposed method shall include P-Δ effect.

8.7 Summary of chapter 7

The objective of this chapter was to investigate the effect of P-Δ and propose a new procedure to include this effect in DDBD, since, in chapter 6, it was found that the design using the new proposed method yielded drifts that were higher than the design drift when NLTHA was used taking into account the effect of P-Δ.

It was found that, for the 15 and 20 story frames, the drift profile obtained from the NLTHA exceeds from levels two to level five and a new equation was needed to represent the P-Δ effect. It was found that, for the 30 story frame, the existing method is suitable for regular and irregular frames when P-Δ is considered. A new proposed method was proposed to consider the P-Δ effect. NLTHAs were performed on the design using the new proposed method for displacement shape, load distribution and P-Δ effect equation.
The results of the NLTHA show that for the regular 15 story building there is some minor exceedance of the design drift of approximately 10%, this exceedance, may be consider acceptable since the drift is very close to the design drift. For the irregular building the results of the NLTHA show that some exceedance occurs in stories 3 and 4, this exceedance is approximately 8%, most of the drifts were below the design drift. This showed that the new proposed procedure was adequate to consider the P-Δ effect.

8.8 Conclusions

The literature survey showed that the previous authors had not considered the effect of the earthquake intensity levels.

- The study using NLTHA of the design using the existing method showed that the resultant drifts exceeded the design drift; the maximum drift does not occur at the first floor.
- A new method to consider the effect of earthquake intensity level was needed, it was developed and presented.
- Various buildings designed using the new proposed displacement and force distribution were analyzed using NLTHA
- It was found that using the existing method to consider the P-Δ effect the drift obtained was still above the design drift for building below 30 stories.
- A new method to consider the P-Δ effect was developed, and NLTHAs were performed to check the adequacy of the new method.
It was concluded that although the new method gave larger sections and reinforcement ratios the drifts were kept below the design drift.

8.9 Recommendations

8.9.1 New displacement profile shape, base displacement response spectrum for design and load distribution

The following is the proposed equation for displacement shape pattern

$$\delta_i = \left( \frac{h_i}{H_n} \right)^{1 - NS_i/165} \text{ for } N > 4 \quad \text{Equation 8-1}$$

Where:

- $i =$ level $i$ of the structure
- $N =$ Number of story of building
- $n =$ top level of the structure
- $S_s =$ mapped Maximum considered earthquake record (MCER), 5 percent damped

Base displacement response spectrum for design

It is recommended for the DDBD, that the base Displacement Response Spectrum (DRS), calculated using $S_s=1.65$ and $S_1=0.68$, $Fa=1.0$ and $Fv=1.3$, $Tc=$corner period $=8$ sec. be used for all cases in conjunction with the proposed equation. Different values of $Fa$ and $Fv$ may be used to consider the soil condition at the site of interest. The following is the proposed lateral force distribution along the height of the building:
e. From the third level and above the distribution is equal to the following equation:

\[ F_i = V_{\text{base}} m_i \Delta_i / \sum_{i=1}^{N} (m_i \Delta_i) \]  

Equation 8-2

f. The distribution for the level below the third floor is equal to the following equation:

\[ F_i = -(N/4) V_{\text{base}} m_i \Delta_i / \sum_{i=1}^{N} (m_i \Delta_i) \]  

Equation 8-3

Where:

N = number of story of the fame or building being analyzed

\( V_{\text{Base}} \) = Base Shear from DDBD analysis

\( m_i \) = Mass of story i

\( \Delta_i \) = Displacement value at level i

g. At level 3 use \( F=0.0 \)

h. The unbalance base shear from equations above should be placed at the top story

i. **New proposed modification to consider the effect of P-\( \Delta \)**

\[ F = K_{\Delta_g} (1 + C\theta) \]  

Equation 8-4

Where \( \theta \) is the stability index of the structure calculated with the following equation:
\[ \theta_d = \frac{P \Delta_d}{VH_e} \]  

Equation 8-5

Where \( P \) is equal to the effective mass times gravity, \( H_e \) is the effective height of the equivalent structure, \( \Delta_d \) is the design displacement, and \( V \) is the base shear.

\( C=1.0 \) for building up to 30 stories.
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Appendix A

A-1 Force Base Design (FBD), Equivalent Lateral Force (ELF) Procedure ASCE-7-05

The Force Based Design or Equivalent Lateral Load is a procedure where an equivalent lateral load is applied to the structure, this lateral load should be applied in each direction according to equation A-1, and distributed vertically in accordance to equation A-11

\[ V = C_s W \]  

Equation A-1

Where:

\( C_s \) = the seismic response coefficient determined in accordance with Sec. 12.8.1.1 (ASCE-7-05) and

\( W \) = the total dead load and applicable portions of other loads listed below:

a. 25 percent of the floor live load is included in \( W \). Some live loads are exempted such as garages and parking structures live loads.

b. A minimum weight should be included of 10 psf.

c. Total operating weight of permanent equipment should be included.

d. Snow loads in flat roof may be ignored if it is less than 30psf, when greater than 30 psf it may be reduced by 20%.
Equation A-2 is used to calculate the seismic coefficient $C_s$, this coefficient shall be
determined in accordance with the following equation:

$$C_s = \frac{S_{DS}}{R/I}$$  \hspace{1cm} Equation A-2

Where:

$S_{DS}$= the design spectral response acceleration parameter in the short period range as
determined from Sec. 11.4.4 or 11.4.7 (ASCE-7-05)

$R$= the response modification factor from Table 12.2-1, (ASCE-7-05) and

$I$= the occupancy importance factor determined in accordance with Sec. 11.5.1 (ASCE-
7-05).

The value of the seismic response coefficient computed in accordance with Equation
A-2 need not exceed the following:

$$C_s = \frac{S_{DI}}{T(R/I)} \quad \text{for} \quad T \leq T_L$$  \hspace{1cm} Equation A-3

and
\[ C_s = \frac{S_{DL} T_L}{T^2 R/I} \text{ for } T > T_L \]  \hspace{1cm} \text{Equation A-4}

Where \( R \) and \( I \) are as defined above and

\( S_{DL} \) = the design spectral response acceleration parameter at a period of 1.0 second as determined from Sec. 11.4.4 or 11.4.7, (ASCE-7-05)

\( T \) = the fundamental period of the structure (in seconds) determined in Sec. 12.8.2, (ASCE-7-05) and

\( T_L \) = long-period transition period (in seconds) determined in Sec. 11.4.5 (ASCE-7-05)

\( C_s \) shall not be taken less than 0.01.

For buildings and structures located where \( S_1 \) is equal to or greater than 0.6g, \( C_s \) will be calculated using equation A-5:

\[ C_s = \frac{0.5S_1}{R/I} \]  \hspace{1cm} \text{Equation A-5}

Where \( R \) and \( I \) are as defined above and

\( S_1 \) = the mapped maximum, considered earthquake spectral response acceleration parameter determined in accordance with Sec. 11.4.1 or 11.4.7. (ASCE-7-05)

A soil-structure interaction reduction is allowed where determined using Chapter 19 (ASCE-7-05)
A.1.1 Basic Procedure:

Period determination. The structural properties and deformational characteristics of the resisting elements are used to determine the fundamental elastic period of the structure, $T$, in the direction of the analysis. The fundamental period, $T$, shall not exceed the product of $C_u$, from Table 12.8-1 (ASCE-7-05) and $T_a$, calculated in accordance to equation A-6 as an alternative to performing a modal analysis to determine the fundamental period of the structure, $T$,

a) Approximate fundamental period. Equation A-6 is used to calculate the approximate fundamental elastic period, $T_a$, in seconds:

$$T_a = C_r h_n^x$$  \hspace{0.5cm} \text{Equation A-6}

Where

$hn$ is the height in feet (meters) above the base to the highest level of the structure and the values of $C_r$ and $x$ shall be determined from Table 12.8-2 (ASCE-7-05)

With restrictions to structures not exceeding 12 stories in height, the approximate fundamental period, $T_a$, in seconds, may be determined from equation A-7 for concrete and steel moment resisting frame structures and having a minimum story height of not less than 10 ft.

$$T_a = 0.1 N$$ \hspace{0.5cm} \text{Equation A-7}
Where

\( N = \text{number of stories.} \)

For other type of structures such as masonry or concrete shear walls, the approximate fundamental elastic period, \( T_a \), in seconds, is calculated using equation A-8:

\[
T_a = \frac{0.0019 \ h_n}{\sqrt{C_w}} \quad \text{Equation A-8}
\]

Where

\( C_w \) is a coefficient related to the effective shear wall area and \( h_n \) is as defined above.

The metric equivalent of Equation A-8 is:

\[
T_a = \frac{0.0062 \ h_n}{\sqrt{C_w}} \quad \text{Equation A-9}
\]

The coefficient \( C_w \) shall be calculated from equation A-10:

\[
C_w = \frac{100}{A_B} \sum_{i=1}^{x} \left( \frac{h_i}{h_n} \right)^2 \frac{A_i}{\left[ 1 + 0.83 \left( \frac{h_i}{D_i} \right)^2 \right]} \quad \text{Equation A-10}
\]

Where:

\( A_B = \text{the base area of the structure,} \)

\( A_i = \text{the area of shear wall } i, \)
Di = the length of shear wall i,

hn = the height above the base to the highest level of the structure,

hi = the height of shear wall i, and,

x = number of shear walls in the building effective in resisting lateral forces in the direction under consideration.

b) After calculating the approximate fundamental elastic period of the structure, the Cs coefficient is determined using corresponding equations A-2 to A-5.

c) Base shear is determined using equation A-1

d) Vertical distribution of seismic forces. The lateral force, Fx, at any level shall be determined from equation A-11:

\[ F_x = C_{vx} V \]  \hspace{1cm} \text{Equation A-11}

And

\[ C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \]  \hspace{1cm} \text{Equation A-12}

Where:

C_{vx} = vertical distribution factor,

V = total design lateral force or shear at the base of the structure,
wi and wx = the portion of the total gravity load of the structure, W, located or assigned to Level i or x,

hi and hx = the height from the base to Level i or x, and

k = an exponent related to the effective fundamental period of the structure as follows:

For structures having a period of 0.5 seconds or less, k = 1

For structures having a period of 2.5 seconds or more, k = 2

For structures having a period between 0.5 and 2.5 seconds, k shall be determined by linear interpolation between 1 and 2 or may be taken equal to 2.

e) Horizontal shear distribution. The seismic design story shear in any story, Vx, is determined from equation A-13

\[ V_x = \sum_{i=x}^{n} F_i \]  

Equation A-13

Where

Fi = the portion of the seismic base shear, V, induced at Level i.

f) Structural analysis is performed to distribute the seismic design story shear, Vx, of each floor to the structure vertical elements of the seismic-force-resisting system.

g) Buildings irregularities such as inherent torsion, accidental torsion, shall be taken in consideration when distributing the lateral force over the height.
h) Other irregularities such as vertical discontinuities, soft story and short columns must be avoided, or specially designed when considered in the structural analysis.

i) Dynamic amplification of torsion. For structures assigned to Seismic Design Category C, D, E, or F, where Type 1 torsional irregularity exists as defined in Table 12.3-1, (ASCE-7-05) the effects of torsional irregularity shall be accounted for by multiplying the sum of Mt plus Mta at each level by a torsional amplification factor, Ax, determined from equation A-14:

\[ A_x = \left( \frac{\delta_{\text{max}}}{1.2\delta_{\text{avg}}} \right)^2 \]

Equation A-14

Where:

\( \delta_{\text{max}} \) = the maximum displacement at Level x, and

\( \delta_{\text{avg}} \) = the average of the displacements at the extreme points of the structure at Level x.

The torsional amplification factor, Ax, is not required to exceed 3.0. The most severe loading for each element shall be considered for design.

j) Overturning. The structure shall be designed to resist overturning effects

The overturning moments at Level x, Mx, shall be determined from Equation A-15 as follows:
\[ M_x = \sum_{i=x}^{n} F_i (h_i - h_x) \]  \hspace{1cm} \text{Equation A-15}

Where:

Fi= the portion of the seismic base shear, V, induced at Level i, and

hi and hx = the height from the base to Level i or x.

The foundations of structures, except inverted pendulum-type structures, shall be permitted to be designed for three-fourths of the foundation overturning design moment, M_0, determined using Equation. A-15 at the foundation-soil interface.

k) Drift determination and P-\(\Delta\) limit. Structural analysis is performed to determine floor displacements and from those floor displacements story drift may be calculated. Story drifts shall be determined in accordance to code prescribe values, in case they exceed those code values, elements should be redesign, and the structure reanalyzed until compliance.

When the structure is analyzed the following consideration have to be considered:

1. Stiffness properties of reinforced concrete and masonry elements shall consider the effects of cracked sections and

2. For steel moment resisting frame systems, the contribution of panel zone deformations to overall story drift shall be included.
Story drifts determination. The design story drift, \( \Delta \), is computed as the difference of the deflections at the center of mass at the top and bottom of the story under consideration. The deflections of Level \( x \), \( \delta x \), shall be determined in accordance to equation A-16:

\[
\delta_x = \frac{C_d \delta_{xe}}{I}
\]

Equation A-16

Where:

- \( C_d \) = the deflection amplification factor from Table 12.2-1,
- \( \delta_{xe} \) = the deflections determined by an elastic analysis, and
- \( I \) = the occupancy importance factor determined in accordance with Sec. 11.5.1.

1) P-\( \Delta \) limit. Stability coefficient, \( \theta \), as determined for each level of the structure by equation A-17.

\[
\theta = \frac{P_x \Delta I}{V_x h_{le} C_d}
\]

Equation A-17

Where:

- \( W \) = the total vertical design load at and above Level \( x \). Where calculating the vertical
- \( P_x \) = design load for purposes of determining P-\( \Delta \) effects, the individual load factors, need not exceed 1.0.
\( \Delta \) = the design story drift calculated in accordance with Sec. 12.8.6. (ASCE-7-05)

I = the occupancy importance factor determined in accordance with Sec. 11.5.1, (ASCE-7-05)

\( V_x \) = the seismic shear force acting between Level x and x - 1.

\( h_{sx} \) = the story height below Level x.

\( C_d \) = the deflection amplification factor from Table 12.2-1., (ASCE-7-05)

The stability coefficient \( (\theta) \) shall not exceed \( \theta_{\text{max}} \) determined as follows:

\[
\theta_{\text{max}} = \frac{0.5}{\beta C_d} \leq 0.25
\]

Equation A-18

Where:

\( \beta \) is the ratio of shear demand to shear capacity for the story between Levels x and x - 1. This ratio is permitted to be conservatively taken as 1.0.

Where the stability coefficient \( (\theta) \) is greater than 0.10 but less than or equal to \( \theta_{\text{max}} \), the incremental factor related to P-\( \Delta \) effects on displacements and member forces shall be determined by rational analysis. Alternatively, it is permitted to multiply displacements and member forces by \( 1.0 / (1-\theta) \).

Where \( \theta \) are greater than \( \theta_{\text{max}} \), the structure is potentially unstable and shall be redesigned.
Where the P-Δ effect is included, in an automated analysis, Eq. A-18 shall still be satisfied, however, the value of θ computed from Eq. A-17 using the results of the P-Δ analysis is permitted to be divided by (1+θ) before checking Eq. A-18.
Appendix B

B.1 Design of the buildings

B.1.1 DDBD procedure Reinforced concrete Frame example

For the design of frames under study, the High seismicity area selected is Berkeley CA, the response spectra for that region was used as the based for high seismic loadings, (ASCE 7, 2005).

B.1.2 Development of seismic loads and design requirements

B.1.3.1 Seismicity

Using ASCE 7-05 Maps, Figures 22-1 and 22-2 (ASCE 7, 2005) for Berkeley, California.

The short period response acceleration parameters $SS$ is 1.65

The one-second period spectral and $S1$ is 0.68.

For the very dense soil conditions:

Site Class C is appropriate as described in ASCE 7-5 Sec. 20.3.3 (ASCE 7, 2005)

Using $SS = 1.65$ and Site Class C, Provisions Table 11.4-1 (ASCE 7, 2005) lists a short period site coefficient $Fa$ of 1.0.

For $S1 > 0.5$ and Site Class C, Provisions Table 11.4-2 (ASCE 7, 2005) gives a velocity based site coefficient $Fv$ of 1.3.

Using code Provisions Eq. 11.4-1 and 11.4-2 (ASCE 7, 2005), the maximum considered spectral response acceleration parameters for the Berkeley area are:
The design spectral response acceleration parameters are given by *Provisions* Eq. 11.4-3 and 11.4-4 (ASCE 7, 2005):

\[ S_{MS} = F_a S_S = 1.0 \times 1.65 = 1.65 \]

\[ S_{M1} = F_r S_1 = 1.3 \times 0.68 = 0.884 \]

The transition period \( T_s \) for the Berkeley response spectrum is:

\[ T_s = \frac{S_{DI}}{S_{DS}} = \frac{0.589}{1.10} = 0.535 \text{ Sec} \]

\( T_s \) is the period where the horizontal (constant acceleration) part of the design response spectrum intersects the descending (constant velocity or acceleration inversely proportional to \( T \)) portion of the spectrum.
Figure B.1 Acceleration response spectrum used as base seismic design for DDBD

B.1.3.2 Definitions

**Peak Ground Acceleration (PGA):** Defined as the maximum acceleration experienced at ground level due to an earthquake. PGA is the value at period equal to zero of an acceleration response spectra.

**Moment Magnitude (Mw):** Measure of the total energy released by an earthquake. The magnitude is directly related to the size of the rupture and the amount of slip on the rupture surface, and not based solely on empirical studies of the observed instrumental recordings. Hanks and Kanamori (1979) proposed an equation to determine the moment magnitude of earthquakes, which is given by:
\[ M_w = \frac{2}{3} \log_{10}(M_o) - 10.73 \]  

Equation B-1

Where \( M_o \) is the seismic moment (in dyne-cm; dyne-cm = 10E-7Nm) which is related to the area of rupture, the average slip across the rupture length, and the shear modulus of the rock.

**Corner Period (\( T_c \)):** It is the period at which the displacement spectra have its maximum value after which it becomes essentially constant, or reducing.

Several researchers have proposed expressions to obtain the corner period such as Faccioli et al (2004), which is presented in Equation B-2. This expression depends only on the moment magnitude (\( M_w \)). For earthquakes with moment magnitudes less than 5.7, the corner period (\( T_c \)) has a value of 1.0 sec.

\[ T_c = 1.0 + 2.5(M_w - 5.7) \]  

Equation B-2

In addition, the Federal Emergency Management Agency, NEHRP (1997) has recommended another expression to calculate the corner period, which is given by:

\[ \log_{10} T_c = -1.25 + 0.3M_w \]  

Equation B-3

Equation based on seismology theory.
**Corner Displacement (Dc):** It is the peak spectral displacement or displacement at the corner period at 5% damping. Faccioli et al (2004) have proposed an expression to obtain the peak spectral displacement, which is given by:

\[
\delta_{\text{max}} = \Delta_c = C_s \frac{10^{(M_s - 3.2)}}{r}
\]

**Equation B-4**

Where \( r \) (km) is the epicentral distance or nearest distance to the fault plane for a large earthquake, and \( C_s \) are soil coefficients. Some recommended values for \( C_s \) (Priestley et al, 2007) are:

- **Rock:** \( C_s = 0.7 \)
- **Firm Ground:** \( C_s = 1.0 \)
- **Intermediate Soil:** \( C_s = 1.4 \)
- **Very Soft Soil:** \( C_s = 1.8 \)

**Damping Reduction Factor (Rx):** This is a factor applied to the elastic displacement spectrum (5%) to account for the increased damping level. Eurocode EC8 (1998) proposed an expression to determine \( Rx \), which are presented next.

Using Faccioli’s et al (2004) expressions:

\[
R_x = \left( \frac{7}{2 + \xi} \right) \quad \text{Equation B-5}
\]
The displacement response spectrum is calculated, after calculating the accelerations response spectrum for the area sites, The Berkeley area will be calculated using the near field equation (Equation B-6 from Eurocode 8 (Eurocode 8, 1998) to determine the displacement response spectrum

\[ R_s = \left( \frac{\sqrt{7}}{\sqrt{2+\xi}} \right) \]  

Equation B-6

\[ S_d = S_{d,5\%} \left( \frac{\sqrt{7}}{\sqrt{2+\xi}} \right) \]  

Equation B-7

\[ S_d = S_{d,5\%} \left( \frac{4}{\sqrt{2+\xi}} \right) \]  

Equation B-8

From de acceleration response spectrum and using equations from dynamics of structures the displacement response spectrum can be evaluated using the following equation:

\[ S_{d,5\%} = S_{d,5\%} \frac{T^2}{4\pi^2} g \]  

Equation B-9

Corner period are calculated using equations:

\[ T_c = 1.0 + 2.5(M_w - 5.7) \]  

Equation B-10
Equation B-11

\[ \log_{10} T_c = -1.25 + 0.3M_w \]

If using a \( M_w = 7.0 \) for calculating the corner period at the Berkeley area.

\[ T_c = 1.0 + 2.5(7 - 5.7) = 4.25 \]

\[ \log_{10} T_c = -1.25 + 0.3 \times 7 = 0.85 \quad T_c = 7.08 \quad \text{Sec.} \]

As shown on equations above the values of \( T_c \) range from 4.25 Sec and 7.08 for the Berkeley area. ASCE 7-05 Figures 22-12 map the long period for the Berkeley location as \( T_c = 8 \) sec.

The value of \( T_c = 8 \) sec. will be used for calculating the base Displacement response spectrum for the analysis of the structures under study.
Figure B.2 Base Displacement response spectrum for DDBD

B.1.3.3 Structural and design Analysis

The structural analysis and design of the frame structures were performed using SAP2000 computer program from Computer and Structures Inc. (2012), and for the design of the members ACI 318-05 provision were considered.

B.1.3.4 DDBD Force Calculation

Example of base shear calculation using existing method of the DDBD for the 8 story frame structure:
a) Displacement Profile

\[ \Delta_i = \delta_i \cdot \left( \frac{\Delta_i}{\delta_i} \right) \]

\[ \delta_i = \frac{H_i}{H_n} \text{ for } N \leq 4 \]

\[ \delta_i = \frac{4}{3} \cdot \left( \frac{H_i}{H_n} \right) \cdot \left( 1 - \frac{H_i}{4H_n} \right) \text{ for } N > 4 \]
Target displacement is calculated as follows:

\[ \Delta_x = \frac{\sum_{i=1}^{n} (m_i \Delta_i^2)}{\sum_{i=1}^{n} (m_i \Delta_i)} = 0.36 \text{ m} \]

Effective mass and effective height are as follows:

\[ m_e = \frac{\sum_{i=1}^{n} m_i \Delta_i}{\Delta_t} = 44,786 \text{ Kgf-} \text{seg}^2/\text{m} \]

85.54% of total mass
\[ H_e = \sum_{i=1}^{n} m_i \Delta H_i / \sum_{i=1}^{n} m_i \Delta_e = 17.20 \text{ m} \]

68.80% of total height

Yield curvature and yield displacement

\[ \phi_y = 0.5 \varepsilon_y \frac{L_b}{h_s} = 0.014 \]

\[ \Delta_y = \phi_y H_e = 0.24 \text{ m.} \]

\[ \mu = \frac{\Delta_d}{\Delta_y} = 1.50 \quad \text{Displacement ductility} \]

\[ \xi_{eq} = 0.05 + 0.565 \left( \frac{\mu - 1}{\mu \pi} \right) = 0.11 \quad \text{Equivalent damping} \]

Figure B.4 DRS for design example
\[ T_{\text{eff}} = 3.38 \text{ sec.} \]

Effective period

\[ K_e = \frac{4\pi^2 m_e}{T_e^2} = 156,745 \text{ Kgf/m.} \]

Effective stiffness

\[ F_{\text{Base}} = V_{\text{Base}} = K_e \Delta_d = 56,600 \text{ Kgf} \]

Base shear

The vertical, lateral load distribution is distributed the following way: 90% of Base shear over height of the frame with the above equation, and, 10% of base shear at the top of the frame. As suggested by Priestley et al. The program SAP2000 was used to find the member forces and the design of members. SAP2000 were used to find the necessary ratios of steel for the columns and beams and to check that the capacity exceeded the demand for all elements.
Appendix C

Material and Properties definitions

**Material Properties:** The stress-strain relationship for confined and unconfined concrete in the elements is described as proposed by Mander et al. (1988) which is applicable to both circular and rectangular sections. The model for the confined concrete includes the enhancement of the strength on the core concrete due to confining pressure provided by the stirrups or ties. The relationship is described by the following equations:

\[
f_c = \frac{f'_{cc}xr}{r - 1 + x'}
\]

Equation C-1

Where:

\[
x = \frac{\varepsilon_c}{\varepsilon_{cc}}
\]

Equation C-2

\[
\varepsilon_{cc} = \varepsilon_{co} \left[1 + 5 \left(\frac{f'_{cc}}{f'_{co}} - 1\right)\right]
\]

Equation C-3

\[
r = \frac{E_c}{E_c - E_{sec}}
\]

Equation C-4
\[ E_{sec} = \frac{f'_{cc}}{\varepsilon_{cc}} \]  
Equation C-5

\[ f'_{cc} = f'_{co} \left( -1.254 + 2.254 \sqrt{1 + \frac{7.97 f'_{i}}{f'_{co}} - 2 \frac{f'_{i}}{f'_{co}}} \right) \]  
Equation C-6

In which, \( \varepsilon_c \) is the longitudinal compressive concrete strain.

\( \varepsilon_{cc} \) is the confined concrete strain.

\( f'_{cc} \) is the compressive strength of confined concrete.

\( f'_{co} \) is the unconfined concrete strength.

\( \varepsilon_{co} \) is the unconfined concrete strain (typically assumed as 0.002).

\( f_l \) is the effective lateral confining pressure.

\( E_e \) is the tangent modulus of elasticity of the concrete which is defined by:

\[ E_e = 5000 \sqrt{f'_{co}} \quad \text{Units: [MN,m]} \]  
Equation C-7

The effective confining pressure is evaluated for rectangular sections as follows:

\[ f'_{lx} = k_e \rho_x F_{yh} \]  
Equation C-8

\[ f'_{ly} = k_e \rho_y F_{yh} \]  
Equation C-9
\[ \rho_x = \frac{A_{sx}}{sd_c} \]  
\[ \rho_y = \frac{A_{sy}}{sd_c} \]

Equation C-10
Equation C-11

Where: \( \rho_x \) and \( \rho_y \) are the transverse steel ratios in the x and y directions, respectively.

\( s \) is the distance between hoops (center to center).

\( b_c \) and \( d_c \) are the core dimensions to the centerline of the perimeter hoop in x and y directions, respectively.

\( k_e \) is the confinement effectiveness coefficient. This factor modifies the lateral pressure provided by the transverse steel to take into account that the confining pressure can only be exerted effectively on parts of the concrete core where confining stress has fully developed due to arching action.

\[ k_e = \frac{1 - \sum_{i=1}^{n} \left( \frac{w_i}{6b_c d_c} \right)^2 \left( 1 - \frac{s'}{2b_c} \right) \left( 1 - \frac{s'}{2d_c} \right)}{1 - \rho_{cc}} \]

Equation C-12

The steel reinforcement behavior was modeled using King Program (1986), the stress strain relationship for the reinforcing steel and the parameters that define the curve are found using the equations below. Set of equations presented to model the behavior of the steel reinforcement:
To model the hysteretic behavior of concrete sections (Beams and Columns), a modified Takeda (1970), Model was used as shown on Figure C-1a) and Figure C-2b). The strength and stiffness of the beams were estimated not taking into account for the contribution of the slab. The strength was computed without the use of strength reduction factor. Miranda and Bertero (1991) remark that experimental results have shown that when the beam gets further and further into its inelastic range of behavior, the reinforcement of the slab outside the effective
width starts contributing more and more to the negative flexural strength of the beam. The amount of reinforcement of the slab that needs to be considered to compute the flexural strength of the beam depends on the level of inelastic deformation suffered by the beam, and thus it is very difficult to capture adequately the flexural behavior of the beam.

CUMBIA (Montejo and Kowalsky, 2007) program was used to calculate the moment curvature of concrete sections, not considering the slab contribution, as well as the P-M interaction of concrete columns, the stiffness of beams and columns were calculated from the moment curvatures using the following relationship

\[ EI = \frac{M_y}{\phi_y} \]

Equation C-18

\[ a) \text{ Large Takeda} \]
b) Thin Takeda

Figure C.1 Takeda Hysteresis model

Beams and Columns were modeled using Giberson’s beam and Columns models shown on Figures C.2 and C.3.
Figure C.2 Giberson one component beam model (Ruaumoko, 2001)

Figure C.3 Concrete beam-column yield interaction surface (Ruaumoko, 2001)
**Beams:** The moment-curvature relationships for the beams were computed using the material properties described above and using the assumptions that plane sections remain plane after flexural deformation and that there exists complete compatibility of strains between steel and concrete. The moment-curvature relationship was approximated by a bilinear curve. The beam yield moment, \( M'_y \), and yield curvature, \( \phi'_y \), were defined as the moment and curvature at which any bar of the section reaches first yielding. The yielding moment is extrapolated to the point where \( \phi_y = (M_n/M'_y) \phi'_y \). The ultimate bending moment, \( M_u \), was defined as the moment when a) the maximum strain is reached in the concrete, b) the ultimate strain is reached in any bar, or c) the buckling stress is reached in any bar. The ultimate curvature, \( \phi_u \), was defined as the curvature when \( M_u \) is reached. By connecting the origin to the point defined by \( (M'_y, \phi'_y) \) and continuing to the point where \( \phi_y \) is reached, with a straight line, and then this point to the point defined by \( (M_u, \phi_u) \) with another straight line, the bilinear moment curvature diagram was defined. Figure C.4 shows a typical moment curvature curve for a beam or a column element.

3 Rigid joints were not considered in the analysis; all inelastic deformation of a member is concentrated at discrete points, usually at the ends of the member. This is a reasonable modeling assumption for the beams of a structure subjected to lateral loads, where the maximum moments are usually concentrated at the ends. Inertias in beams range from 0.25\( I_g \) to 0.35 \( I_g \) in the analysis made.
Columns: The moment of inertia, to model the columns, was computed using the same process as for the beams, but considering a high axial compression force for the lower story columns and a lower axial compression force as the columns were going up in the building. The vast majority of the columns remain under axial compression even under the effect of lateral loads. Small axial tension forces are expected only in the columns at the ends of the frames. For this reason, one to two zones of columns was considered for each of the buildings analyzed. With this assumption, redistribution of forces in the columns, due to possible changes in their stiffness which varies depending on the value of the axial force induced in the column, were tried to be captured.

Inertias in columns range from $0.50I_g$ to $0.55I_g$ in the analysis made.
Appendix D

D.1. DDBD Direct displacement base design of buildings

Base Shear Calculation for a 20 story frame

a) Using Priestley et al. deformed shape equation

b) New displacement profile equation and new vertical force distribution.

Figure D.1 20 story concrete frame

a. Existing Method
\[ \Delta_i = \delta_i \left( \frac{A_i}{\delta_c} \right) \]
\[ \delta_i = H_i / H_n \text{ for } N \leq 4 \]
\[ \delta_i = \frac{4}{3} \left( \frac{H_i}{H_n} \right) \left( 1 - \frac{H_i}{4H_n} \right) \text{ for } N > 4 \]

### Table D-1 DDBD 20 story regular frame using Priestly et al. equation

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<th>mass, mi (Kgf)</th>
<th>( \delta_i )</th>
<th>( \Delta_i )</th>
<th>( m\Delta_i )</th>
<th>( m\Delta_i^2 )</th>
<th>( m\Delta_i Hi )</th>
<th>( F_i )</th>
<th>( V_{si}(m\Delta_i/\Sigma m\Delta_i) )</th>
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\[ \Delta_d = \sum_{i=1}^{n} \left( m_i \Delta_i^2 \right) / \sum_{i=1}^{n} (m_i \Delta_i) = 0.836 \text{ m} \] Target Displacement

\[ m_e = \sum_{i=1}^{n} m_i \Delta_i / \Delta_d = 107,565 \text{ Kgf-seg}^2/\text{m} \] Effective mass is 81.72 % of total mass

\[ H_i = \sum_{i=1}^{n} m_i \Delta_i H_i / \sum_{i=1}^{n} m_i \Delta_i = 40.52 \text{ m} \] Effective height is 66.42 % of total height

\[ \phi_y = 0.5 \varepsilon_y \frac{L_b}{h_b} = 0.014 \] Yield curvature

\[ \Delta_y = \phi_y H_i = 0.5672 \text{ m} \] Yield displacement

\[ \mu = \frac{\Delta_d}{\Delta_y} = 1.473 \] Displacement ductility

\[ \xi_{eq} = 0.05 + 0.565 \left( \frac{\mu - 1}{\mu \pi} \right) = 0.1078 \] Equivalent damping
Figure D.2 Base DRS

$$T_{ef} = 7.726 \text{ sec.}$$ Effective period

$$K_e = 4\pi^2 \frac{m_e}{T_e^2} = 71,141.28 \text{ Kgf/m}$$ Effective stiffness

$$F_{Base} = V_{Base} = K_e \Delta_d = 59,462.00 \text{ Kgf.}$$ Base shear

Distributed 90% of Base shear over height with the following equation, 90% of Base shear + 10% of base shear at the top of the frame.

$$F_i = V_{Base_i} (m_i \Delta_i) / \sum_i (m_i \Delta_i) \quad \text{Columns 9 of table 1a)}$$

b. New displacement profile proposed method

$$\Delta_i = \delta_i \left( \frac{\Delta_i}{\Delta_e} \right)$$

$$\delta_i = H_i / H_n \quad \text{for } N \leq 4$$
b. New Equation

\[ \delta_i = \left( \frac{h_i}{H_n} \right)^{1 - NS_s / 165} \]

For this case \( N=20, S_s=1.65 \)

\[ 1-(S_s)(N/165) = 1 - 1.65(20)/165 = 0.80 \]

Then for this case \( \delta_i = \left( \frac{h_i}{H_n} \right)^{0.8} \)
Table D-2 DDBD 20 story regular frame using Proposed equation

<table>
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<th>mass, mi (KN)</th>
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<th>$\Delta_i$</th>
<th>$m\Delta_i$</th>
<th>$m\Delta_i^2$</th>
<th>$m\Delta_i\mu$</th>
<th>$F_i$</th>
<th>$\mu$</th>
<th>vs($m\Delta_i/\Sigma m\Delta_i$)</th>
<th>Ductility</th>
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$\delta_i = \left( \frac{h_i}{H_n} \right)^{1-NS_i/165}$

Adjusted to design drift

$$\Delta_d = \frac{\sum_{i=1}^{n} (m_i\Delta_i^2)}{\sum_{i=1}^{n} (m_i\Delta_i)} = 0.624 \text{ m.}$$

Target displacement
\[ m_e = \sum_{i=1}^{n} m_i \Delta_i / \Delta_e = 109,448 \text{ Kgf-seg}^2/\text{m.} \quad 83.15\% \text{ of total mass} \]

\[ H_e = \sum_{i=1}^{n} m_i \Delta_i H_i / \sum_{i=1}^{n} m_i \Delta_i = 40.14 \text{ m.} \quad 65.80\% \text{ of total height} \]

\[ \phi_y = 0.5 \varepsilon_y \frac{L_b}{h_b} = 0.0114 \quad \text{note: } h_b \text{ had to be increased to 0.7 m.} \]

\[ \Delta_y = \phi_y H_e = 0.4587 \text{ m.} \quad \text{Yield displacement} \]

\[ \mu = \frac{\Delta_d}{\Delta_y} = 1.36 \quad \text{Displacement ductility} \]

\[ \xi_{eq} = 0.05 + 0.565 \left( \frac{\mu - 1}{\mu \pi} \right) = 0.0976 \quad \text{Equivalent damping} \]

![Figure D.3 Base DRS for analysis](image)

\[ T_{eff} = 5.501 \text{ sec.} \quad \text{Effective period} \]
\[ K_e = \frac{4\pi^2 m_e}{T_e^2} = 142,782.09 \text{ Kgf/m.} \quad \text{Effective stiffness} \]

\[ F_{\text{base}} = V_{\text{base}} = K_e \Delta d = 89,107.20 \text{ Kgf.} \quad \text{Base shear} \]

Distributed with the following equation

\[
F_i = V_{\text{base}} (m_i \Delta_i) / \sum_{i=1}^{n} (m_i \Delta_i)
\]

\[
F_i = (N/4)V_{\text{base}} (m_i \Delta_i) / \sum_{i=1}^{n} (m_i \Delta_i)
\]

**Figure D.4 Vertical force distribution**

The resulting force from the distributed over the height is located at the top of the frame as shown in column 9 table D-2.

Base shear comparison with to methods is as follows:

Vb = 59,462 Kgf with existing displacement profile equation

Vb = 89,107 Kgf with new displacement profile equation
The results shown above correspond to 100% intensity for use on the new displacement profiles equation, other analyses were done using 200% and 50% intensity and the results are as follows:

<table>
<thead>
<tr>
<th>Table D-3 Base shear results for 100% intensity</th>
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<td>Base shear</td>
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<td>Vb existing eq.</td>
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<tr>
<td>Vb new eq.</td>
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Appendix E

E-1 Design of the buildings

E-1.1 DDBD procedure

The following pages show an example of the design using the existing method for DDBD considering P-Δ effect, in this example the base shear is calculated using DDBD for a 15 story regular frame structure.

Figure E.1 Example of 15 Story frame geometry

The steps for the analysis using the existing procedure, are as follows:

b) Determine the displacement Profile

\[
\Delta_i = \delta_i \cdot \left( \frac{\Delta_i}{\delta_i} \right)
\]
The following equations are used to define the displacement profile of the structure, which is then adjusted for the design drift using the equation above:

\[ \delta_i = H_i / H_n \quad \text{for } N \leq 4 \]

\[ \delta_i = \frac{4}{3} \left( \frac{H_i}{H_n} \right) \left( 1 - \frac{H_i}{4H_n} \right) \quad \text{for } N > 4 \]

Determine the target design displacement

\[ \Delta_d = \frac{\sum_{i=1}^{n} (m_i \Delta_i^2) / \sum_{i=1}^{n} (m_i \Delta_i)}{\sum_{i=1}^{n} (m_i \Delta_i)} = 0.63 \text{ m.} \]

c) Determine the effective mass and effective height of the structure

\[ m_e = \sum_{i=1}^{n} m_i \Delta_i / \Delta_i = 81,490 \text{ Kgf-seg2/m.} \quad 82.65 \% \text{ of total mass} \]

\[ H_e = \sum_{i=1}^{n} m_i \Delta_i H_i / \sum_{i=1}^{n} m_i \Delta_i = 30.79 \text{ m.} \quad 66.93 \% \text{ of total height} \]

d) Determine the yield curvature of SDOF equivalent system

\[ \phi_y = 0.5 \varepsilon_y \frac{L_h}{H_h} = 0.014 \]

e) Determine the yield displacement, the displacement ductility and the effective damping of equivalent SDOF
\[ \Delta_v = \phi_y H_v = 0.43 \text{ m.} \]

\[ \mu = \frac{\Delta_d}{\Delta_y} = 1.48 \]

\[ \xi_{eq} = 0.05 + 0.565 \left( \frac{\mu - 1}{\mu \pi} \right) = 0.1082 \]

\[ T_{eff} = 5.89 \text{ sec} \]

Teff is the effective period of the equivalent SDOF and is calculated above using the target displacement and the modified to the effective damping of the Design response spectrum for the location in consideration.

f) Determine the effective stiffness of the equivalent SDOF

\[ K_e = 4\pi^2 m_e / T_{eff}^2 = 93,049 \text{ Kgf/m.} \]

g) With the target displacement and the effective stiffness the base force (base shear) is calculated

\[ F_{base} = V_{base} = K_e \Delta_d = 59,329 \text{ Kgf} \]

h) The base force is laterally distributed using the following equation:
\[ F_i = \frac{V_{bas}(m_i \Delta_i)}{\sum_{i=1}^{n}(m_i \Delta_i)} \]

**Table E-1 DDBD example for finding Base Shear existing method**

<table>
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<tr>
<th>Story, I</th>
<th>Height H (m)</th>
<th>mass, mi (KN)</th>
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<th>( \Delta_i )</th>
<th>( m\Delta_i )</th>
<th>( m\Delta_i^2 )</th>
<th>( m\Delta_i\mu_i )</th>
<th>( \frac{V_{vb}(m\Delta_i/\Sigma m\Delta_i)}{m\Delta_i} )</th>
<th>Ductility</th>
<th>( \mu )</th>
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Using the **new proposed method** modifying the displacement shape the design steps are as follow:
i) Determine the displacement Profile

\[ \Delta_i = \delta_i \cdot \left( \frac{\Delta_i}{\delta_i} \right) \]

The following equations are used to define the displacement profile of the structure, which is then adjusted for the design drift using the equation above:

\[ \delta_i = H_i / H_n \quad \text{for } N \leq 4 \]

\[ \delta_i = \left( \frac{h_i}{H_n} \right)^{1-NS_e/165} \quad \text{for } N > 4 \]

j) Determine the target design displacement

\[ \Delta_d = \sum_{i=1}^{n} \frac{(m_i \Delta_i^2)}{\sum_{i=1}^{n} (m_i \Delta_i)} = 0.56 \text{ m} \]

k) Determine the effective mass and effective high of the structure

\[ m_e = \sum_{i=1}^{n} m_i \Delta_i / \Delta_u = 81,502 \text{ Kgf-seg2/m} \quad 82.65 \% \text{ of total mass} \]

\[ H_e = \sum_{i=1}^{n} m_i \Delta_i H / \sum_{i=1}^{n} m_i \Delta_i = 30.80 \text{ m} \quad 66.93 \% \text{ of total height} \]

l) Determine the yield curvature of SDOF equivalent system
\[ \phi_y = 0.5 \xi \frac{L_0}{\eta_0} = 0.014 \]

m) Determine the yield displacement, the displacement ductility and the effective damping of equivalent SDOF

\[ \Delta_y = \phi_y H_e = 0.43 \text{ m.} \]

\[ \mu = \frac{\Delta_d}{\Delta_y} = 1.48 \]

\[ \xi_{eq} = 0.05 + 0.565 \left( \frac{\mu - 1}{\mu \pi} \right) = 0.0916 \]

\[ T_{eff} = 4.89 \text{ sec} \]

\( T_{eff} \) is the effective period of the equivalent SDOF and is calculated above using the target displacement and the modified to the effective damping of the Design response spectrum for the base DRS.

n) Determine the effective stiffness of the equivalent SDOF
\[ K_e = 4\pi^2 \frac{m_e}{T_e^2} = 137,406 \text{ Kgf/m.} \]

o) With the target displacement and the effective stiffness the base force (base shear) is calculated

\[ F_{\text{base}} = V_{\text{base}} = K_e \Delta_d = 75,507 \text{ Kgf} \]

Considering P-\(\Delta\)

\[ F = K_e \Delta_d + C \frac{P\Delta_d}{H} = 91,671 \text{ Kgf} \text{ and using} \]

\[ C = 1.00 \]

p) The base force is laterally distributed using the following equation:

\[ F_i = V_{\text{base}} \left( m_i \Delta_i \right) / \sum_{j=1}^{n} \left( m_i \Delta_i \right) \text{ distributed from the 4th level and above} \]

\[ F_i = 0.00 \text{ at third level} \]

\[ F_i = -(N/4) V_{\text{base}} \left( m_i \Delta_i \right) / \sum_{j=1}^{n} \left( m_i \Delta_i \right) \text{ at the second and first level} \]
Table E-2 DDBD example for finding Base Shear proposed method

<table>
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<tr>
<th>Story, I</th>
<th>Height H (m)</th>
<th>mass, mi (KN)</th>
<th>δi</th>
<th>Δi</th>
<th>mΔi</th>
<th>mΔi2</th>
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Total 98593.28 45734.48 25663.64 1408707.00 84387.99 91670.92

C=0.5  C=1.0
When using the new proposed method described in chapter five the design displacement $\Delta_d = 0.56$ m., the equivalent damping $\xi_e = 9.16$ % and $T_{eff}$ is 4.89 sec. and the design base shear $V_b = 75,507$ Kgf. and the new proposed vertical load distribution was used for the analyses of the frames.

**E-2 Existing design methods considering P-Δ using DDBD Priestley et al. (2007)**

As structures displace laterally, as proposed, for example, in the single-degree-of freedom approximation of Figure E-2, gravity loads induce overturning moments, in addition to those resulting from, lateral inertia forces. Using the nomenclature of Figure E-2, the base moment $M$ is:

$$M = FH + P\Delta$$

Equation E-2
Assuming that the sum of the moment capacity of all columns at the base $M_D$ is reached during the earthquake, then the lateral inertia force that can be resisted decreases as the displacement increases, according to the relationship:

$$ F = \frac{M_D - P\Delta}{H} $$

Equation E-2

This effect is illustrated in Figure E-3, where it is apparent that the P-Δ effect not only reduces the lateral force capacity of the structure, but also modifies the entire lateral force-displacement characteristic. The effective initial stiffness is reduced, and the post-yield stiffness may become negative.

The importance of P-Δ effects is recognized in most seismic design codes and is typically quantified by some form of "stability index, $\theta$". This stability index compares the magnitude of the P-Δ effect at either nominal yield, or at expected maximum displacement, to the design base moment capacity of the structure. Since the DDBD considers the capacity displacement as a measure for design at the performance level and the P-Δ effect is of maximum importance at the design level of seismic response, we relate the stability index to conditions at maximum response, as recommended in (Priestley, Calvi and Kowalsky, 2007):

$$ \theta_D = \frac{P\Delta_{\text{Max}}}{M_D} $$

Equation E-3
In conventional force-based design, (FBD) one of two different approaches is typically adopted to account for P-Δ effects. One approach is to increase the expected design displacement to $\Delta^{*}\text{max}$:

$$
\Delta^{*}_{\text{max}} = \frac{\Delta_{\text{max}}}{1 - \theta_{D}} = \frac{\mu \Delta_{y}}{1 - \theta_{D}}
$$

Equation E-4

Where $\mu$ is the ductility of the system,

In the case of FBD the design base shear force has to be increased in order to reach the $\Delta^{*}\text{max}$, for the case of DDBD the approach could be to reduce the target displacement by the factor $(1-\theta_{D})$, to increase the effective stiffness of the system in order to have a higher base shear for designing.

An alternate method is to increase the strength stiffness of the system, in an attempt to avoid an increase in the expected design displacement. Paulay and Priestley (1992) discussing the design of reinforced concrete frame buildings, recommend that when the stability index is less than $\theta_{\Delta}=0.085$, P=Delta effects may be ignored. For higher values of the stability index, equal energy approach is adopted to determine the required strength increase, as suggested in Figure E-2. This implies that the required nominal strength increases, ignoring P-Δ effects is somewhat greater than 50% of the calculated P-Δ effect.

The treatment of P-Δ effects in DDBD is comparatively straightforward and is illustrated in Figure E-2. Unlike conditions for force-based design, the design displacement is known at
the start of the design process, and therefore the P-Δ moment is also 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 (see Ke in Figure E-2) that must match the required stiffness. Therefore, Equation E-5 defines the required residual strength. The initial strength, corresponding to zero displacement, is thus given by equation E-6. This equation E-6 is the proposed equation by Priestley et Al. for designing considering the P-Δ effect when using the DDBD procedure.

\[ F = V_{\text{base}} = K_e \Delta_d \]  \hspace{1cm} \text{Equation E-5}

\[ F = K_e \Delta_d + C \frac{P \Delta_d}{H} \]  \hspace{1cm} \text{Equation E-6}

Where C=0.5 for concrete structures and C=1.00 for Steel structures as proposed by Priestley et Al. C is factor that depends on the type of material of the structure and hence the required base-moment capacity is

\[ M_B = K_e \Delta_d H + CP \Delta_d \]  \hspace{1cm} \text{Equation E-7}

\textbf{In the case of concrete structures}: When the structural stability index defined by Equation E-3 exceeds 0.10, the design base moment capacity should be amplified for P-Δ effects as indicated in Equation E-6, taking C=0.5 as suggested by Priestley et Al. for concrete
structures. This is represented by line 2 in Figure E-3. This behavior corresponds to the upper line marked "strength enhancement for P-Δ" in Figure E-2. For lesser values of the stability index, P-Δ effects may be ignored, for both steel and concrete structures, it is recommended that the Stability Index, given by Equation E-3 should not exceed 0.33 (Priestley, Calvi and Kowalsky, 2007).

\[
(K_e + P/H)\Delta_d
\]

\[
(K_e + 0.5P/H)\Delta_d
\]

Figure E.3 Required P-Δ Strength enhancement in Displacement-Based Design (Priestley, Calvi and Kowalsky 2007)