EVALUATION OF DEFLECTION AMPLIFICATION FACTOR FOR ESTIMATING SEISMIC DEFORMATIONS OF RC FRAMES

Mohammad Emdadul Haque

MASTER OF SCIENCE IN CIVIL ENGINEERING (STRUCTURAL)

DEPARTMENT OF CIVIL ENGINEERING

BANGLADESH UNIVERSITY OF ENGINEERING AND TECHNOLOGY

DHAKA-1000, BANGLADESH

November, 2020

Evaluation of Deflection Amplification Factor for Estimating Seismic Deformations of RC Frames

A Thesis

By

Mohammad Emdadul Haque

A Thesis Submitted in partial fulfillment of the requirement for the degree of

Master of Science in Civil Engineering (Structural)

Department of Civil Engineering

Bangladesh University of Engineering and Technology

Dhaka-1000, Bangladesh

November, 2020

The Thesis Titled "Evaluation of Deflection Amplification Factor for Estimating Seismic Deformations of RC Frames", submitted by Mohammad Emdadul Haque, Roll No.1018042335 F, Session: October 1018; has been accepted as satisfactory in partial fulfillment of the requirement for the degree of Master of Science in Civil Engineering (Structural) on 25th November, 2020

BOARD OF EXAMINERS

Dr. Syed Ishtiaq Ahmad Professor Department of Civil Engineering BUET, Dhaka-1000	Chairman (Supervisor)
Dr. Md. Delwar Hossain Professor and Head Department of Civil Engineering BUET, Dhaka-1000	Member (Ex-Officio)
Dr. Md. Shafiul Bari Professor Department of Civil Engineering BUET, Dhaka-1000	Member
Dr. Md. Zakaria Ahmed Professor Department of Civil Engineering BUET, Dhaka-1000	Member
Brigadier General Mohammad Nur Hossain, BSP, psc Brigadier General (Senior Instructor) Department of Civil Engineering MIST, Mirpur Cantonment	Member

DECLARATION

It is hereby declared that this thesis is the research work carried out by the author under supervision of Dr. Syed Ishtiaq Ahmad, Professor, Department of Civil Engineering, BUET. Neither this thesis nor any part thereof has been submitted or is being concurrently submitted elsewhere for any other purposes except for publication.

November, 2020

(Mohammad Emdadul Haque)

Roll No: 1018042335

DEDICATED

To

My Parents, My Supervisor and Freedom Fighters of Liberation War

ACKNOWLEDGEMENT

ALL PRAISE IS OF ALMIGHTY ALLAH (SWT). My sincere gratitude to ALMIGHTY ALLAH (SWT) for being able to complete this research work effectively.

The author expresses his profound gratitude and heartiest thanks to his thesis supervisor Professor Dr. Syed Ishtiaq Ahmad, Department of Civil Engineering, Bangladesh University of Engineering and Technology (BUET) for his constant guidance, supervision, keen interest as well as resource management in making this paper success. His cooperation and guidance have benefited the author greatly.

The author expresses his heartfelt ever remembered gratefulness to Brigadier General Md Mamun Or Rashid, BSP, ndc, psc (LPR), Ex Director of Movement and Quartering of Bangladesh Army. His contribution, support and trustworthiness cannot be expressed just by few words.

The author is grateful to the members of thesis defence committee Professor Dr. Md Delwar Hossain, Professor Dr. Md Shafiul Bari, Professor Dr. Md Zakaria Ahmed and Brigadier General Mohammad Nur Hossain, BSP, psc for their advice and help in reviewing this thesis.

The author expresses his deepest gratitude to Dr. Md. Jahangir Alam, Professor, Department of Civil Engineering, BUET, for his neverending support over the years. The author expresses deepest gratitude to Colonel Md Ehsanul Haque, BSP (bar), afwc, psc of Bangladesh Army for his ever-remembered support and encouragement.

The author is also very thankful to Professor Dr. Mehedi Ahmed Ansary, Professor Dr. Raquib Ahsan and Assistant Professor Radin Md Mahirul Hoque. The author is very thankful to Lieutenant Colonel Dr Khondaker Sakil Ahmed, PEng, M. ASCE, Associate Professor, Department of Civil Engineering, MIST. The author is also very thankful to Mohammed Salauddin and Md. Tariqul Islam Aunto, MSc Student of BUET.

The author is very grateful to his family members and friends for their unconditional love, encouragement, blessings and cooperation.

ABSTRACT

Elastic displacement is multiplied by deflection amplification factor (C_d) to get maximum displacement under severe seismic conditions. The elastic displacement is determined either by equivalent static analysis or linear time history analysis. In general code specifies certain value of C_d which may be used to estimate maximum inelastic deformation of building. The approved Bangladesh National Building Code (BNBC) has specified 4.5 as C_d value for Intermediate Reinforced concrete Moment Resisting Frames (IMRF). Theoretically, C_d value should vary according to building height, span, stiffness and other properties. To examine the range over which C_d varies for building in different regions of Bangladesh, in this work, 24 Reinforced Concrete buildings with different vertical and plan configuration has been examined. For analysis of these frames, three-dimensional finite element software ETABS has been used.

The 24 buildings considered in this work had bay width of 5 meter and height of 3.2 meter. Building frames of 2, 3 and 4-bay were examined in this work. Each bay has 8 frames having 2, 3, 4, 5, 6, 8, 10, 12 story. At first, structural analysis of these frames has been performed using equivalent static analysis as per BNBC (2020) which is based on zone specified seismic force and weight of the building. Through equivalent static analysis the building is designed and different structural member dimension was selected. Using same geometrical, material and loading data, nonlinear time history analysis was on the same buildings. Nonlinear time history was been done for PGA $0.2g$, $0.3g$ and $0.36g$ using the imperial valley earthquake. As peak ground acceleration of Imperial valley earthquake is 0.2605g (at 0.43 sect), it was scaled so that PGA becomes $0.2g$, $0.3g$ and $0.36g$.

For 12 storied building, nonlinear displacement was found 115.2 mm, 100.4mm and 102.5 mm for 2,3 and 4-bay building respectively. Whereas for 8 Story building had displacement was of 63.3 mm, 55.1 mm and 55.8 mm for 2, 3 and 4-bay building respectively. As storey was reduced, top nonlinear displacement was found to decrease whereas as number of bay was increased, nonlinear displacement was found to decrease. Likewise, nonlinear storey drift was found to increase as number of storeys was increased.

For 12 storied building, linear displacement was found 25.2 mm, 22.6 mm and 22.8 mm for 2,3 and 4-bay building respectively. Whereas for 8 Story building, top displacement reduced to 14.9 mm, 15.2 mm and 15.4 mm for 2, 3 and 4-bay building respectively. For four storey building, top displacement was found to decrease further. Hence, as storey height was reduced, top linear displacement was found to decrease. Further, as number of bays was increased, linear displacement was found to decrease.

C_d value was estimated from non-linear and linear top displacement found from the respective analysis. For 12,10 and 8 storied building had C_d value ranged from 4.57 to 4.23, 4.5 to 3.63 and 4.5 to 3.63 for 2,3 and 4-bay building respectively. BNBC specify a flat C_d value of 4.5 for buildings of all configuration. However, from this analysis work, it is evident that $C_d$value is very much dependent on building height, number of bays etc. C_d values suggested in this work may be useful for practicing engineers and researchers to estimate ultimate displacement of buildings due to severe earthquake from linear analysis of building for different regions of Bangladesh.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1

INTRODUCTION

1.1 General

Earthquakes are vibrations caused by sudden displacement of land. An earthquake is a sudden movement of the earth-crust, caused by the abrupt release of strain, accumulated over a long time. For hundreds of millions of years, the forces of plate tectonic wave shaped the earth, as the huge plates that form the earth's surface move slowly over, under, and past each other. Sometimes the movement is gradual. At other times, the plates are locked together, unable to release the accumulated energy. When the accumulated energy grows strong enough, the plates break free.

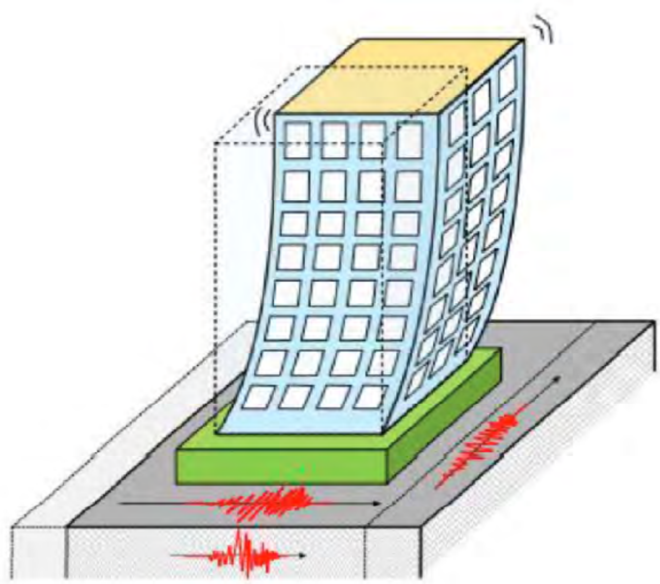


Figure 1.1. Earthquake Behavior of Building (**Source:** www.iitk.ac.in/nicee/IITK-GSDMA)

Bangladesh is situated in a seismically active region. Part of the country extended from Sylhet to Chittagong is in the high seismic zone whereas Dhaka is in the moderate seismic zone. Major metropolitan cities of our country are under serious threat because of inadequacies in design and construction of structures. Rapid urbanization creates a great demand on human shelter especially in big cities like

Dhaka, Chittagong etc. As a result, a lot of multi-storeyed buildings are built in order to fulfil the demand.

Figure 1.2. Seismic Zoning Map of Bangladesh (BNBC, 2020)

Materials remain elastic at certain level. The ground motion applied on a structure exerts force which becomes nonlinear at some point. To bring down the ground motion force at inelastic level, response modification factor is applied. Again, for assessing for total displacement and energy absorbed deflection amplification factor is applied. For seismic design, it is important to estimate maximum displacement (Δ_{max}) for several reasons; these include: (1) estimating minimum building separation to avoid pounding; (2) estimating maximum story drifts; (3) checking deformation capacity of critical structural members (e.g., shear links in eccentrically braced frames); (4) checking P-delta effects; and (5) detailing connections for non-structural components and so on. This Δ_{max} divided by elastic displacement (Δ_e) is the deflection amplificatory factor (Uang and Maaruf ,1994).

The equivalent static lateral force method is a simplified technique to substitute the effect of dynamic loading of an expected earthquake by a static force distributed laterally on a structure for design purposes. The total applied seismic force V is generally evaluated in two horizontal directions parallel to the main axes of the building (Fig. 1.2). It assumes that the building responds in its fundamental lateral mode. For this to be true, the building must be low rise and must be fairly symmetric to avoid torsional movement underground motions. The structure must be able to resist effects caused by seismic forces in either direction, but not in both directions simultaneously.

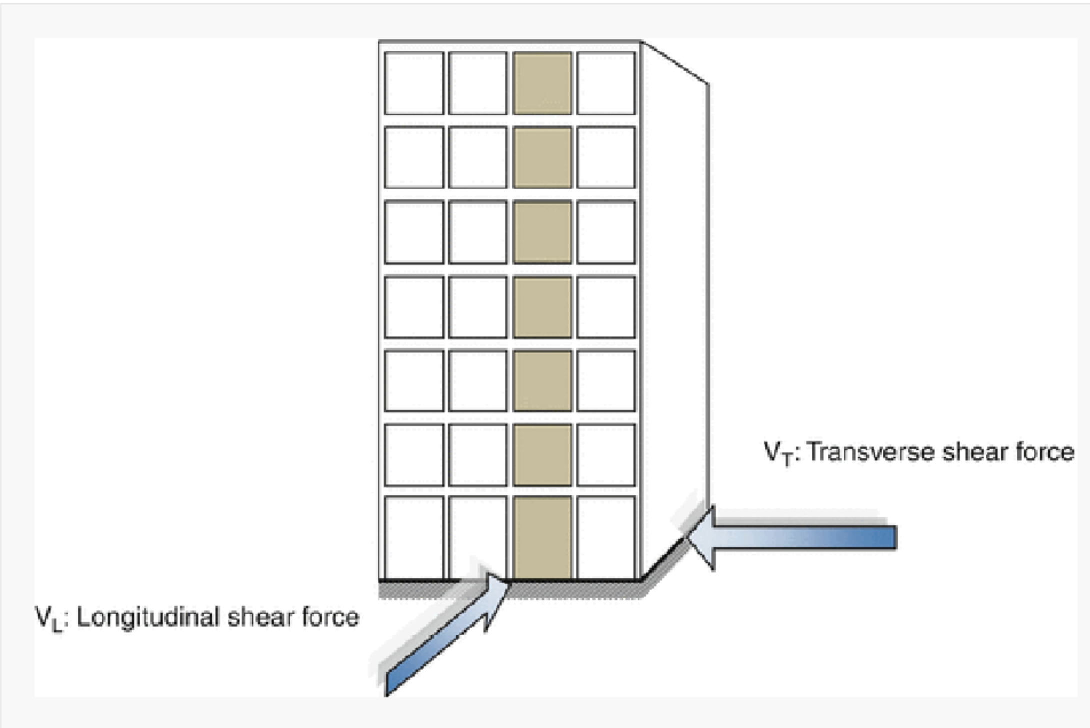


Figure 1.3. Equivalent Static Analysis of Structures Subjected to Seismic Actions (**Source:** Encyclopaedia of Earthquake Engineering)

This approach defines a series of forces acting on a building to represent the effect of earthquake ground motion, typically defined by a seismic design response spectrum. The response is read from a design response spectrum, given **the natural frequency of the building** (either calculated or defined by the building code). To account for effects due to "yielding" of the structure, many codes apply modification factors that reduce the design forces (e.g., force reduction factors).

A building has the potential to 'wave' back and forth during an earthquake. This is called the 'fundamental mode', and is the lowest frequency of building response. Most buildings, however, have higher modes of response, which are uniquely activated during earthquakes. The figure just shows the second mode, but there are higher 'shimmy' (abnormal vibration) modes. Nevertheless, the first and second modes tend to cause the most damage in most cases. Assumption in equivalent static analysis is that building responds in its fundamental modes.

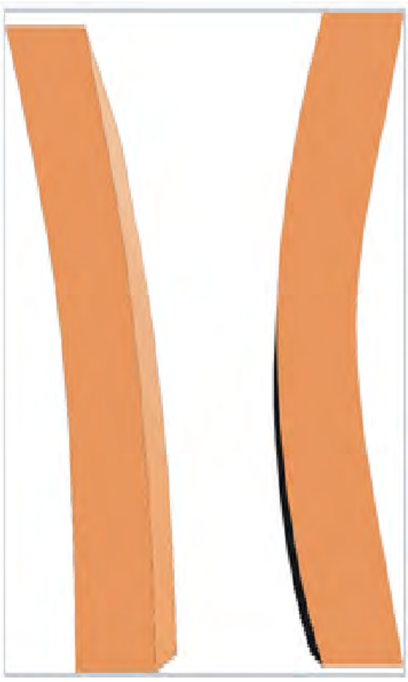


Figure 1.4. First and Second Modes of Building Seismic Response (**Source:** Seismic Analysis-Wikipedia)

In the force-based seismic design procedures, the response modification factor (R) is the one used to reduce the linear elastic response spectra to the inelastic ones. In other word, response modification factor is the ratio of strength required to maintain the structural elasticity.

Conventional seismic design in codes of practice is entirely force-based, with a final check on structural displacements. Force-based design is suited to design for actions that are permanently applied. Members are designed to resist the effects of these actions. Seismic design follows the same procedure, except for the fact that inelastic deformations may be utilized to absorb certain levels of energy leading to reduction in the forces for which structures are designed. This leads to the creation of the Response Modification Factor (R); the all-important parameter that accounts for over-strength, energy absorption and dissipation as well as structural capacity to redistribute forces from inelastic highly stressed regions to other less stressed locations in the structure. The concept of Response Modification Factor or also commonly known as Force Reduction Factor, has emerged as a single most important number, reflecting the capability of the structure to dissipate energy through inelastic behavior. This factor is unique and different for different type of structures and materials used.

Materials remain elastic at certain level. The ground motion applied on a structure exerts force which becomes nonlinear at certain level. To bring down the ground motion force at inelastic level, Response modification factor is applied. Again, for assessing for total displacement and energy absorbed deflection amplification factor is applied.

It is fair to say that nonlinearity is viewed as a detriment by most designers and experimentalists. Stiffness is constant in linear systems. Time variant loads that induce considerable inertial and damping forces may warrant dynamic analysis. Linear static analysis is independent of time. All structures behave non linearly in one way or other beyond a particular level of loading. In some cases, linear analysis may be adequate but, in many cases, the linear analysis may produce erroneous results as the assumptions on which linear analysis is done may be violated in real time structure. Nonlinear analysis is needed if the loading produces significant changes in the stiffness.

To correct for the too-low displacement predicted by the reduced force elastic analysis, the “computed design displacement” is multiplied by the factor C_d. This factor is always less than the R factor because R contains ingredients other than pure ductility.

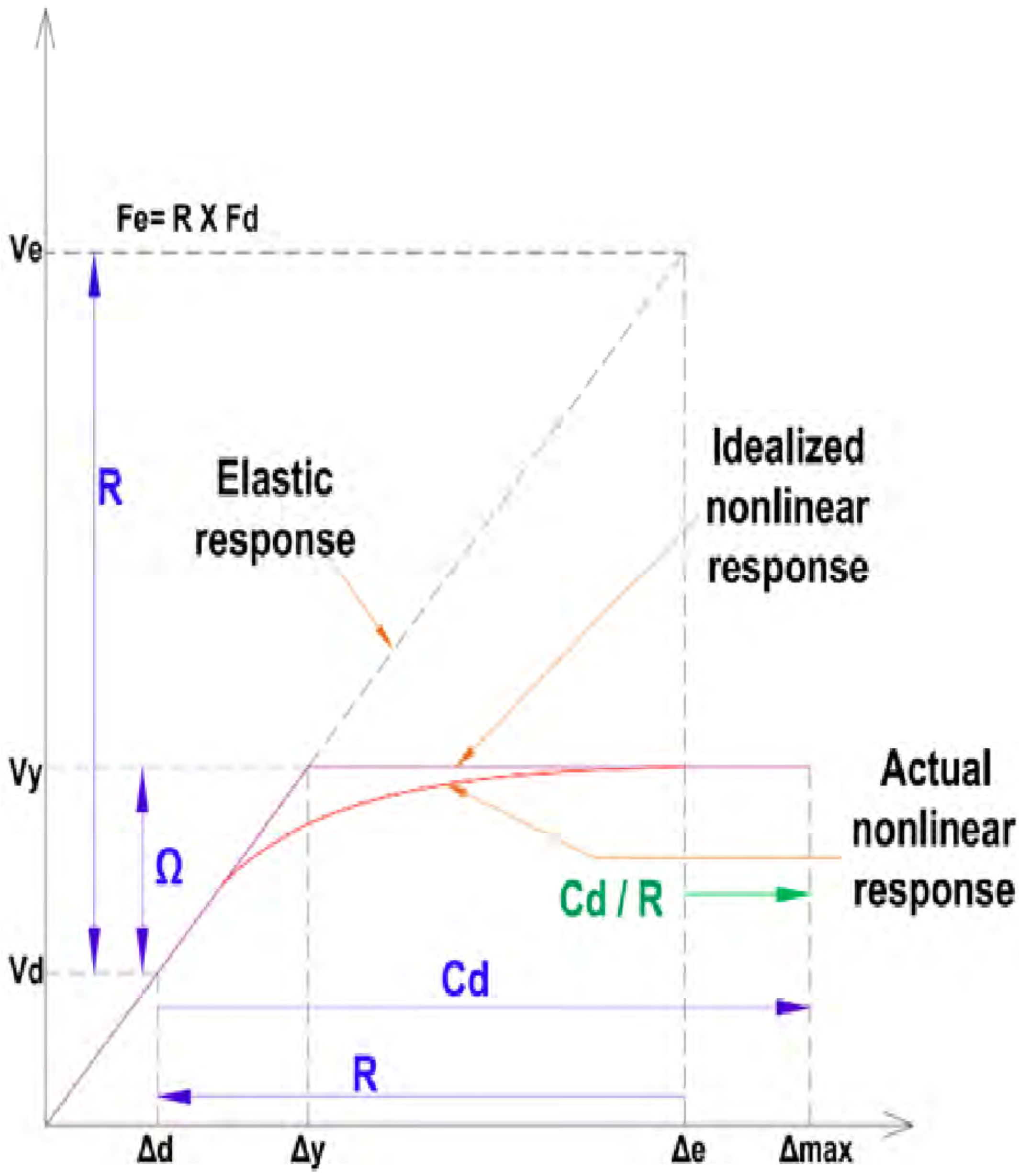


Figure 1.5. Relation Between $\boldsymbol{C_d}$ and $\boldsymbol{R}$ (Astrid et al., 2014)

It is well known that modern seismic design provisions reduce design seismic forces significantly to take advantage of the structure's capacity to dissipate earthquake input energy. This concept is explained by using Fig. 1.5, where the typical response envelope of base shear (V) versus lateral drift (Δ) of a ductile system is shown. The force V_e represents the required level of design seismic forces if the structure were to respond elastically in a major earthquake. For strength design, the NEHRP Recommended Seismic Provisions (NEHRP, 1991) reduces this elastic force level by a force reduction factor (FRF), R.

To estimate the maximum inelastic deflections $\Delta_{\max}$ may develop in a major earthquake, the design deflections computed from an elastic structural analysis are amplified by a deflection amplification factor (DAF) as follows:

$$\text{NEHRP: } \Delta_{\max} = \Delta_s \times C_d \qquad (1.1)$$

The elastic deformations calculated under reduced design forces are then amplified by the deflection amplification factor C_d to estimate the expected deformations likely to be experienced in response to the design ground motion. Different structure having variation in height and bay width has different amplification factor. Therefore, has different inelastic displacement.

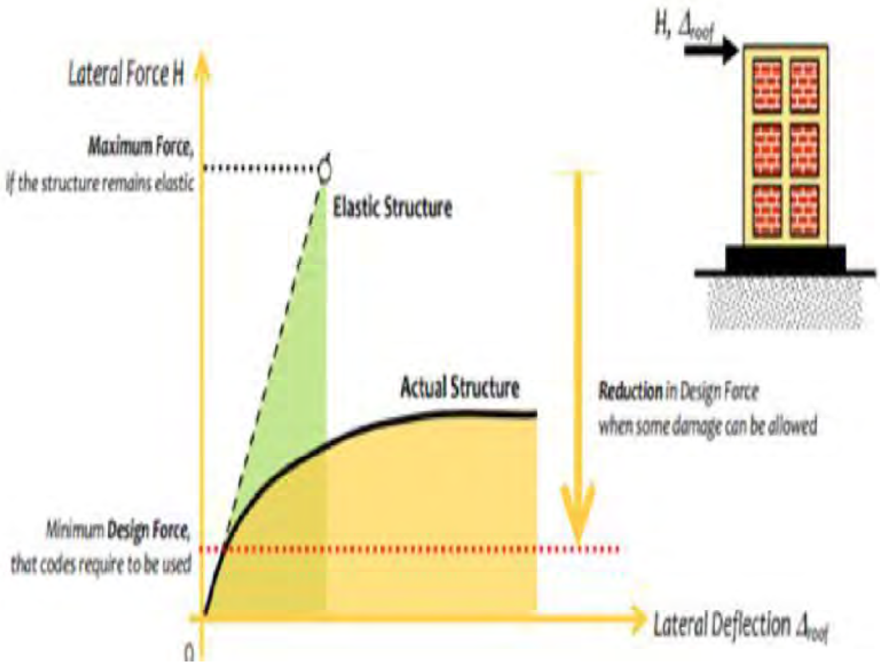


Figure 1.6. Basic Strategy of Earthquake Design: Calculate Maximum Elastic Forces and Reduce by a Force to Obtain Design Forces (**Source:** www.iitk.ac.in>nicee>IITK-GSDMA)

The lateral displacement should be controlled to limit possible damage to structural and non-structural components and also to avoid pounding between adjacent structure.

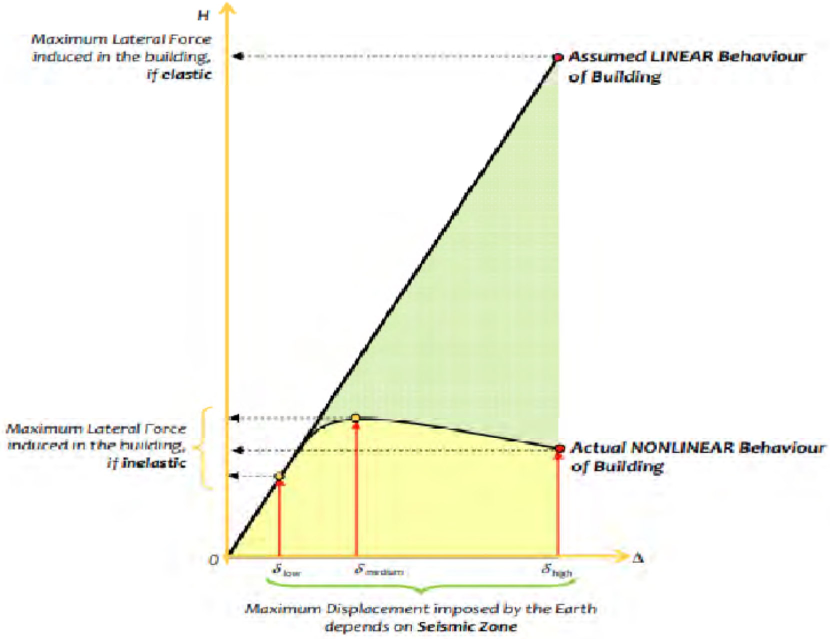


Figure 1.7. Actual Linear and Nonlinear Behaviour of Building (**Source:** www.iitk.ac.in>nicee>IITK-GSDMA)

For estimating the maximum inelastic deflection that might occur during an earthquake, the design deflections computed from an elastic analysis are usually amplified by a deflection amplification factor (C_d). This process is referred to as force-based design method.

1.2 Background and Present State of the Problem

There was no written building code in Bangladesh until 1993. In 1993, Bangladesh National Building Code (BNBC) was published by Housing and Building Research Institute (HBRI) which is commonly known as BNBC. The seismic design provisions of BNBC were based on the UBC (UBC, 1991). For the regular structures, the Code defines a simple method to represent earthquake induced inertia forces by Equivalent Static Force for static analysis. For very tall structure, the Code provisions require Time History Analysis. All these methods detailed in BNBC are force-based methods. As in many other codes, the level of forces prescribed by BNBC for a structure is rather arbitrarily set and aimed at damage control performance objectives i.e., no damage under small earthquake and no collapse under extreme earthquake. The code approach is to design seismic load resisting system on the basis of a pseudo-seismic load obtained by dividing the actual load by response modification factor, R. The R value is specified by the code for each structural system without explicitly defining the level of element (i.e., beam, column, connection etc.) ductility required for each system. The code implicitly assumes that the enhanced ductile detailing would result in seismic energy dissipation and hence a reduced demand would result. This BNBC was gazetted in 2006 as BNBC 2006. Bangladesh National Building Code 2020 (BNBC 2020) is the latest version of building code in Bangladesh. This code is very much similar to ASCE 7-05.

The purpose of this study is to investigate deflection amplification factor of intermediate RC moment frames based on the ratio of maximum inelastic to equivalent static and linear time displacement. This study also identifies maximum seismic story drifts in intermediate RC frames and consequently which stories can be susceptible to maximum inelastic story distortions. Therefore, nonlinear, linear and equivalent static analysis are performed on the 24 models of reinforced concrete moment frames to determine the maximum seismic deflections at all the stories when subjected to the scaled earthquake records.

The nonlinear displacement is divided by the equivalent static displacement to find the Deflection Amplification Factor, C_d. The design ground motions assumed to occur along any horizontal direction of a building structure. The elastic deformations calculated under these reduced design forces are multiplied by the deflection amplification factor, C_d to estimate the deformations likely to result from the design earthquake.

There are many researches on $C_{d,}$ but as per proposed BNBC no research has been found. In the newly proposed Bangladesh National Building Code (BNBC, 2020), different values for C_d are proposed for different types of RC structures. However, a more detail analysis may be performed to show the variation of C_d due to changes in different RC building parameters like story height (Typical storey height 3.2 meter and bottom storey height 2.13 meter), plan geometry, distribution of mass etc. This would be helpful in understanding the inelastic behaviour of RC building of different configuration designed as per BNBC (2020).

It is to be remembered that the length to breadth ratio of the building in plan not to be higher than 4. In buildings, floors (including the roof) act as horizontal diaphragms that collect and transmit the inertia forces to the vertical structural systems and ensure that those systems act together in resisting the horizontal seismic action.

1.3 Objectives and Scope of the Study

The main objectives of the thesis are:

i. To determine elastic deformation and storey drift characteristics of RC buildings of different configuration having variation in storey height and plan dimension as per BNBC (2020).

ii. To determine inelastic deformation and inelastic storey drift characteristics of RC buildings of different configuration having variation in storey height and plan dimension as per BNBC (2020).

iii. To Determination of elastic and inelastic displacement ratio (C_d) and elastic to inelastic storey drift ratio for RC building of different configuration and provide relationship between variations of C_d with different building parameters.

1.4 Outline of Methodology

Structural modelling of RC buildings of different configuration to be done in finite element analytical software. 24 intermediate reinforced concrete moment frames to be designed based on BNBC (2020) and ACI 318 (2002) (where needed) provisions. To cover wide range of building geometries, several frame models with two, three and four bays will be selected. Height range of building will vary from 2 to 12 stores, in 1-storey increments except for the frames more than 6 stories, which will have 2-storey increment. The typical bay span and story height will be 5 and 3.2 meters, respectively.

Elastic deformation of these buildings will be determined as per BNBC prescribed procedure.

Inelastic deformation will be determined using procedure like push over and non-linear time history analysis.

Ratio of elastic to inelastic deformation would be determined. Elastic to inelastic storey drift ratio will be also be evaluated and relationship to be proposed.

1.5 Organization of the Thesis

Chapter 1 is a general introduction to the themes that to be dealt: the main topic is described, the objective and scope of the study including the outline of methodology are set.

Chapter 2 presents concept of deflection amplification factor including inelastic displacement ratio and review the previously published literature in the field of deflection amplification factor. Bangladesh National Building Code related to deflection amplification factor is also discussed in this Chapter. Essential articles related to deflection amplification factor is also included here.

Chapter 3 Methodology has dimensions of models including loading, procedure of analysis and different parameters.

In Chapter 4 Results of analysis are given in detail that include deflection amplification factor, drift pattern, inelastic displacement profile and hinge result.

Chapter 5 presents the conclusions of the thesis and the objectives achieved including some recommendations.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Reinforced Concrete (RC) Structures inherently shows nonlinear behaviour at certain stage subjected to external loading. Traditionally structures are designed following linear equivalent elastic analysis. However nonlinear behaviour of RC structures has to be taken into account. A brief review on existing analysis procedure as per BNBC, deflection amplification factor including inelastic displacement ratio, related BNBC code provisions, related existing research reviews, related definitions and concepts and material nonlinearities have been summarized in this chapter.

2.2 Existing Analysis and Design Procedure

2.2.1 Equivalent Static Analysis

The evaluation of the seismic loads starts with the calculation of the design base shear which is derived from the design response spectrum, S_a. The building period in the two main horizontal directions to be smaller than both $4T_c$ and 2 seconds and the building does not possess irregularity in elevation.

SC type soil is used in analysis. The parameters in BNBC and Etabs of SC soil has difference. That is why site class F has been used in Etabs for Equivalent static analysis. The parameters of Soil type SC has been taken from Appendix C of BNBC.

Table 2.1. Site Dependent Soil Factor and Other Parameters Defining Elastic Response Spectrum (Table 6.2.16 of BNBC)

Sl. No.	Soil Type	S	T_B(s)	T_c(s)	T_d(s)
1	SA	1.0	0.15	0.40	2.0
2	SC	1.15	0.20	0.60	2.0

Table 2.2. Spectral Response Acceleration Parameter S_s and S_1 (Appendix C of BNBC)

Parameters	Zone-1	Zone-2	Zone-3	Zone-4
S_s	0.3	0.5	0.7	0.9
S_1	0.12	0.2	0.28	0.36

Table 2.3. Site coefficient F_a (Appendix C of BNBC)

Sl. No.	Soil Type	Zone 2 & Zone 4
1	SC	1.15

Table 2.4. Site coefficient F_v (Appendix C of BNBC)

Sl. No.	Soil Type	Zone 2 & Zone 4
1	SC	1.725

Table 2.5. Spectral Response Acceleration Parameter S_{ds} (Appendix C of BNBC)

Sl. No.	Soil Type	Zone 2
1	SC	0.383

Table 2.6. Spectral Response Acceleration Parameter S_{d1} (Appendix C of BNBC)

Sl. No.	Soil Type	Zone 2
1	SC	0.23

2.2.1.1 Design Base Shear

The seismic design base shear force in a given direction to be determined from the following relation:

$$V = S_a W \qquad (2.1)$$

where,

S_a= Design spectral acceleration (in units of g) corresponding to the building period T.

W = Total seismic weight of the building.

2.2.1.2 Building Period

The fundamental period T of the building in the horizontal direction under consideration to be determined using following guidelines:

The building period T (in secs) may be approximated by the following formula:

$$T = C_t(h_n)^m \qquad (2.2)$$

where,

h_n = Height of building in meters from foundation or from top of rigid basement.

Table 2.7. Values for Coefficients to Estimate Approximate Period

Sl.	Structure Type	C_t	m	Remarks
1	Concrete moment resisting frames	0.0466	0.9	Resist seismic force

2.2.1.3 Seismic weight

Seismic weight, W is the total dead load of a building or a structure, including partition walls, and applicable portions of other imposed loads listed below:

i. For live load up to and including 3 kN/m^2, a minimum of 25 percent of the live load to be applicable.

ii. For live load above 3 kN/m^2, a minimum of 50 percent of the live load to be applicable.

iii. Total weight (100 percent) of permanent heavy equipment or retained liquid or any imposed load sustained in nature to be included.

2.2.1.4 Vertical Distribution of Lateral Forces

Base shear V, to be considered as the sum of lateral forces F_x induced at different floor levels, these forces may be calculated as:

$$F_x = V \frac{w_x {h_x}^k}{\sum_{i=1}^{n} w_i {h_i}^k} \qquad (2.3)$$

where,

F_x = Part of base shear induced at level x.

w_i and w_x = Part of the total effective seismic weight of the structure (W) assigned to level i or x.

h_i and h_x = the height from the base to level i or x.

k = 1 for structure period less or equal to 0.5 Sec.

= 2 for structure period greater or equal to 2.5 Sec.

= Linear interpolation between 1 and 2 for other periods.

2.2.1.5 Storey Shear and its Horizontal Distribution

The design story shear Vx, at any storey x is the sum of the forces Fx in that storey and all other stories above it.

$$V_x = \sum_{i=x}^{n} F_i \qquad (2.4)$$

where,

F_i = Portion of base shear induced at level i.

2.2.1.6 Deflection and Story Drift

The deflections of level x at the center of the mass to be determined in accordance with the following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I} \qquad (2.5)$$

where,

C_d = Deflection amplication factor.

δ_{xe} = Deflection determined by an elastic analysis.

The design storey drift at storey x to be computed as the difference of the deflections at the centers of mass at the top and bottom of the story under consideration:

$$\Delta_x = \delta_x - \delta_{x-1} \qquad (2.6)$$

2.2.2 Dynamic Analysis Methods

Spectral acceleration is measured in g that describes the maximum acceleration in an earthquake on an object. Dynamic analysis should be performed for regular buildings with height greater than 40m in Zones 2,3,4 and greater than 90 m in Zone 1. Dynamic analysis may be carried out through response spectrum and linear and nonlinear time history analysis.

2.2.3 Non Linear Time History Analysis (NTHA)

It consists of analysis of a structure to determine its response, through methods of numerical integration, to ground acceleration time histories compatible with the design response spectrum for the site. The structure to be fixed at the base. The acceleration time history (ground motion) is applied at the base of the structure. The advantage of this procedure is that actual time dependent behavior of the structural response considering inelastic deformations in the structure can be obtained.

2.2.3.1 Modeling- Non Linear Time History Analysis

A structure to be framed such that represents the spatial distribution of mass throughout the structure. Strength based on expected values considering material over-strength, strain hardening and hysteretic strength degradation. The structure should have a fixed base. Regular structures are with independent orthogonal seismic-force-resisting systems.

2.2.3.2 Ground Motion

For inelastic analysis method, R = 1 and I = 1. The real design acceleration response spectrum is the true representation of the expected ground motion(design basis earthquake) including local soil effects and corresponds to a peak ground acceleration(PGA) value of $\frac{2}{3}ZS$.

$$S_a = \frac{2}{3}\frac{ZI}{R}C_s \qquad (2.7)$$

At least three appropriate acceleration time histories to be used in the analysis.

2.2.3.3 Three-Dimensional Analysis

Ground motions are scaled. An **SRSS** spectrum (ground motion's) to be constructed by taking the square root of the sum of the squares of the five percent damped response spectra for the components(where an identical scale factor is applied to both components of a pair). Each pair of motions to be scaled such that for each period between 0.2T and 1.5T (Where T is the natural period of the fundamental mode of the structure) the average of the SRSS spectra from all horizontal component pairs is not less than 1.3 times the corresponding ordinate of the real design accelertion response spectrum.

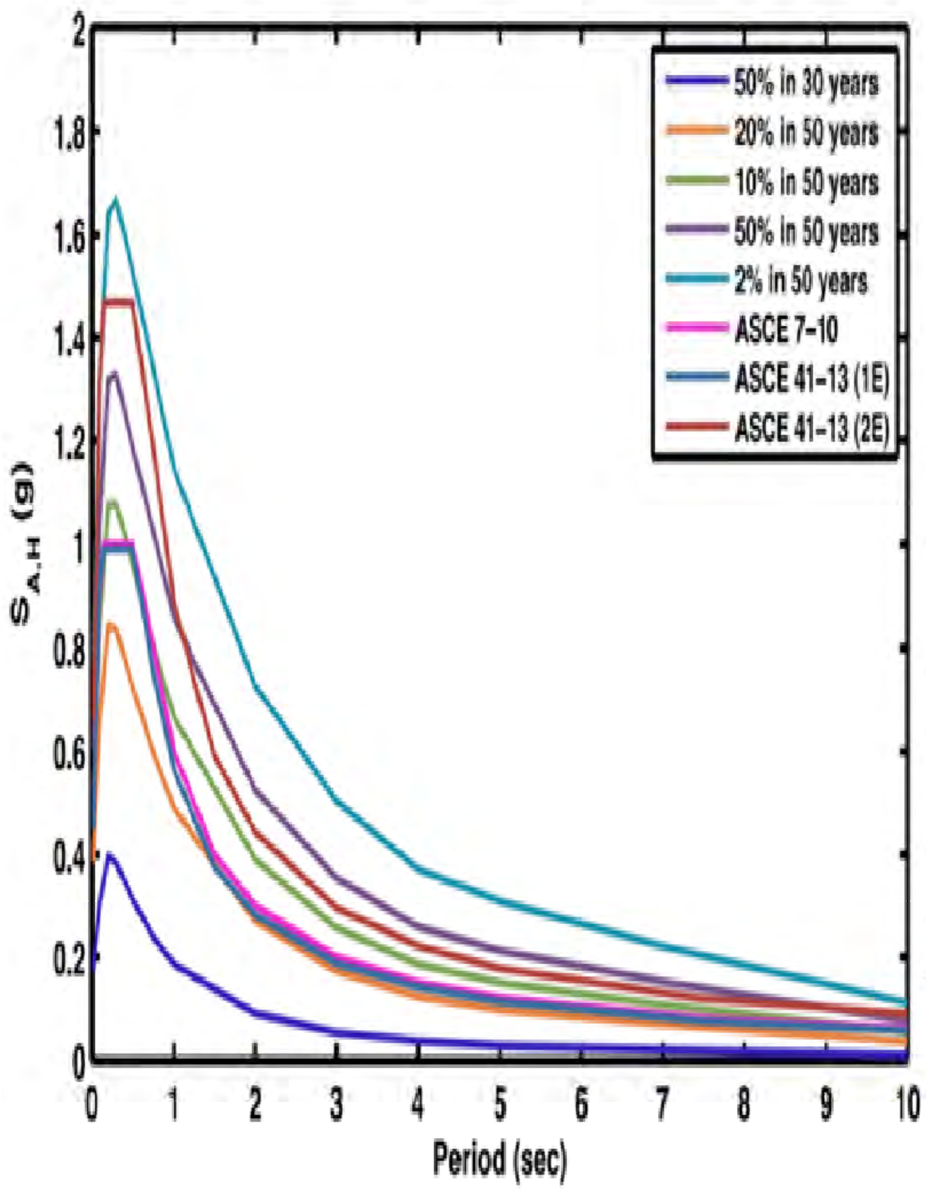


Figure 2.1. Target Horizontal Acceleration Response Spectra at 5% Critical Damping (Shanshan Wang, 2018)

2.2.3.4 Structure Response (Non Linear Time History Analysis)

Maximum base shear obtained from the nonlinear time history analysis. If number of earthquake records used is less than seven, the maximum structural response to be considered as the design value. If the number is at least seven, then the average of maximum structural responses to be considered as the design value. Since real expected earthquake motion input is used,the results as obtained are directly used (no scaling as in LTHA or RSA is required) for interpretation and design.

2.2.3.5 Structure Member Design (Non Linear Time History Analysis)

Gravity and other loads on member deformation capacity should be considered. Member deformation should not exceed two thirds of the smaller of: the value that results in loss of ability to carry gravity loads or the value at which member strength has deteriorated or less than 67 percent of peak strength.

2.2.4 Non-Linear Static Analysis (NSA)

NSA, known as pushover analysis, is a method of directly evaluating nonlinear response. It is an alternative to NTHA. The building is subjected to monotonically increasing static horizontal loads under constant gravity load.

2.3 Related BNBC Provisions

2.3.1 Design Response Spectrum

The earthquake ground motion is represented by the design response spectrum. Both static and dynamic analysis methods are based on this response spectrum. This spectrum represents the spectral acceleration for which the building has to be designed as function of the building period, taking into account the ground motion intensity. The spectrum is based on elastic analysis but in order to account for energy dissipation due to inelastic deformation and benefits of structural redundancy, the spectral accelerations are reduced by the response modification factor, R. For important structures, the spectral accelerations are increased by the importance factor I. The design basis earthquake (DBE) ground motion is selected at a ground shaking level that is 2/3 of the maximum considered earthquake (MCE) ground motion. The effect of local soil conditions on the response spectrum is incorporated in the normalized acceleration response spectrum (C_s).

2.3.2 Design Spectrum for Elastic Analysis

For site classes SA to SE, the design acceleration response spectrum for elastic analysis methods is obtained using following equation to compute S_a (in units of g) as a function of period T. The design acceleration response spectrum represents the expected ground motion (Design Basis Earthquake) divided by the factor R/I.

2.3.3 Design Spectrum for Inelastic Analysis

For inelastic analysis methods,the anticipated ground motion(Design Basis Earhquake) is directly used. Corresponding real design acceleration response spectrum is used,which is obtained by using R = 1 and I = 1 in Eq. 1. The 'real design acceleration response spectrum' is equal to 'design acceleration response spectrum' multiplied by R/I.

2.3.4 Building Categories

2.3.4.1 Importance Factor

Buildings are classified in four occupancy categories.

Table 2.8. Importance Factors for Buildings and Structures for Earthquake Design

Sl. No.	Occupancy Category	Importance Factor
1	I, II	1.00

2.3.4.2 Seismic Design Category

Buildings to be assigned as seismic design category among B, C or D based on seismic zone, local site conditions and importance class of building, as given in the table.

Table 2.9. Seismic Design Category of Buildings

Sl. No.	Site Class	Occupancy Category I,II and III	Occupancy Category IV
		Zone 2	Zone 2
1	SC	C	D

2.3.4.3 Type of Structural Systems

The basic lateral and vertical seismic force-resisting system shall conform to one of the types A to G indicated in Table 2.10 where R and C_d to be used in determining the design base shear and design story drift.

Table 2.10. $\boldsymbol{R}$, $\boldsymbol{C_d}$ and Height Limitations for Different Structural Systems

Sl. No.	Seismic Force Resisting System	R	System Overstrength Factor	C_d	Seismic Design Category B	Seismic Design Category C	Seismic Design Category D
					Height Limit(m)		
1	Intermediate Reinforced Concrete Moment Frame	5	3	4.5	NL	NL	NP

2.3.5 Drift and Deformation.

2.3.5.1 Storey Drift Limit

The design storey drift of each storey should not exceed the allowable storey drift (Table 2.11) for any story.

Table 2.11. Allowable Storey Drift Limit

Sl.	Structure	Occupancy Category		
		I and II	III	IV
1	All other Structure	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

hsx is the storey height below level. For nonlinear time history analysis (NTHA), the storey drift obtained should not exceed 1.25 times the storey drift limit specified above for linear elastic analysis procedures.

2.4 Previous Study on Deflection Amplification Factor and Existing Literature on Nonlinear Analysis for IMRF

2.4.1 Previous Study on Deflection Amplification Factor Ratio

A number of researchers have carried out study on deflection amplification factor and inelastic displacement ratio. Some of the research works have been summarized in this section:

Uang and Maarouf (1993) had publication on Deflection amplification factor for seismic design provisions. Seismic design provisions estimate the maximum roof and story drifts occurring in major earthquakes by amplifying the drifts computed from elastic analysis at the prescribed design seismic force level with a deflection amplification factor (DAF). An analytical study of the seismic responses of four instrumented buildings confirmed that drifts developed in major earthquakes are much higher than those predicted by the UBC or NEHRP approach. It is recommended that the deflection amplification factor be increased to at least the seismic force reduction factor (R_w in UBC and R in NEHRP) for estimating maximum drifts. The effects of the ratio between building period and earthquake predominant period, types of yield mechanisms, and structural overstrength on the (DAF) are also presented.

For estimating the maximum inelastic deflection that might occur during an earthquake, the design deflections computed from an elastic analysis are amplified by a deflection amplification factor (C_d) (Uang, 1991; Mohammadi, 2002). This process is referred to as force-based design method.

The deflection amplification factor of RC buildings has been evaluated through a statistical procedure by Hwang and Jaw (1989). The ratio of the corresponding nonlinear and linear displacements was defined as the C_d factor. They obtained structural response data from dynamic analyses was used to extract an empirical equation for the C_d factor as a function of the maximum story ductility ratio. They concluded that generally, the calculated design story drifts based on specified C_d factors in NEHRP provisions are overestimated. The ratio of C_d/R was also calculated for two types of steel structures and two special reinforced concrete moment frames (Uang and Maarouf, 1994). This ratio was evaluated as a function of ductility-reduction-factor (R_d). It was observed that the ratio of inelastic to elastic roof displacement increases with an increase in R_d factor. They indicated that within the practical range of ductility reduction factors for the analyzed frames, the ratio of inelastic to elastic interstory drifts varies from 1.0 to 1.5, which can be even higher for the structures with a weak first story.

Samimifar et al. (2015) assessed the inelastic and elastic lateral deformations, 24 intermediate reinforced concrete moment frames were designed based on the Iranian Seismic Design Code (Standard No. 2800-2005), and ACI 318 (2002) provisions. To cover a wide range of building geometries, several frame models with two, three, and four bays were selected. They ranged in height from 2 to 12 stories, in 1-story increments except for the frames with more than 6 stories, which had 2-story increments. The typical bay span and story height were 5 and 3.2 meters, respectively. Few Conclusions are as follows:

i. The proposed value for the C_d/R ratio, which is calculated based on the ratio of maximum inelastic to elastic roof drift for intermediate RC frames, is equal to 1.0 when computed according to the stories' maximum inelastic to elastic displacement ratio.

ii. It was observed that generally, the ratio of maximum inelastic to elastic displacements increases along the height of RC frames and shows a milder ascending trend when the number of frame stories increases, whereas changing the number of bays does not significantly affect this ratio.

2.4.2 Existing Literature on Nonlinear Analysis for IMRF

Abou-Elfath (2019) studied to evaluate the inelastic displacement ratio (ρ) for moment resisting steel frames (MRSFs) designed according to Egyptian Code. Four MRSFs having 2, 4, 8 and 12 stories are designed and are analyzed under the effect of two sets of ground motion records. The results obtained in this study indicate that the consideration of ρ for both the roof drift ratios (RDRs) and the maximum story drift ratios (MSDRs) equal to **1.0** is a reasonable estimation for MRSFs designed according to the Egyptian code.

Another approximate method was presented by Miranda (2002) for investigating the maximum roof and inter story drifts in a simplified model of multistory buildings with non-uniform lateral stiffness. It was a continuation of his research (Miranda, 1999) in which some amplification factors were defined to estimate inelastic deformations of these models with uniform stiffness as functions of number of stories and displacement ductility ratio. However, only the first mode contribution was considered for the models during an earthquake. Miranda (2002) concluded that the difference between the spectral displacement and the maximum roof displacement increases with the number of stories as well as overall deflections developed by flexural behavior.

2.5 Summary on Nonlinearities

2.5.1 Lateral Stiffness is Equal to Force/Length

The structure shown in Fig. 2.2a with no dynamic excitation is subjected to an externally applied static force f_s along the DOF u. It is desired to determine the relationship between the force f_s and the relative displacement u associated with deformations during oscillatory motion. This force–displacement relation would be linear at small deformations but would become nonlinear at larger deformations (Fig. 2.2c); both nonlinear and linear relations are considered (Fig. 2.2c and 2.2d).

To determine the relationship between f_s and u is a standard problem in static structural analysis.

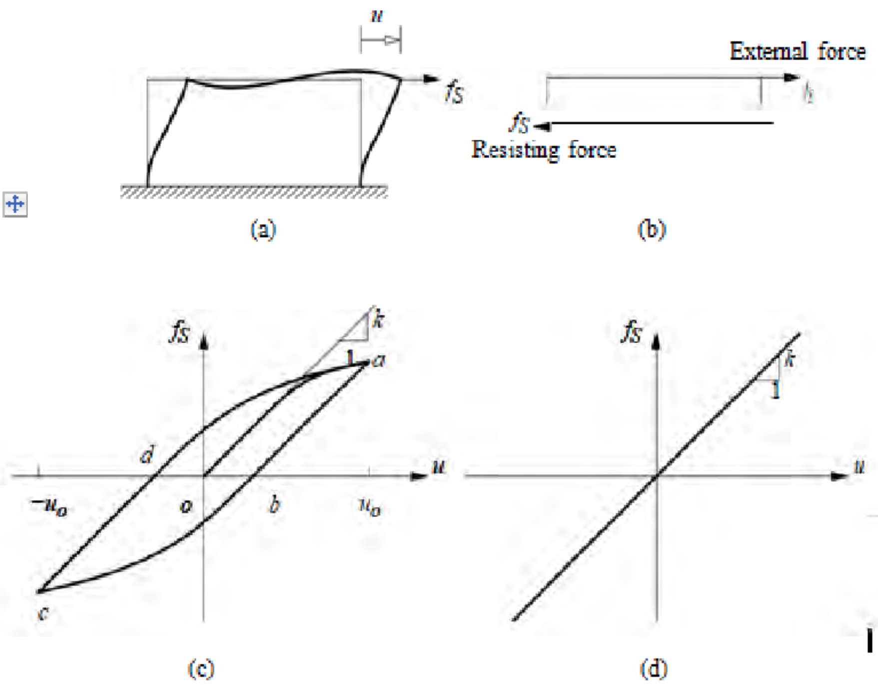


Figure 2.2. Relationship between Force and Displacement (Chopra, 2012)

2.5.1.1 Behaviour and Basic Response of Elasto Plastic material

The basic behavior of an elasto-plastic material is summarized in Fig. 2.3. The behavior is linear elastic with stiffness E until the initial yield stress σ_{yo} is reached; after that plastic strains develop. Unloading from point A, Fig. 2.3, occurs elastically with the stiffness E so that at complete unloading to point B, the residual strain amounts to the plastic strain ε^p developed at point A. Therefore, at point A, the total strain Consists of the sum of the elastic and plastic strain, i.e.

$$\varepsilon = \varepsilon^e + \varepsilon^p \qquad (2.8)$$

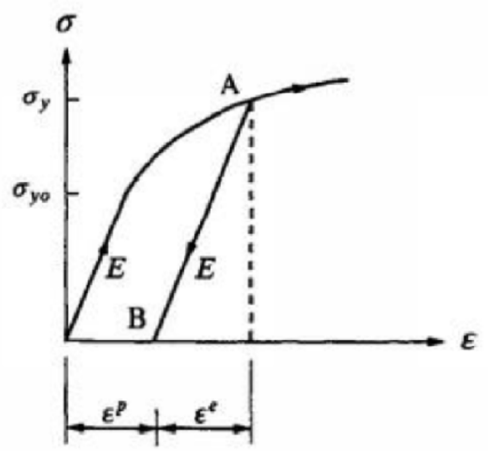


Figure 2.3. Basic Response of Elasto-Plastic Material (Ottosen and Ristinmaa, 2005)

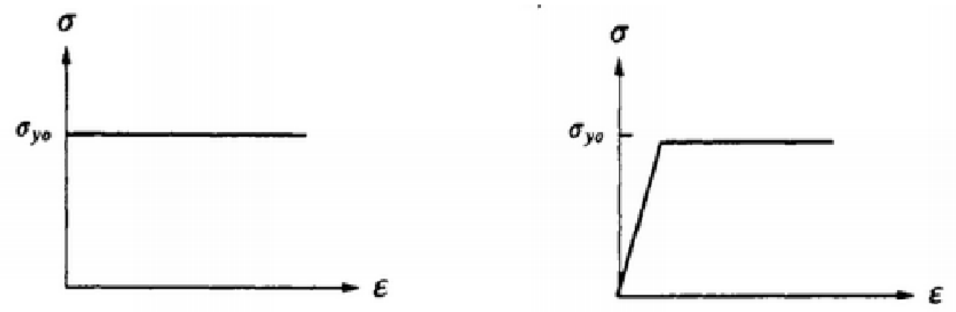


Figure 2.4. (a) Stiff-Ideal Plastic Behavior; (b) Elastic-Ideal Plastic Behavior (Ottosen and Ristinmaa, 2005)

2.5.2 Lateral Stiffness of Frame is Independent of Beam Length

For a linear system the relationship between the lateral force f_s and resulting deformation u is linear, that is,

$$f_s = ku \qquad (2.9)$$

where, k is the lateral stiffness of the system; its units are force/length. Implicit in Eq. (2.9) is the assumption that the linear $f_s - u$ relationship determined for small deformations of the structure is also valid for larger deformations. This linear relationship implies that f_s is a single-valued function of u (i.e., the loading and unloading curves are identical). Such a system is said to be elastic; hence we use the term linearly elastic system to emphasize both properties.

Let consider the frame of Fig. 2.5 a with bay width L, height h, elastic modulus E and **second moment of the cross-sectional area** I (or moment of inertia) about the axis of bending = I_b and I_c for the beam and columns, respectively; the columns are clamped (or fixed) at the base. The lateral stiffness of the frame can readily be determined for two extreme cases: If the beam is rigid [i.e., flexural rigidity $EI_b = \infty$] (Fig. 2.5 b),

$$k = \sum_{columns} \frac{12EI_c}{h^3} = 24\frac{EI_c}{h^3} \qquad (2.10)$$

On the other hand, for a beam with no stiffness [i.e., $EI_b = 0$ (Fig. 2.5 c)],

$$k = \sum_{columns} \frac{3EI_c}{h^3} = 6\frac{EI_c}{h^3} \qquad (2.11)$$

It is observed that for the two extreme values of beam stiffness, the lateral stiffness of the frame is independent of L, the beam length or bay width.

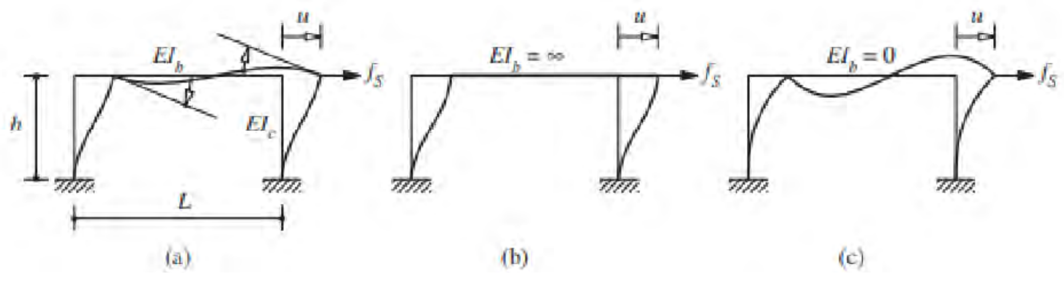


Figure 2.5. Lateral Stiffness of Frame is Independent of Length (Chopra, 2012)

2.5.3 Inelastic Systems

The initial loading curve is nonlinear at the larger amplitudes of deformation, and the unloading and reloading curves differ from the initial loading branch; such a system is said to be inelastic. This implies that the force deformation relation is path dependent, i.e., it depends on whether the deformation is increasing or decreasing. Thus, the resisting force is an implicit function of deformation:

$$f_s = f_s(u) \qquad (2.12)$$

The force–deformation relation for the idealized one-story frame (Fig. 2.5 a) deforming into the inelastic range can be determined in one of two ways. One approach is to use methods of nonlinear static structural analysis.

2.5.4 Direct Integration Time History Analysis

Direct-integration time-history analysis is a nonlinear, dynamic analysis method in which the equilibrium equations of motion are fully integrated as a structure is subjected to dynamic loading. Analysis involves the integration of structural properties and behaviours at a series of time steps which are small relative to loading duration. The equation of motion under evaluation is given as follows:

$$\mathrm{M\ddot{u}(t) + C\dot{u}(t) + Ku(t) = F(t)} \qquad (2.13)$$

Integration is performed at every time step of the input record, regardless of the output increment.

2.5.5 Inelastic Displacement of Structure

Inelastic displacement is the actual displacement, obtained by elastic analysis multiplied by a deflection amplification factor, $\mathrm{C_d}$. The inelastic displacement is obtained from the following equation:

$$\Delta_{\mathrm{x}} = \frac{\mathrm{C_d}\Delta_{\mathrm{xe}}}{\mathrm{I_e}} \qquad (2.14)$$

where,

Δ_{x} = Inelastic Displacement

Δ_{xe} = Elastic Displacement

C_d = Deflection Amplification Factor

I_e = Importance Factor

As illustrated in Figure 2.6, the deflection amplification factor, C_d, accounts for the increase in displacement due to the inelastic response of a structure that is not determined by elastic analysis and corrects for the reduction of forces introduced by response modification factor R.

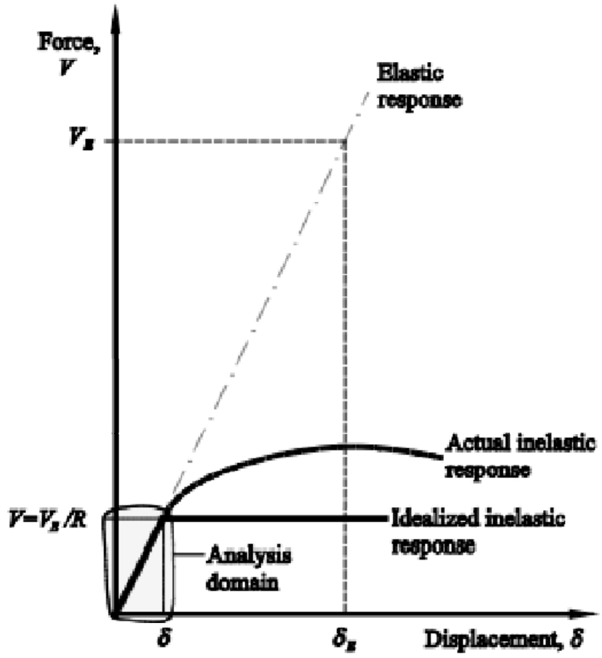


Figure 2.6. Displacement used to compute drift (FEMA P-1050-1/2015 Edition)

If a structure remains elastic during an earthquake, the forces developed in the building are elastic (not reduced by R) which results in elastic displacements that donot account for system ductility and overstrength. If a structure is inelastic with initial stiffness equal to that of an elastic structure; then its maximum displacement will be similar to that of an elastic system.

2.5.6 Scaled Time History

Time-domain spectral matching of an earthquake ground motion consists of iteratively adding sets of wavelets to an acceleration history until the resulting response spectrum sufficiently matches a target spectrum. The spectral matching procedure is at its core a nonlinear problem because the addition of a wavelet often causes shifting in the time of peak response or creation of a larger second peak at a different time.

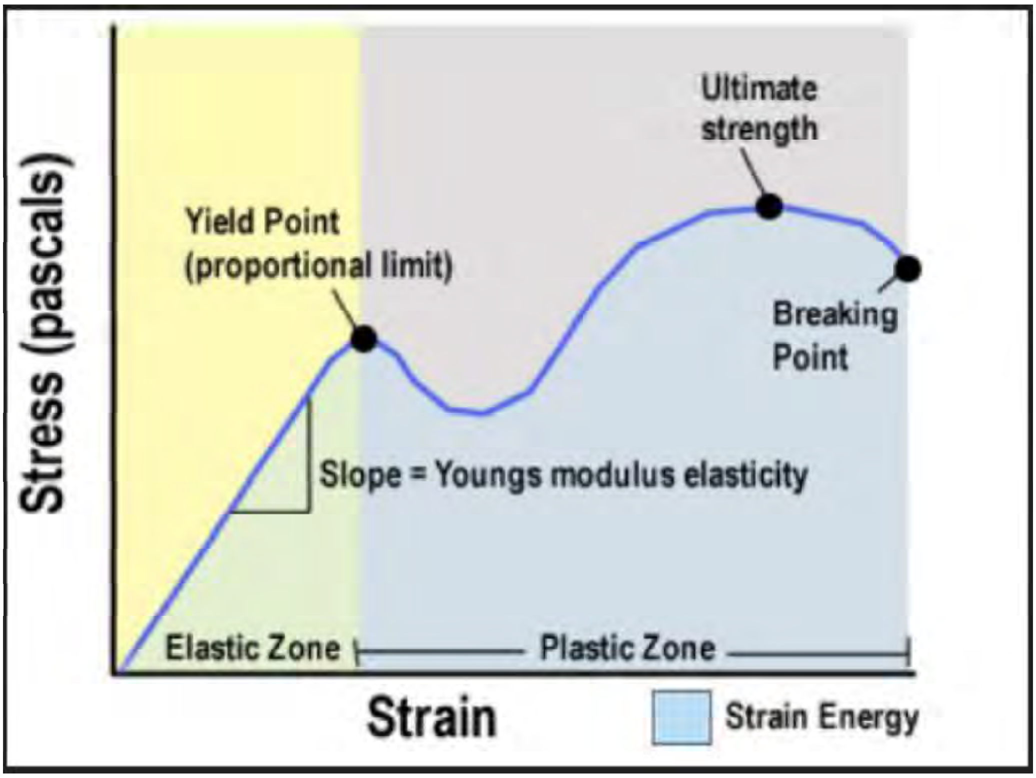


Figure 2.7. Material Strength (**Source:** Orthobullets.com)

2.5.7 Deflection Amplification Factor (C_d) and Inelastic Displacement Ratio (ρ)

Static analysis cannot be used when the structure undergoes large displacements and stresses. Static studies neglect inertial and damping forces. Practically loads are not applied slowly or they change with time or frequency therefore dynamic analysis is used. When the frequency of a load is larger than 1/3 of the lowest (fundamental) frequency, a dynamic study should be used.

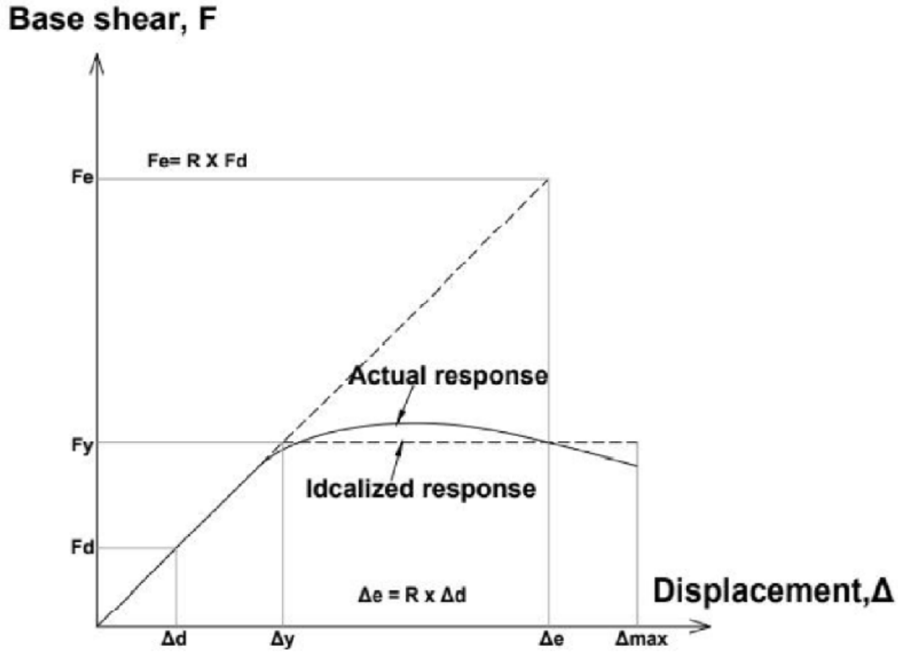


Figure 2.8. General Structural Response under the Effect of Lateral Loading (Elfath, 2019)

Figure shows global inelastic response of a structure under the effect of lateral loading. The actual inelastic response is idealized by a bilinear relation between the base shear and a lateral displacement component of the structure. Δ_{max} represents the maximum displacement demands under inelastic earthquake analysis. The elastic force and displacement demand F_e and Δ_e are related to the design force and displacement demands F_d and Δ_d according to the following relation:

$$\frac{F_e}{F_d} = \frac{\Delta_e}{\Delta_d} = R \qquad (2.15)$$

According to the definition presented by FEMA P695 (2009) and by various researchers such as Uang and Maarouf (1994), the inelastic displacement ratio ρ is calculated according to the following equation:

$$\rho = \frac{DAF}{R} = \frac{(\Delta_{max}/\Delta_d)}{R} = \frac{\Delta_{max}}{\Delta_e} \qquad (2.16)$$

This indicates that the inelastic displacement ratio ρ is equal to the ratio of inelastic to elastic displacements of the multi-story structures under the effect of earthquake loading. The mean values of the inelastic displacement ratio (ρ) are calculated based on the ratios of inelastic to elastic displacements of the multi-story frames.

2.5.8 Inelastic Displacement Ratio Value

The value of the ρ-ratio specified by the European code (Euro code 8, 2004) and the Canadian code (NBCC, 2010) is equal to 1.0, while it equals to 0.7 in the Egyptian code (ECP-201, 2012). The ASCE 7-10 specification (ASCE 7-10, 2010) assigns different values to ρ depending on the type of the structural system.

2.5.9 Stress-Strain Curve of Isotropic Material

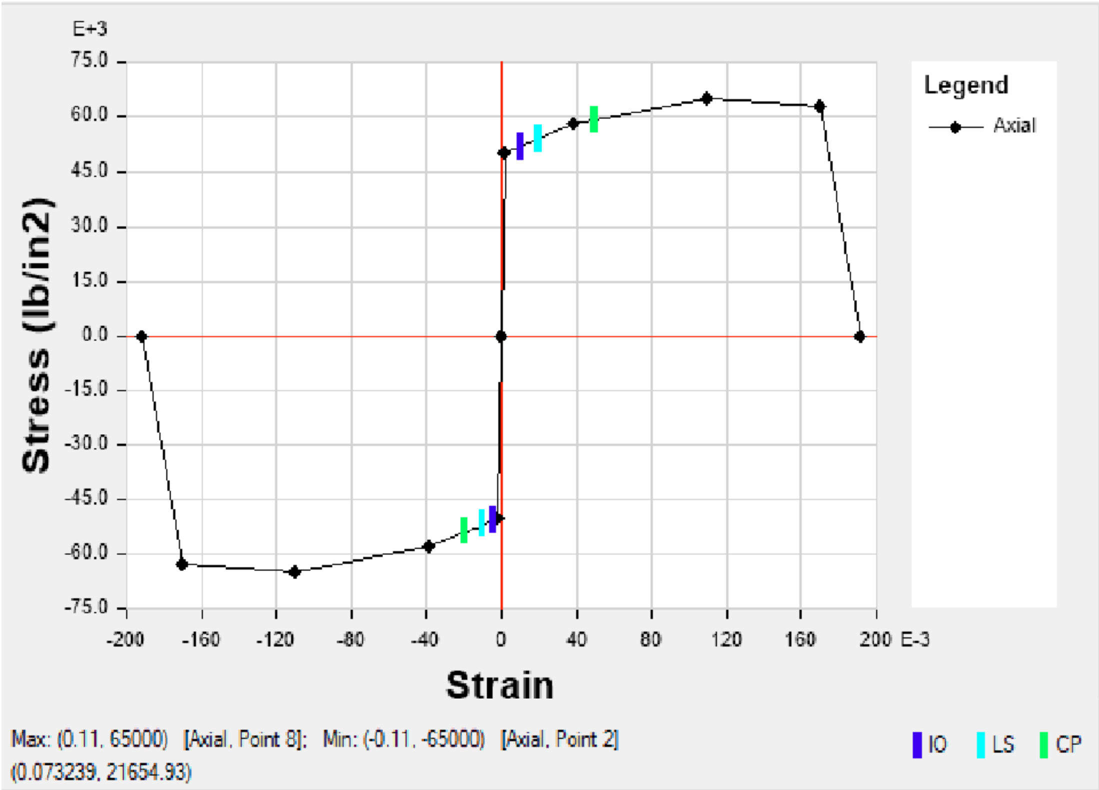


Figure 2.9. Stress-strain curve of steel as an isotropic material (Etabs 2016)

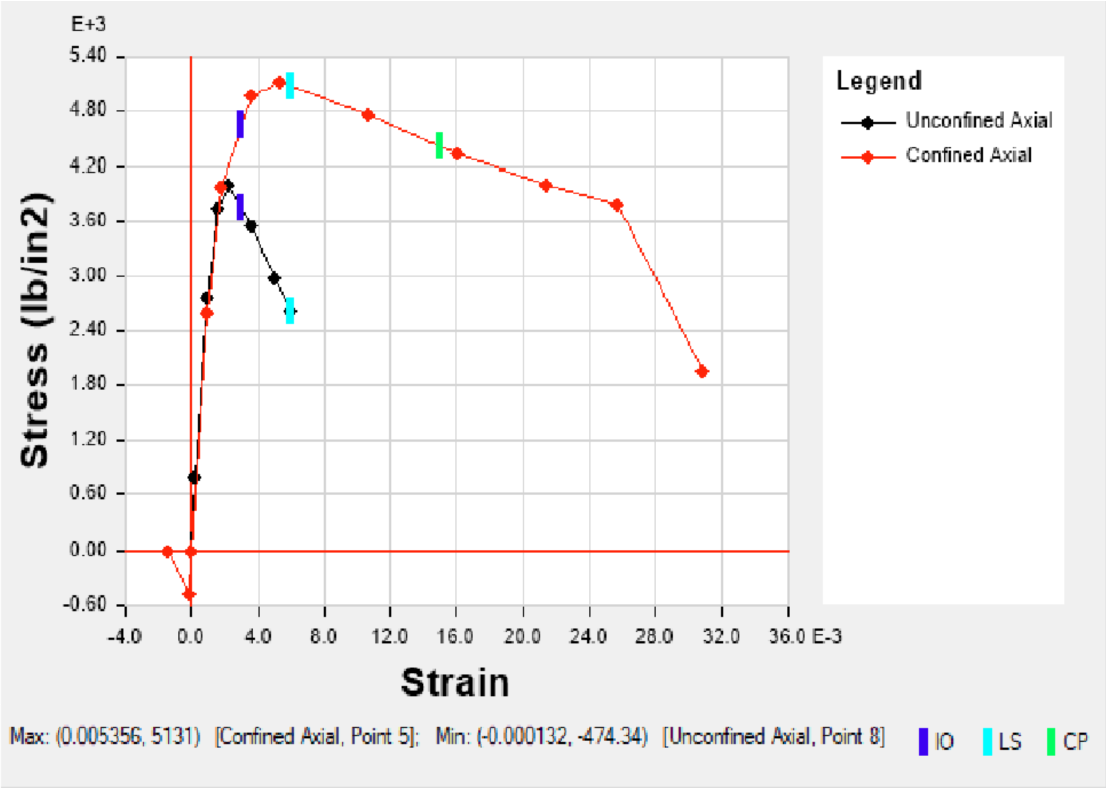


Figure 2.10. Stress-strain curve of concrete as an isotropic material (Etabs 2016)

CHAPTER 3

METHODOLOGY

3.1 General

Numerical modelling of Reinforced Concrete frames has been presented in this chapter. In this research, total 24 reinforced concrete frame structures have been analysed using finite element method. Firstly, structural analysis of these frames has been performed using equivalent static analysis as per BNBC 2020. Using same geometrical, material and loading data, nonlinear time history analysis has been performed for all cases. There are mainly two methods to evaluate deflection amplification factor. Ratio of nonlinear time history displacement with Equivalent static displacement and ratio of nonlinear time history displacement with linear time history displacement (BNBC, 2020).

Basic analysis and design consideration for equivalent static analysis and modelling criteria, hinge properties and loading criteria for nonlinear time history analysis has been discussed in this chapter.

3.2 Equivalent Static Analysis

Basic design considerations(material properties,loading,boundary conditions etc) and design outputs of equivalent static analysis to be discussed in this section.

3.2.1 Seismic Load

Table 3.1. List of Load Cases

Name	Type	Self-Weight Multiplier	Auto Load
Dead	Dead	1	
Live	Live	0	
E_x	Seismic	0	ASCE 7-05
E_y	Seismic	0	ASCE 7-05
SD	Superimposed Dead	0	

The evaluation of the seismic loads starts with the calculation of the design base shear which is derived from the design response spectrum This Section presents different computations relevent to the equivalent static analysis procedure. The load cases are as follows:

3.2.2 Design Considerations

Structural analysis and design have been performed according to BNBC (2020). Other codes, standards, specifications have been utilized as required in structural design.

A. Structural Geometry Considerations

Initially shape, size, story height and number of stories of the building have been considered as per design requirement and checked as per BNBC (2020). The buildings having bay width of 5 meter and height of 3.2 meter. There are frames of 2,3 and 4 bay. Each bay has 8 frames having 2, 3, 4, 5, 6, 8, 10, 12 story. Typical column location, beam location is shown in the following layout (Figure 3.1).

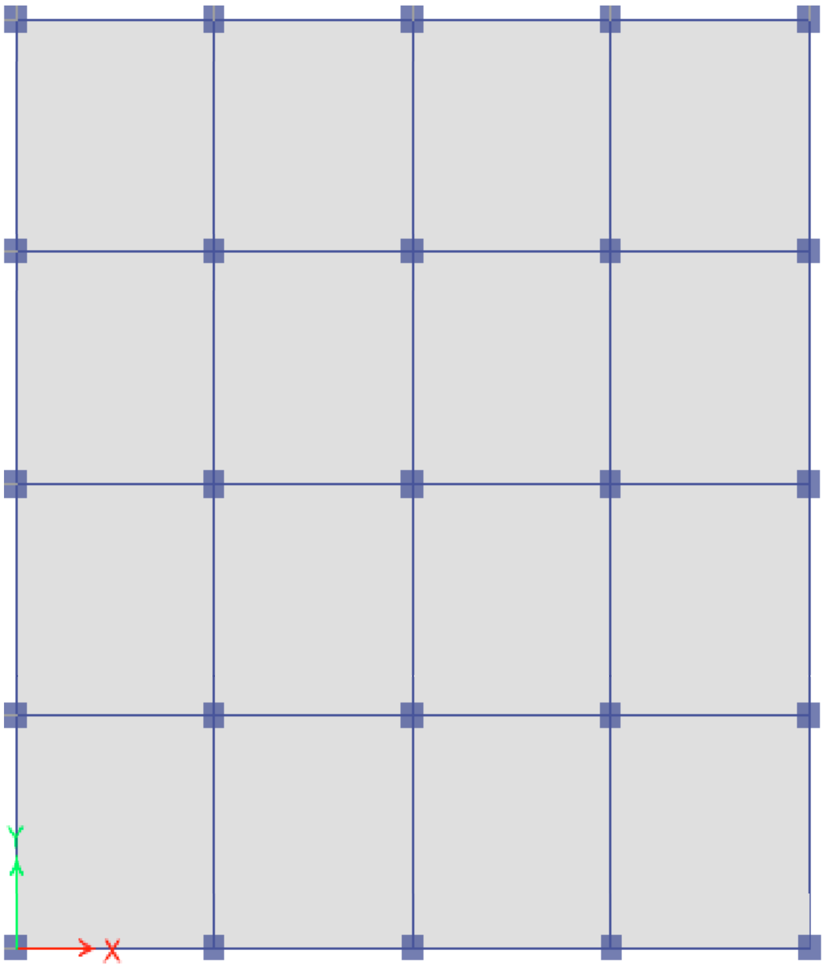


Figure 3.1. Column Layout of Four Bay Twelve Story

Table 3.2. Model Types and Dimension

RC Frame (Shape)	Story Height (in meter)	No of Story							
Model-1 10m×10m	3.2m	2	3	4	5	6	8	10	12
Model-2 15m×15m	3.2m	2	3	4	5	6	8	10	12
Model -3 20m×20m	3.2m	2	3	4	5	6	8	10	12

B. Material Specification

The grade of steel and concrete strength considered is as follows:

Specified Concrete Compressive Strength : 27.6 MPa (4000 psi)

Minimum Yield Stress of Steel, F_y : 34.5 MPa (50000 psi)

Minimum Yield Strength of Rebar, Uniaxial : 41.4 MPa (60000 psi)

Table 3.3. Material Specification

Name	Type	E (GPa)	v	Unit Weight (kN/m³)	Design Strengths
4000Psi	Concrete	24.9	0.2	23.5631	F_c = 27.6 MPa
A615Gr60	Rebar	200	0.3	76.9729	F_y = 414 MPa F_u = 621 MPa

C. Loading Criteria

The building has been analyzed for possible load actions such as Gravity and Lateral Loads. Gravity Loads, such as dead load and live loads applied at the floors and roofs of the building according to the provision of Chapter 2, Part 6 of BNBC (2020) are as follows:

Dead Loads,

Self-Weight of Concrete = 23.55 kN/m^3 (150 pcf)

Superimposed Dead Load = 3.36 kN/m^2 (70 psf)

Live Load = 2.02 KN/m^2 (42 psf)

Seismic Load Consideration Parameters,

Seismic Zone (Z)	: Zone II (Dhaka, Bangladesh)
	: 0.2 [Table 6.2.14]
Response Modification Coefficient (R)	: 5 for IRMF [Table 6.2.19]
Structural Importance Factor	: 1 [Table 6.2.17]
Site Coefficient (S)	: 1.15 [Table 6.2.16]
Numerical Coefficient (C_t)	: 0.0466 (for 'h' in 'meter')
Building Seismic Design Category, C	: IMRF [Table 6.2.18]
Building Period, (T for 12 Story)	: 1.305 sec
Basement to Ground Floor Height	= 2.13 meter (7 Feet)

D. Structural Period

Estimate of Approximate Period

C_t = 0.0466; for Concrete moment resisting frame

m = 0.9; for Concrete moment resisting frame

h_n = Height of building in meters

For 12 Storied building, each floor having 10.5 feet and ground floor having 7 feet height

h_n = 7+(12×10.5) =133 feet = (133×0.3048) = 40.5384 meter

T = 0.0466(40.5384)×0.9 = 1.30456

Long-Period Transition Period, T_L [ASCE 11.4.5] T_L = 1 sec.

E. Boundary Conditions

To simulate structural behaviour, Column base supports have been considered as fixed supports in 3D model of super structure. Joint assignments (base fixed) including restraints for column base are as follows:

Table 3.4. Restraints in Global Directions of Column Base

Item	Condition
Translation in X Direction	Restrained
Translation in Y Direction	Restrained
Translation in Z Direction	Restrained
Rotation about X Direction	Restrained
Rotation about Y Direction	Restrained
Rotation about Z Direction	Restrained

F. Selection of Analysis Type

Structural analysis has been performed in a single step using the equivalent linear static analysis method and finite element method.

3.2.3 Demand Capacity Ratio Check

Design checking of the structure done after equivalent static analysis is completed. After that clicking on Display Design Info,Column P-M-M Interaction Ratios is checked. The ratio should be less than 1. If the ratio is more than 1 then column size to be increased.

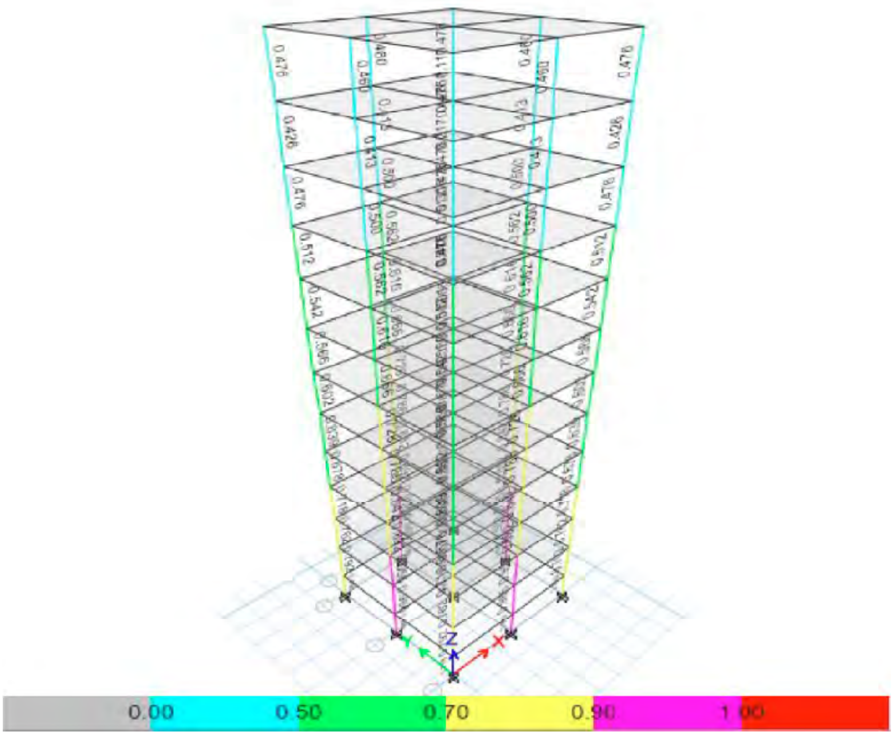


Figure 3.2. Demand-Capacity Ratio

3.3 Nonlinear Time History Analysis

3.3.1 Structural Modelling

The intermediate moment resisting frame(IMRF) whose performance would be between that of the ductile moment resisting frame and the ordinary moment resisting frame could be considered for application in moderate seismic zone. The seismic detailing requirements of this type would be less stringent than that required for special RC moment resisting frames. IMRF perform at a lower level of toughness with an appropriate compensating modification in the response reduction factor.

Member deformation should not exceed two thirds of the smaller of: the value that results in loss of ability to carry gravity loads or the value at which member strength has deteriorated or less than 67 percent of peak strength.

For nonlinear time history analysis (NTHA), the storey drift obtained should not exceed 1.25 times the storey drift limit specified above for linear elastic analysis procedures.

3.3.2 Load Case Data

Nonlinear dynamic load case name is given. Load case type/subtype is Time History/Nonlinear Direct Integration. Mass source is as previous. Zero initial condition which starts from unstressed state. Loads applied are pair of ground motion which are acceleration type load. Scale factor in loads applied is I_g/R where R = 1. Acceleration unit is in/sec^2. Geometric Nonlinearity Option is taken as none. Number of Output Time Steps is taken 100. Output Time Step Size is 0.1 Sec. So, ground motion run time is 100×0.1 = 10 sec. If output time step size is 0.01 sec then ground motion run time is going to be 100×0.01 = 1 sec. Direct Integration Damping is taken as Viscous Proportional Damping where in Direct Specification Mass Proportional Coefficient is 0.05 and Stiffness Proportional Coefficient is 0.05. Time Integration Method is Hiber Hughes-Taylor in which Gamma = 0.5, Beta = 0.25, Alpha = 0.

3.3.2.1 Geometric Nonlinearity Option

None (Neither P-Delta or P-Delta plus Large Displacement)

3.3.2.2 Material Nonlinearity Parameters

The material nonlinearity parameters are as follows:

Table 3.5. Material Nonlinearity Parameters

Frame Tension or Compression Only	Yes
Frame Hinges	Yes
Link Tension or Compression Only	Yes
Link Other Nonlinear Properties	Yes
Time Dependent Material Properties	No

3.3.3 Load and Deformation Criteria

The Loadings and their deformation criteria are as follows:

Table 3.6. Load and Deformation Criteria

Name	**Type**
Dead	Linear Static
Live	Linear Static
E_x	Linear Static
E_y	Linear Static
SD	Linear Static
NONLINEARDYNAMIC	Nonlinear Direct Integration History

The other few loading parameters are as follows:

Superimposed dead load for floor finish = 1.5 kN/m^2

Live load = 2 kN/m^2

Seismic weight = Total dead load + 25% of total live load

No wind load was considered

3.3.4 Hinge Properties

A hinge property is a named set of nonlinear properties that can be assigned to points along the length of one or more frame elements. Yielding and post-yielding behaviour can be modelled using discrete user-defined hinges. Hinges can be assigned to a frame element at any location along the clear length of the element. Uncoupled moment, torsion, axial force and shear hinges are available. There are also coupled P-M2-M3 hinges which yield based on the interaction of the axial force and bi-axial bending moments at the hinge location. Subsets of these hinges may include P-M2, P-M3, and M2-M3 behavior.

More than one type of frame hinge can exist at the same location, for example, we may assign both an M3 (moment) and V2 (shear) hinge to the same end of a frame element. Hinge properties can be computed automatically from the element material and section properties (FEMA-356 of FEMA, 2000 or ACSE 41-13). Hinges only affect the behaviour of the structure in nonlinear static and nonlinear time history analysis.

3.3.4.1 Frame Assignment-Hinges

Table 3.7. Frame Hinge Assignment Data

Hinge Property	Relative distance
Moment	0.1
Moment	0.9

3.3.4.2 Moment Vs Rotation Curve

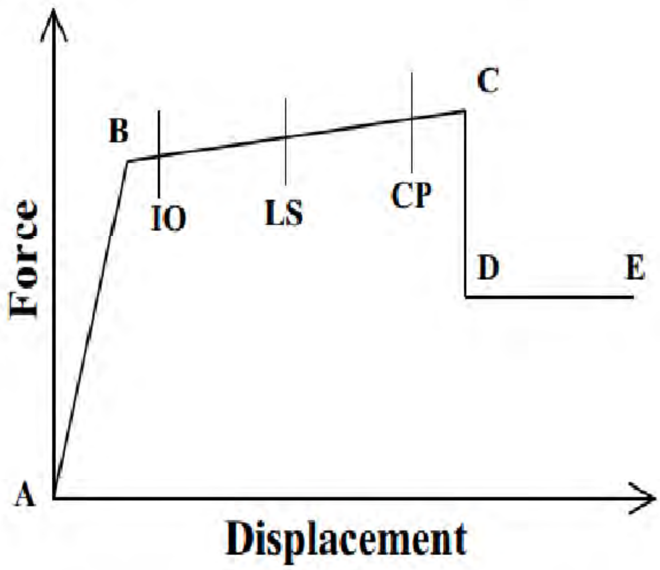


Figure 3.3. Moment Vs Rotation Curve (or, Force Vs Displacement)

3.3.4.3 Displacement Control Parameters (Moment Rotation Type)

Displacement Control parameter for moment rotation type is as follows:

Table 3.8. Moment Rotation Parameters

Point	Moment/SF	Rotation/SF
A	0	0
B	1	0
C	1.1	0.015
D	0.2	0.015
E	0.2	0.025

Load carrying capacity beyond point E drops to zero

3.3.4.4 Scaling for Moment and Rotation

Yield moment is used

3.3.4.5 Acceptance Criteria (Plastic Rotation/SF)

Acceptance Criteria for Plastic Rotation is as follows:

Table 3.9. Acceptance Criteria for Plastic Rotation

Level	Positive	Negative
Immediate Occupancy	0.003	-0.003
Life Safety	0.012	-0.012
Collapse Prevention	0.015	-0.015

3.4 Ground Motion and Analysis Procedure

3.4.1 Peak Ground Acceleration of Imperial Valley Earthquake

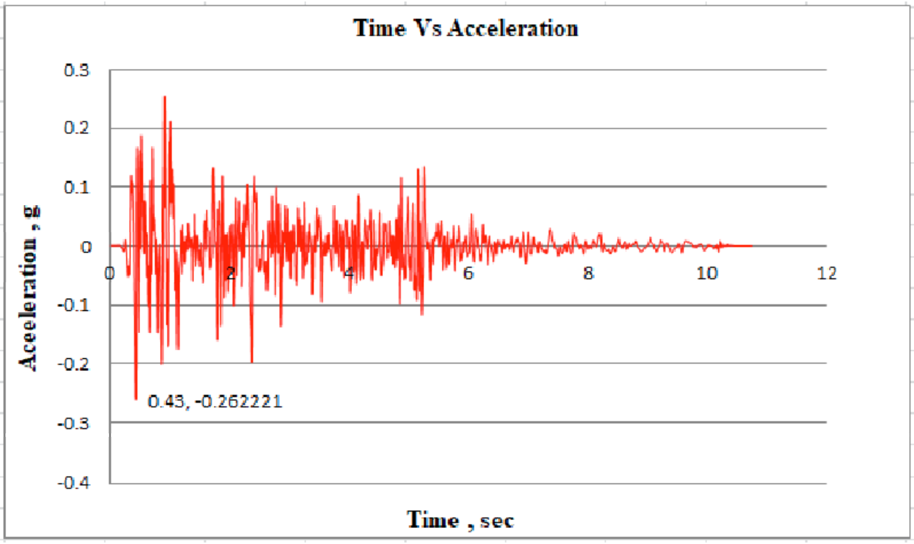


Figure 3.4. Peak Ground Acceleration of Imperial Valley Earthquake

PEER NGA Strong Motion Database Record

Imperial Valley-02, 5/19/1940, El Centro Array #9, 180

Acceleration Time Series in units of g.

No of Points = 5372, Difference of Time = .0100 Sec

Scale Factor = I_g/R

Here, g = 32.2 ft/sec^2 = 386.4 in/sec^2

Scale Factor of Nonlinear Analysis , I_g/R_x = (1x386.4)/1=386.4

Linear Time History Analysis, I_g/R_x = 1×386.4/5 = 77.28

3.4.2 Scale Factor for Dhaka Region

Peak Ground Acceleration/ Maximum Considered Earthquake of Dhaka = 0.2g

Peak Ground Acceleration of Ground Motion = 0.2605g;

To scale down acceleration to 0.2g (Peak Ground Acceleration of Dhaka); let equalizing factor is A.

$0.2605g \times A = 0.2g$

So, $A = 0.768$

So, in Nonlinear Time History Analysis, Scale Factor = (386.4) (0.768) = 296.66.

In Linear Time History Scale Factor for 0.2g = I_g/R = 296.66/5 = 59.3

3.4.3 Scale Factor for Sylhet Region

Peak Ground Acceleration of Sylhet Region=0.36g

$0.2605g \times A = 0.36g$

So, $A = 1.382$

Scale Factor = 386.4×1.382 = 533.9

In Linear Time History Scale Factor for 0.36g = I_g/R=533.9/5=106.78

3.4.4 Selecting/Forming Target Response Spectrum

Nonlinear Time History Analysis is a very complex phenomenon. To analyse a structure against a specific structure first equivalent static analysis is done. Equivalent static analysis is done to check that the structure is resistant against earthquake. After several trial for beam and column the structure is designed and checked. Once design check is complete then Demand capacity ratio (Column P-M-M interaction ratio) is checked. Demand capacity ratio should be less than one.

Then Target Response Spectrum is formed for specified site class and zone. Spectral Acceleration for 0.2 sec and 1 sec is termed as S_s and S_1. Soil type is taken sandy clay (SC) for which to put value as per Code, site class F has been taken. Once Site coefficient is put then calculated values for Response Spectrum Curve is generated which match with the code. A specific earthquake is matched with this Target Response Spectrum.

3.4.5 Earthquake Database-Time History Function Definition Imperial Valley Data

Imperial earthquake database is downloaded from the Pacific Earthquake Engineering Research Center data base. Modern approaches to assessing seismic performance of infrastructure rely on good information about likely ground shaking at a site. Historically, access to earthquake ground motion data has been hampered by difficult access to the large body of data, as well as by the inconsistency in how the data are gathered and stored.

In the late 1990s Pacific Earthquake Engineering Research Center recognized the need to improve access to earthquake ground motion data and thus embarked on an effort to create a web-based searchable database of strong ground motion data. The first step was to collect the most important ground motion records worldwide. The second step was to ensure that all the data had been processed consistently and reliably. The following step was to gather related metadata such as earthquake magnitude, various site-to-source distance measures, style of faulting, local site conditions at the recording stations, and other relevant engineering parameters. Finally, Pacific Earthquake Engineering Research created the online database to make all the information available to the public

Imperial valley earthquake data is browsed from file. Earthquake data once incorporated from file to ETABS. Then data can be converted to user defined format.

Time history function graph once found is converted to user defined graph by right clicking above/below the acceleration graph. Thereby the user defined graph can be found by clicking on Display as Resizable Graph.

Thereby the User Defined Graph is Displayed. From this graph it is found that Peak Ground Acceleration is $0.26g$ at 0.43 Sec.

Once clicking on User Defined Graph, Graph Plot Function Data is displayed. In this box Horizontal scale factor and vertical scale factor is 1. For scaling up or down, the scale factor can be multiplied by the coefficient or value.

The CSI calculator appears by clicking right side rectangular box of vertical scale factor. Putting the required coefficient in formula box peak ground acceleration can be scaled down or up.

User Defined graph (Above/below) once clicked show property grid appears. By Clicking on show property grid, plot functions appear. On right side of plot function, once clicked plot function data (points 1075 defined) appears. On right side of 1075 once clicked Time history data appears. Then this data can be copied to Excel to have graphs.

3.4.6 Time History is Matched to Response spectrum

Target Response spectrum is identified. Then time history is matched to Response spectrum There are two types of method to use for spectral matching. spectral matching in frequency domain and spectral domain in time domain. Here spectral matching in Time domain is done.

3.4.7 Defining Mass source

The weight of the structure used in the calculation of automatic seismic loads is based on the specified mass of the structure, and is termed mass source in ETABS 2016.

3.4.8 Defining Nonlinear Dynamic Load and Putting Value (0.05) for Direct Integration Damping

Nonlinear Dynamic load is defined where time history is a type and nonlinear direct integration is subtype. Load type is acceleration and function are matched time history. Direct integration damping is 0.05.

3.4.9 Hinge Overwrites

Strength loss is permitted in the hinge properties, and in fact the FEMA and ASCE hinges assume a sudden loss of strength. However, this feature should be used judiciously. Sudden strength loss is often unrealistic and can be very difficult to analyse, especially when elastic snap-back occurs. It is necessary to consider strength loss only when necessary, to use realistic negative slopes. To have the convergence, the program automatically limits the negative slope of a hinge to be no stiffer than 10% of the elastic stiff ness of the Frame element containing the hinge. If steeper slopes needed, we can assign a hinge over write that automatically meshes the frame element around the hinge. By reducing the size of the meshed element, we can increase the steepness of the drop-off.

A hinge-overwrite option is available through the Assign > Frame > Hinge Overwrites menu such that users may specify steeper strength degradation by using a small relative length on the order of 0.02.

3.4.10 Cockie Cut

At first quadrilaterals to be divided at intersections with visible grids. Slab section and slab to be selected from properties which is subset of select. Once slab is selected from assigning shell floor auto mesh option to be selected. From floor meshing options once selecting auto cookie cut object into structural elements of following items:

A. Mesh at Beams Other Meshing Lines (Applies to Horizontal Floors Only)

B. Mesh at Vertical/Inclined Wall Edges (Applies to Horizontal Floors Only)

C. Further Mesh Where Needed to Maximum Element Size of 24 inch

3.4.11 Effect of Hysteretic Behavior on the Nonlinear Response of Frames-Isotropic Hysteresis Model

The energy dissipation which occurs during time-history analysis may be modeled using hysteretic links. Links are useful for capturing dynamic loading and unloading because of their multi-axial response. Isotropic, kinematic, Takeda, and pivot hysteresis models are available for single DOF hinges. For isotropic hysteresis, hinges unload elastically, parallel to the initial stiffness tangent (A-B slope), while for other hysteresis types, unloading follows a more complex nonlinear relationship.

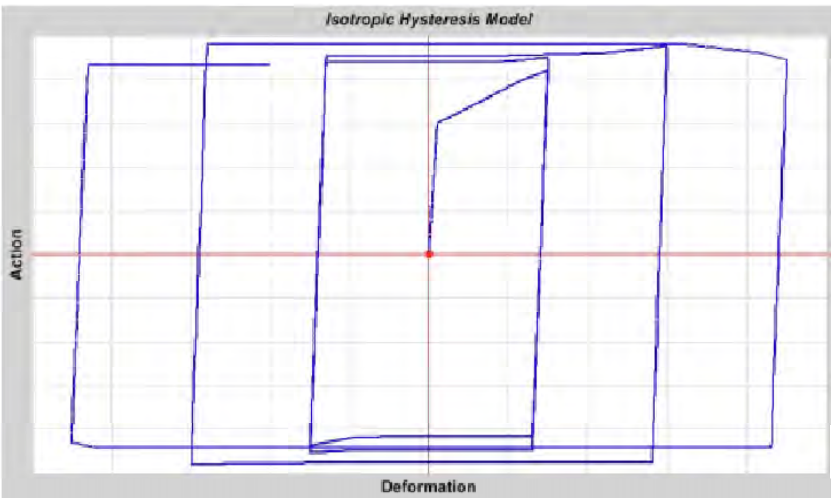


Figure 3.5. Isotropic Hysteresis Model under Increasing Cyclic Load(Medina and Krawinkler, 2004)

Isotropic hysteresis model's plastic deformation in one direction 'pushes' the curve for the other direction away from it, so that both directions increases in strength simultaneously. The backbone curve itself does not increase in strength, only the unloading and reverse loading behaviour. Matching pairs of points are linked. No additional parameters are required for this model. Unloading and reverse loading occur along a path parallel to the elastic line until the magnitude of the action in the reverse direction equals that of backbone curve at the same amount of deformation in the reverse direction, and then continues along a horizontal secant to the backbone curve. Symmetrical pairs of points to be linked even if the curve is not symmetrical. This allows for some control over the shape of the hysteretic loop. This model dissipates the most energy of all the models.

3.5 Analysing the frame

At first section properties taken from set view options after the hinge overwrite is done. Frame is analysed keeping Dead Load, Live load, equivalent static load in load case.

3.6 Yield Check through Pushover Analysis

From nonlinear time history analysis, it is found that Nonlinear displacement is 167.64 mm (6.60 inch). From push over analysis, it is found that for target displacement of 127 mm (5 inch), at 126.746 mm (4.99-inch) structure has formed hinge. That means structure has been formed into yield state at 126.746 mm (4.99 inch). So, the structure at nonlinear time history has transformed into nonlinear state at 167.64 mm (6.60 inch).

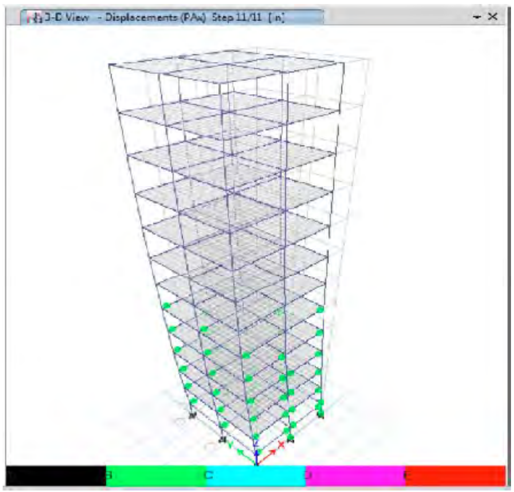


Figure 3.6. Yield Check through Pushover Analysis

CHAPTER 4

RESULT AND DISCUSSION

4.1 Introduction

This chapter highlights the analysis and results of 24 different finite element models prescribed in chapter 3. The result includes structural response from equivalent static analysis and nonlinear time history analysis. Results from equivalent static analysis for 24 frames are presented. Nonlinear behavior of 24 frames are discussed including computaion of deflection amplification factor and drift ratio. There are three different geometries/building configurations that includes 10m×10m, 15m×15m and 20m×20m floor area. Deflection Amplification Factor (C_d) and Drift Ratios are determined for all three geometric configuration of finite element.

4.2 Finite Element Analysis Result for 10m×10m Frame Structures

4.2.1 C_d and Drift Ratio of Two Bay Two Story Frame (2-2)

The selected beam of 235.2mm×416.6mm (9.26in×16.4in) and column of 235.2mm×235.2mm (9.26in×9.26in). Rebar percentage is 1%, 1.29% and 2.39% at bottom, first floor and second floor respectively. Demand Capacity Ratio is <1.

A. Deflection Amplication Factor (C_d)

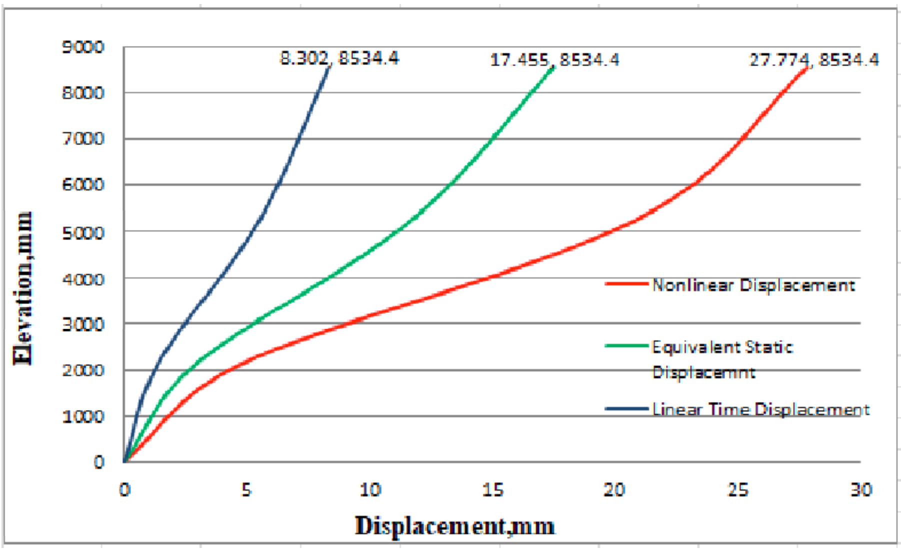


Figure 4.1. Maximum Story Displacement of 2 Bay 2 Story Frame

C_d = 1.59 and 3.35 respectively with respect to equivalent static displacement and linear time displacement.

B. Story Drift

As per BNBC, drift limit is 0.02. Here static linear, linear and nonlinear drifts are less than specified limit.

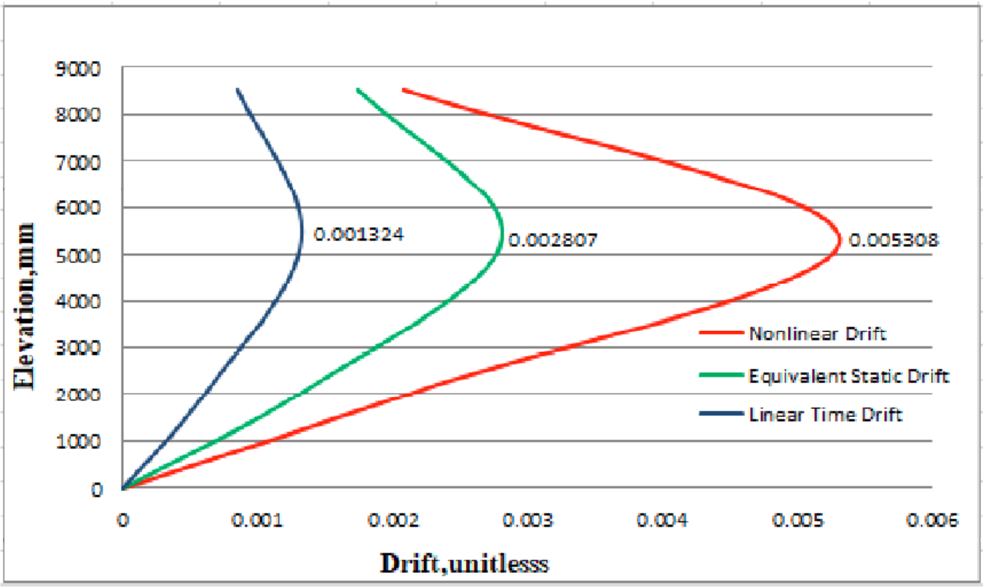


Figure 4.2. Maximum Story Drift of 2 Bay 2 Story Frame

C. Relationship between Displacement and Load

Two bay two story has maximum roof displacement of 27.8 mm where under load condition culumn has maximum displacement of 27.8 mm.

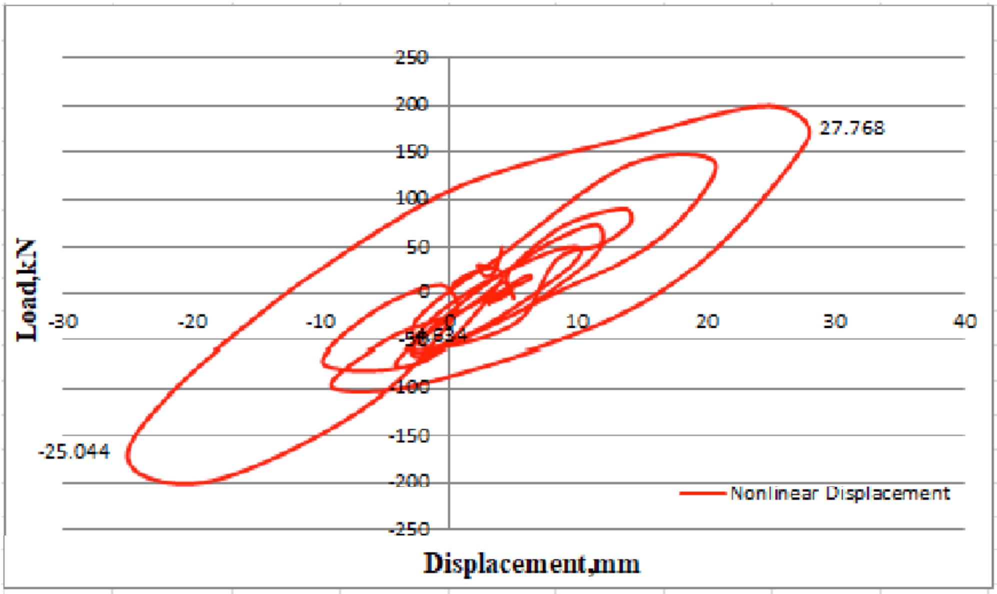


Figure 4.3. Relationship between Displacement and Load of 2 Bay 2 Story Frame

D. Hinge Result

The hinge result (Figure 4.4) shows that it is in plastic zone. Plastic rotation in negative direction.

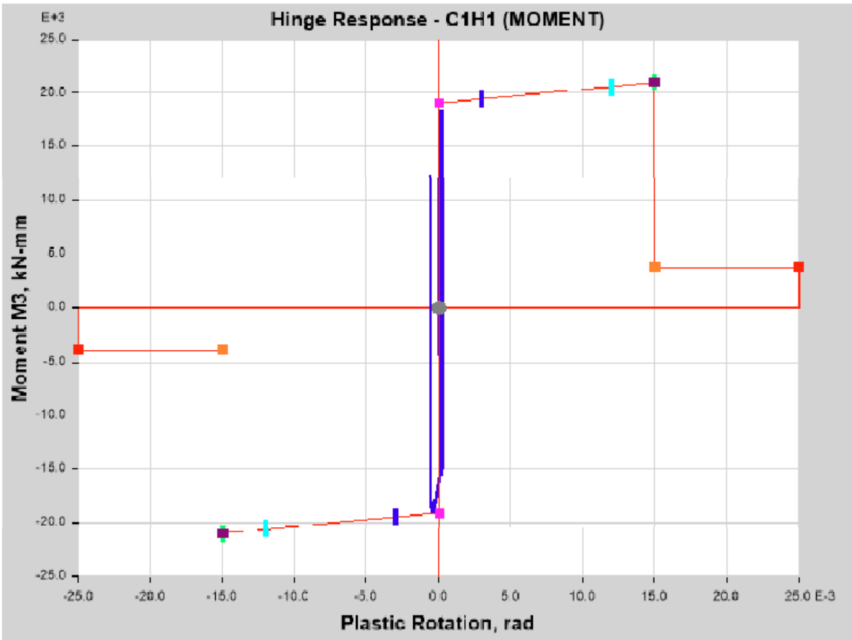


Figure 4.4. Hinge Result of 2 Bay 2 Story Frame

4.2.2 C_d and Drift Ratio of Two Bay Three Story Frame (2-3)

The selected beam of 262mm×416.6mm (10.32in×16.4in) and column of 262mm×262mm (10.32in×10.32in).

A. Deflection Amplication Factor (C_d)

C_d = 39.15/25.610028 = 1.53; 39.15/10.24 =3.82; with respect to equivalent static displacement and linear time displacement. As per BNBC, the nonlinear time history to be done for more than 40 m in zone 2. 3 story is of 11.73 meter only. Therefore, C_d value is much lower than specified value (4.5).

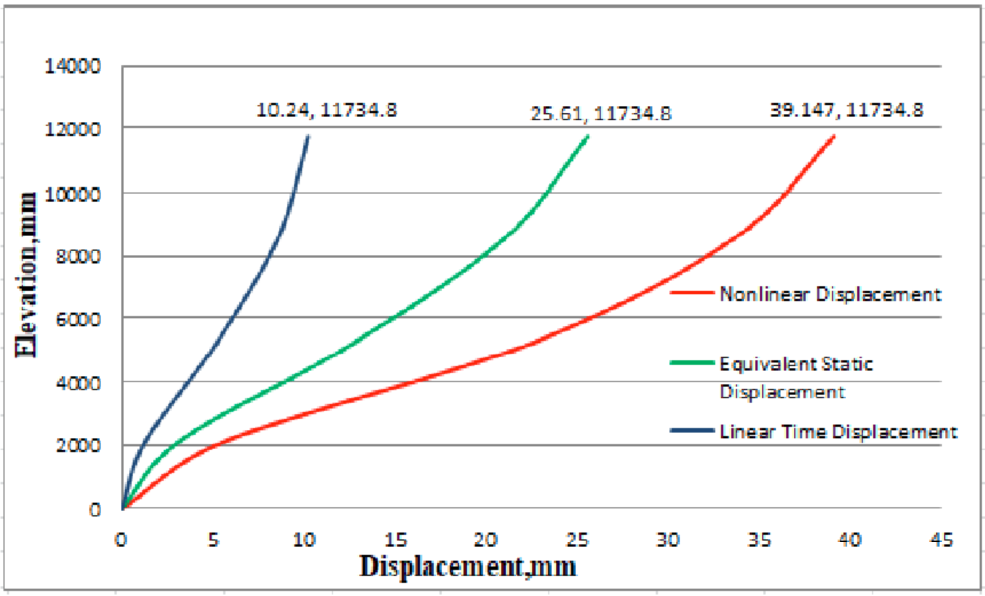


Figure 4.5. Maximum Story Displacement of 2 Bay 3 Story Frame

B. Story Drift

As per BNBC, drift limit is 0.02. Here static linear, linear and nonlinear drifts are less than specified limit.

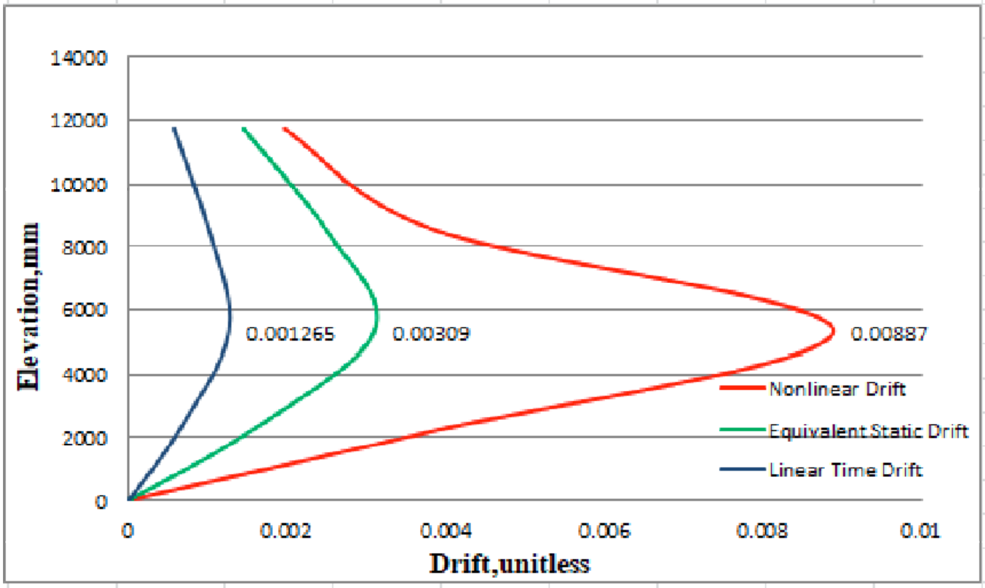


Figure 4.6. Maximum Story Drift of 2 Bay 3 Story Frame

C. Relationship between Displacement and Load

Two bay three story has maximum roof displacement of 39.15 mm where under load condition culumn has maximum displacement of 39.14 mm.

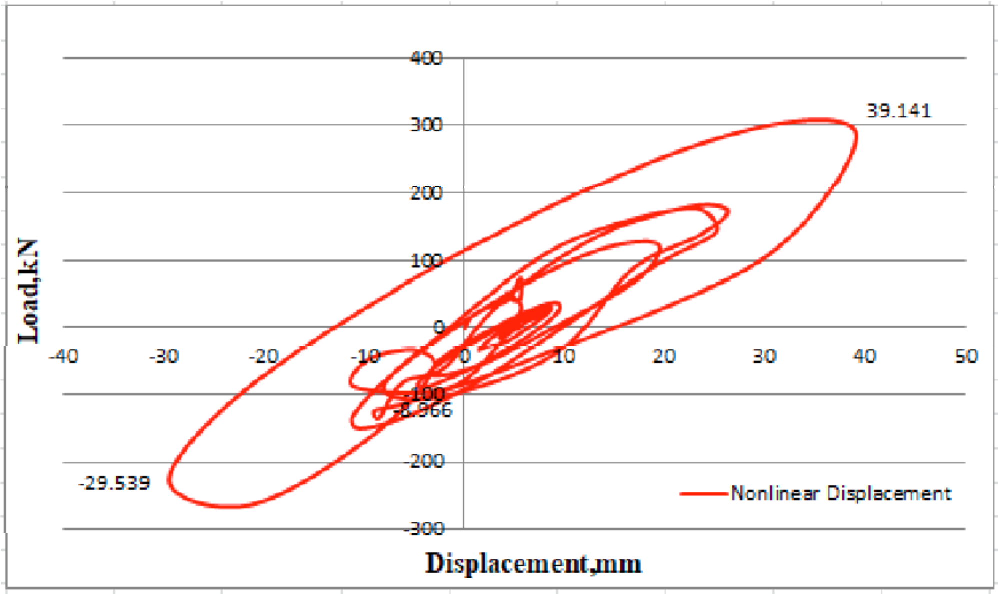


Figure 4.7. Relationship between Displacement and Load of 2 Bay 3 Story Frame

D. Hinge Result

The hinge result (Figure 4.8) shows that it is in plastic zone. Plastic rotation has negative value.

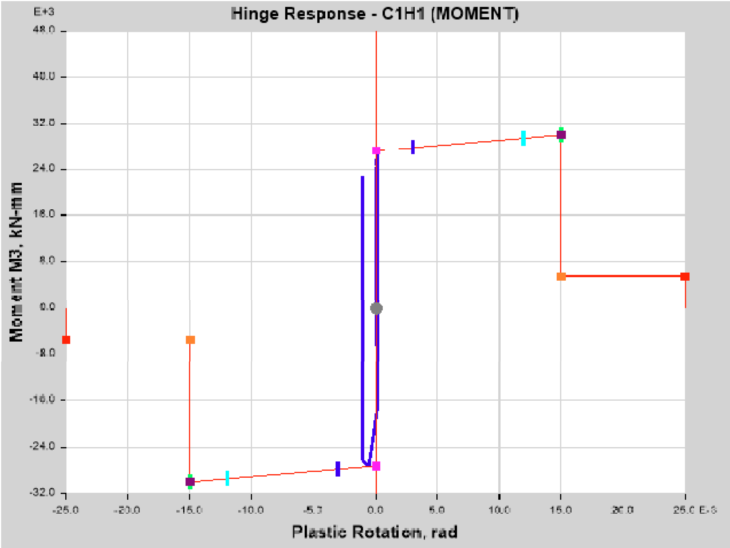


Figure 4.8. Hinge Result of 2 Bay 3 Story Frame

4.2.3 C_d and Drift Ratio of Two Bay Four Story Frame (2-4)

The selected beam of 284.5mm×284.5mm (11.2in×16.4in) and column of 284.5mm×284.5mm (11.2in×11.2in).

A. Deflection Amplication Factor (C_d)

C_d = 1.38 and 3.95; with respect to equivalent static displacement and linear time displacement. As per BNBC, the nonlinear time history to be done for more than 40 m in zone 2. 4 story is of 14.93 meter only.

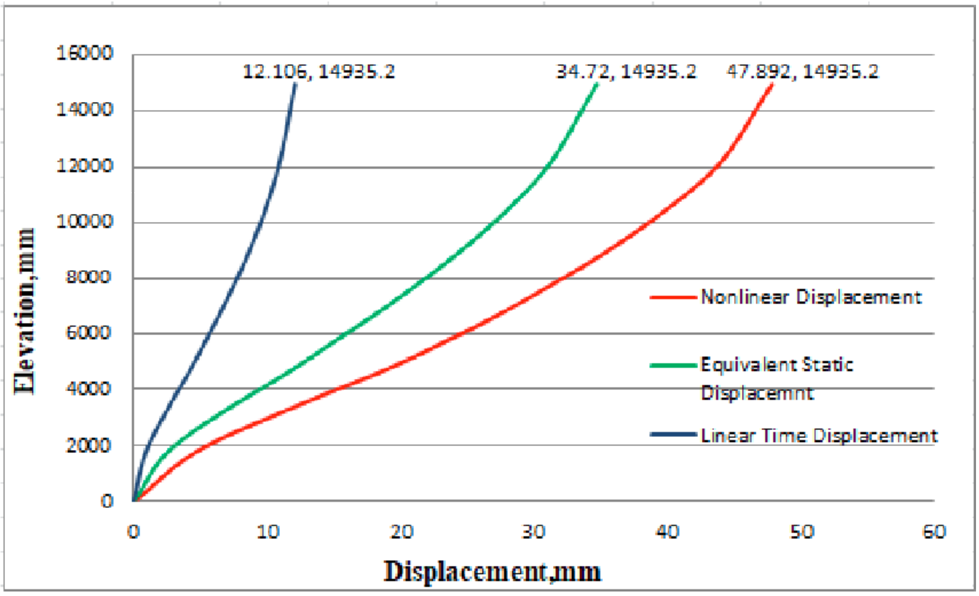


Figure 4.9. Maximum Story Displacement of 2 Bay 4 Story Frame

B. Story Drift

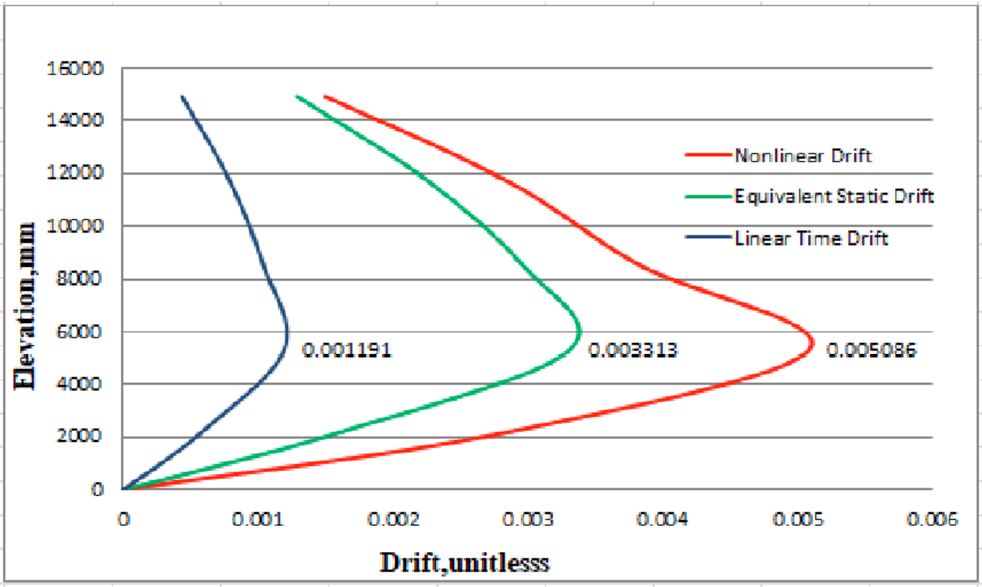


Figure 4.10. Maximum Story Drift of 2 Bay 4 Story Frame

As per BNBC, drift limit is 0.020. Here static linear, linear and nonlinear drifts are less than specified limit.

C. Relationship between Displacement and Load

Two bay three four story has maximum roof displacement of 47.9 mm where under load condition culumn has maximum displacement of around 47.9 mm.

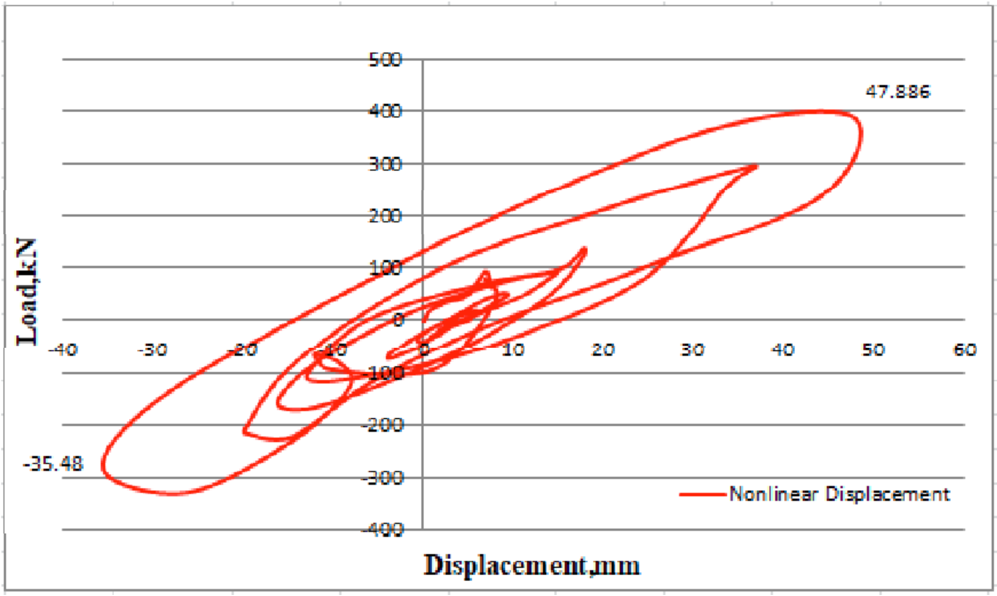


Figure 4.11. Relationship between Displacement and Load of 2 Bay 4 Story Frame

D. Hinge Result

The hinge result (Figure 4.12) shows that it is in plastic zone. Plastic rotation has negative value.

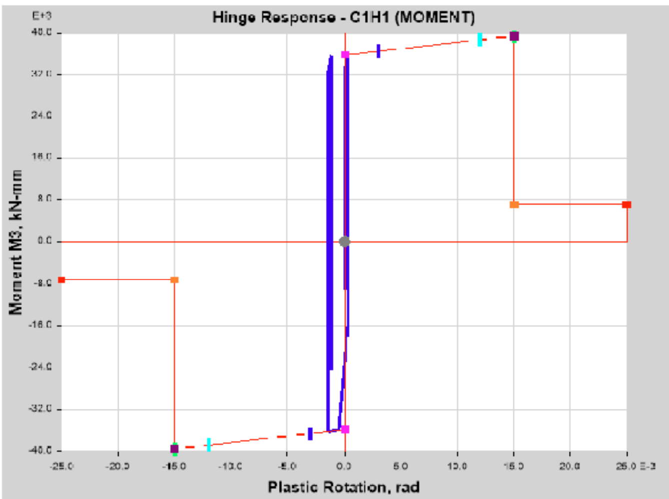


Figure 4.12. Hinge Result of 2 Bay 4 Story Frame

4.2.4 C_d and Drift Ratio of Two Bay Five Story Frame (2-5)

The selected beam of 286.8mm×431.8mm (11.29in×17in) and column of 286.8mm×286.8mm (11.29in×11.29in).

A. Deflection Amplication Factor (C_d)

C_d = 1.71 and 3.66; with respect to equivalent static displacement and linear time displacement.

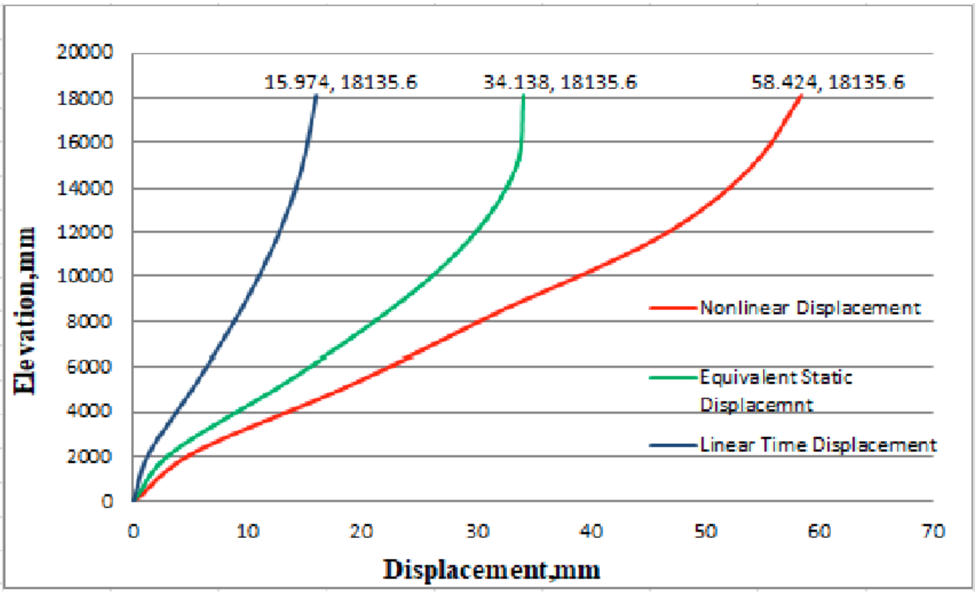


Figure 4.13. Maximum Story Displacement of 2 Bay 5 Story Frame

B. Story Drift

As per BNBC, drift limit is 0.020. Here static linear, linear and nonlinear drifts are less than specified limit.

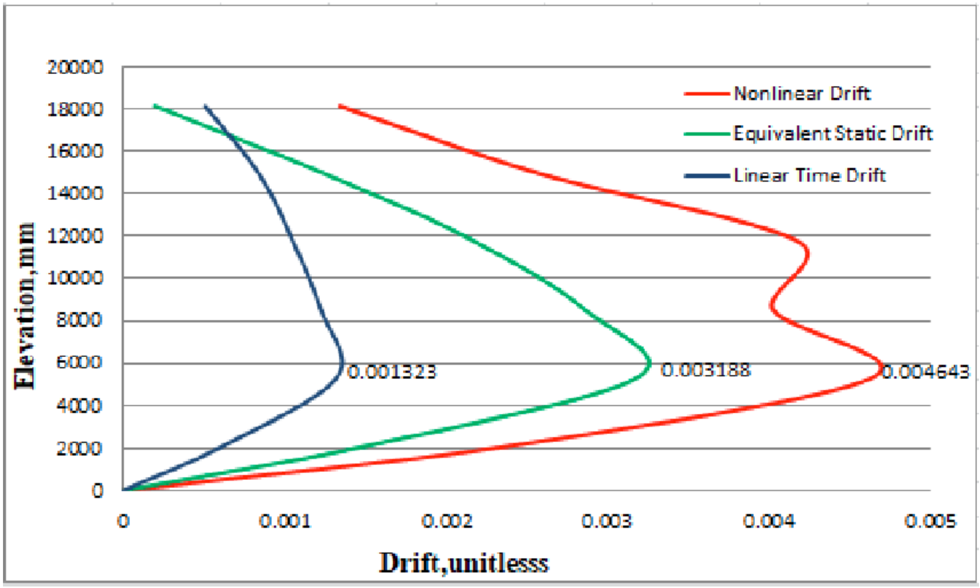


Figure 4.14. Maximum Story Drift of 2 Bay 5 Story Frame

C. Relationship between Displacement and Load

Two bay five story has maximum roof displacement of 58.4 mm where under load condition culumn has maximum displacement of 58.4 mm.

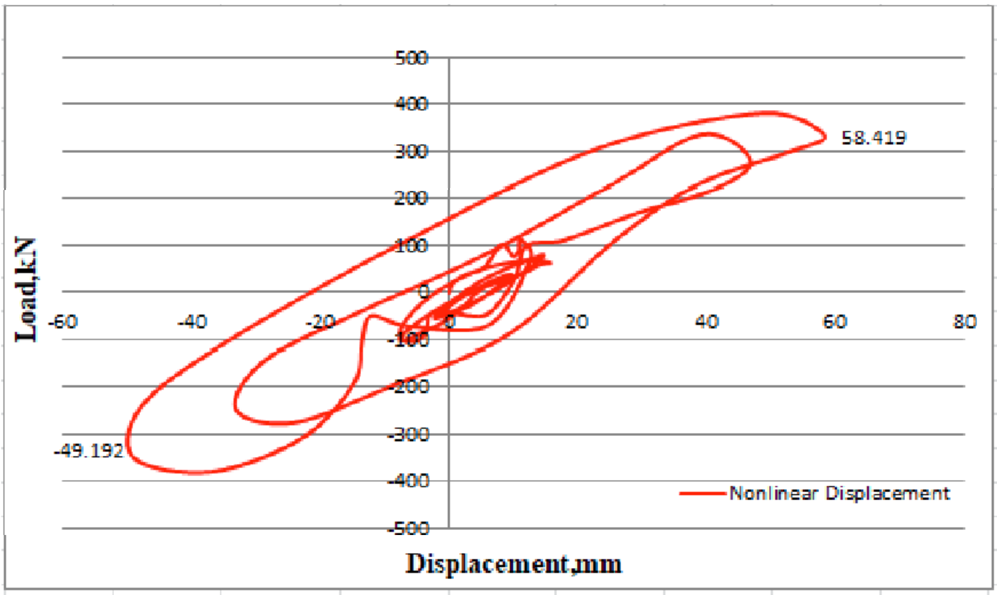


Figure 4.15. Relationship between Displacement and Load of 2 Bay 5 Story Frame

D. Hinge Result

The hinge result (Figure 4.16) shows that it is in plastic zone. It has positive Plastic rotation.

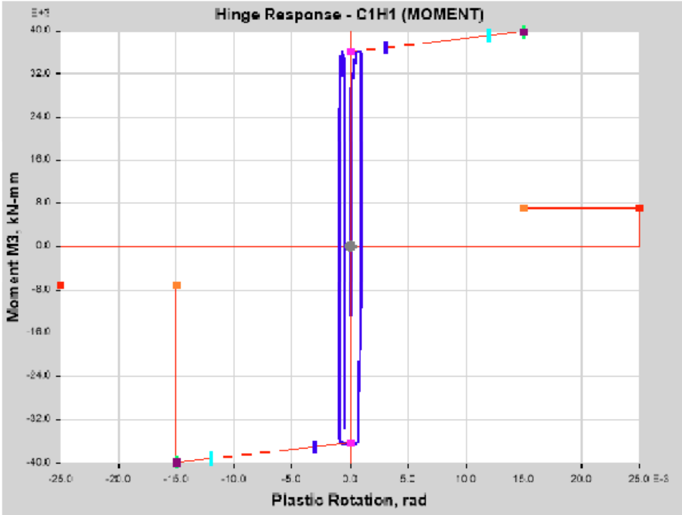


Figure 4.16. Hinge Result of 2 Bay 5 Story Frame

4.2.5 C_d and Drift Ratio of Two Bay Six Story Frame (2-6)

Selected Beam of 338.3mm×622.3mm (13.32in×24.5in) and column of 339.8mm×339.8 (13.38in×13.38in).

A. Deflection Amplification Factor (C_d)

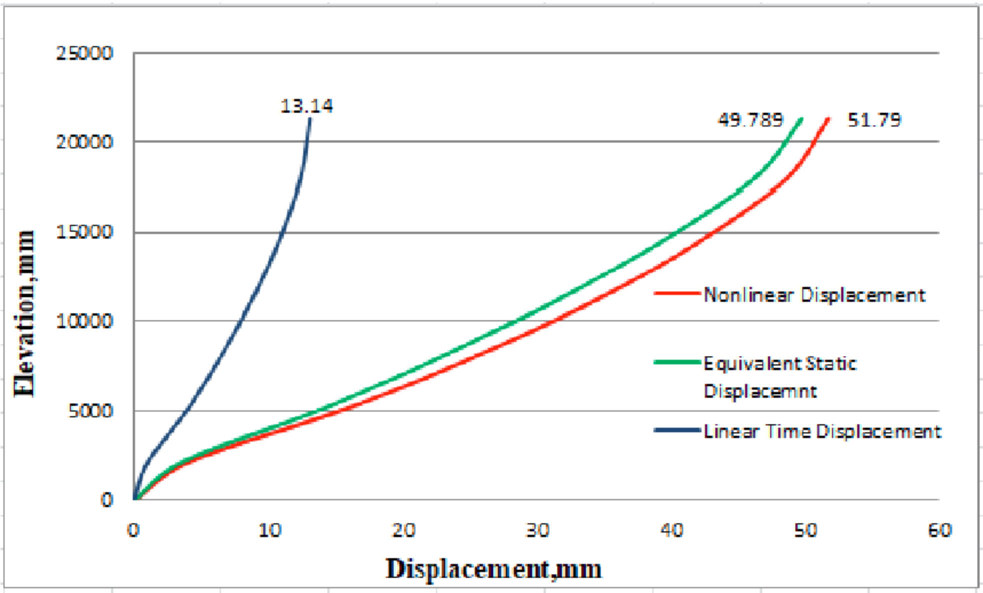


Figure 4.17. Maximum Story Displacement of 2 Bay 6 Story Frame

C_d = 1.04 and 3.94; with respect to equivalent static displacement and linear time displacement.

B. Story Drift

As per BNBC, drift limit is 0.020. Here static linear, linear and nonlinear drifts are less than specified limit.

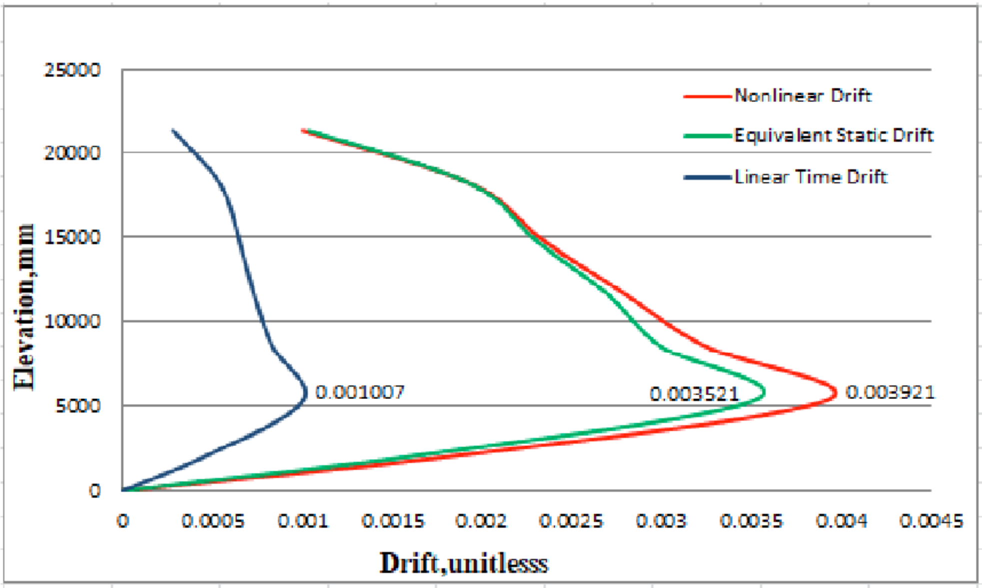


Figure 4.18. Maximum Story Drift of 2 Bay 6 Story Frame

C. Relationship between Displacement and Load

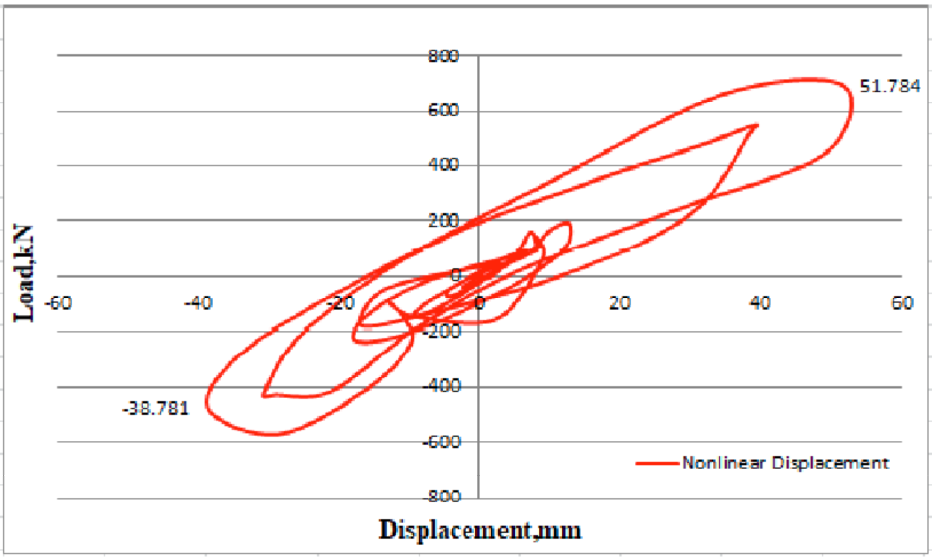


Figure 4.19. Relationship between Displacement and Load of 2 Bay 6 Story Frame

Two bay six story has maximum roof displacement of 51.8 mm where under load condition culumn has maximum displacement of 51.8 mm.

D. Hinge Result

The hinge result (Figure 4.20) shows that it is in plastic zone. Plastic rotation has occurred.

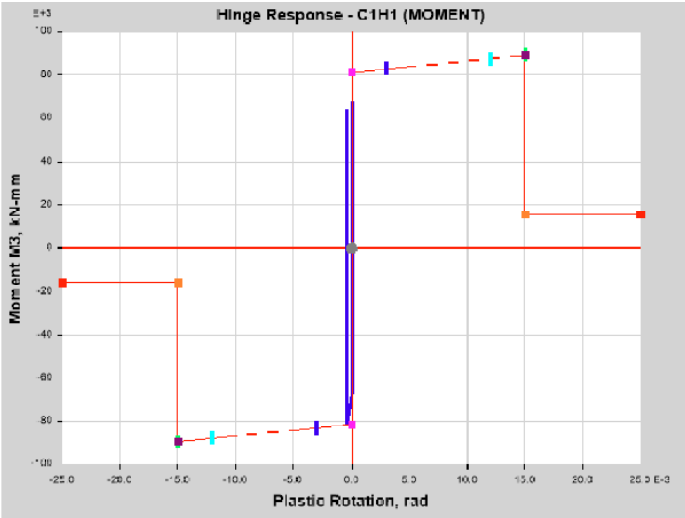


Figure 4.20. Hinge Result of 2 Bay 6 Story Frame

4.2.6 C_d and Drift Ratio of Two Bay Eight Story Frame (2-8)

Selected Beam of 315.7mm×622.3mm (12.43in×24.5in) and Column of 380.7mm×380.7mm (14.99in×14.99in)

A. Deflection Amplification Factor (C_d)

C_d = 1.19 and 4.23; with respect to equivalent static displacement and linear time displacement.

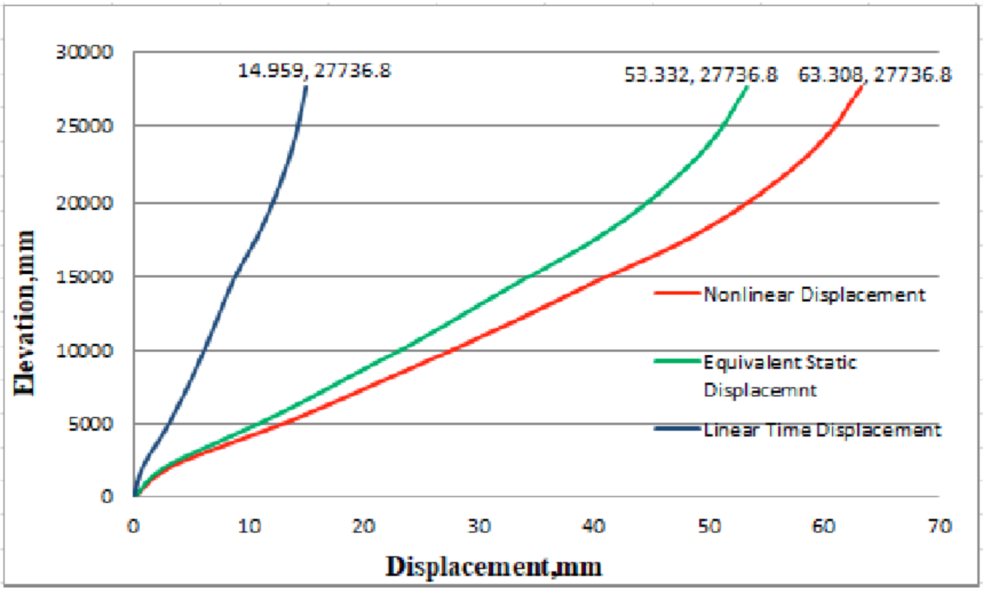


Figure 4.21. Maximum Story Displacement of 2 Bay 8 Story Frame

B. Story Drift

As per BNBC, drift limit is 0.020. Here static linear, linear and nonlinear drifts are less than specified limit.

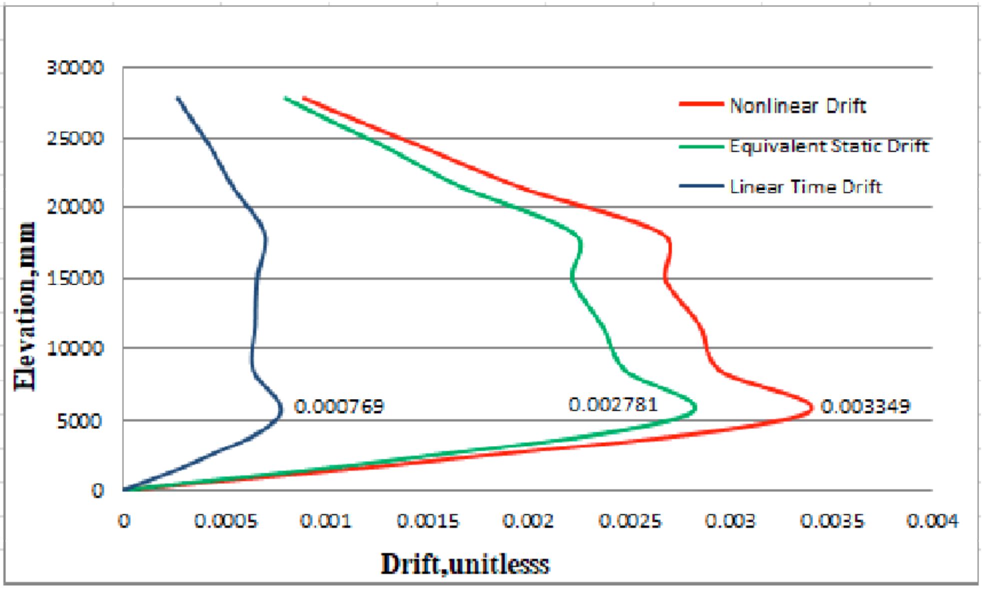


Figure 4.22. Maximum Story Drift of 2 Bay 8 Story Frame

C. Relationship between Displacement and Load

Two bay three story has maximum roof displacement of 63.31 mm where under load condition culumn has maximum displacement of around 61 mm. The values are nearer since under load condition the value is of column.

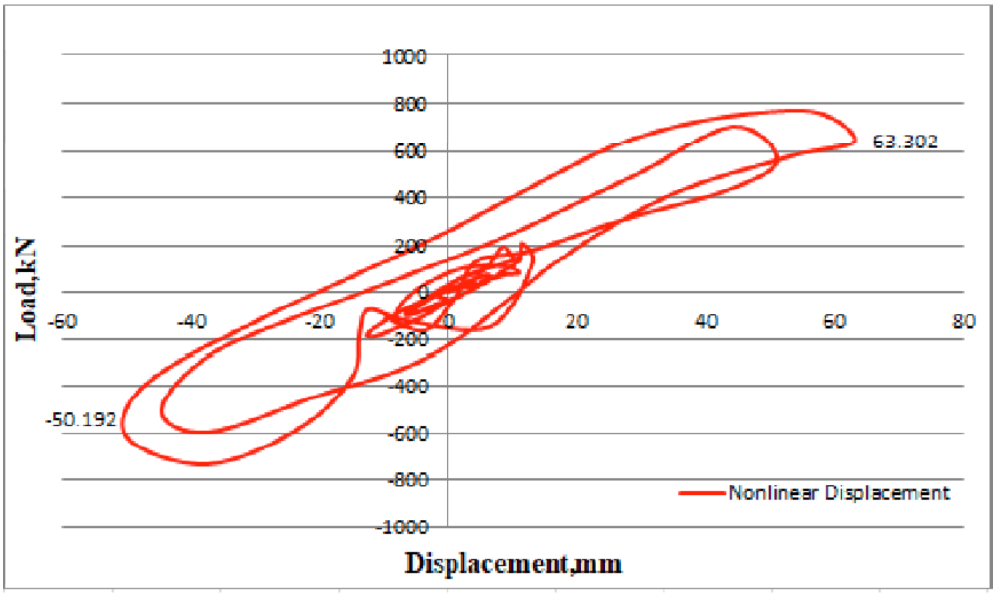


Figure 4.23. Relationship between Displacement and Load of 2 Bay 8 Story Frame

D. Hinge Result

The hinge result (Figure 4.24) shows that it is in plastic zone. Plastic rotation has occurred.

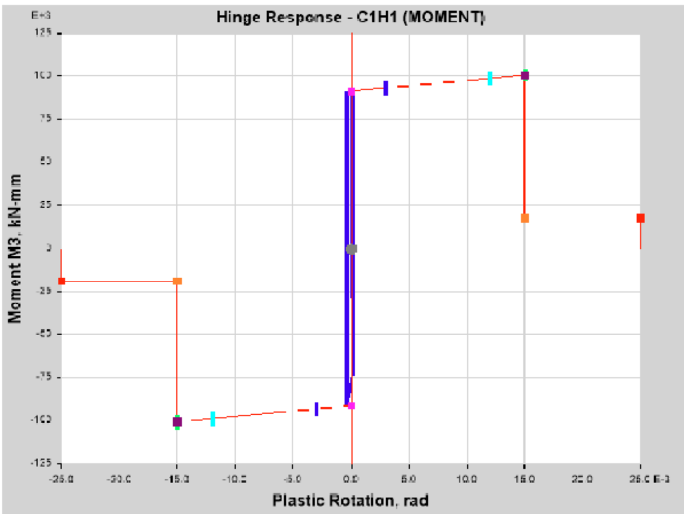


Figure 4.24. Hinge Result of of 2 Bay 8 Story Frame

4.2.7 C_d and Drift Ratio of Two Bay Ten Story Frame (2-10)

The selected beam 315.7mm×631.2mm (12.43in×24.85in) and column of 380.7mm×380.7mm (14.99in×14.99in).

A. Deflection Amplification Factor (C_d)

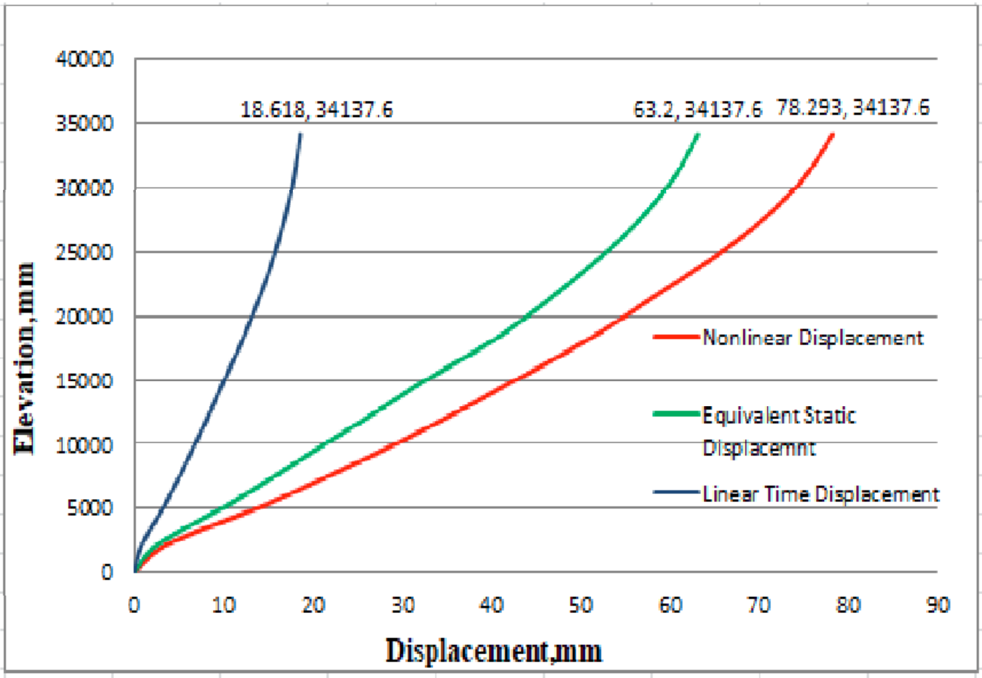


Figure 4.25. Maximum Story Displacement of 2 Bay 10 Story Frame

C_d = 1.24 and 4.21; with respect to equivalent static displacement and linear time displacement.

B. Story Drift

As per BNBC, drift limit is 0.020. Here static linear, linear and nonlinear drifts are less than specified limit.

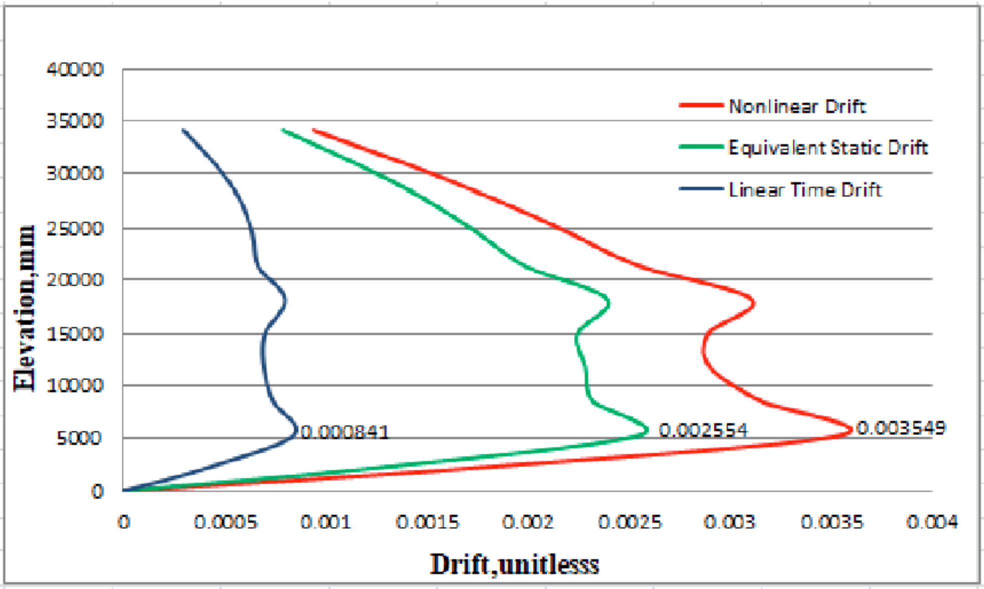


Figure 4.26. Maximum Story Drift of 2 Bay 10 Story Frame

C. Relationship between Displacement and Load

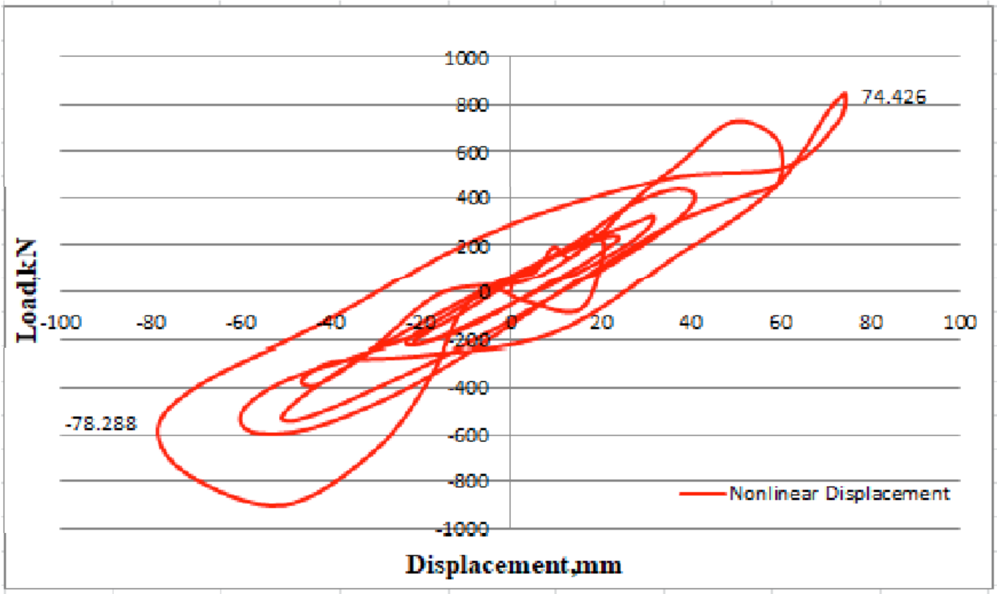


Figure 4.27. Relationship between Displacement and Load of 2 Bay 10 Story Frame

Two bay ten story has maximum roof displacement of 78.3 mm where under load condition culumn has maximum displacement of around 78.3 mm.

D. Hinge Result

The hinge result (Figure 4.28) shows that it is not in plastic zone. Plastic rotation has occurred.

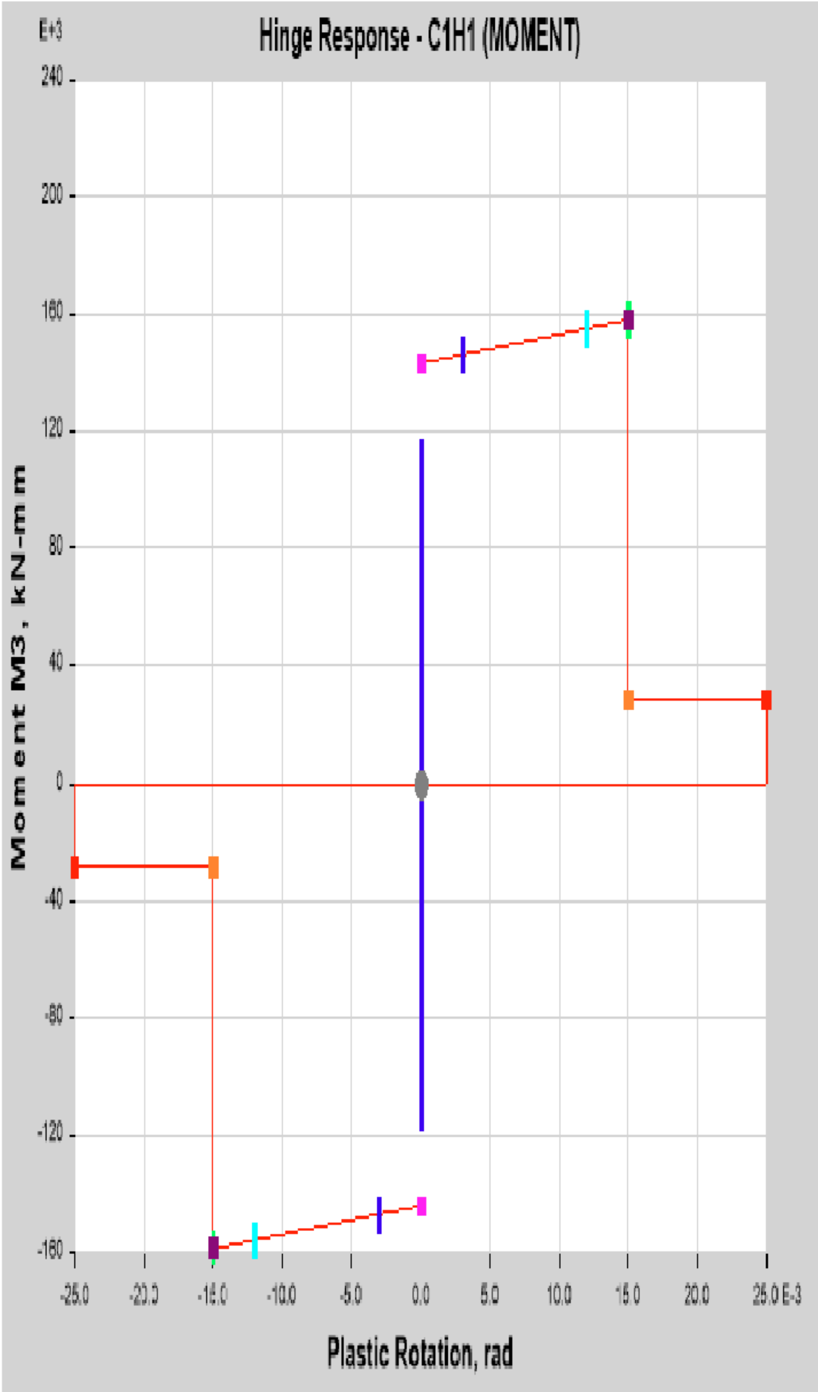


Figure 4.28. Hinge Result of 2 Bay 10 Story Frame

4.2.8 C_d and Drift Ratio of Two Bay Twelve Story Frame (2-12)

Selected Beam of 304.8mm× 439.7mm (12in×17.31in) and column 418.8mm× 418.8mm (16.49in×16.49in).

A. Deflection amplification factor (C_d)

C_d = 1.64 and 4.57; with respect to equivalent static displacement and linear time displacement.

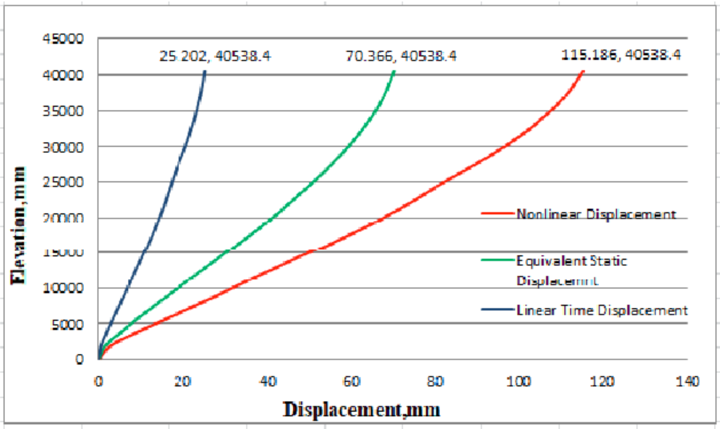


Figure 4.29. Maximum Story Displacement of 2 Bay 12 Story Frame

B. Story Drift

As per BNBC, drift limit is 0.020. Here static linear, linear and nonlinear drifts are less than specified limit.

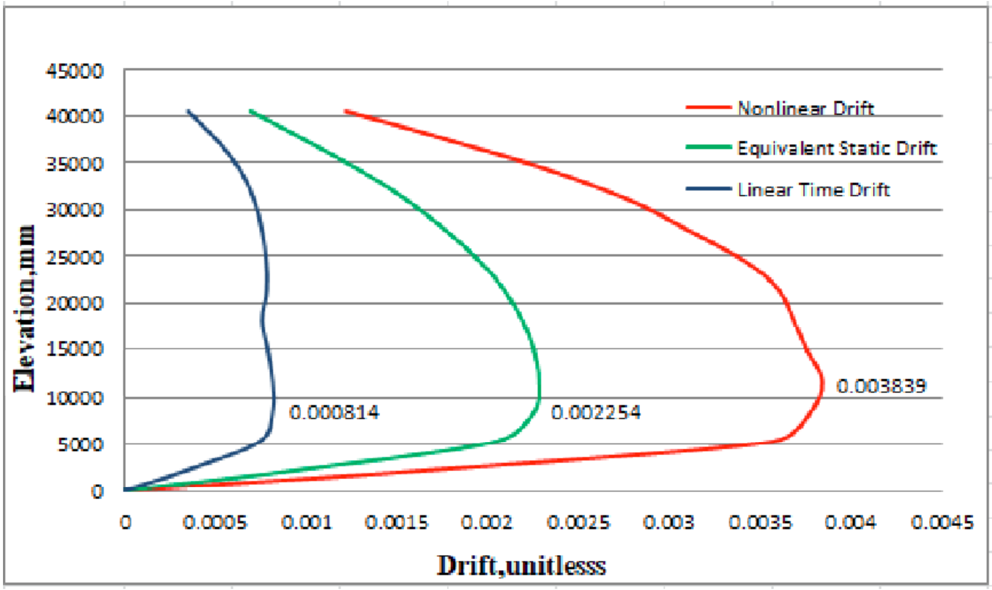


Figure 4.30. Maximum Story Drift of 2 Bay 12 Story Frame

C. Relationship between Displacement and Load

Two bay twelve story has maximum roof displacement of 115.186 mm where under load condition culumn has maximum displacement of 115.181 mm. The values are nearer since under load condition the value is of column.

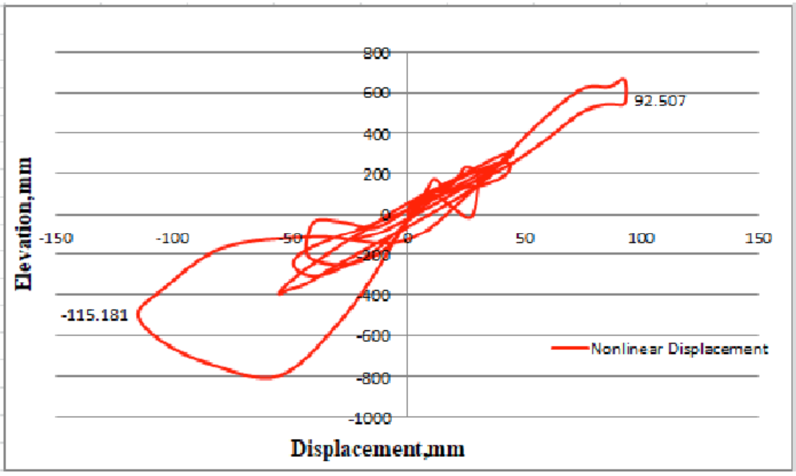


Figure 4.31. Relationship between Displacement and Load of 2 Bay 12 Story Frame

D. Hinge Result

The figure 4.31 and figure 4.32 shows that though it has nonlinearity but plastic rotation has not occurred.

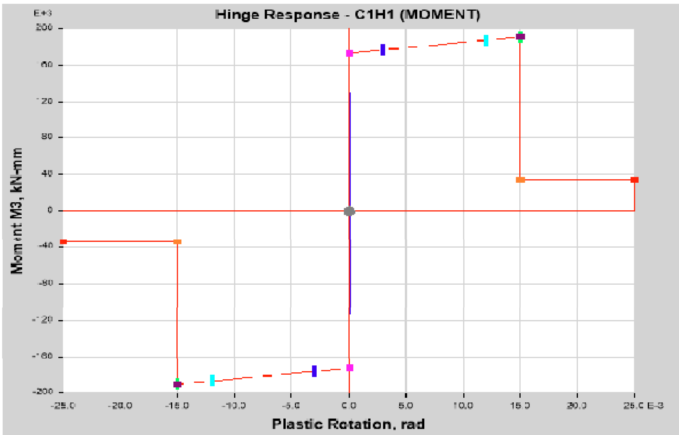


Figure 4.32. Hinge Result of 2 Bay 12 Story Frame

4.3 Finite Element Analysis Result for 15m×15m Frame Structures

4.3.1 C_d and Drift Ratio of Three Bay Two Story Frame (3-2)

The three bay two story frame is having Beam of 251.4mm×485.6mm (9.9in×19.12in) and Column of 251.4mm×251.4mm (9.9in×9.9in).

A. Deflection Amplification Factor (C_d)

C_d = 1.38 and 2.92; with respect to equivalent static displacement and linear time displacement.

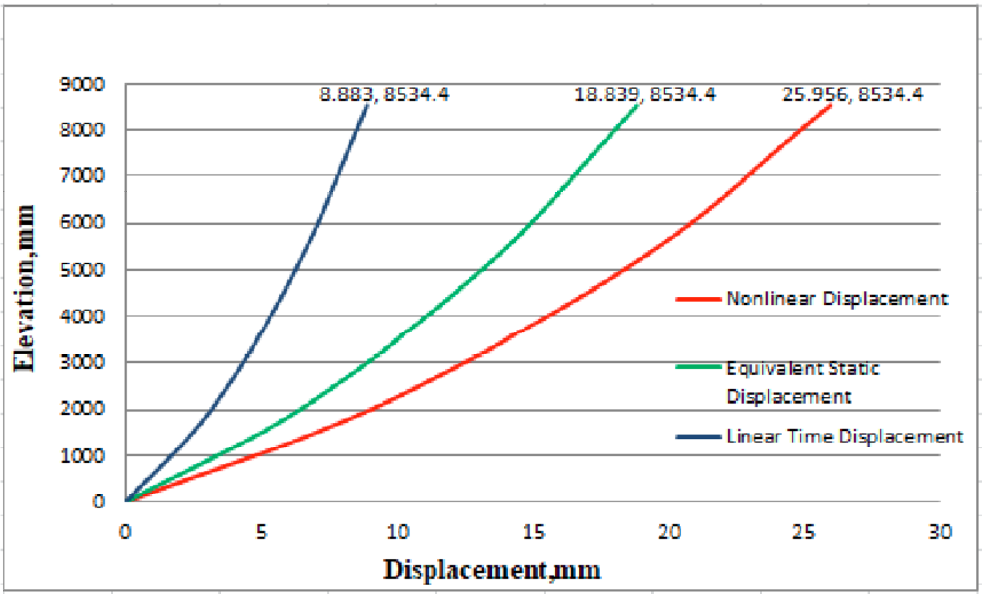


Figure 4.33. Maximum Story Displacement of 3 Bay 2 Story Frame

B. Story Drift

As per BNBC, drift limit is 0.020. Here static linear, linear and nonlinear drifts are less than specified limit.

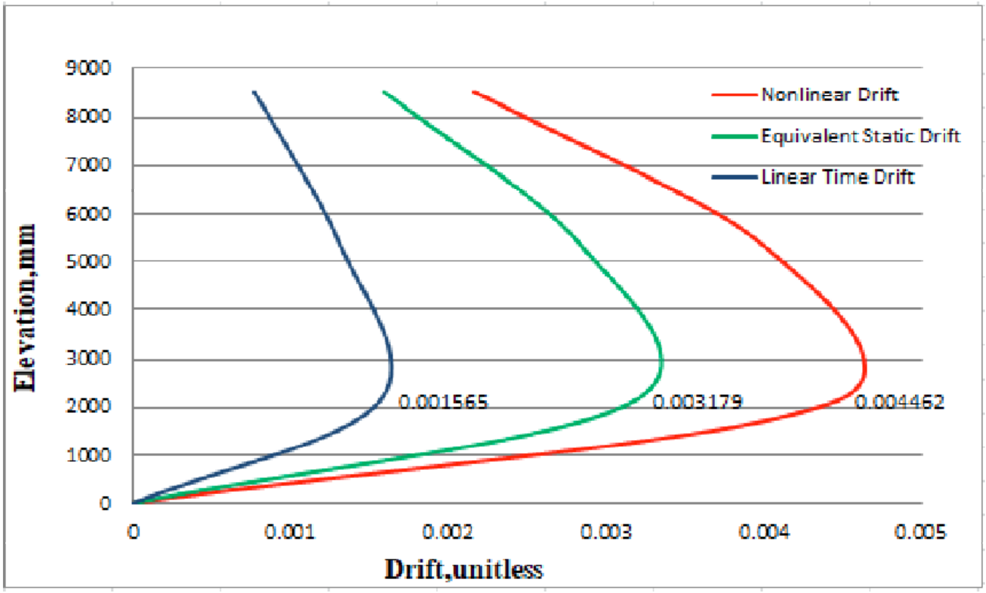


Figure 4.34. Maximum Story Drift of 3 Bay 2 Story Frame

C. Relationship between Displacement and Load

Three Bay Two story has maximum roof displacement of 25.96 mm where under load condition culumn has maximum displacement of 25.04 mm. The values are nearer since under load condition the value is of column.

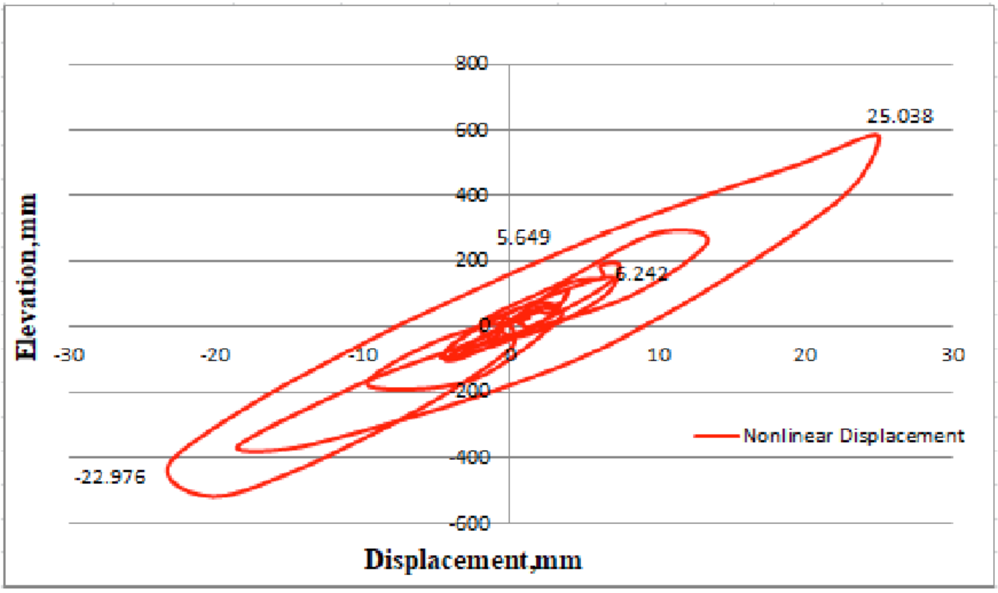


Figure 4.35. Relationship between Displacement and Load of 3 Bay 2 Story Frame

D. Hinge Result

The hinge result (Figure 4.36) shows that it is in elastic limit. Plastic rotation has not occurred

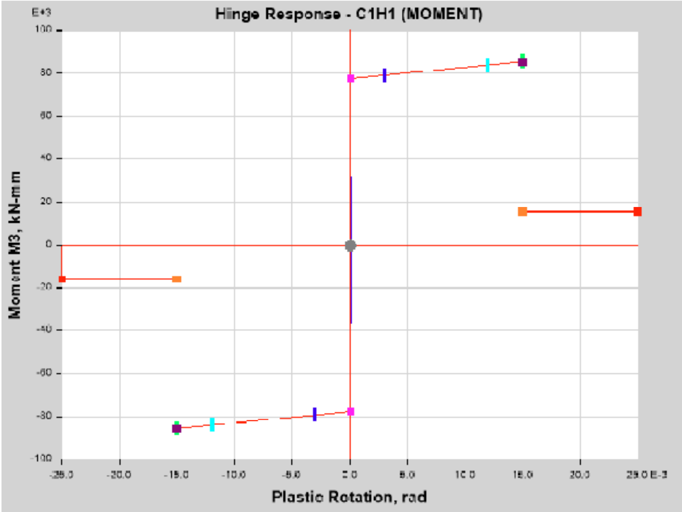


Figure 4.36. Hinge Result of 3 Bay 2 Story Frame

4.3.2 C_d and Drift Ratio of Three Bay Three Story Frame (3-3)

The three bay three story frame is having Beam of 254mm×524mm (10in×20.64in) and Column of 294.4mm×294.4mm (11.59in×11.59in).

A. Deflection Amplification Factor (C_d)

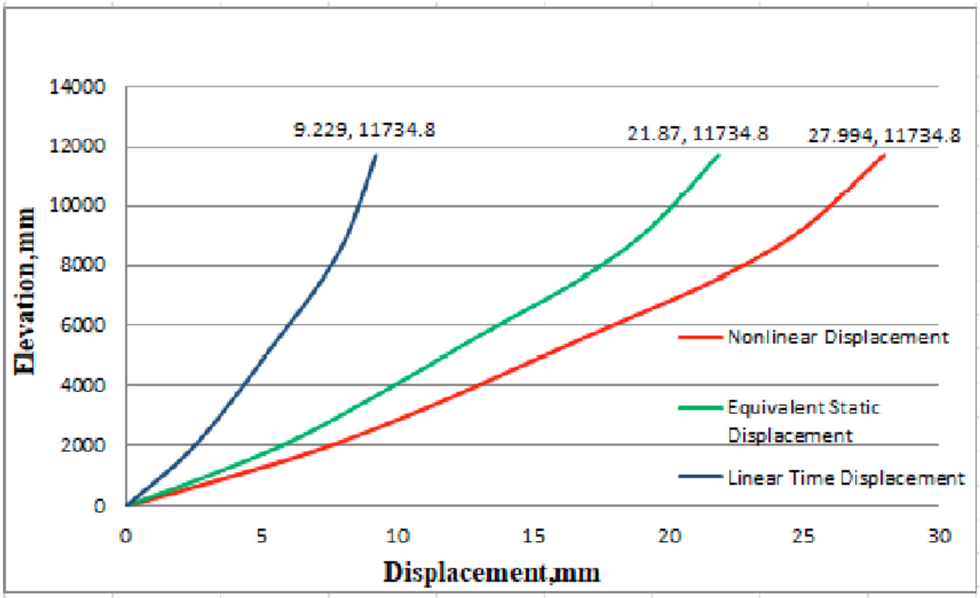


Figure 4.37. Maximum Story Displacement of 3 Bay 3 Story Frame

C_d = 1.28 and 3.03; with respect to equivalent static displacement and linear time displacement.

B. Story Drift

As per BNBC, drift limit is 0.020. Here static linear, linear and nonlinear drifts are less than specified limit.

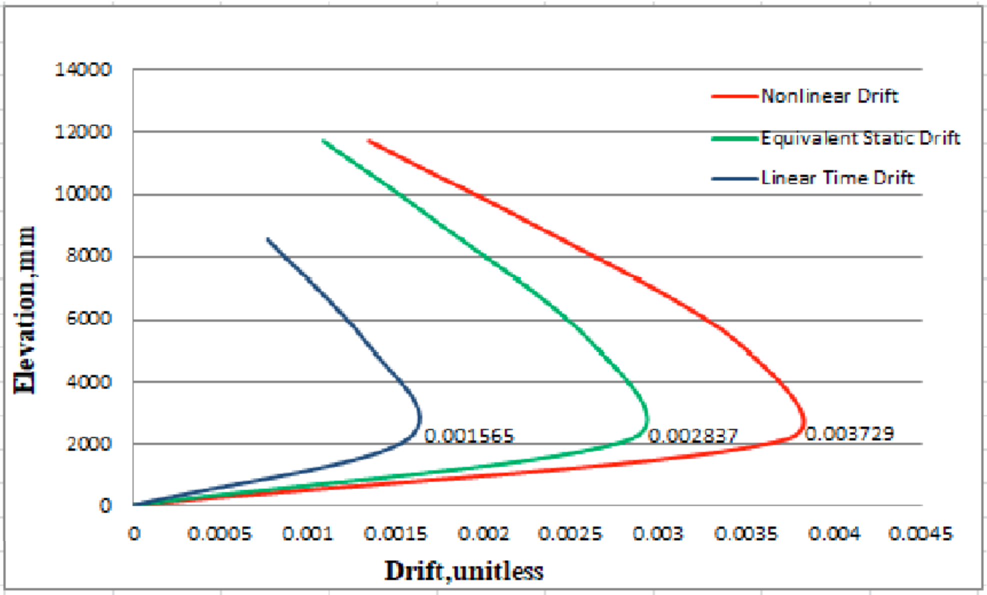


Figure 4.38. Maximum Story Drift of 3 Bay 3 Story Frame

C. Relationship between Displacement and Load

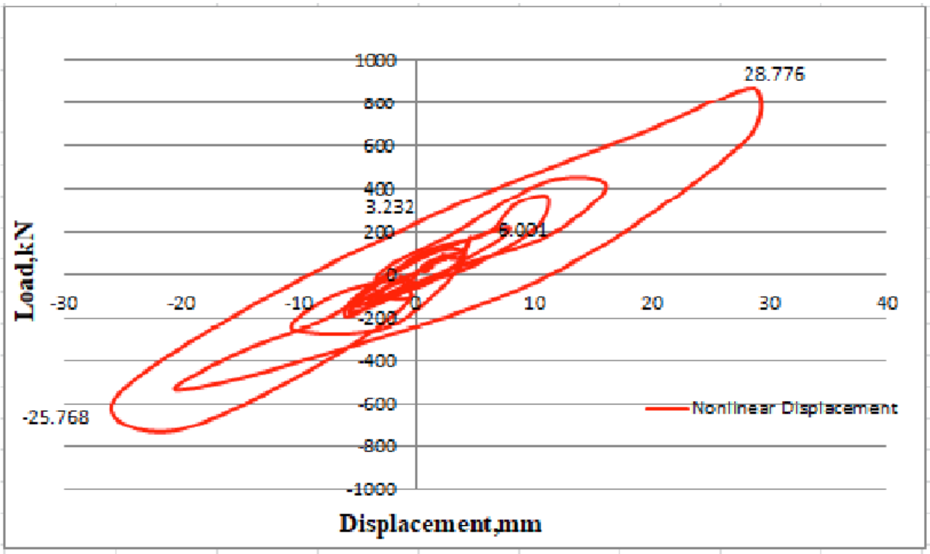


Figure 4.39. Relationship between Displacement and Load of 3 Bay 3 Story Frame

Three Bay three story has maximum roof displacement of 27.9 mm where under load condition culumn has maximum displacement of 28.8 mm. The values are nearer since under load condition the value is of column.

D. Hinge Result

The hinge result (Figure 4.40) shows that it is in elastic limit. Plastic rotation has not occurred.

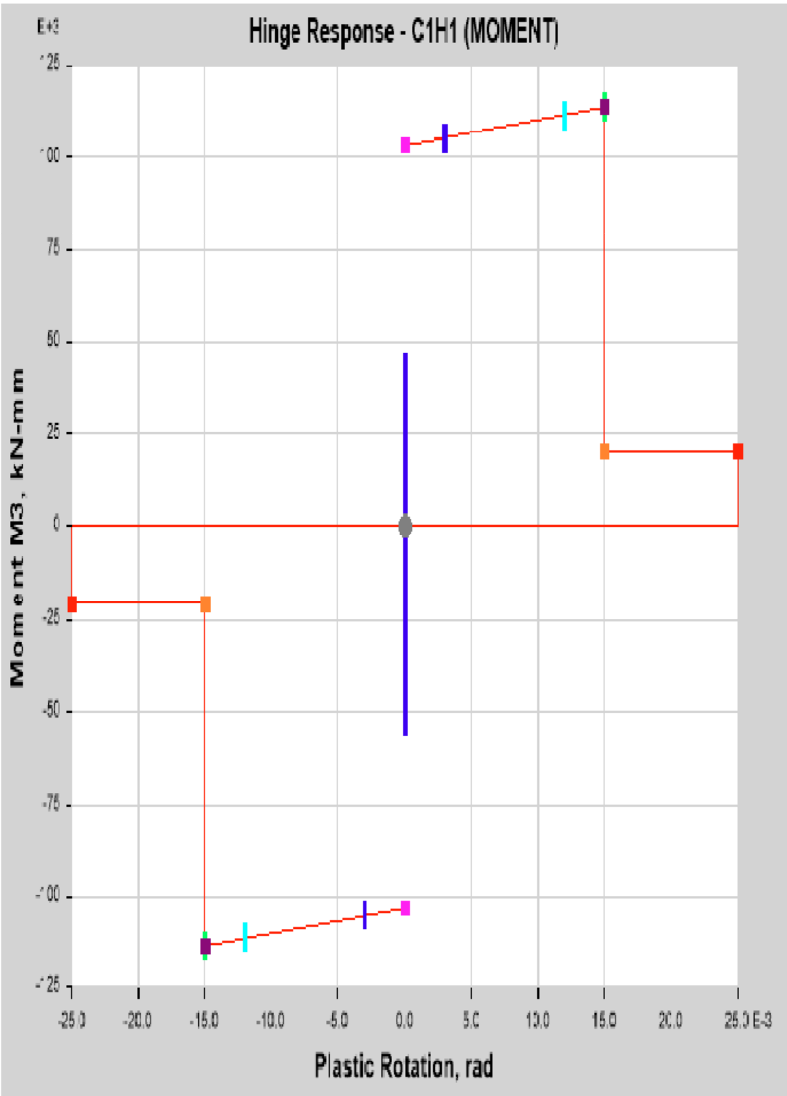


Figure 4.40. Hinge Result of 3 Bay 3 Story Frame

4.3.3 C_d and Drift Ratio of Three Bay Four Story Frame (3-4)

The selected Beam of 304.8mm×460.8mm (12in×18.14in) and Column of 342.4mm×342.4mm (13.48in×13.48in).

A. Deflection Amplification Factor (C_d)

C_d = 1.16 and 3.42; with respect to equivalent static displacement and linear time displacement.

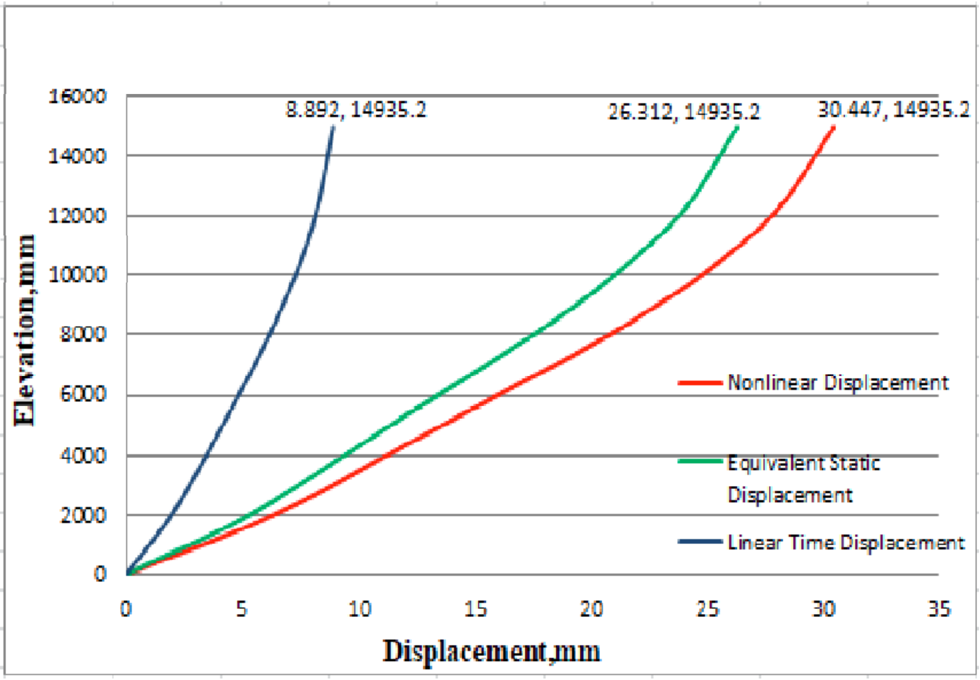


Figure 4.41. Maximum Story Displacement of 3 Bay 4 Story Frame

B. Story Drift

As per BNBC, drift limit is 0.020. Here static linear, linear and nonlinear drifts are less than specified limit.

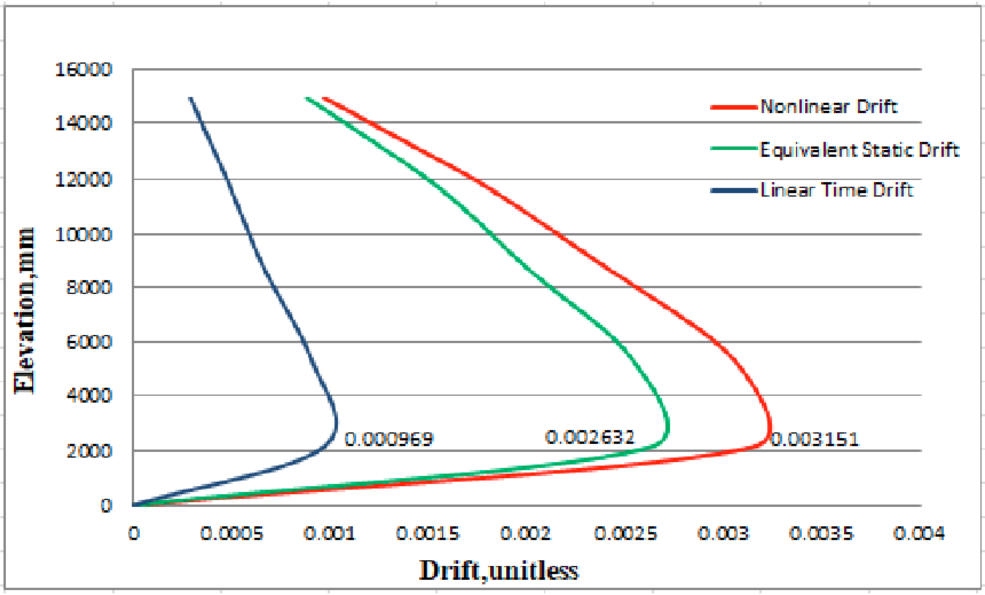


Figure 4.42. Maximum Story Drift of 3 Bay 4 Story Frame

C. Relationship between Displacement and Load

Three Bay four story has maximum roof displacement of 30.45 mm where under load condition culumn has maximum displacement of 32 mm. The values are nearer since under load condition the value is of column.

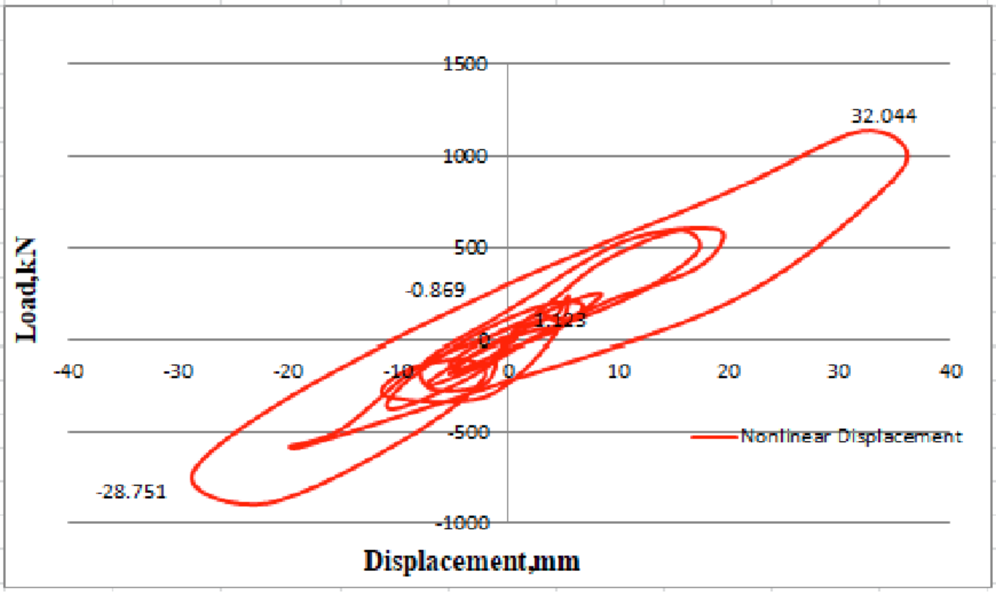


Figure 4.43. Relationship between Displacement and Load of 3 Bay 4 Story Frame

D. Hinge Result

The hinge result (Figure 4.44) shows that it is in elastic limit. Plastic rotation has not occurred.

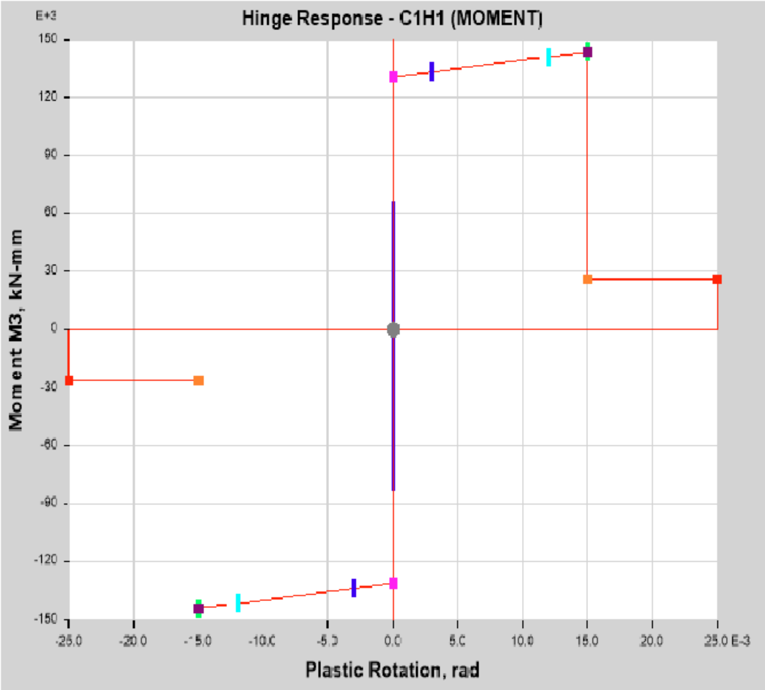


Figure 4.44. Hinge Result of 3 Bay 4 Story Frame

4.3.4 C_d and Drift Ratio of Three Bay Five Story Frame (3-5)

The selected beam of 304.8mm×475.7mm (12in×18.73in) and column of 377.7mm×377.7mm (14.87in ×14.87in).

A. Deflection amplification Factor (C_d)

C_d = 1.12 and 3.05; with respect to equivalent static displacement and linear time displacement.

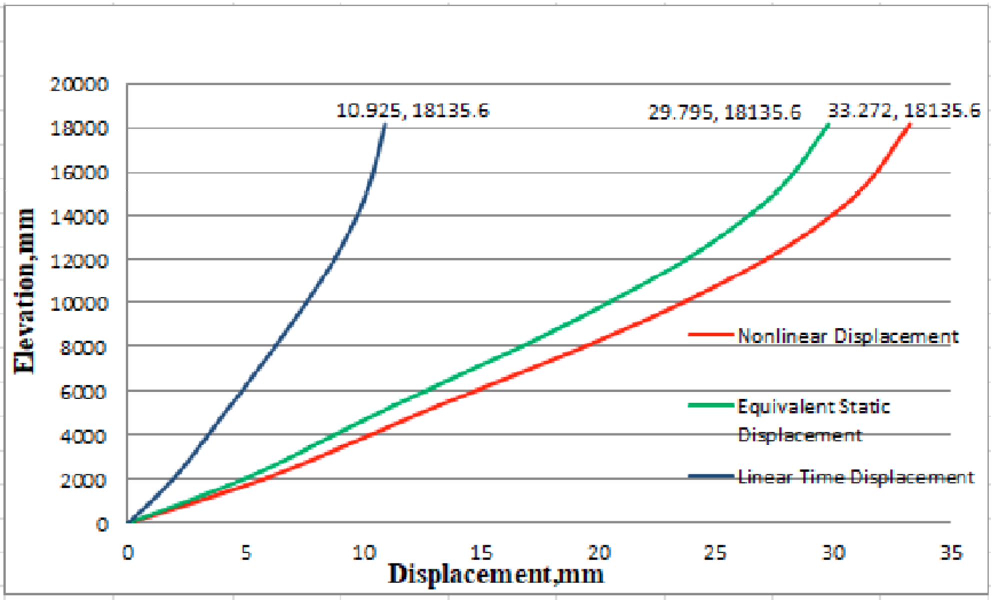


Figure 4.45. Maximum Story Displacement of 3 Bay 5 Story Frame

B. Story Drift

As per BNBC, drift limit is 0.020. Here static linear, linear and nonlinear drifts are less than specified limit.

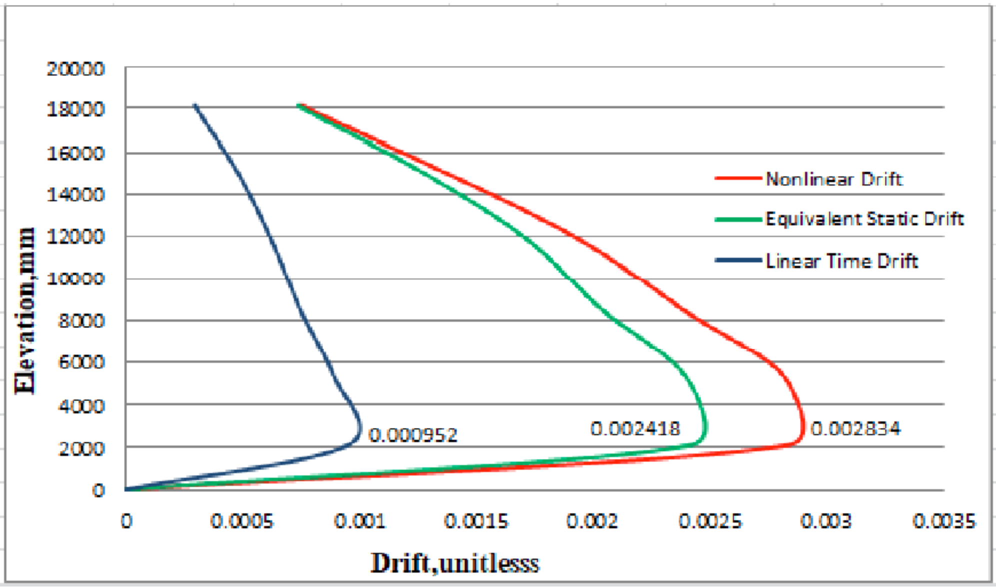


Figure 4.46. Maximum Story Drift of 3 Bay 5 Story Frame

C. Relationship between Displacement and Load

Three Bay five story has maximum roof displacement of 33.3 mm where under load condition culumn has maximum displacement of 42.3 mm. The values are not nearer since under load condition the value is of column.

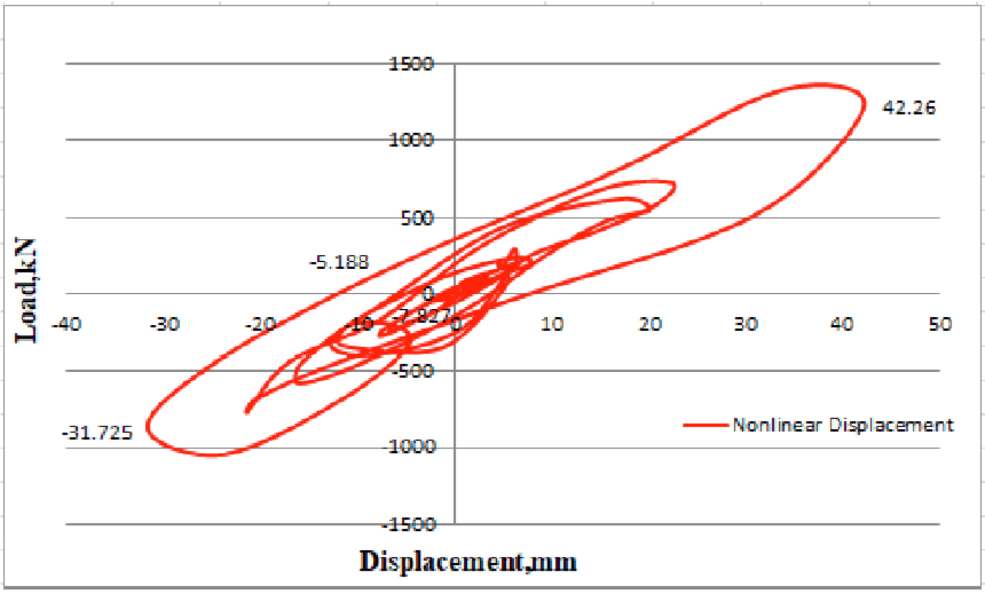


Figure 4.47. Relationship between Displacement and Load of 3 Bay 5 Story Frame

D. Hinge Result

The hinge result (Figure 4.48) shows that it is not in plastic zone. Plastic rotation has not occurred.

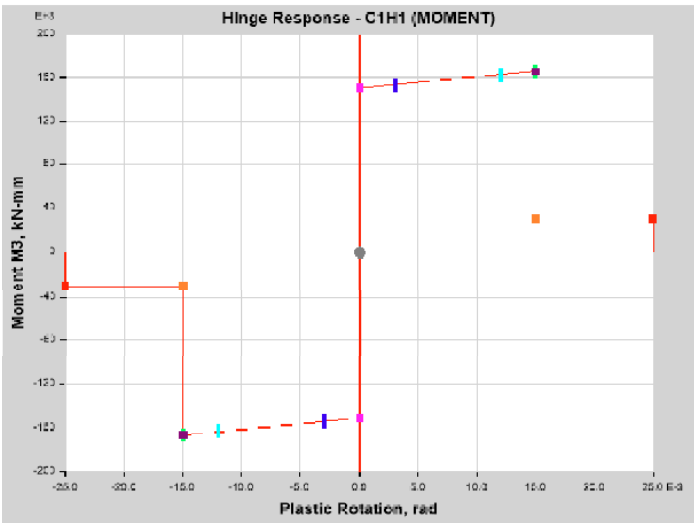


Figure 4.48. Hinge Result of 3 Bay 5 Story Frame

4.3.5 C_d and Drift Ratio of Three Bay Five Story Frame (3-6)

The selected beam of 304.8mm×480.6mm (12in×18.92in) and Column of 400.3mm×400.3mm (15.76in×15.76in).

A. Deflection Amplification Factor (C_d)

C_d = 1.20 and 3.62; with respect to equivalent static displacement and linear time displacement.

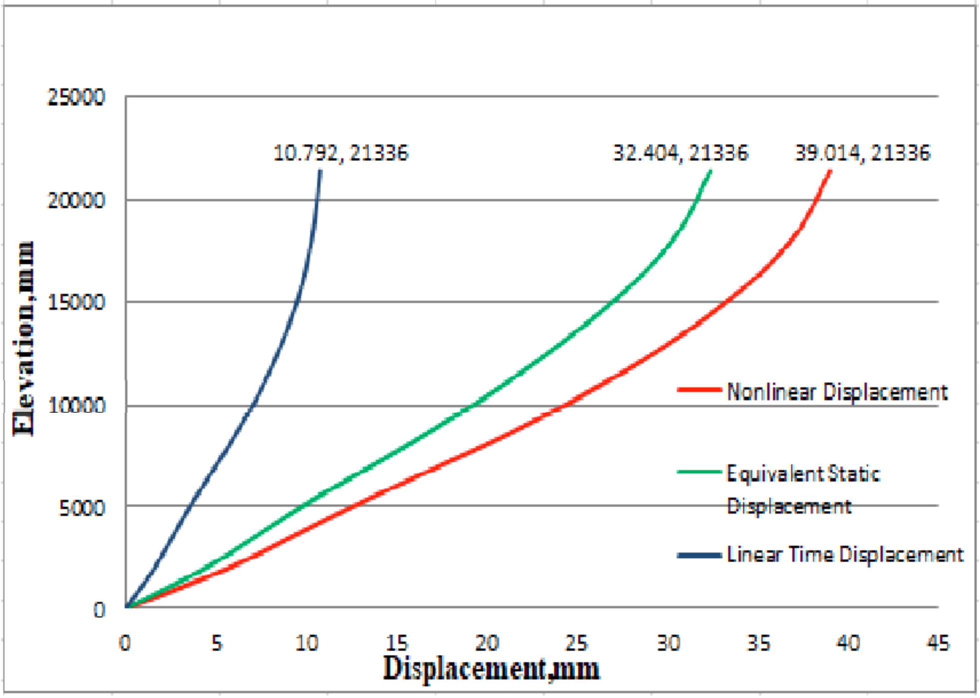


Figure 4.49. Maximum Story Displacement of 3 Bay 6 Story Frame

B. Story Drift

As per BNBC, drift limit is 0.020. Here static linear, linear and nonlinear drifts are less than specified limit.

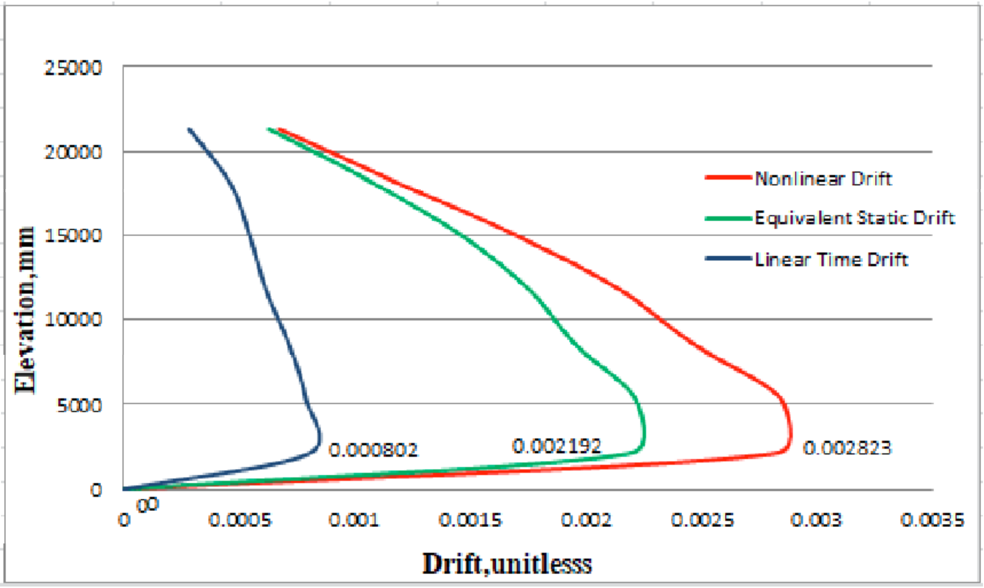


Figure 4.50. Maximum Story Drift of 3 Bay 6 Story Frame

C. Relationship between Displacement and Load

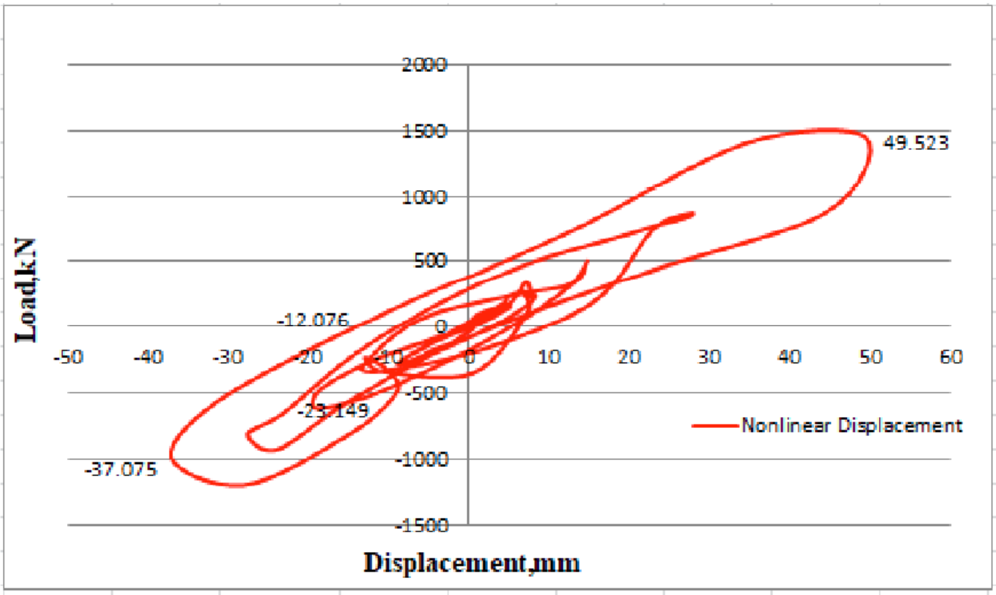


Figure 4.51. Relationship between Displacement and Load of 3 Bay 6 Story Frame

Three Bay six story has maximum roof displacement of 39 mm where under load condition culumn has maximum displacement of 49.5 mm. The values are nearer since under load condition the value is of column.

D. Hinge Result

The hinge result (Figure 4.52) shows that it is not in plastic zone.

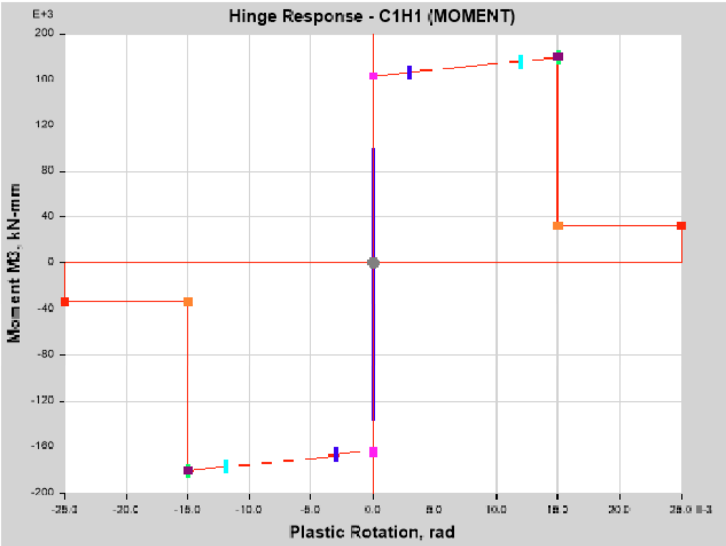


Figure 4.52. Hinge Result of 3 Bay 6 Story Frame

4.3.6 C_d and Drift Ratio of Three Bay Eight Story Frame (3-8)

The selected beam of 304.8mm×490.5mm (12in×19.31in) and column of 442.7mm×442.7mm (17.43in×17.43in).

A. Deflection Amplification Factor (C_d)

C_d = 55.103/37.717=1.46; with respect to equivalent static displacement and C_d = 3.63 with respect to linear time displacement.

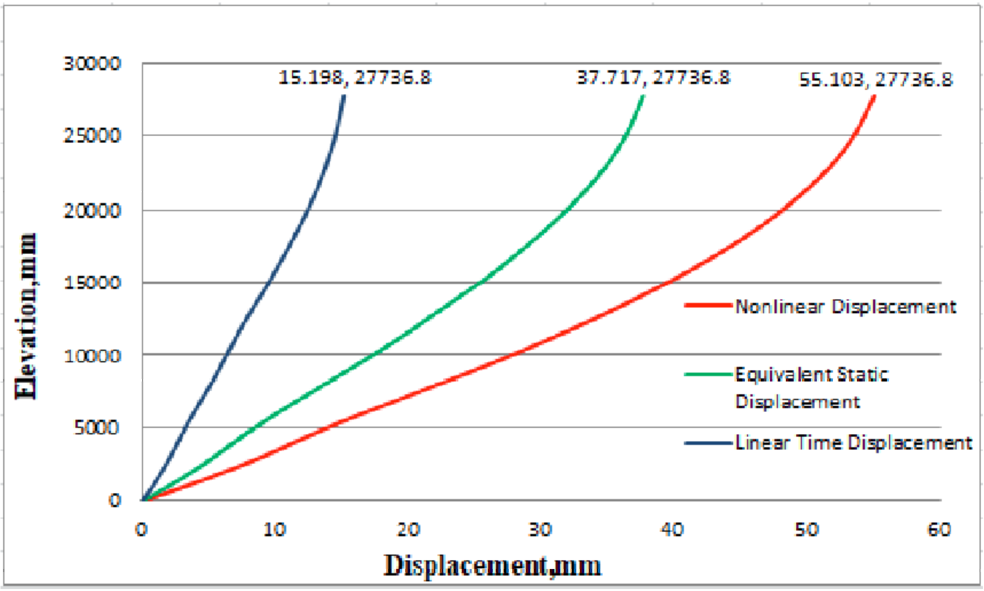


Figure 4.53. Maximum Story Displacement of 3 Bay 8 Story Frame

B. Story Drift

As per BNBC, drift limit is 0.020. Here in three cases drift is less than specified limit.

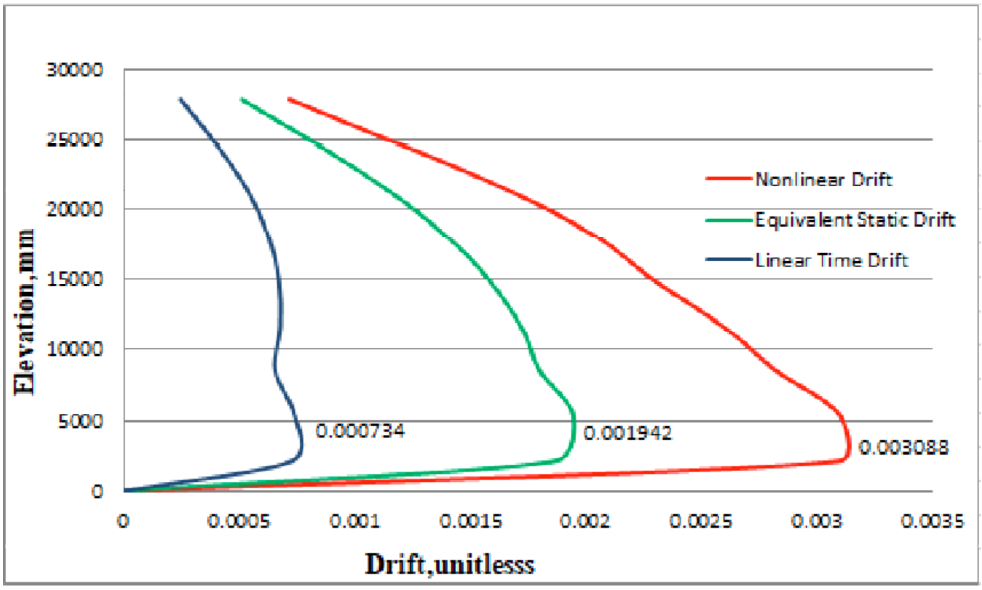


Figure 4.54. Maximum Story Drift of of 3 Bay 8 Story Frame

C. Relationship between Displacement and Load

Three bay eight story has maximum roof displacement of 55.1 mm where under load condition culumn has maximum displacement of 64.3 mm. The values are nearer since under load condition the value is of column

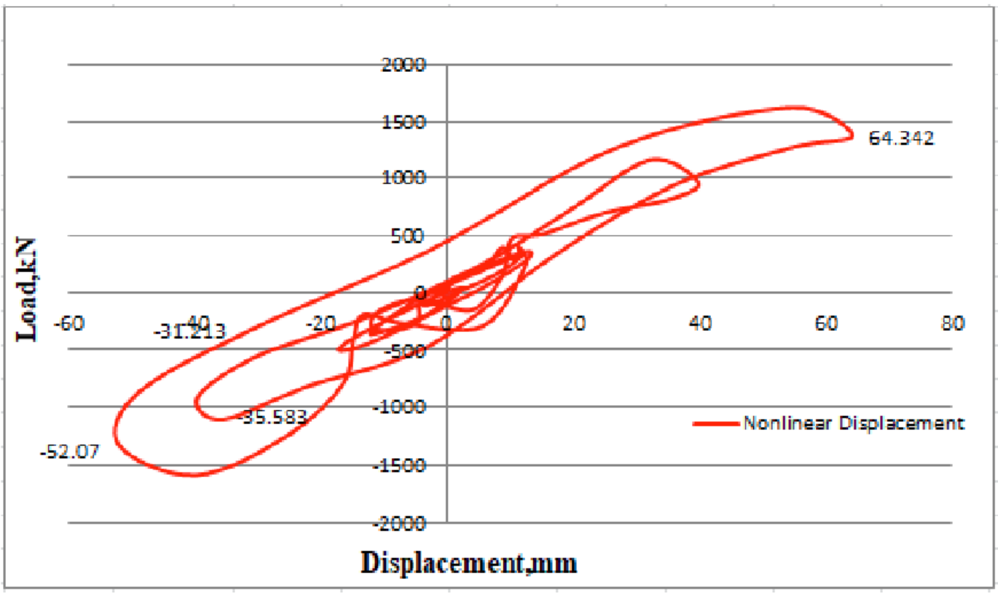


Figure 4.55. Relationship between Displacement and Load of 3 Bay 8 Story Frame

D. Hinge Result

The hinge result (Figure 4.56) shows that it is not in plastic zone.

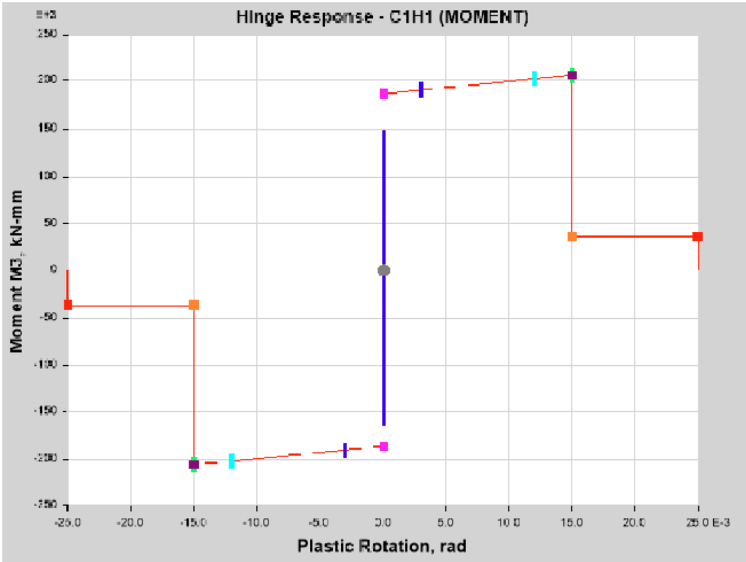


Figure 4.56. Hinge Result of 3 Bay 8 Story Frame

4.3.7 C_d and Drift Ratio of Three Bay Ten Story Frame (3-10)

The selected beam of 304.8mm×486.6mm (12in×19.15in) and column of 479mm×479mm (18.86in×18.86in).

A. Deflection Amplification Factor (C_d)

C_d = 78.931/40.537=1.95; with respect to equivalent static displacement and C_d = 4.22 with respect to linear time displacement. As per BNBC (2020), the nonlinear time history to be done for more than 40 m in zone 2. 10 story is much lower than 40 meters. Therefore, C_d value is lower than specified value (4.5).

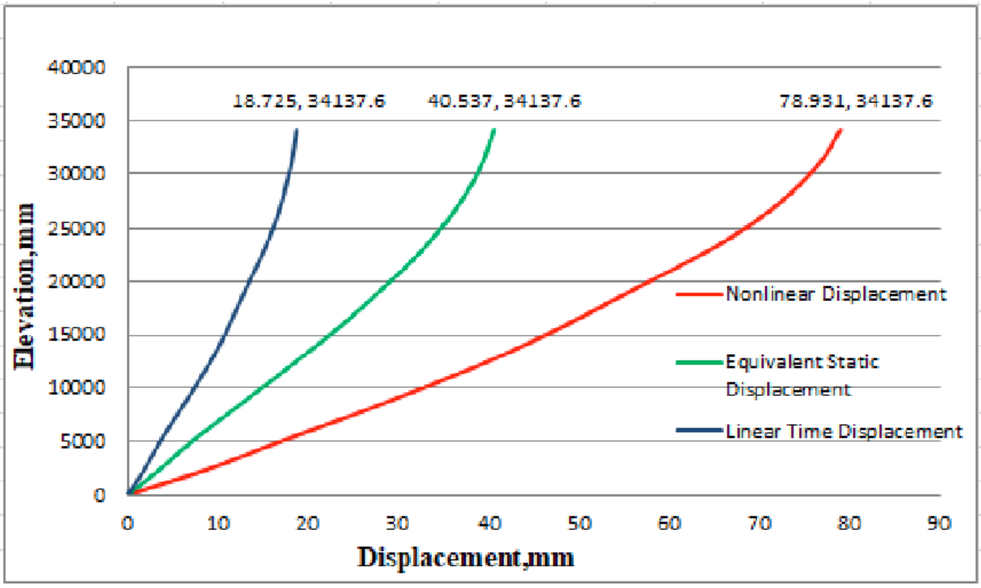


Figure 4.57. Maximum Story Displacement of 3 Bay 10 Story Frame

B. Story Drift

As per BNBC, drift limit is 0.020. Here in three cases drift is less than specified limit.

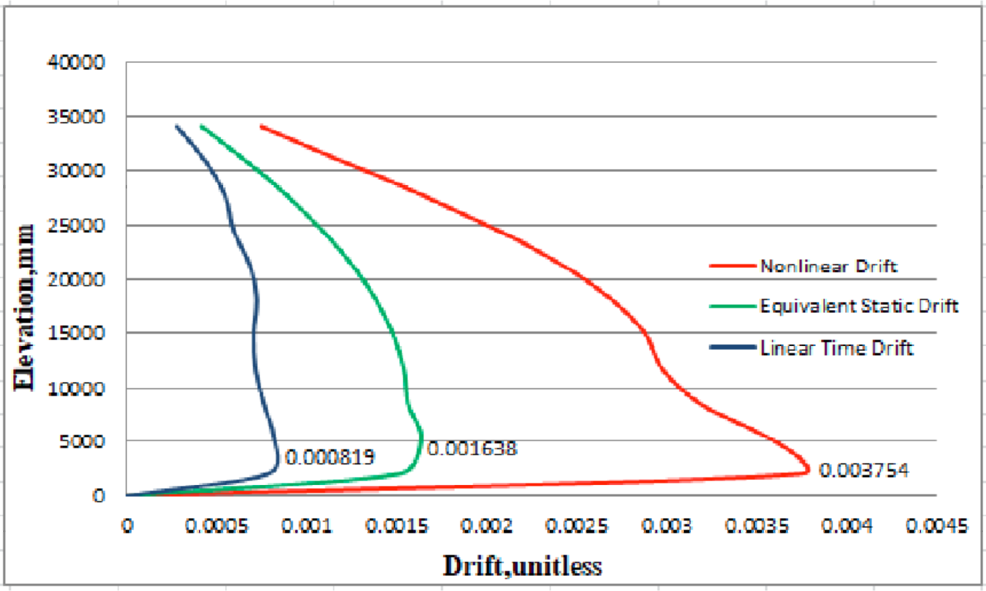


Figure 4.58. Maximum Story Drift of 3 Bay 10 Story Frame

C. Relationship between Displacement and Load

Three bay ten story has maximum roof displacement of 78.9 mm where under load condition culumn has maximum displacement of 74.4 mm. The values are nearer since under load condition the value is of column

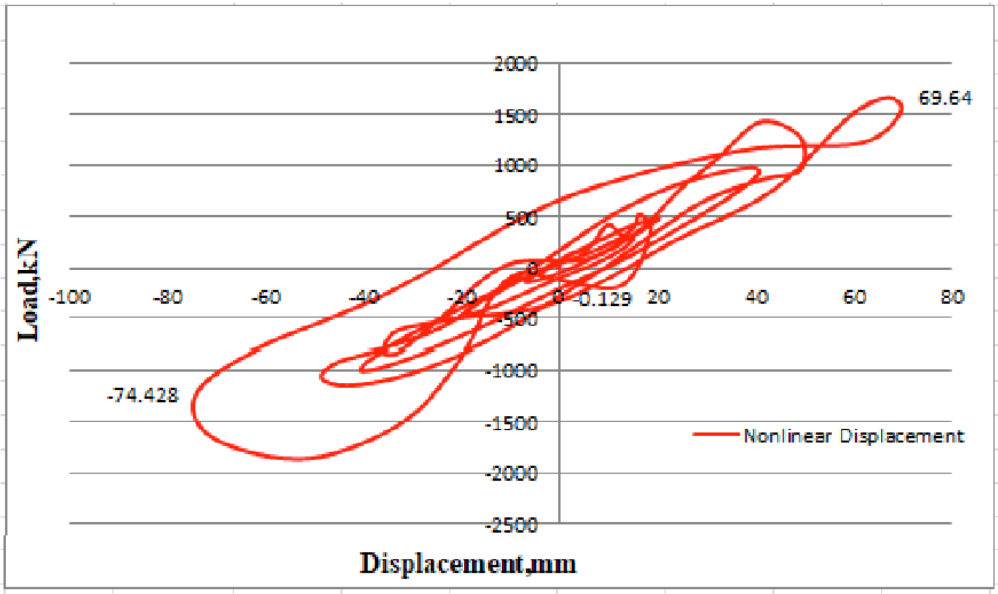


Figure 4.59. Relationship between Displacement and Load of 3 Bay 8 Story Frame

D. Hinge Result

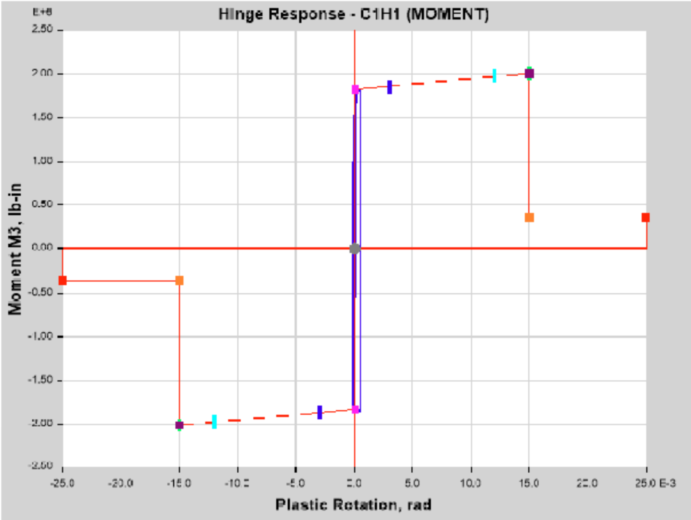


Figure 4.60. Hinge Result of 3 Bay 10 Story Frame

The hinge result (Figure 4.60) shows that it is in plastic zone. Plastic rotation has occurred. Plastic rotation value is positive.

4.3.8 C_d and Drift Ratio of Three Bay Twelve Story Frame (3-12)

A. Deflection Amplification Factor (C_d)

Equivalent static analysis and nonlinear time history analysis has been done for 3 bay 12 story frame. The parameters including building dimension: 15 meter by 15 meter; number of story:12; story height 3.2 meter; Base to ground floor 2.13 meter (7 feet); are considered in equivalent static and nonoliear time history analysis and design.The selected beam is 312mm×483mm (12.3in×19in) and column is 521mm×521mm (20.5in×20.5in).The evaluated nonlinear displacement is 97.09 mm where the linear time history displacement is 21.9 4mm. Therefore deflection amplification is found to be 4.43 with respect to linear time history displacement.

C_d, story drift ratio, inelastic displacement profile, hinge result, time vs acceleration, time vs load and displacement vs load has been summerized in this section.

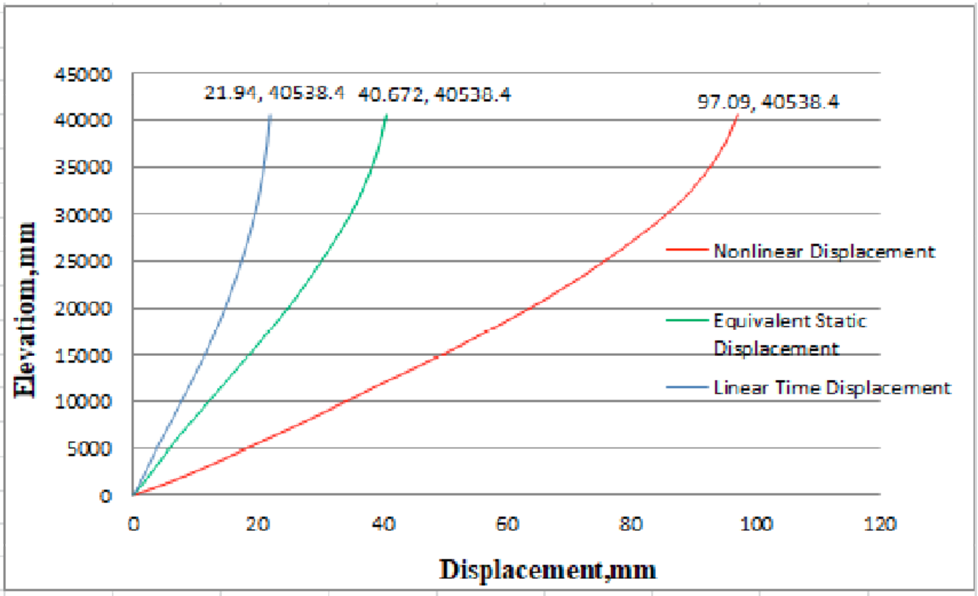


Figure 4.61. Maximum Story Displacement of 3 Bay 12 Story Frame

C_d = 97.09/21.94 = 4.43; With respect to linear time history displacement.

C_d = 97.09/40.672 = 2.39; With respect to linear time history displacement.

B. Story Drift

Evaluated nonlinear drift 0.00397, Linear drift 0.000845 and Equivalent static drift 0.001323. Drift limit is 0.020. So the drift found is less than the drift limit.

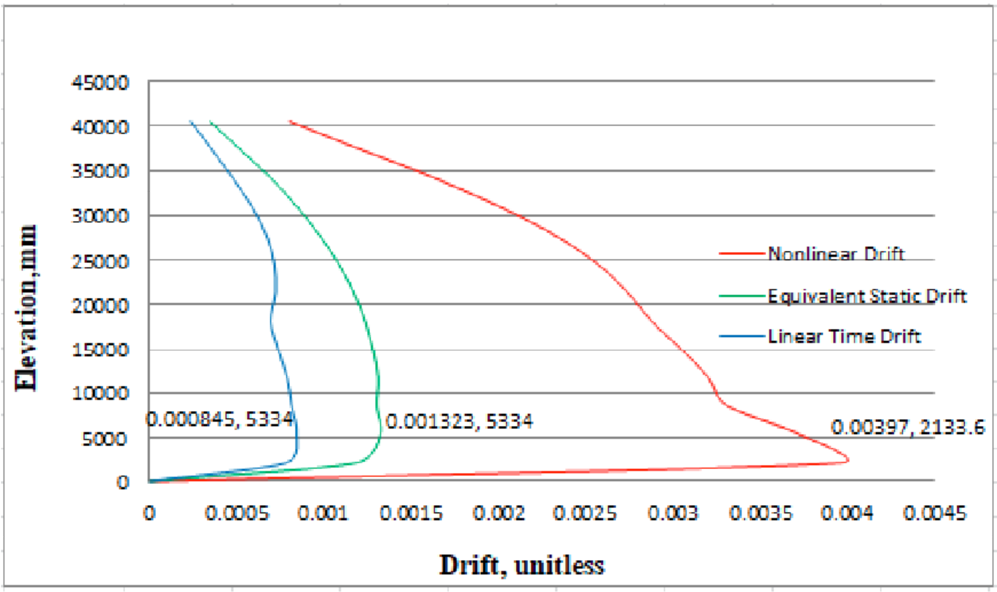


Figure 4.62. Maximum Story Drift of 3 Bay 12 Story Frame

C. Relationship between Displacement and Load

The nonlinear roof displacement is 97.09 mm and displacemnt of frame (column) under load is 93.37 mm. This indicates that evaluated result is correct.

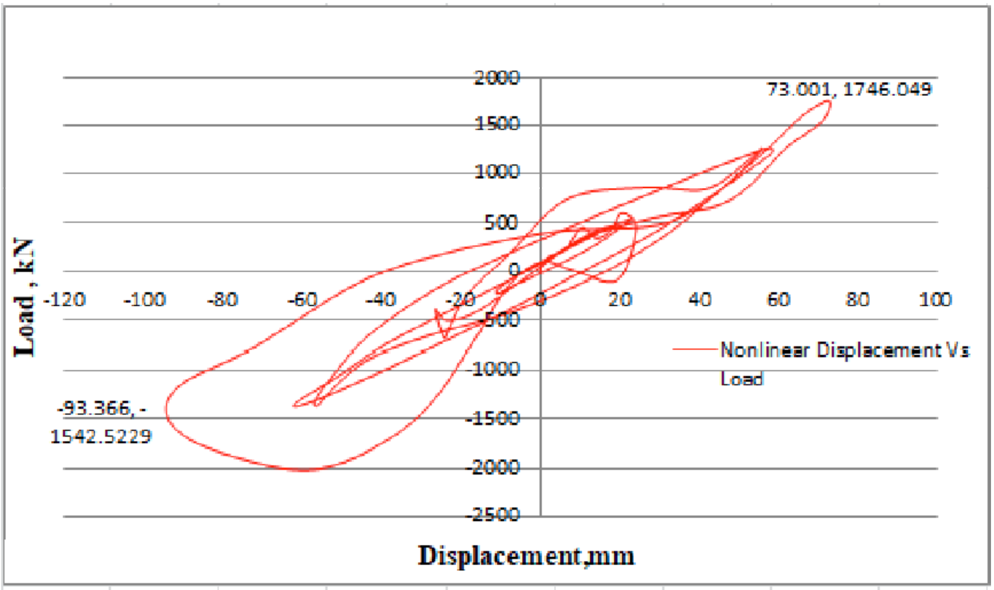


Figure 4.63. Relationship between Displacement and Load of 3 Bay 12 Story Frame

D. Hinge Result

The Load case is nonlineardynamic. The column is in ground floor. Hinge degree of freedom M3. Hinge relative distance is 0.1. The hinge has moved to plastic zone. Rotation is in positive direction.

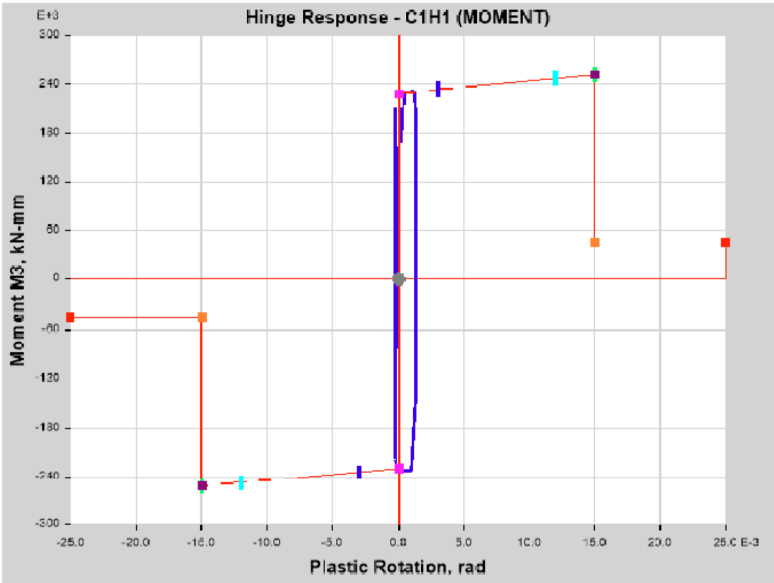


Figure 4.64. Hinge Result of of 3 Bay 12 Story Frame

4.3.9 Three Bay Twelve Story - Reduced Beam and Column Size

Selected beam and column were 12.3in×19in and 20.5in×20.5in respectively. Beam was reduced to around 12.3in×18.5in. At this moment capacity of beam was sufficient. When column was reduced to around 20.4in×20.4in at that time column showing sufficient but beam was showing insufficient. Then column was increased to 20.43 in, then demand capacity ratio was 1. At that time beam was increased to 12.3in×20.53in, the beam was sufficient. Finding is that when reducing column affect the beam, then column should not be reduced if column capacity is sufficient. Keeping column capacity exactly as required, beam capacity to be increased unless beam is sufficient. Then selected beam is 12.3in×18.54in and selected column is 20.43in×20.43in.

A. Deflection Amplification Factor (C_d)

C_d = 2.37; with respect to equivalent static displacement and C_d=4.45 with respect to linear time displacement.

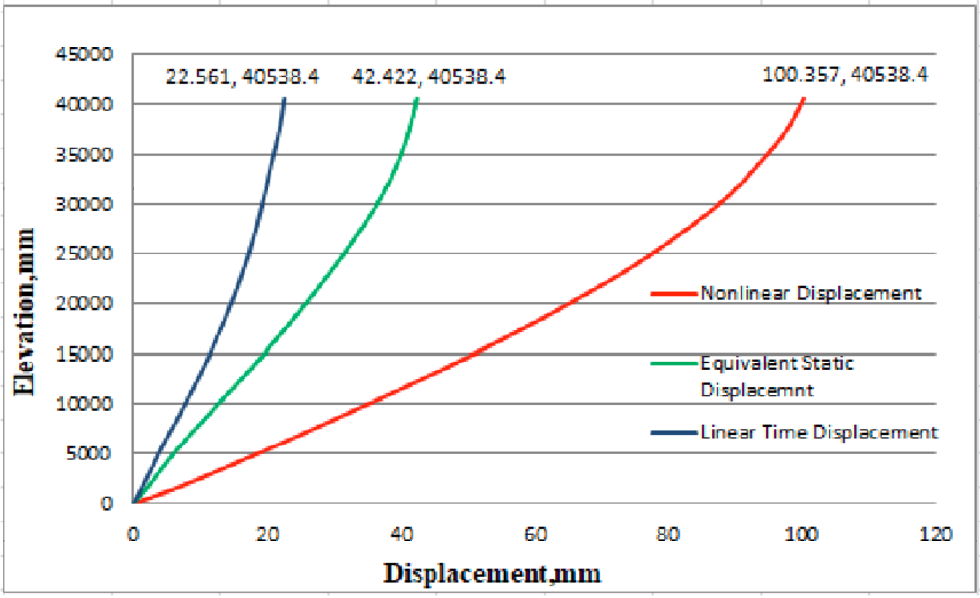


Figure 4.65. Maximum Story Displacement of 3 Bay 12 Story Frame

B. Story Drift

As per BNBC, drift limit is 0.020. Here in three cases drift is less than specified limit.

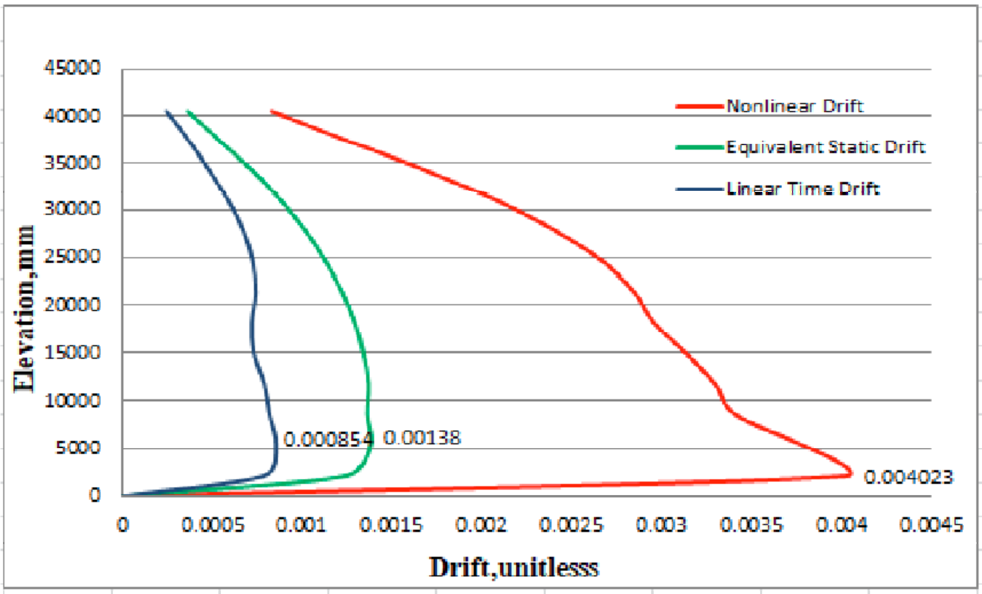


Figure 4.66. Maximum Story Drift of 3 Bay 12 Story Frame

C. Relationship between Displacement and Load

The nonlinear roof displacement is 100.357 mm and displacemnt of frame (column) under load is 94.95 mm. This indicates that evaluated result is correct.

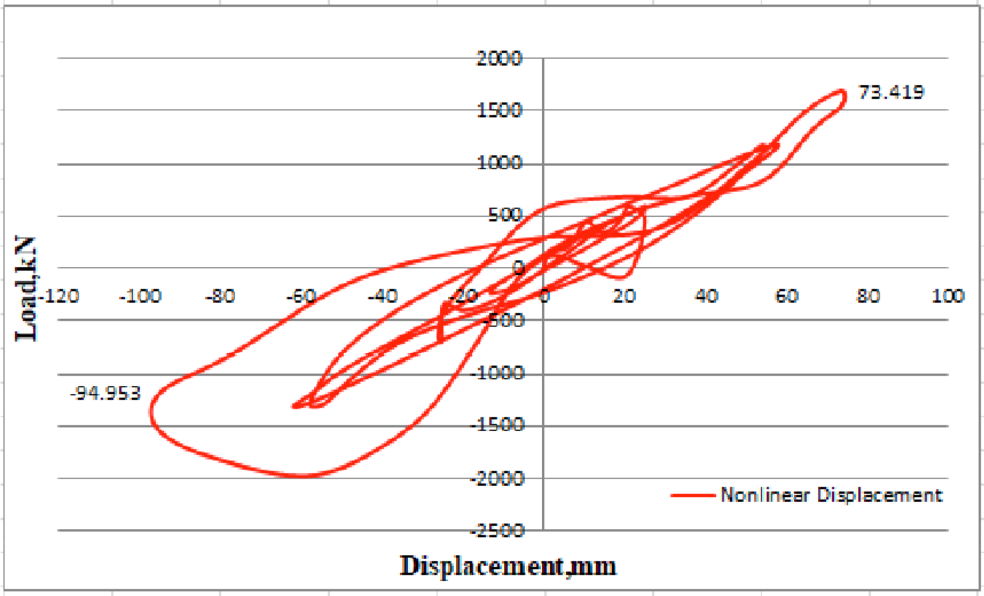


Figure 4.67. Relationship between Displacement and Load of 3 Bay 12 Story Frame

D. Hinge Result

The Load case is nonlineardynamic. The column is in ground floor. Hinge degree of freedom M3. Hinge relative distance is 0.1. The hinge has moved to plastic zone. Rotation is in positive direction.

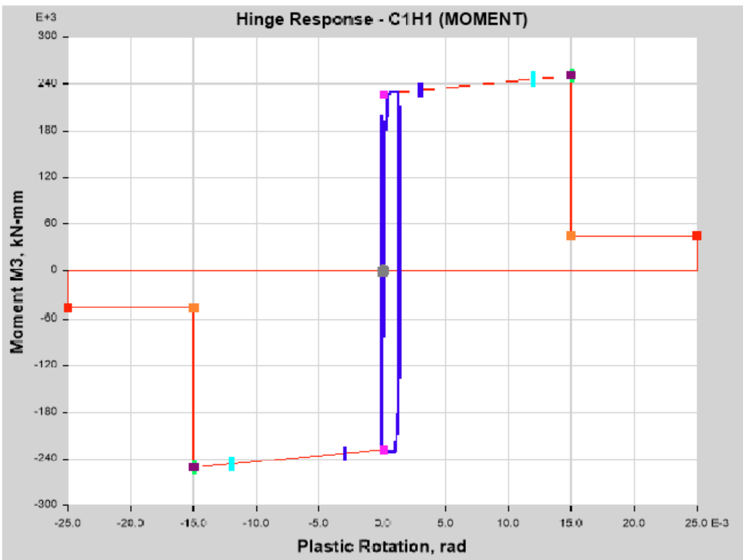


Figure 4.68. Hinge Result of of 3 Bay 12 Story Frame

4.4 Finite Element Analysis Result for 20m×20m Frame Structures

4.4.1 C_d and Drift Ratio for Four Bay Two Story Frame (4-2)

The beam having width of 265.4mm (10.45 inch) and depth of 464.8 mm (18.3 inch). The column selected having width of 265.4mm (10.45 inch) and height of 265.4 (10.45 inch).

A. Deflection Amplication Factor (C_d)

C_d = 1.35 and 2.92 respectively with respect to equivalent static displacement and linear time displacement.

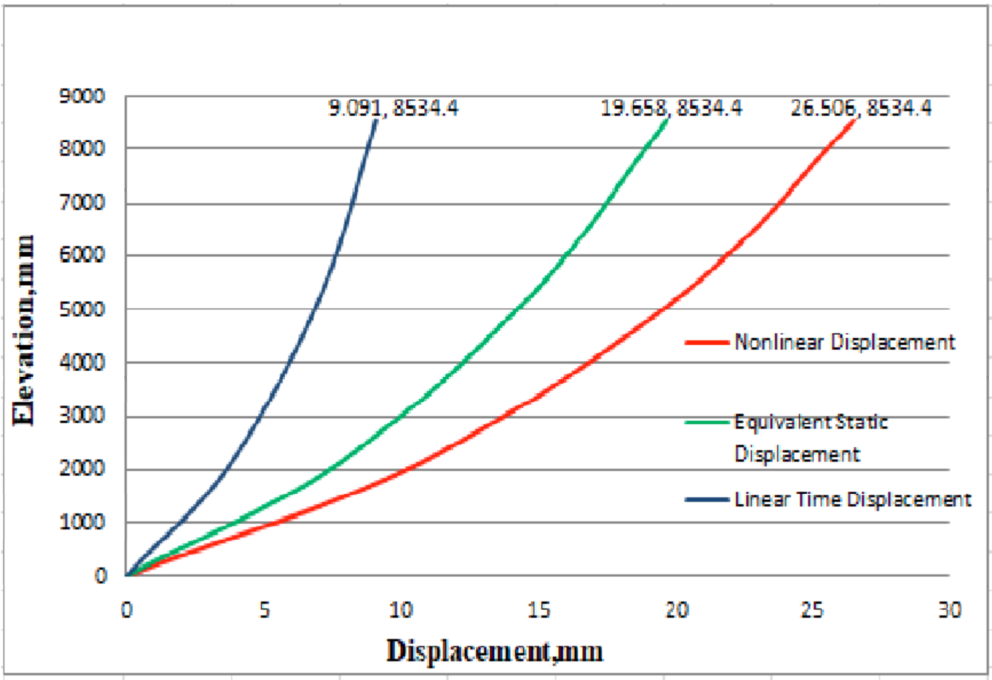


Figure 4.69. Maximum Story Displacement of 4 Bay 2 Story Frame

B. Story Drift

As per BNBC drift limit is 0.020. Here static linear, linear and nonlinear drifts are less than specified limit.

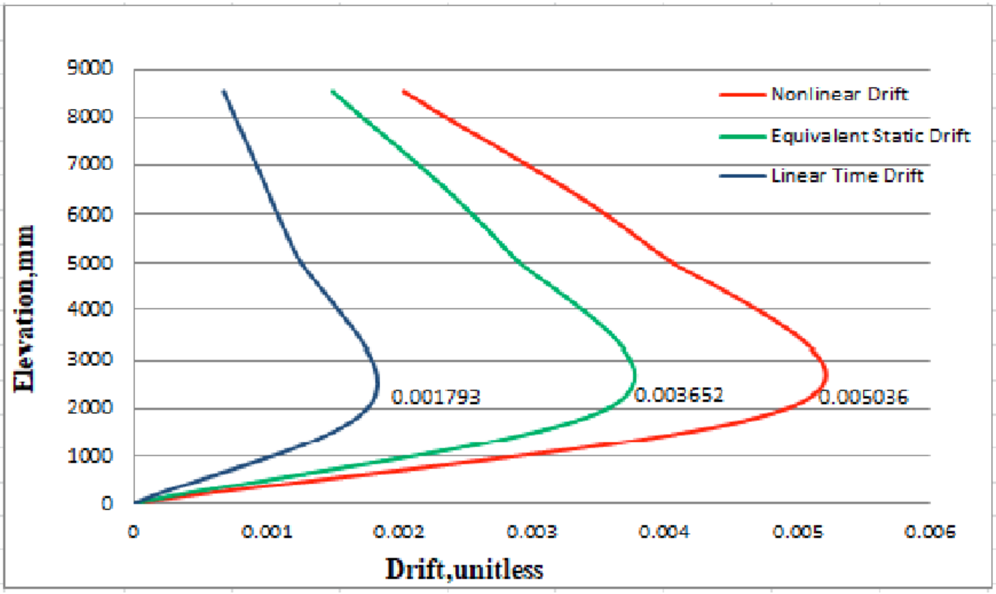


Figure 4.70. Maximum Story Drift of 4 Bay 2 Story Frame

C. Relationship between Displacement and Load

Four Bay Two story has maximum roof displacement of 26.506 mm where under load condition culumn has maximum displacement of 24.9 mm. The values are nearer since under load condition the value is of column.

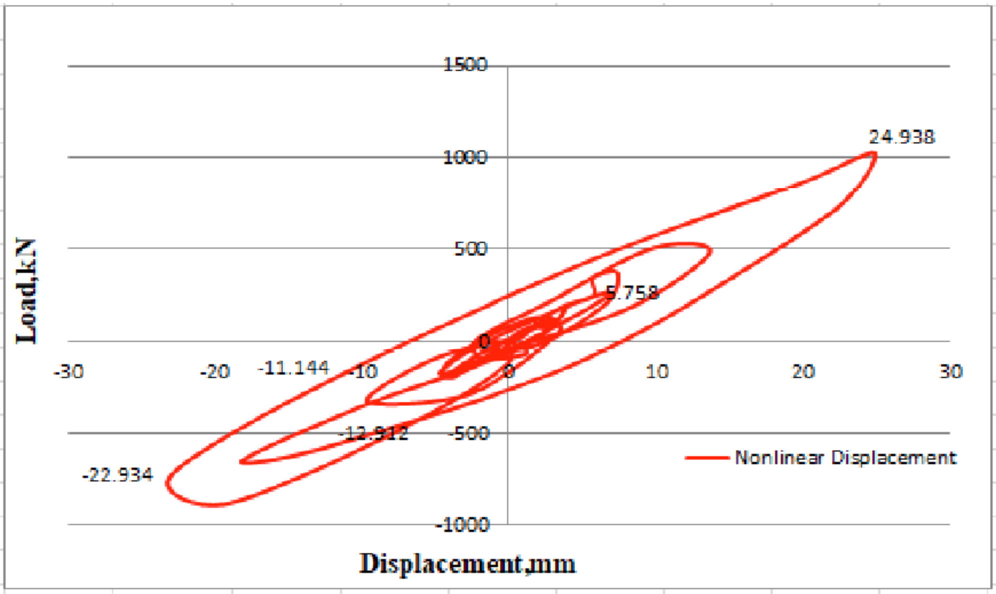


Figure 4.71. Relationship between Displacement and Load of 4 Bay 2 Story Frame

D. Hinge Result

The hinge result (Figure 4.72) shows that it is in elastic limit. Plastic rotation has not occurred.

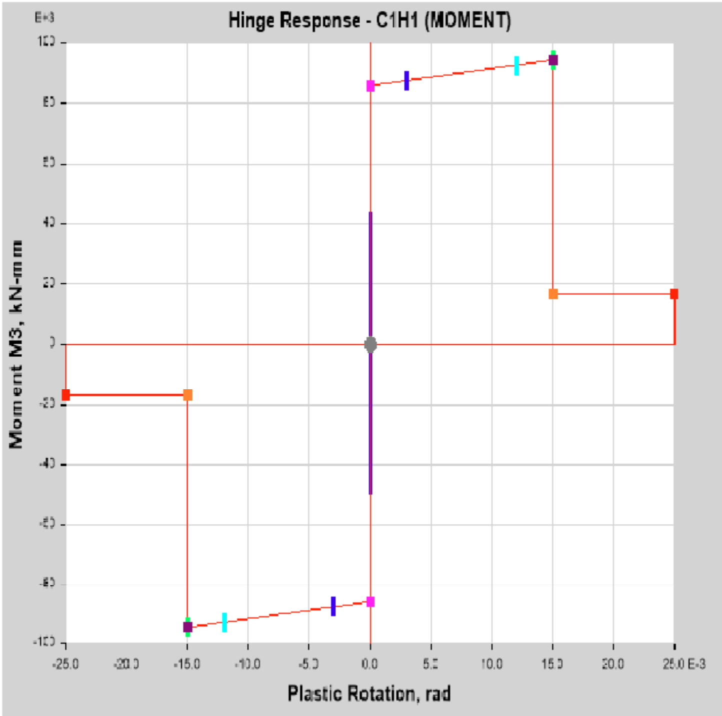


Figure 4.72. Hinge Result of 4 Bay 2 Story Frame

4.4.2 C_d and Drift Ratio for Four Bay Three Story Frame (4-3)

The beam having width of 278.6mm (10.97 inch) and depth of 482.6 (19 inch). The column selected having width of 316mm (12.44 inch) and height of 316mm (12.44 inch).

A. Deflection Amplication Factor (C_d)

C_d = 1.27 and 3.03 respectively with respect to equivalent static displacement and linear time displacement.

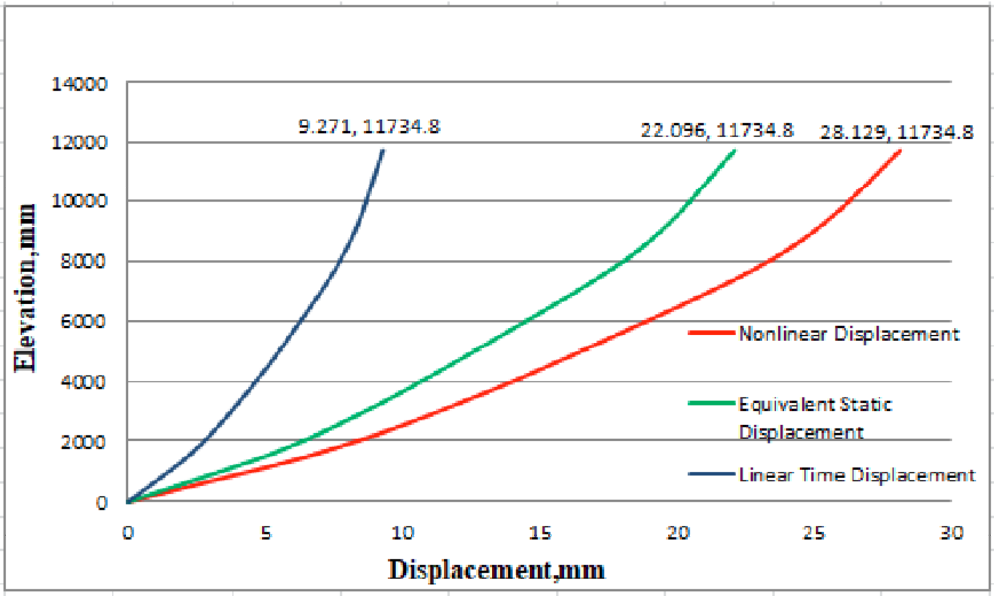


Figure 4.73. Maximum Story Displacement of 4 Bay 3 Story Frame

B. Story Drift

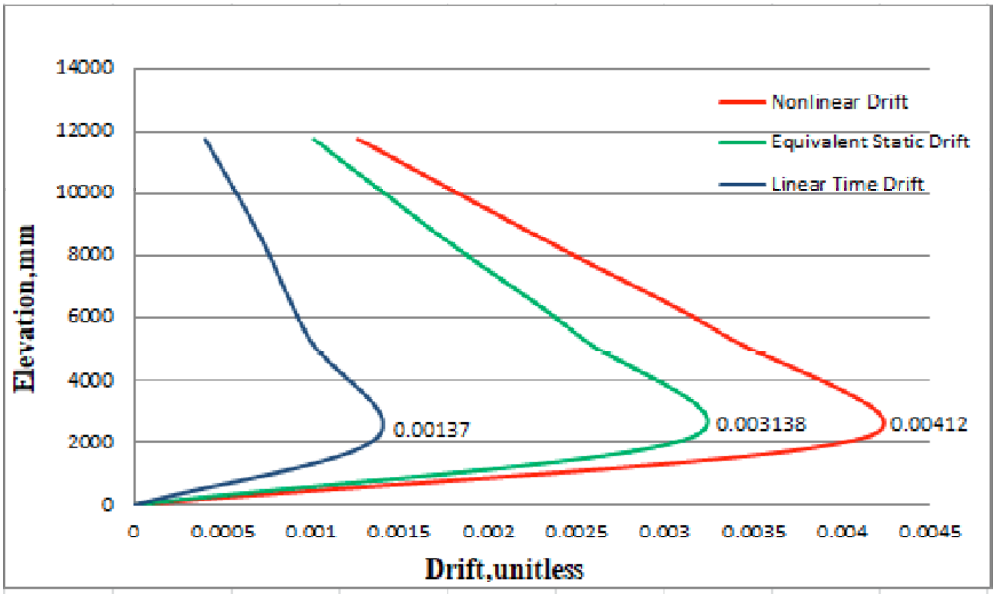


Figure 4.74. Maximum Story Drift of 4 Bay 3 Story Frame

As per BNBC drift limit is 0.02. Here static linear, linear and nonlinear drifts are less than specified limit.

C. Relationship between Displacement and Load

Four Bay Three story has maximum roof displacement of 28.13 mm where under load condition culumn has maximum displacement of 28.2 mm. The values are nearer since under load condition the value is of column.

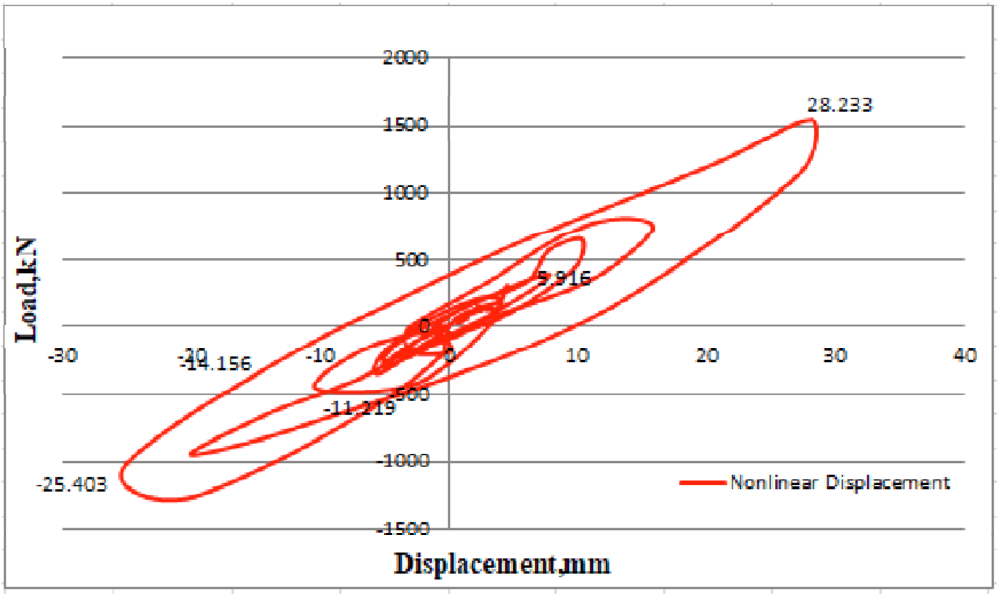


Figure 4.75. Relationship between Displacement and Load of 4 Bay 3 Story Frame

D. Hinge Result

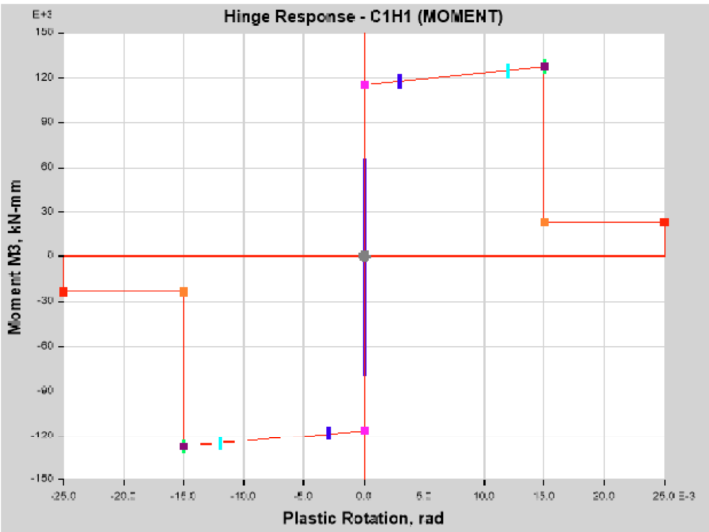


Figure 4.76. Hinge Result of 4 Bay 3 Story Frame

The hinge result (Figure 4.76) shows that it is not in inelastic limit. Plastic rotation has not occurred.

4.4.3 C_d and Drift Ratio for Four Bay Four Story Frame (4-4)

The beam having width of 304.8mm (12 inch) and depth of 482.6mm (19 inch). The column selected having width of 366mm (14.41 inch) and height of 366mm (14.41 inch).

A. Deflection Amplication Factor (C_d)

C_d = 1.2 and 3.35; respectively with respect to equivalent static displacement and linear time displacement.

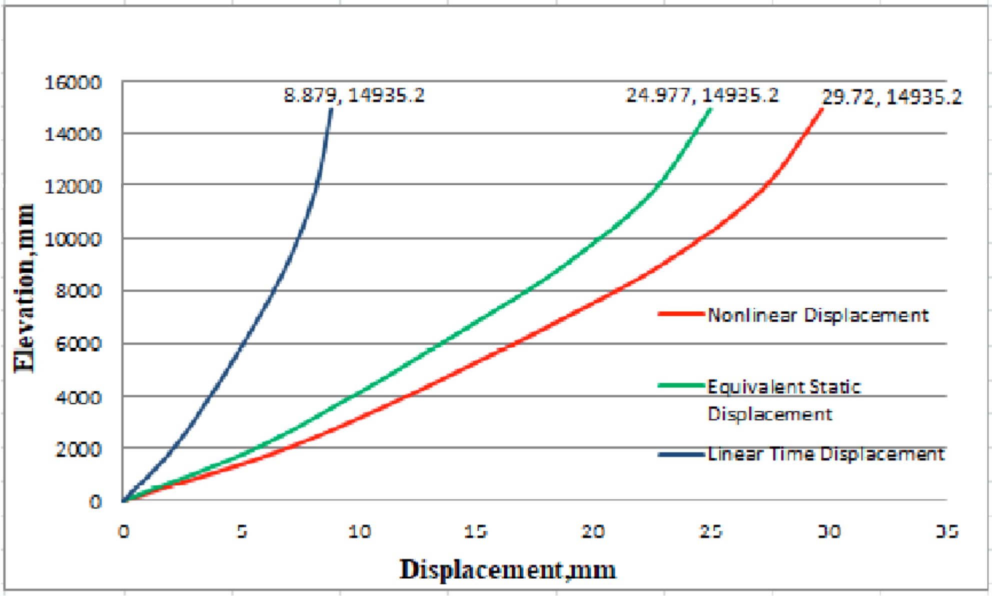


Figure 4.77. Maximum Story Displacement of 4 Bay 4 Story Frame

B. Story Drift

As per BNBC drift limit is 0.020. Here static linear, linear and nonlinear drifts are less than specified limit.

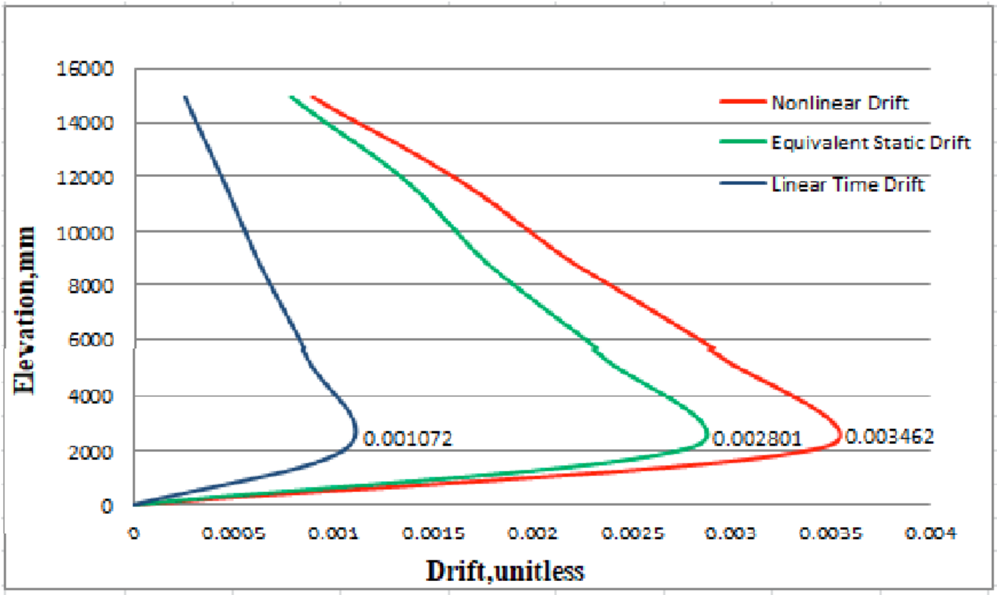


Figure 4.78. Maximum Story Driftt of of 4 Bay 4 Story Frame

C. Relationship between Displacement and Load

Four Bay four story has maximum roof displacement of 29.72 mm where under load condition culumn has maximum displacement of 30.875 mm. The values are nearer since under load condition the value is of column.

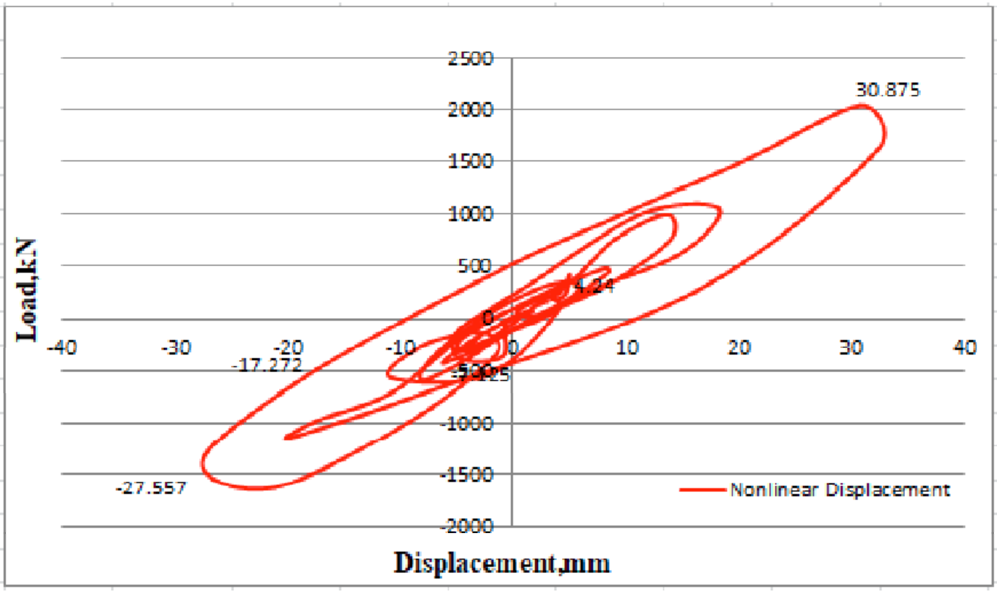


Figure 4.79. Relationship between Displacement and Load of 4 Bay 4 Story Frame

D. Hinge Result

The hinge result (Figure 4.80) shows that it is not in inelastic limit. Plastic rotation has not occurred.

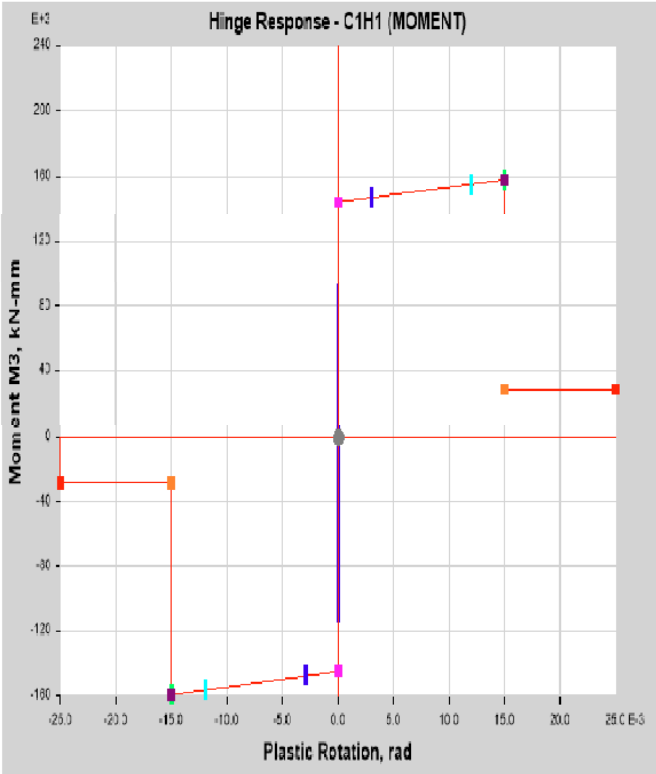


Figure 4.80. Hinge Result of 4 Bay 4 Story Frame

4.4.4 C_d and Drift Ratio of Four Bay Five Story Frame (4-5)

The beam having width of 304.8mm (12 inch) and depth of 479.8mm (18.89 inch). The column selected having width of 406mm (15.98 inch) and height of 406mm (15.98 inch).

A. Deflection Amplification Factor (C_d)

C_d = 1.13 and 2.93; respectively with respect to equivalent static displacement and linear time displacement

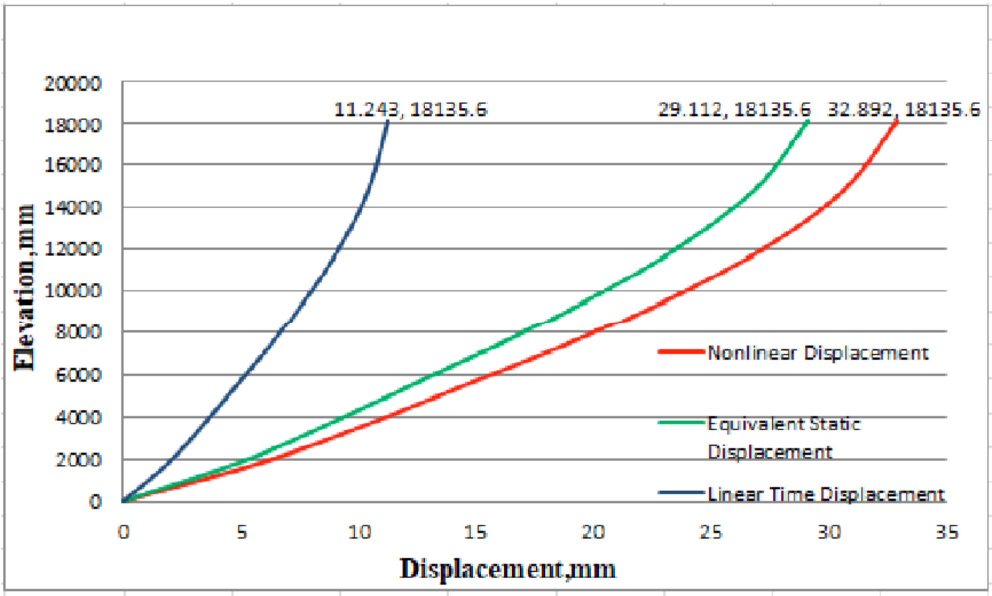


Figure 4.81. Maximum Story Displacement of 4 Bay 5 Story Frame

B. Story Drift

As per BNBC drift limit is 0.020. Here static linear, linear and nonlinear drifts are less than specified limit.

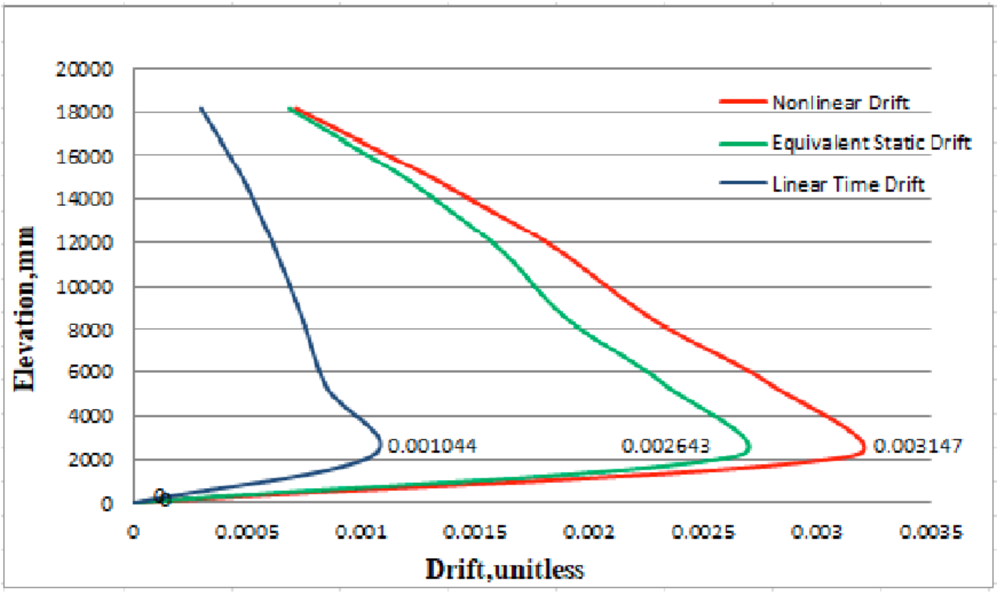


Figure 4.82. Maximum Story Drift of 4 Bay 5 Story Frame

C. Relationship between Displacement and Load

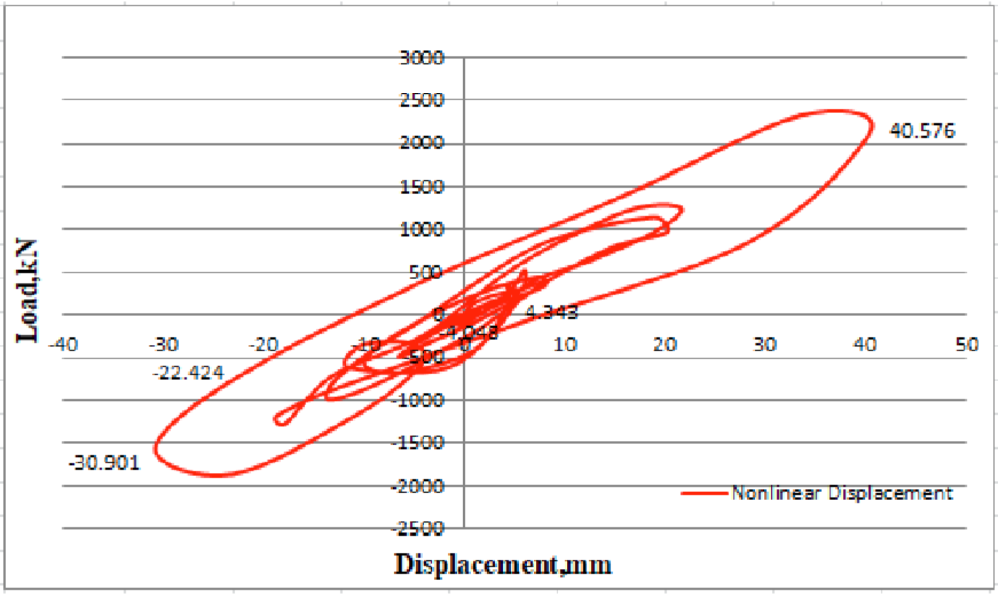


Figure 4.83. Relationship between Displacement and Load of 4 Bay 5 Story Frame

Four Bay five story has maximum roof displacement of 32.9 mm where under load condition culumn has maximum displacement of 40.6 mm. The values are nearer since under load condition the value is of column.

D. Hinge Result

The hinge result (Figure 4.84) shows that it is in inelastic limit. It has gone to plastic zone. The Plastic rotation is having negative Value.

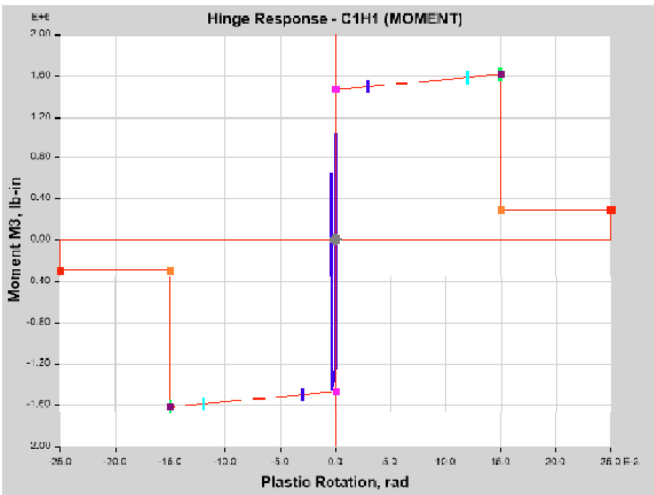


Figure 4.84. Hinge Result of 4 Bay 5 Story Frame

4.4.5 C_d and Drift Ratio of Four Bay Six Story Frame (4-6)

The selected beam of 305.6mm×482.3mm (12.03in×18.99in) and column of 444.5mm×444.5mm (17.5in×17.5in).

A. Deflection Amplification Factor (C_d)

C_d = 1.22 and 3.12; respectively with respect to equivalent static displacement and linear time displacement

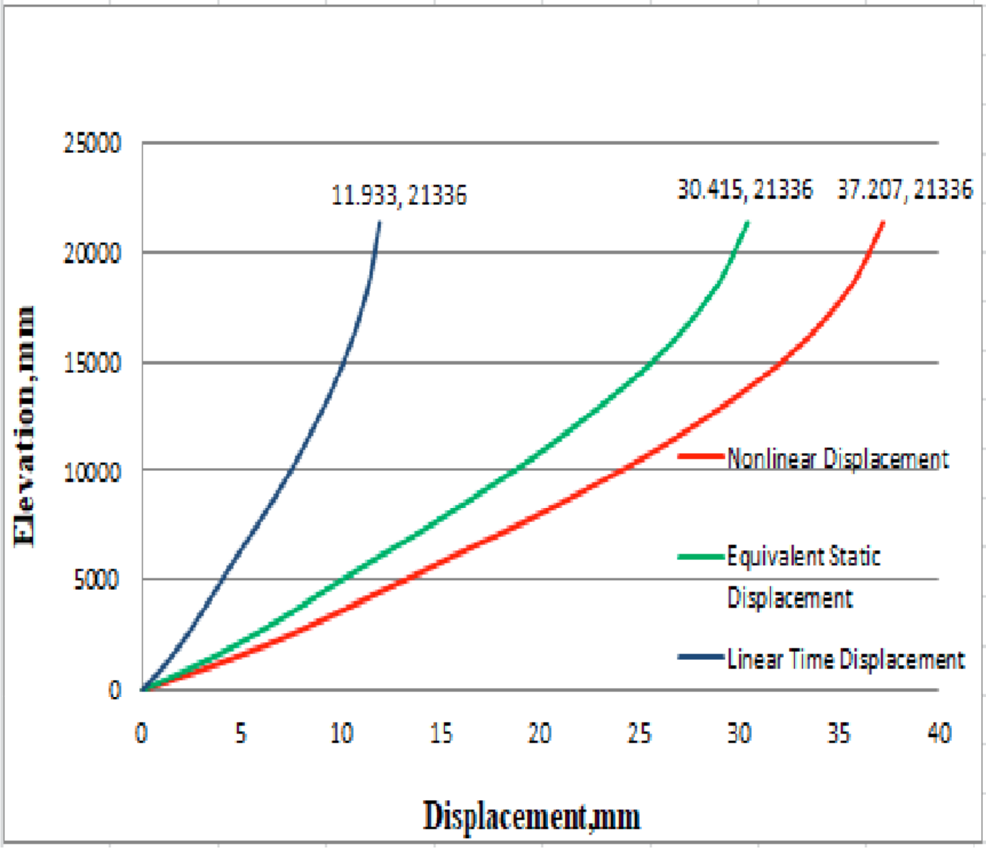


Figure 4.85. Maximum Story Drift of 4 Bay 6 Story Frame

B. Story Drift

As per BNBC drift limit is 0.020. Here static linear, linear and nonlinear drifts are less than specified limit.

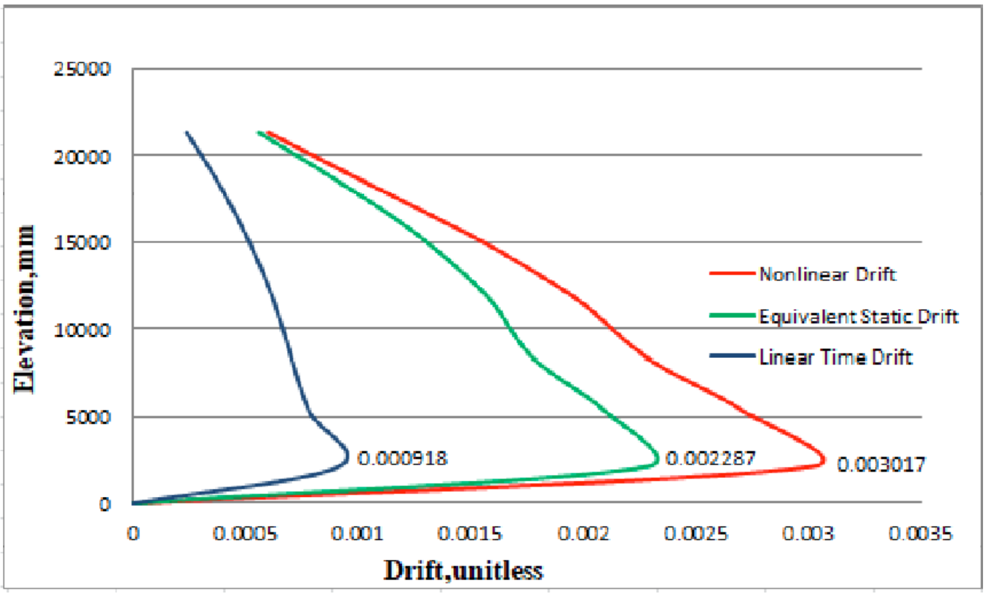


Figure 4.86. Maximum Story Drift of 4 Bay 6 Story Frame

C. Relationship between Displacement and Load

Four Bay six story has maximum roof displacement of 37.21 mm where under load condition culumn has maximum displacement of 46.7 mm. The values are nearer since under load condition the value is of column.

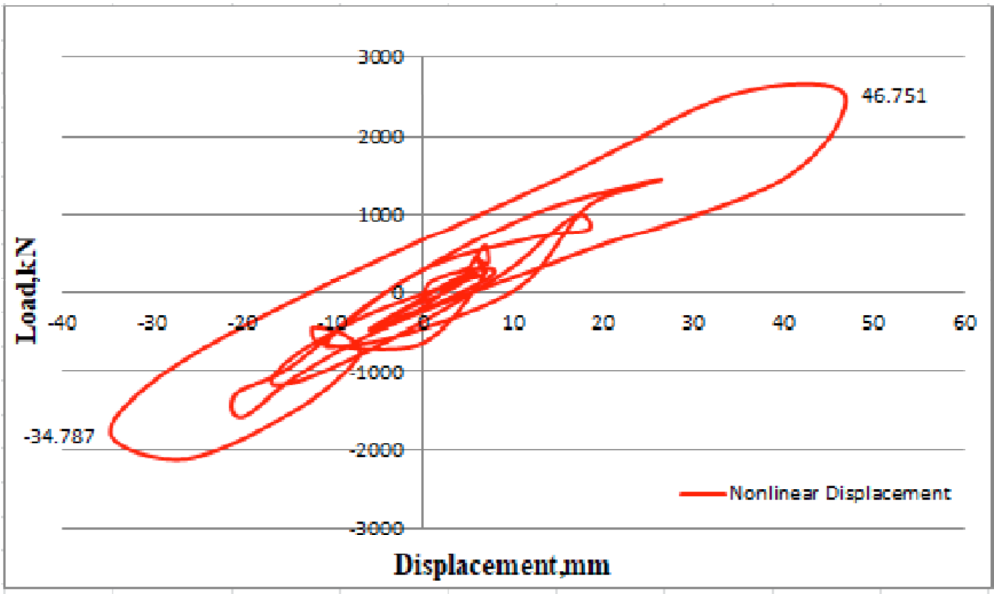


Figure 4.87. Relationship between Displacement and Load of 4 Bay 6 Story Frame

D. Hinge Result

The hinge result (Figure 4.88) shows that it is in inelastic limit. It has gone to plastic zone. The Plastic rotation is having negative value.

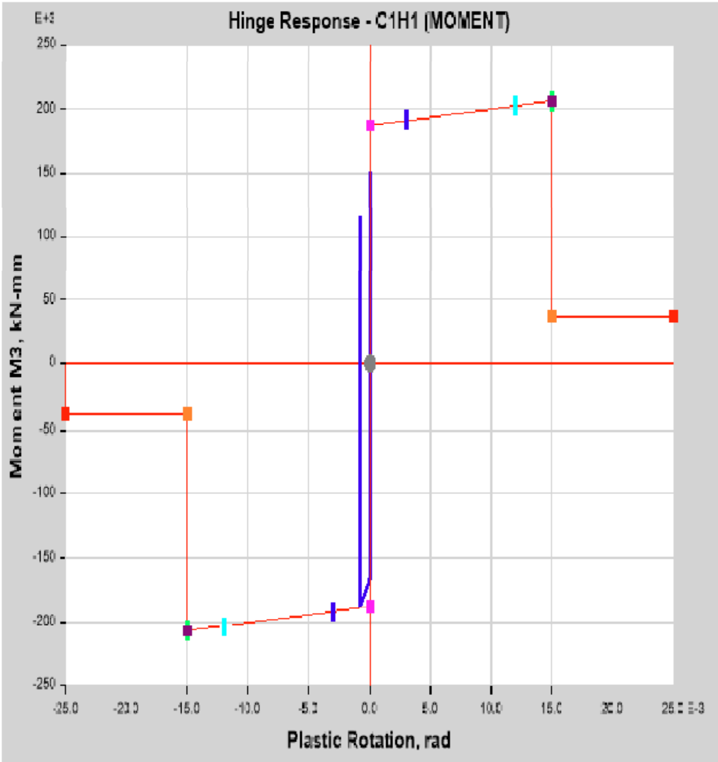


Figure 4.88. Hinge Result of 4 Bay 6 Story Frame

4.4.6 C_d and Drift Ratio of Four Bay Eight Story Frame (4-8)

The selected beam of 311mm×311mm (12.24in×19in) and column of 478mm×478mm (18.82in×18.82in).

A. Deflection Amplification Factor (C_d)

C_d = 55.784/37.772375 =1.48; with respect to equivalent static displacement and C_d = 55.784/15.359 = 3.63; with respect to linear time displacement. As per BNBC, the nonlinear time history to be done for more than 40 m in zone 2. 8 story is much less than 40 meters. Therefore, C_d value is much lower than specified value (4.5).

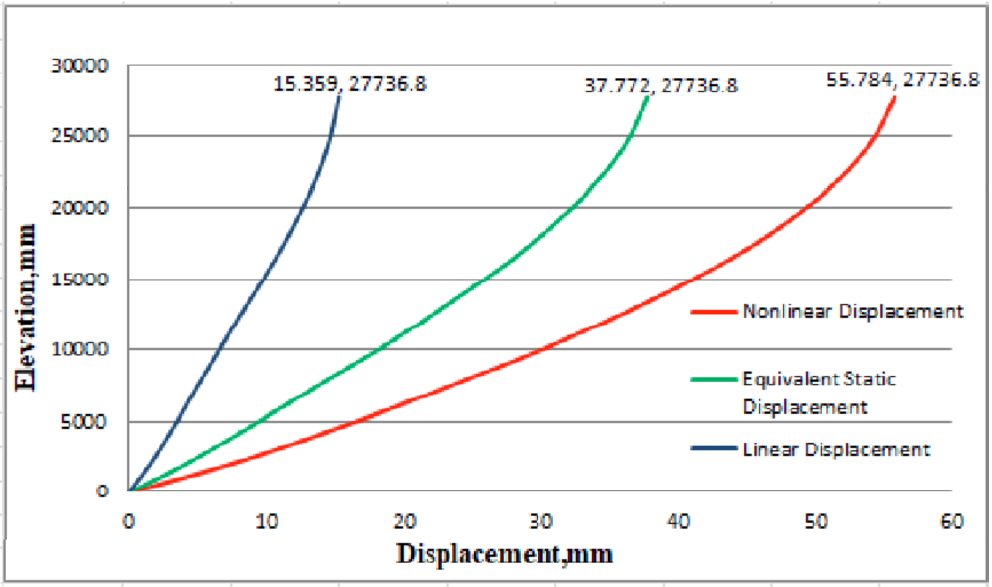


Figure 4.89. Maximum Story Displacement of 4 Bay 8 Story Frame

B. Story Drift

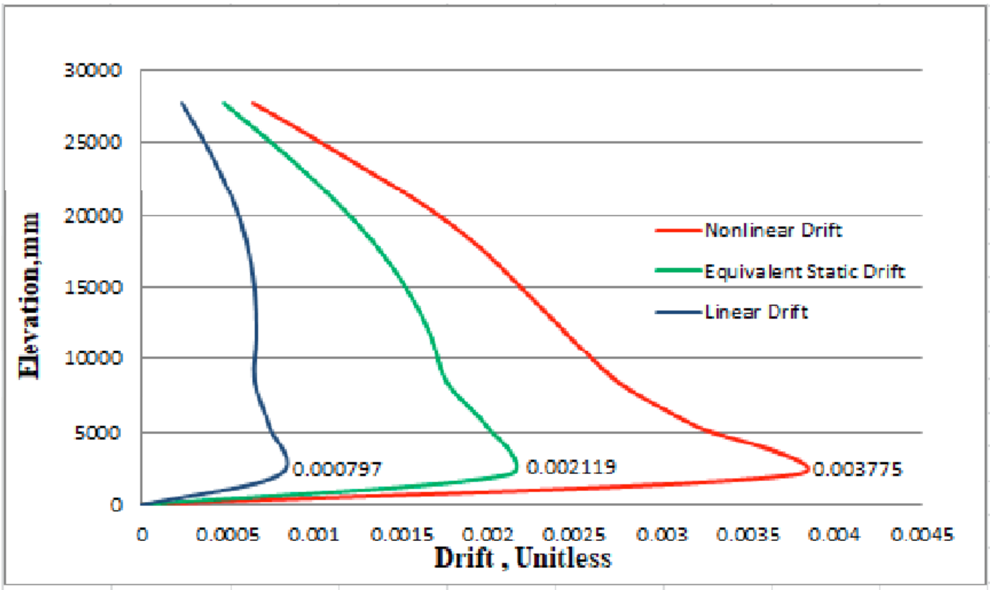


Figure 4.90. Maximum Story Drift of 4 Bay 8 Story Frame

As per BNBC, drift limit is 0.020. Here in this case both static linear and nonlinear drift is less than specified limit.

C. Relationship between Displacement and Load

Four Bay six story has maximum roof displacement of 55.8 mm where under load condition culumn has maximum displacement of 61.4 mm. The values are nearer since under load condition the value is of column.

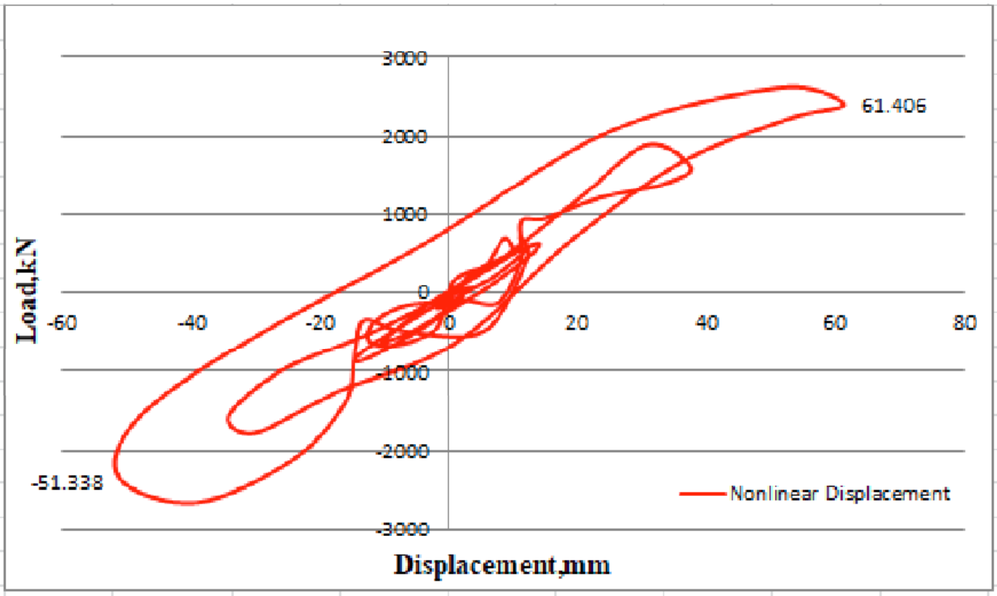


Figure 4.91. Relationship between Displacement and Load of 4 Bay 8 Story Frame

D. Hinge Result

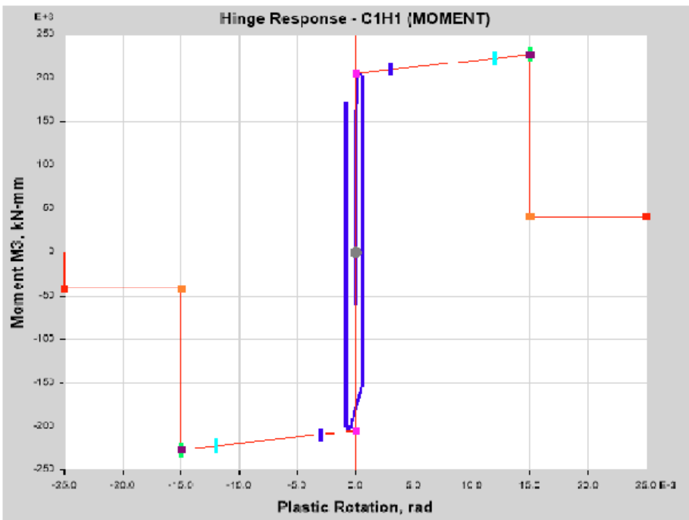


Figure 4.92. Hinge Result of 4 Bay 8 Story Frame

The hinge result (Figure 4.91) shows that it is in inelastic limit. It has gone to plastic zone. The Plastic rotation is having positive and negative value.

4.4.7 C_d and Drift Ratio of Four Bay Ten Story Frame (4-10)

The selected beam of 308mm×482.6mm (12.13in×19in) and column of 507mm×507mm (19.97 in×19.97in).

A. Deflection amplification factor (C_d)

C_d = 81.071/41.154261 = 1.97 with respect to equivalent static displacement. C_d = 81.071/19.843 = 4.09 with respect to linear time displacement.

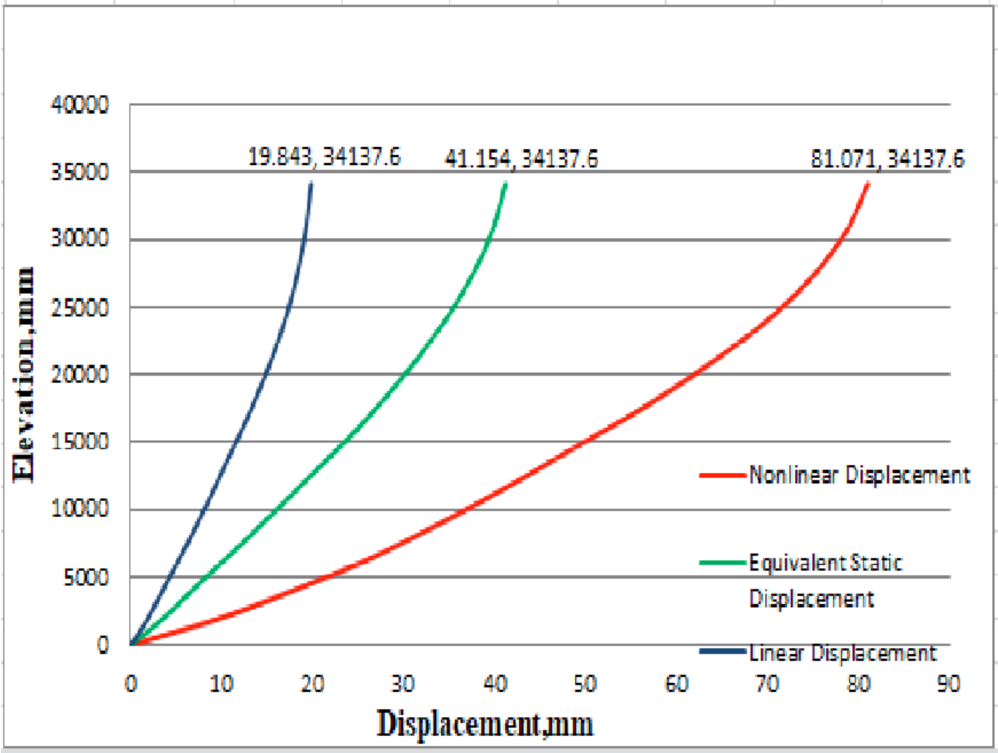


Figure 4.93. Maximum Story Displacement of 4 Bay 10 Story Frame

B. Story Drift

As per BNBC, drift limit is 0.020. Here in this case both static linear, linear and nonlinear drift is less than specified limit.

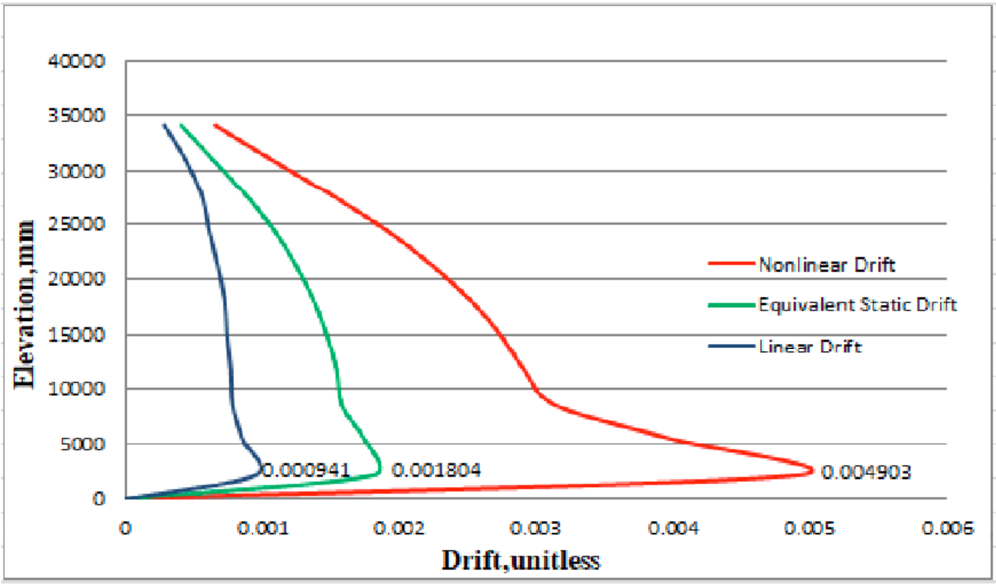


Figure 4.94. Maximum Story Drift of 4 Bay 10 Story Frame

C. Relationship between Displacement and Load

Four Bay six story has maximum roof displacement of 81 mm where under load condition culumn has maximum displacement of 75 mm. The values are nearer since under load condition the value is of column.

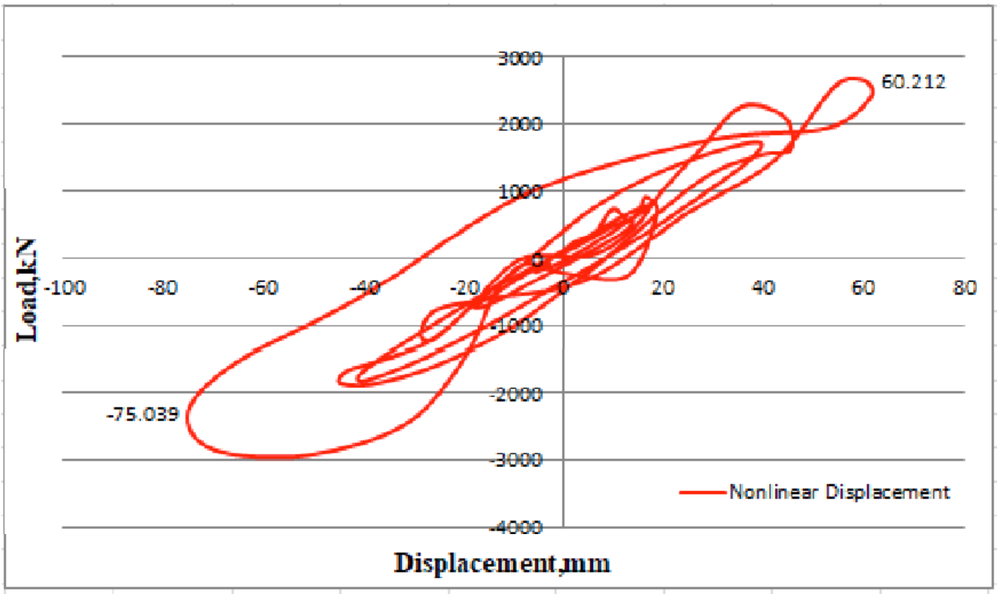


Figure 4.95. Relationship between Displacement and Load of 4 Bay 10 Story Frame

D. Hinge Result

The hinge result (Figure 4.96) shows that it is in inelastic limit. It has gone to plastic zone. The Plastic rotation is having positive and negative value

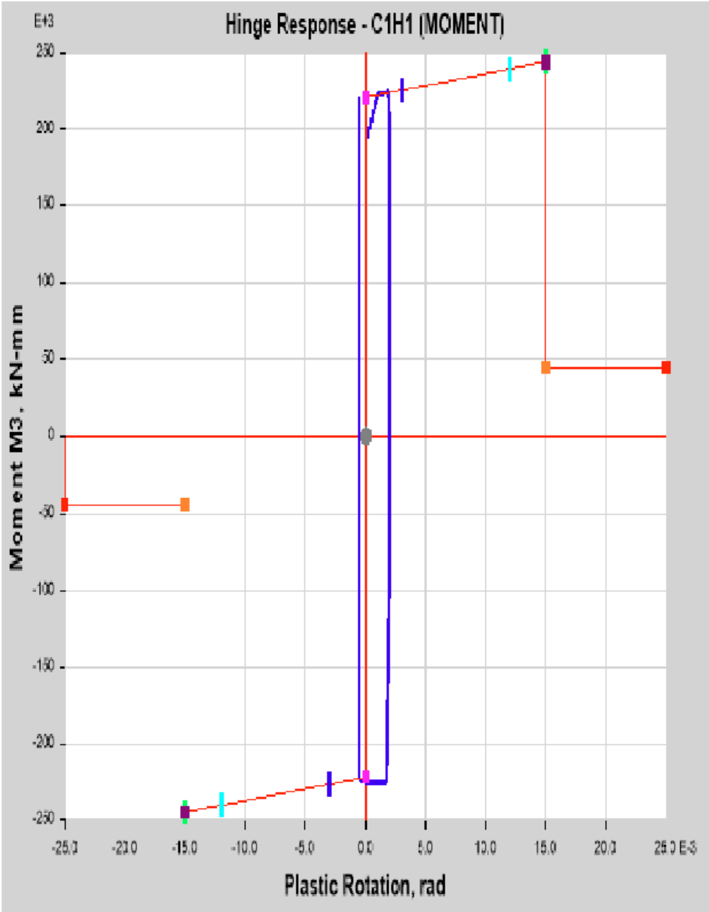


Figure 4.96. Hinge Result of 4 Bay 10 Story Frame

4.4.8 C_d and Drift Ratio for Four Bay Twelve Story Frame (4-12)

The selected beam of 312.4mm× 472.4m (12.30in× 18.60in) and column of 543.6mm×543.6mm (21.40in×21.40in). Linear and nonlinear time history analysis is done to evaluate the deflection amplification factor. Deflection amplification factor, Drift Vs Elevation, Displacement Vs Load and Hinge Result is shown below.

A. Deflection amplification factor (C_d)

The evaluated displacements are shown as first coordinate. C_d = 2.39; with respect to equivalent static displacement. C_d = 4.49; with respect to linear time displacement.

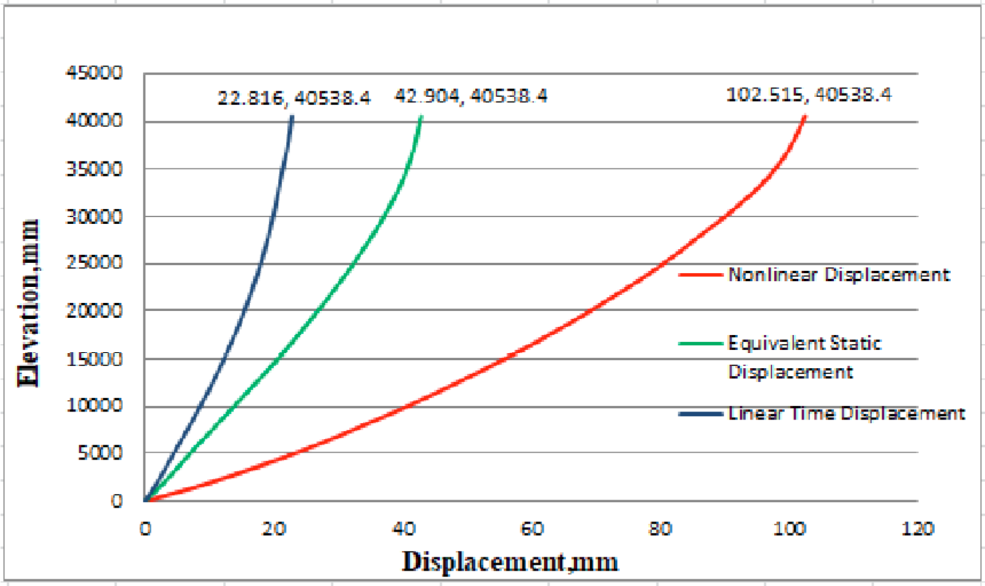


Figure 4.97. Maximum Story Displacement of 4 Bay 12 Story Frame

B. Story Drift

Evaluated drifts found are less than the drift limit 0.02.

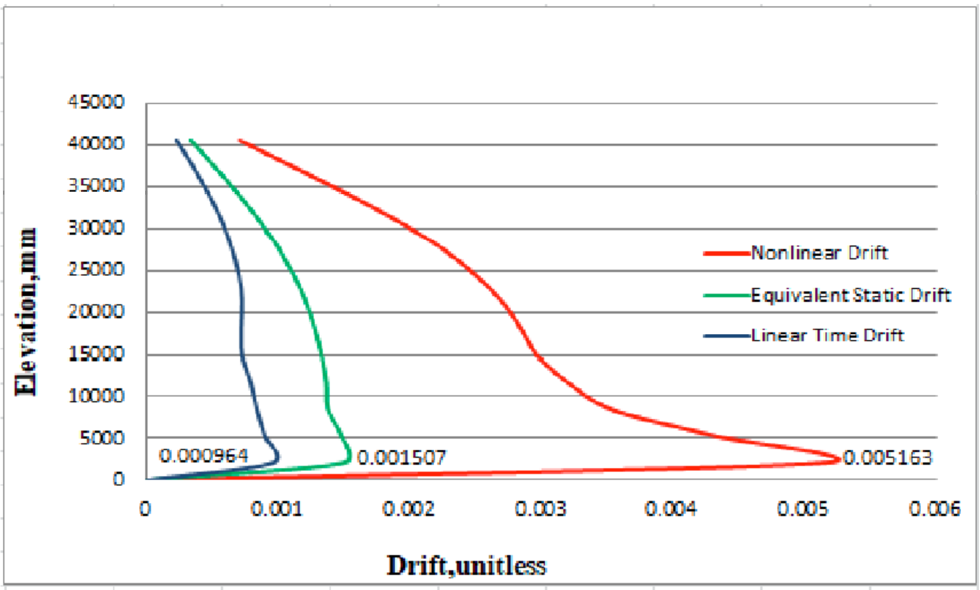


Figure 4.98. Maximum Story Drift of 4 Bay 12 Story Frame

C. Relationship between Displacement and Load

The nonlinear roof displacement is 102.5 mm and displacemnt of frame (column) under load is 95.3 mm.

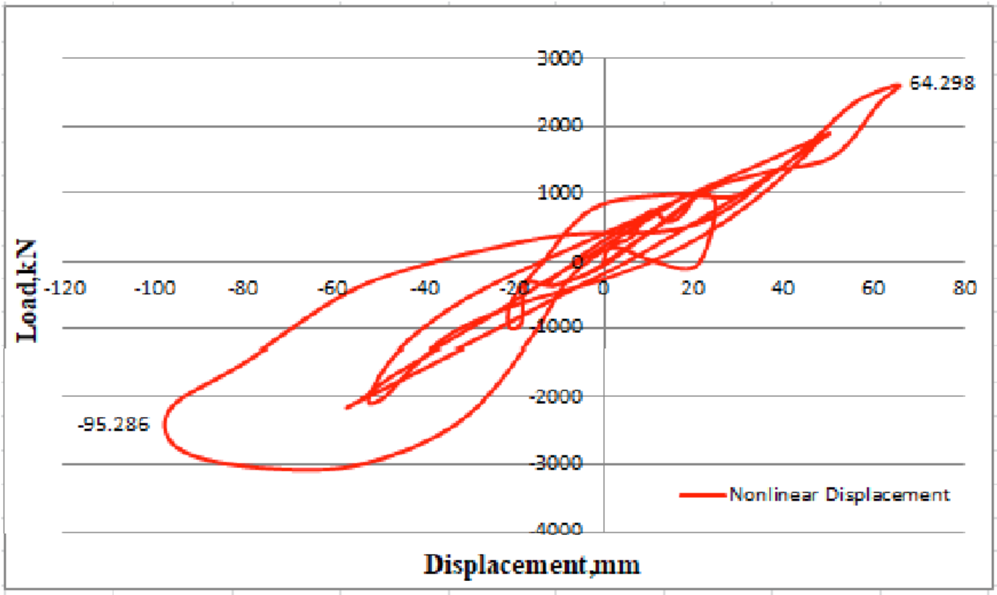


Figure 4.99. Relationship between Displacement and Load of 4 Bay 12 story frame

D. Hinge Result

The Load case is nonlineardynamic. The column is in ground floor. Hinge degree of freedom M3. Hinge relative distance is 0.1. The hinge has moved to plastic zone. Rotation is in positive direction.

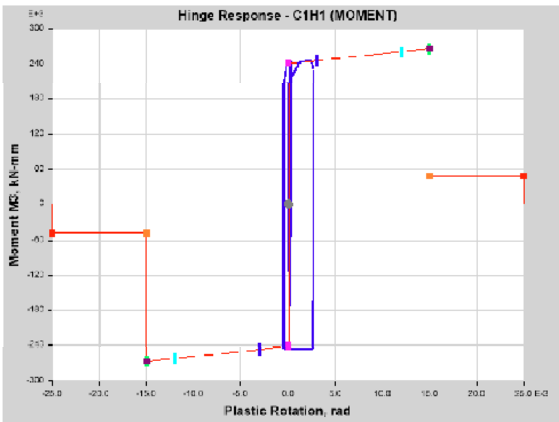


Figure 4.100. Hinge Result of 4 Bay 12 Story Frame

4.5 Evaluation of C_d for Sylhet (PGA = $0.36g$)

4.5.1 Two Bay Two Story in Sylhet (2-2)

The selected beam of 235.2mm×416.6mm (9.26in×16.4in) and column of 235.2mm×235.2mm (9.26in×9.26in).

A. Deflection amplification factor (C_d)

The evaluated nonlinear displacement is 51.497 mm,the equivalent static displacement is 17.455 mm and linear time displacement is 14.944 mm. Therefore deflection amplification is found to be 2.95 and 3.45 with respect to equivalent static displacemnt and linear time displacement respectively.

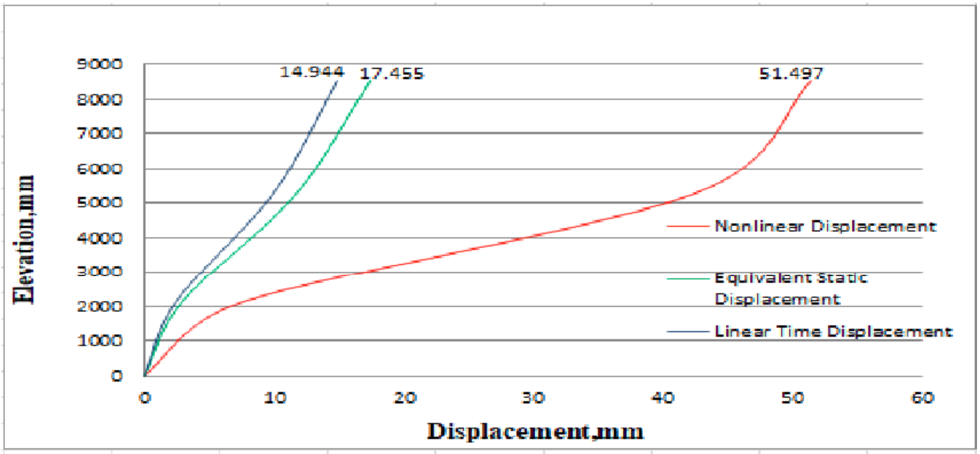


Figure 4.101. Relationship between Displacement and Load of 2 Bay 2 Story Frame

B. Hinge Result in Sylhet Region

The Load case is nonlineardynamic. The column is in ground floor. Hinge degree of freedom M3. Hinge relative distance is 0.1. The hinge has moved to plastic zone. Rotation is in positive and negative direction.

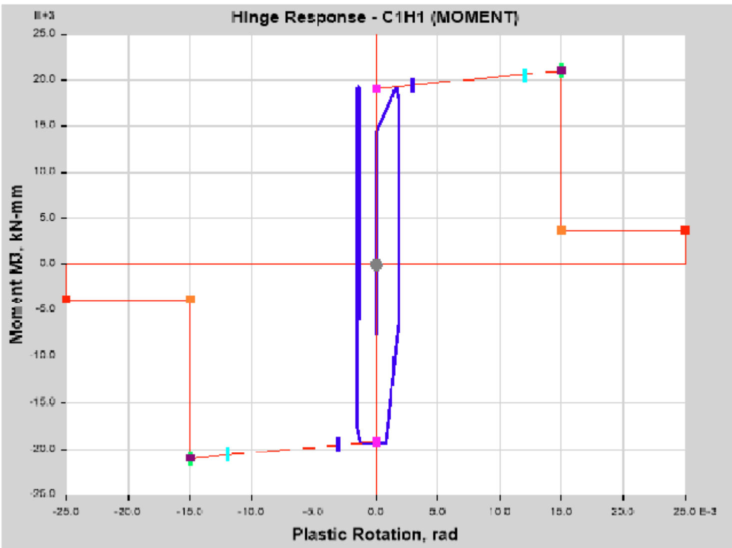


Figure 4.102. Hinge Result of 2 Bay 2 Story in Sylhet Region

4.6 Summary of Result (PGA = 0.2g)

4.6.1 Top Deflection and C_d Value (PGA value = 0.2g)

The top deflection and C_d value for two, three and four bay frames are shown below:

Table 4.1. Top Deflection and C_d Value of Two Bay Frames

Frame	Equivalent Displacement mm	Linear Displacement mm	Nonlinear Displacement mm	C_d with respect to Equivalent Displacement	C_d with respect to Linear time displacement
2 Bay 2 Story	17.5	8.302	27.77	1.59	3.35
2 Bay 3 Story	25.6	10.24	39.15	1.53	3.82
2 Bay 4 Story	34.7	12.106	47.9	1.38	3.95
2 Bay 5 Story	34.14	15.974	58.424	1.71	3.66
2 Bay 6 Story	49.8	13.14	51.8	1.04	3.94
2 Bay 8 Story	53.33	14.96	63.3	1.19	4.23
2 Bay 10 Story	63.2	18.618	78.29	1.24	4.21
2 Bay 12 Story	70.366	25.202	115.186	1.64	4.57

Table 4.2. Top Deflection and C_d Value of Three Bay Frames

Frame	Equivalent Displacement mm	Linear Displacement mm	Nonlinear Displacement mm	C_d with respect to Equivalent Displacement	C_d with respect to Linear time displacement
3 Bay 2 Story	18.839	8.883	25.956	1.38	2.92
3 Bay 3 Story	21.87	9.229	27.994	1.28	3.03
3 Bay 4 Story	26.312	8.892	30.447	1.16	3.42
3 Bay 5 Story	29.795	10.925	33.272	1.12	3.05
3 Bay 6 Story	32.404	10.792	39.014	1.20	3.62
3 Bay 8 Story	37.717	15.198	55.103	1.46	3.63
3 Bay 10 Story	40.537	18.725	78.931	1.95	4.22
3 Bay 12 Story	42.422	22.561	100.357	2.37	4.45

Table 4.3. Top Deflection and C_d Value of Four Bay Frames

Frame	Equivalent Displacement mm	Linear Displacement mm	Nonlinear Displacement mm	C_d with respect to Equivalent Displacement	C_d with respect to Linear Time Displacement
4 Bay 2 Story	19.658	9.091	26.506	1.35	2.92
4 Bay 3 Story	22.096	9.271	28.129	1.27	3.034
4 Bay 4 Story	24.977	8.879	29.72	1.19	3.35
4 Bay 5 Story	29.112	11.243	32.892	1.13	2.93
4 Bay 6 Story	30.415	11.933	37.207	1.22	3.12
4 Bay 8 Story	37.772	15.359	55.784	1.48	3.63
4 Bay 10 Story	41.154	19.843	81.071	1.97	4.09
4 Bay 12 Story	42.904	22.816	102.515	2.39	4.49

4.6.2 Story Drift (PGA Value=0.2g)

The story drift of two, three and four bay frames are shown below:

Table 4.4. Story Drift of Two Bay Frames

Frame	Equivalent Drift	Linear Drift	Nonlinear Drift	Drift Ratio with respect to Equivalent Drift	Drift Ratio with respect to Linear Drift
2 Bay 2 Story	0.0028	0.0013	0.0053	1.89	4.08
2 Bay 3 Story	0.0031	0.0013	0.0089	2.87	6.85
2 Bay 4 Story	0.0033	0.0012	0.0051	1.55	4.25
2 Bay 5 Story	0.0032	0.0013	0.0046	1.44	3.54
2 Bay 6 Story	0.0035	0.0010	0.0039	1.11	3.9
2 Bay 8 Story	0.0028	0.0008	0.0033	1.18	4.125
2 Bay 10 Story	0.0025	0.0008	0.0035	1.4	4.4
2 Bay 12 Story	0.0022	0.0008	0.0038	1.73	4.75

Table 4.5. Story Drift of Three Bay Frames

Frame	Equivalent Drift	Linear Drift	Nonlinear Drift	Drift Ratio with respect to Equivalent Drift	Drift Ratio with respect to Linear Drift
3 Bay 2 Story	0.0032	0.0016	0.0045	1.4	2.8
3 Bay 3 Story	0.0028	0.0016	0.0037	1.32	2.3
3 Bay 4 Story	0.0026	0.0009	0.0032	1.23	3.56
3 Bay 5 Story	0.0024	0.0009	0.0028	1.17	3.11
3 Bay 6 Story	0.0022	0.0008	0.0028	1.27	3.5
3 Bay 8 Story	0.0019	0.0007	0.0031	1.63	4.43
3 Bay 10 Story	0.0016	0.0008	0.0037	2.3	4.6
3 Bay 12 Story	0.0014	0.0008	0.0040	2.86	5

Table 4.6. Story Drift of Four Bay Frames

Frame	Equivalent Drift	Linear Drift	Nonlinear Drift	Drift Ratio with respect to Equivalent Drift	Drift Ratio with respect to Linear Drift
4 Bay 2 Story	0.0036	0.0018	0.0050	1.39	2.8
4 Bay 3 Story	0.0031	0.0014	0.0041	1.3	2.9
4 Bay 4 Story	0.0028	0.0011	0.0035	1.25	3.2
4 Bay 5 Story	0.0026	0.0010	0.0031	1.2	3.1
4 Bay 6 Story	0.0023	0.0009	0.0030	1.3	3.3
4 Bay 8 Story	0.0021	0.0008	0.0038	1.81	4.75
4 Bay 10 Story	0.0018	0.0009	0.0049	2.72	5.44
4 Bay 12 Story	0.0015	0.0009	0.0052	3.5	5.8

4.6.3 Building Height Vs C_d of 24 Frames (PGA value 0.2g)

The building height Vs C_d graph with respect to equivalent static displacement is shown below:

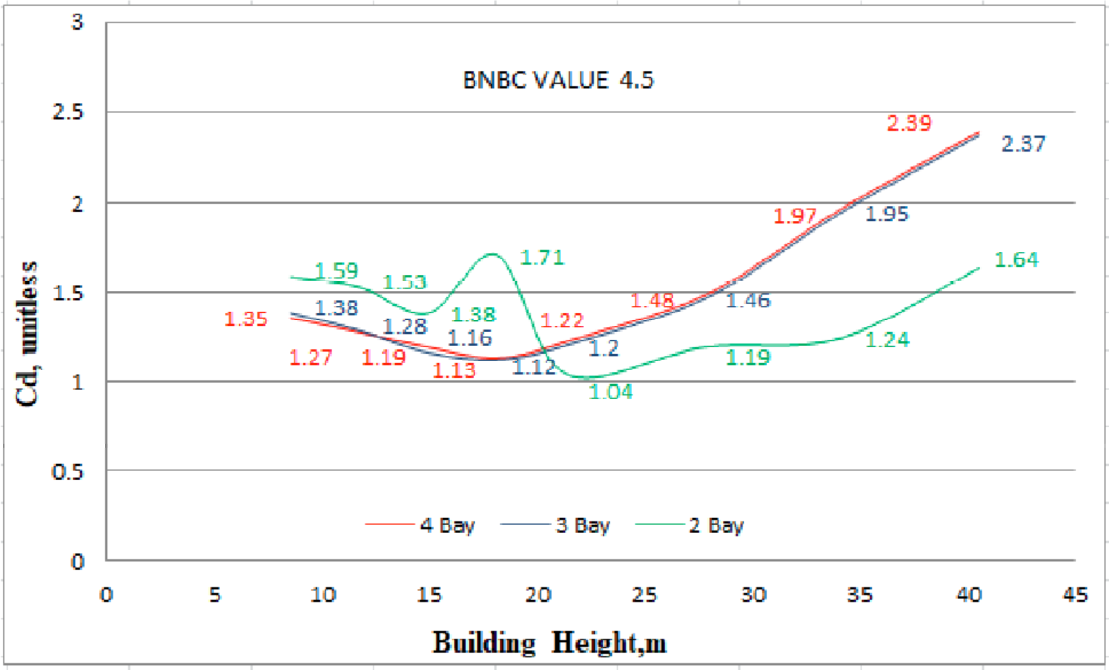


Figure 4.103. Relationship between Building Height and Equivalent Static C_d

The building height Vs C_d graph with respect to Linear Time displacement is shown below:

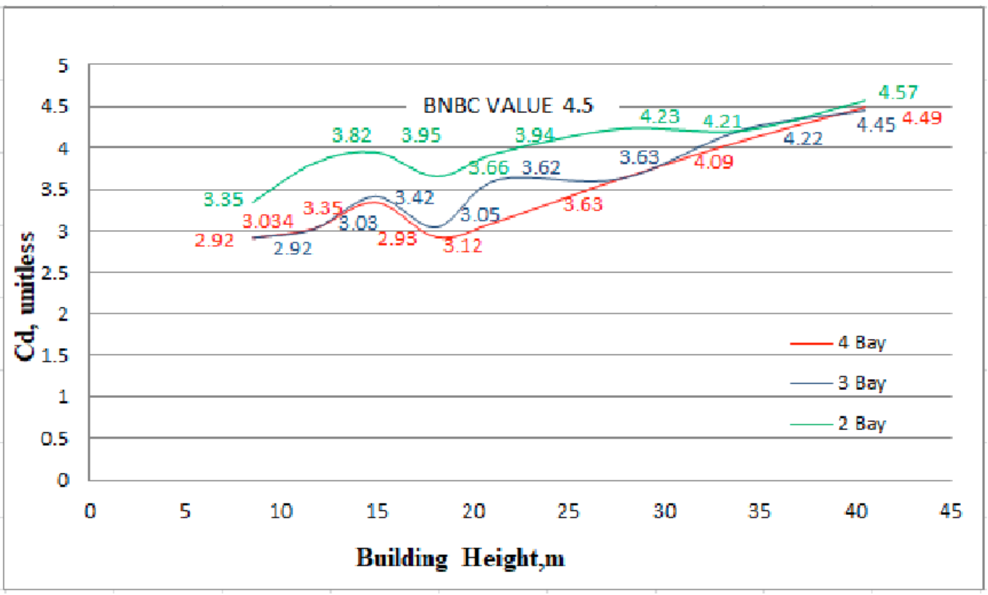


Figure 4.104. Relationship between building height and Linear Time C_d

4.6.4 Building Height Vs Drift Ratio of 24 Frames (PGA value 0.2g)

The building height Vs Drift ratio graph with respect to equivalent static drift is shown below:

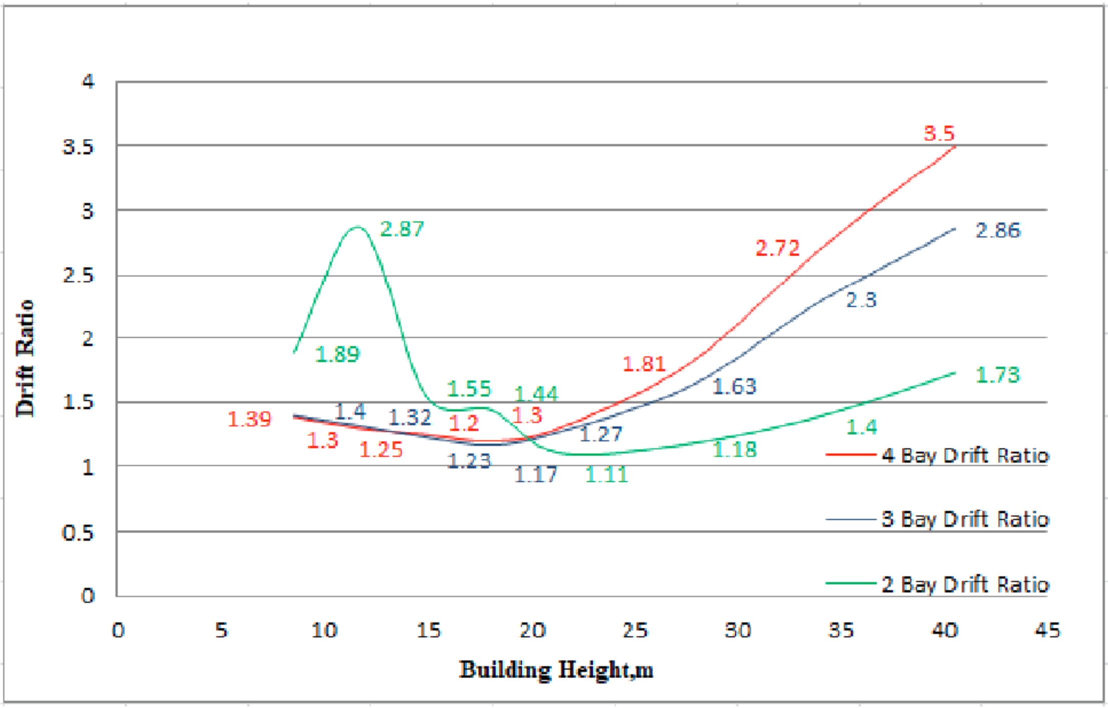


Figure 4.105. Relationship between Building Height and Equivalent Drift Ratio

The building height Vs Drift ratio graph with respect to linear time drift is shown below:

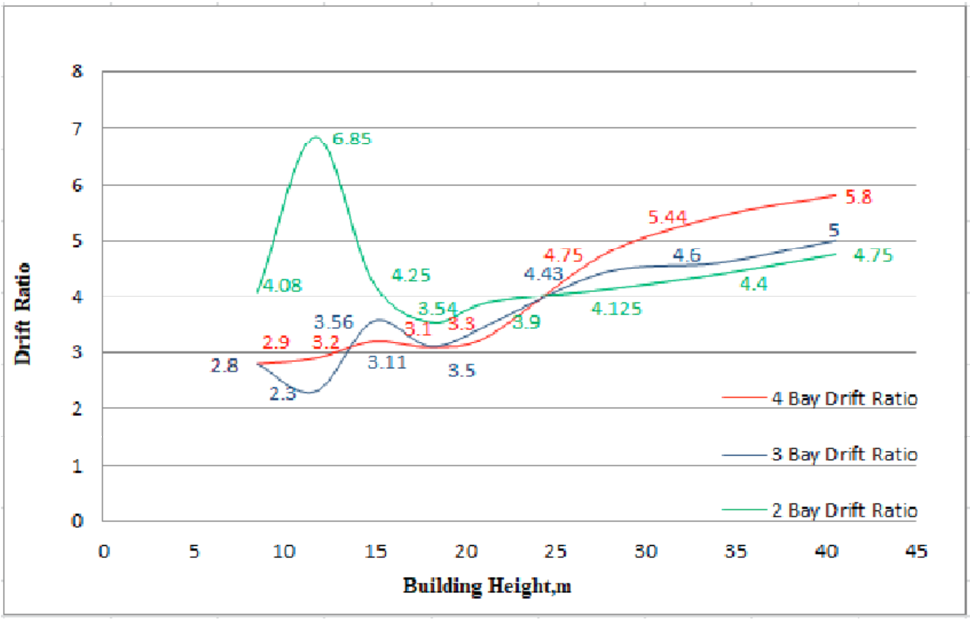


Figure 4.106. Relationship between Building Height and Linear Time Drift ratio

4.6.5 Peak Ground Acceleration Vs C_d of Two Bay Two Story Frame

C_d value is increased when acceleration is increased.

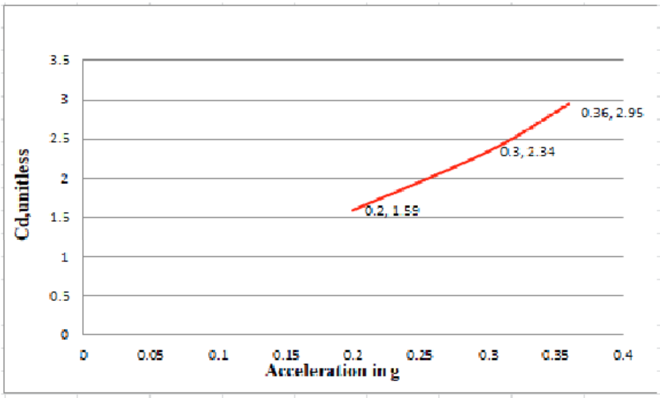


Figure 4.107. Relationship between Peak Ground Acceleration and C_d

4.6.6 Displacement % Vs Story

A. Three Bay Two Story

Percentage of Displacement divided by story height Vs Story is drawn for nonlinear displacement, equivalent static displacement and linear time displacement. Graph shows that displacement increases linearly once story height is increased.

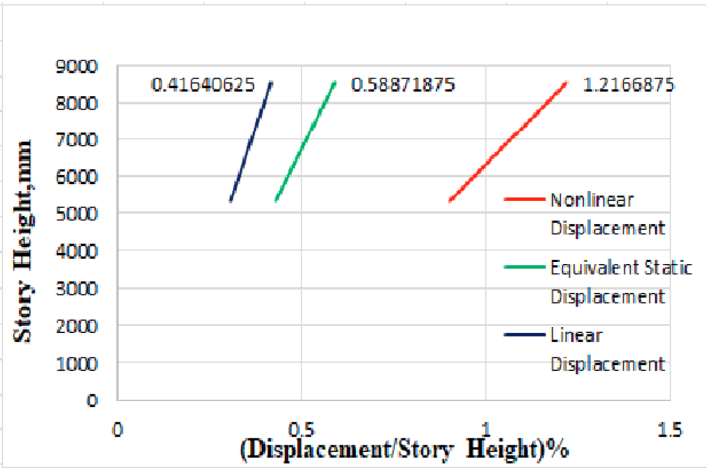


Figure 4.108. Relationship between (Displacement / Story height) % and Height

B. Three Bay Ten Story

Percentage of Displacement divided by story height Vs Story is drawn for nonlinear displacement, equivalent static displacement and linear time displacement. Graph shows that displacement increases once story height is increased.

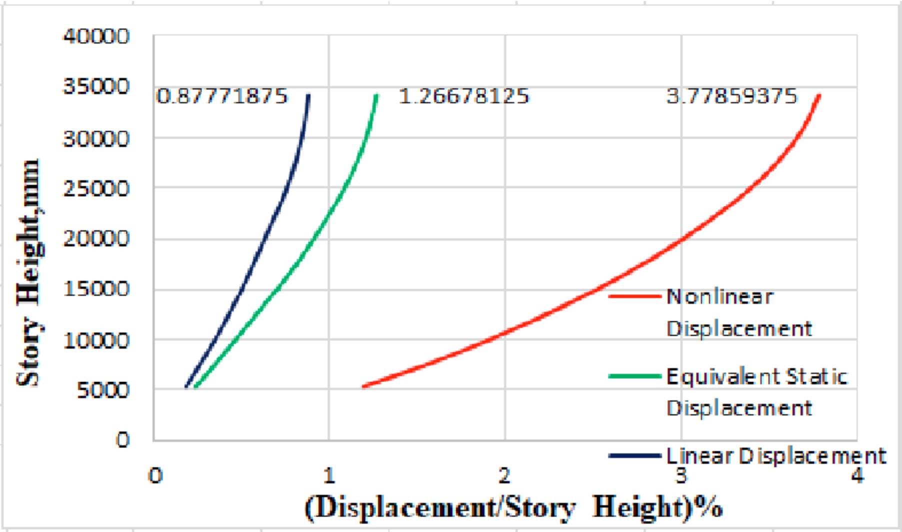


Figure 4.109. Relationship between (Displacement / Story height) % and Height

4.7 Categorization as Per Result with Respect to Equivalent Static (PGA = 0.3g)

Table 4.7. Height Above 40 meters

Sl. No.	Description of Structure	Value of C_d
1	Four Bay Twelve Story	4.1
2	Three Bay Twelve Story	3.9
3	Two Bay Twelve Story	2.9

Table 4.8. Height below 40 meters but Structure Undergoes Plastic Deformation

Sl. No.	Description of Structure	Value of C_d
1	Four Bay Three Story	1.9
2	Four Bay Four Story	1.8
3	Four Bay Five Story	1.9
4	Four Bay Six Story	1.8
5	Four Bay Eight Story	2.3
6	Four Bay Ten Story	3.03
7	Three Bay Four Story	1.7
8	Three Bay Five Story	1.7
9	Three Bay Six Story	1.8
10	Three Bay Eight Story	2.2
11	Three Bay Ten Story	2.9
12	Two Bay Two Story	2.34
13	Two Bay Three Story	1.8
14	Two Bay Four Story	1.6
15	Two Bay Five Story	1.6
16	Two Bay Six Story	1.2
17	Two Bay Eight Story	1.42
18	Two Bay Ten Story	1.9

Table 4.9. Height Below 40 meters but Structures Remain Elastic

Sl. No.	Description of Structure	Value of C_d
1	Four Bay Two Story	2.1
2	Three Bay Two Story	2.1
3	Three Bay Three Story	1.9

4.8 Story Height Vs Inelastic Displacement Ratio (PGA = 0.3g)

4.8.1 Two Bay

The figure shows that as story height goes up inelastic displacement ratio also increases except less than 20 m.

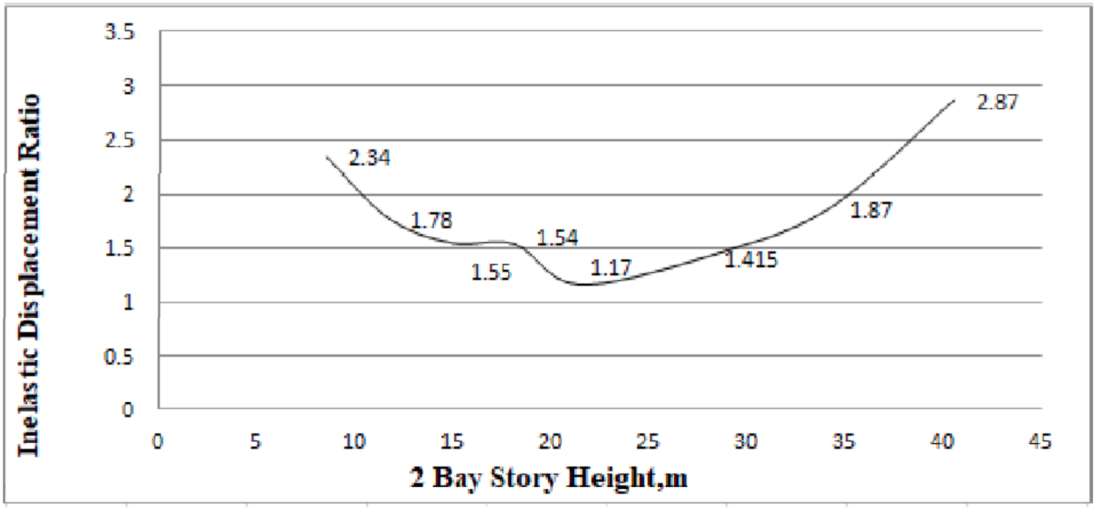


Figure 4.110. Relationship between Story Height and C_d of 2 bay

4.8.2 Three Bay

The figure shows that as story height goes up inelastic displacement ratio also increases except less than 10 m.

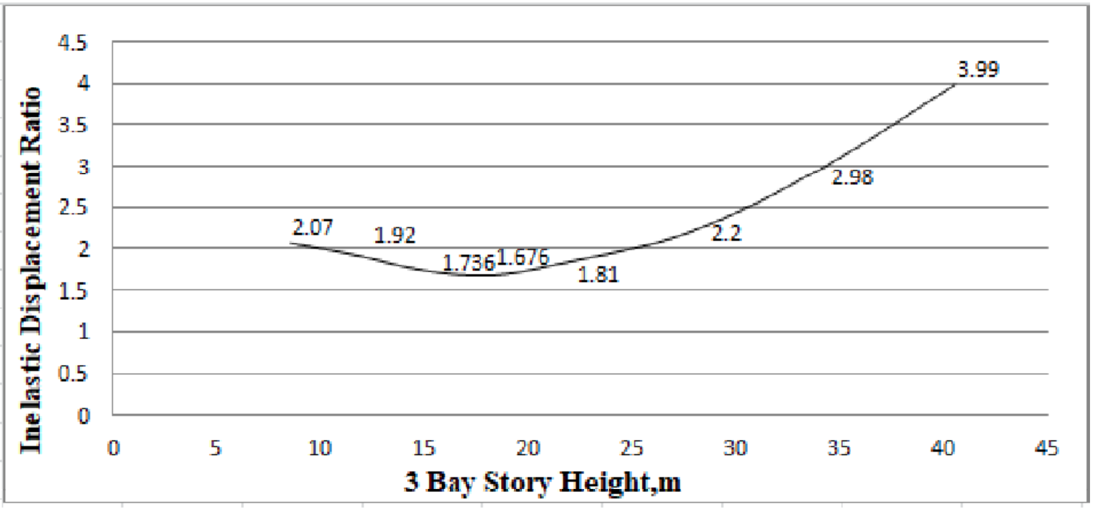


Figure 4.111. Relationship between Story Height and C_d of 3 Bay

4.8.3 Four Bay

The figure shows that as story height goes up inelastic displacement ratio also increases except less than 10 m.

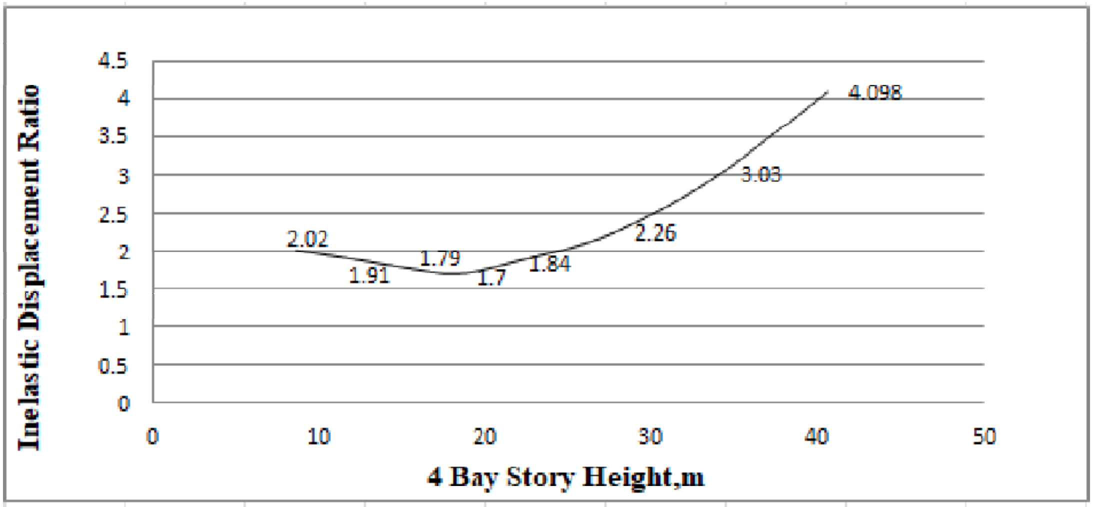


Figure 4.112. Relationship between Story Height and C_d of 4 Bay

4.9 Building Height Vs C_d (PGA = 0.3g)

Building height vs C_d graph (Figure Shown below) shows that increasing story heights and number of stories lead to increased deflection amplification factor (C_d) irrespective of all the bays. But from 10 meter to 20-meter height, deflection amplification factor decreases. The reason is that since base is fixed, the ground motion hit at the base, therefore first attack comes on to the ground floor and nearby floors. Thereby failure from maximum inelastic displacement is very closer from 2nd to 5th floor. It is evident from the study that at 10 m height (2 to 3 Story) normal structured building is very stable compared to four to six story (10m to 20 m) normal structured building. Demand capacity ratio is less in 2 story than 4 story which indicates that two story has more capacity.

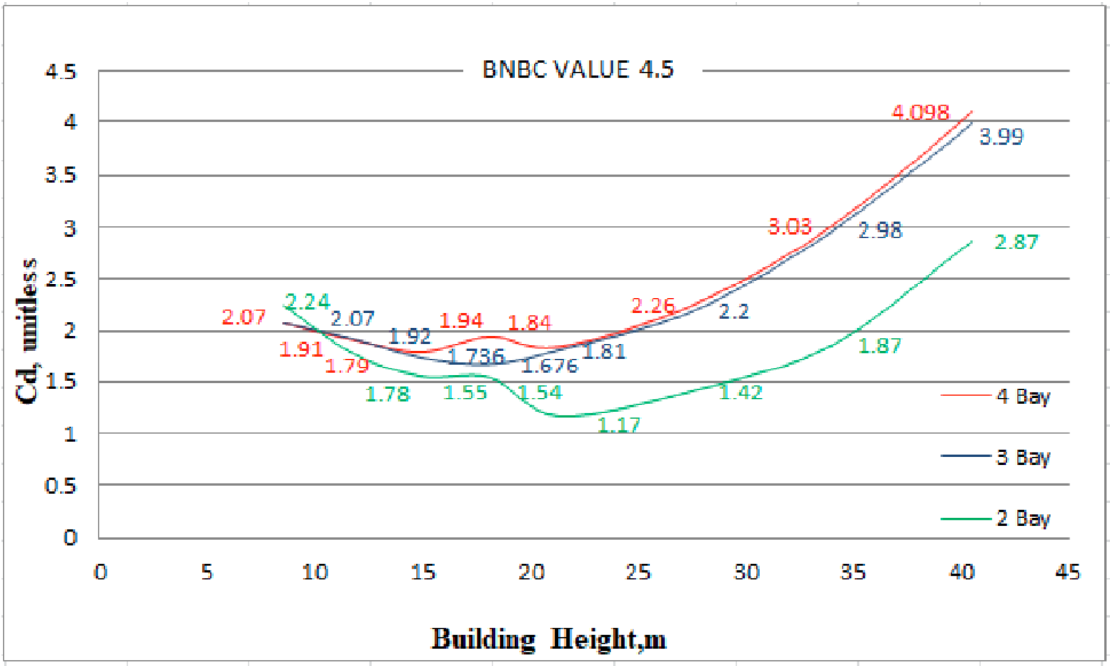


Figure 4.113. Relationship between Building Height and Equivalent Static Cd

4.10 Building Height Vs Maximum Inelastic to Elastic Drift Ratio (PGA = 0.3g)

It can be observed from building height vs Maximum inelastic/elastic drift ratio (figure shown below) that the ratio is increased by increasing the story number. Except that for 2 Bay frame ratios is initially decreased from 10m to 20m. From 20 m onward the ratio is increased irrespective of all bays. From comparing the trend of the ratio of the studied RC frames, it can be concluded that by increasing the frame height, the slope of the graphs is decreased along the structural height towards the top stories. In addition, it is observed that the ratio is not significantly affected by the number of bays up to 20 m.

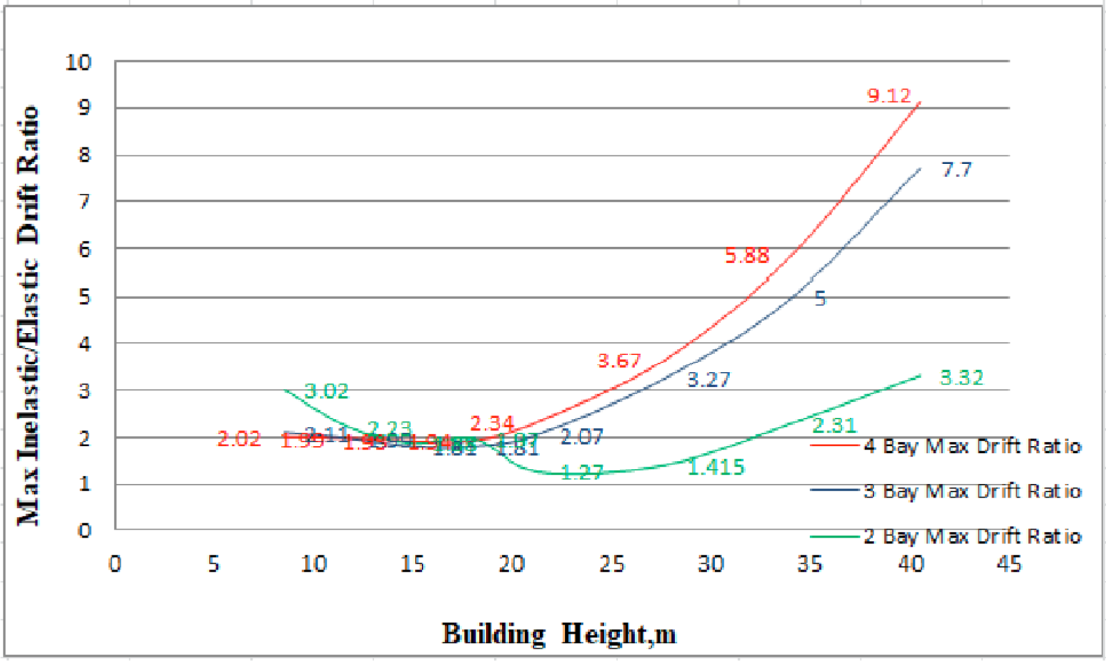


Figure 4.114. Relationship between Building Height and Inelastic Drift Ratio

4.11 Variation Coefficient of Inelastic to Elastic Interstory Displacement Ratio of Four Bay Twelve Story Frame (PGA=0.3g)

The number of frame stories once increased, the difference between the values of inelastic displacement ratios in adjacent stories are decreased and generally, the variation coefficient of these ratios is computed larger for the low to mid-rise structures than for the high-rise frames when the number of stories is more than three.

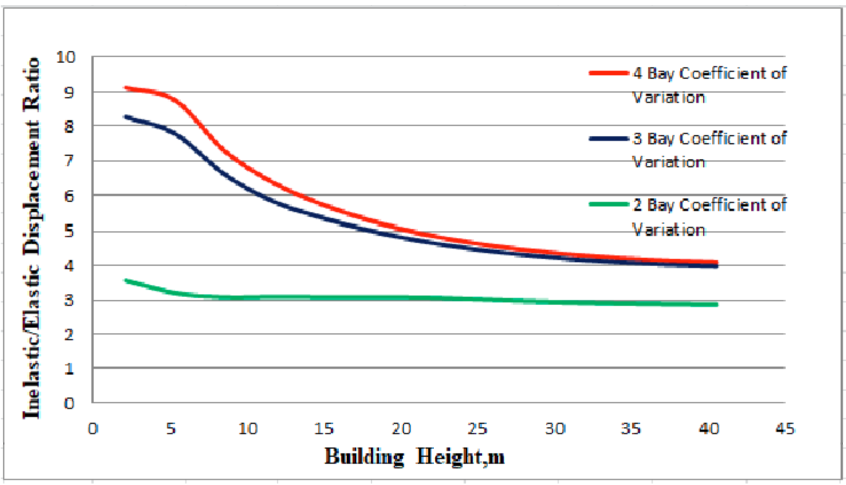


Figure 4.115. Variation Coefficient of Inelastic to Elastic Displacement Ratio

4.12 Variation Coefficient of Inelastic to Elastic Interstory Drift Ratio of Four Bay Twelve Story Frame (PGA = 0.3g)

The number of frame stories once increased, the difference between the values of inelastic drift ratios in adjacent stories are decreased and generally, the variation coefficient of these ratios is computed larger for the low to mid-rise structures than for the high-rise frames when the number of stories is more than three.

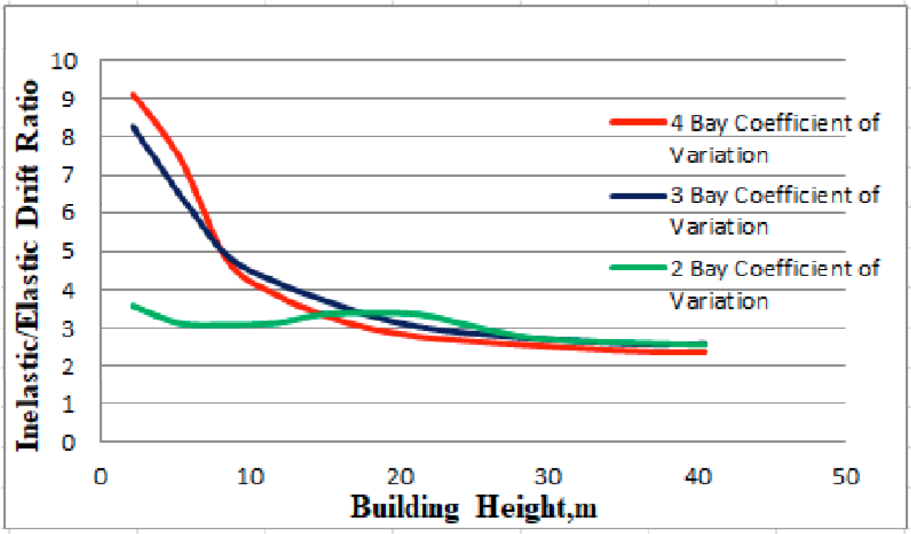


Figure 4.116. Variation Coefficient of Inelastic to Elastic Interstory Drift Ratio

4.13 Summary of C_d with respect to Equivalent Static Displacement and Storey Drift Characteristics (PGA = 0.3g)

Table 4.10. Summary of Inelastic Displacement Ratios and Storey Drift Characteristics

Frame	Analysis	Top Deflection (mm)	Story Drift	Inelastic Displacement Ratio
4 Bay 12 Story	Nonlinear Time History	110.13	0.008834	4.098
	Equivalent Static	26.873	0.000968	
4 Bay 10 Story	Nonlinear Time History	124.788	0.010601	3.03
	Equivalent Static	41.154	0.001804	

4 Bay 8 Story	Nonlinear Time History	85.494	0.00778	2.26
	Equivalent Static	37.772	0.002119	
4 Bay 6 Story	Nonlinear Time History	56.01	0.005339	1.84
	Equivalent Static	30.415	0.002287	
4 Bay 5 Story	Nonlinear Time History	49.443	0.00512	1.7
	Equivalent Static	29.112	0.002643	
4 Bay 4 Story	Equivalent Static	44.671	0.005402	1.79
	Nonlinear Time History	24.977	0.002801	
4 Bay 3 Story	Nonlinear Time History	42.229	0.006231	1.91
	Equivalent Static	22.096	0.003138	
4 Bay 2 Story	Nonlinear Time History	39.769	0.007563	2.02
	Equivalent Static	19.658	0.003652	
3 Bay 12 Story	Nonlinear Time History	103.313	0.006398	3.99
	Equivalent Static	25.888	0.000832	
3 Bay 10 Story	Nonlinear Time History	120.915	0.008206	2.98
	Equivalent Static	40.537	0.001638	
3 Bay 8 Story	Nonlinear Time History	82.916	0.006366	2.2
	Equivalent Static	37.717	0.001942	
3 Bay 6 Story	Nonlinear Time History	58.578	0.004579	1.81
	Equivalent Static	32.404	0.002192	
3 Bay 5 Story	Nonlinear Time History	49.935	0.004383	1.676
	Equivalent Static	29.795	0.002418	
3 Bay 4 Story	Nonlinear Time History	45.682	0.004768	1.736
	Equivalent Static	26.312	0.002632	
3 Bay 3 Story	Nonlinear Time History	41.99	0.005593	1.92
	Equivalent Static	21.87	0.002837	
3 Bay 2 Story	Nonlinear Time History	38.934	0.006694	2.07
	Equivalent Static	18.839	0.003179	
2 Bay 12	Nonlinear Time	173.152	0.006497	2.87

Story	History			
	Equivalent Static	60.388	0.001956	
2 Bay 10 Story	Nonlinear Time History	118.099	0.005895	1.87
	Equivalent Static	63.2	0.002554	
2 Bay 8 Story	Nonlinear Time History	75.478	0.00496	1.415
	Equivalent Static	53.332	0.002781	
2 Bay 6 Story	Nonlinear Time History	58.199	0.004473	1.17
	Equivalent Static	49.789	0.003521	
2 Bay 5 Story	Nonlinear Time History	74.167	0.00739	
	Equivalent Static	48.035	0.03758	
2 Bay 4 Story	Nonlinear Time History	53.85	0.006234	1.55
	Equivalent Static	34.72	0.003313	
2 Bay 3 Story	Nonlinear Time History	45.497	0.006903	1.78
	Equivalent Static	25.61	0.00309	
2 Bay 2 Story	Nonlinear Time History	40.808	0.008468	2.34
	Equivalent Static	17.455	0.002807	

4.14 Base Shear, Weight, Inelastic Displacement Ratios, and Storey Drift of Two Frames (PGA=0.3g)

Table 4.11. Base Shear, Weight, Inelastic Displacement Ratios, and Storey Drift of Two Frames

Frame	Analysis	Top Deflection (mm)	Story Drift	Base Shear, V (KN)	Total Weight, W (KN)	(V/W) %	Axial Load on Column (KN)	Column End, Rebar (%)	Column and Beam Size(in)	Inelastic Displacement Ratio
4 Bay 12 Story	Nonlinear Time History	110.13	0.008834	3728	50645	2.7	58174	0.82	21.75×21.75 and 12.30×24.75	4.098
	Equivalent Static	26.873	0.000968	1368						
4 Bay 10 Story	Nonlinear Time History	124.788	0.010601	2936	39310	3.7	45572	0.98	19.97×19.97 and 12.13×19	3.03

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 General

In this study the deflection amplification factor of Reinforced Concrete frames has been investigated and compared with values suggested in BNBC. For this, 24 frames were modelled in finite element software ETABS. These 24 frames were varied with respect to story height and bay configuration. Eight different story from 2 to 12 were considered in this work. Three types of bay arrangements were used, i.e., two by two bay, three by three bay and four by four bay.

The buildings were first designed using equivalent static analysis as per BNBC. From that beam and column sizes were determined. From this static analysis of 24 frames, maximum top deflection and storey drift were recorded. Afterwards, nonlinear dynamic analysis of these frames was conducted. For this, imperial valley earthquake data was used. This earthquake data was scaled to fit the BNBC recommendation for Dhaka zone, i.e., peak ground acceleration was seated to $0.2g$. For nonlinearity of members i.e., beams and columns, isotropic hysteresis model which is a built-in function of ETABS was used. From the nonlinear analysis, the top deflection, story drift and hinge results were recorded.

5.2 Conclusions

Following Conclusions may be drawn based on the study:

(i) For 12 storied building, nonlinear displacement was found 115.2 mm, 100.4mm and 102.5 mm for 2,3 and 4-bay building respectively. Whereas for 8 Story building had displacement was of 63.3 mm, 55.1 mm and 55.8 mm for 2, 3 and 4-bay building respectively. For 6 storied building, nonlinear displacement was found 51.8 mm, 39 mm and 37.2 mm for 2,3 and 4-bay building respectively. For 4 storied building, nonlinear displacement was found 47.9 mm, 30.4 mm and 29.7 mm for 2,3 and 4-bay building respectively.

(ii) As storey was reduced, top nonlinear displacement was found to decrease whereas as number of bays was increased, nonlinear displacement was found to decrease.

(iii) For 12 storied building, linear displacement was found 25.2 mm, 22.6 mm and 22.8 mm for 2,3 and 4-bay building respectively. Whereas for 8 Story building, top displacement reduced to 14.9 mm, 15.2 mm and 15.4 mm for 2, 3 and 4-bay building respectively. For 6 storied building, linear displacement was found 13.14 mm, 10.8 mm and 11.9 mm for 2,3 and 4-bay building respectively. For 4 storied building, linear displacement was found 12.1 mm, 8.9 mm and 8.9 mm for 2, 3 and 4-bay building respectively.

(iv) As storey height was reduced, top linear displacement was found to decrease. Further, as number of bays was increased, linear displacement was found to decrease.

(v) Nonlinear storey drift values at 2,4,6,10 and 12 storey level were 0.003536, 0.002965, 0.0027, 0.001521 and 0.000711 respectively for four bay 12 storey building. Linear storey drift values at 2,4,6,10 and 12 storey level were 0.000842, 00073, 00073, 00047 and 0.000227 respectively for four bay 12 storey building. As can be seen, when non linearity was considered, storey drift was found to be significantly higher. For building with lower storey considered in this work, similar increase in storey drift value was found for non linear case. These values may provide a better insight into building behavior for structural designer and researcher in this area.

(vi) C_d value was estimated from non-linear and linear top displacement found from the respective analysis. For 12,10 and 8 storied building had C_d value ranged from 4.57 to 4.23, 4.5 to 3.63 and 4.5 to 3.63 for 2,3 and 4-bay building respectively. BNBC specify a flat C_d value of 4.5 for buildings of all configuration. However, from this analysis work, it is evident that $C_d$value is very much dependent on building height, number of bays etc. For most of the cases analyzed in this work, C_d value was found to be much lower than BNBC specified valued. However, for 12 storied building, C_d was found to have value slightly more than 4.5 for 2 bay configuration.

(vii) C_d values suggested in this work may be useful for practicing engineers and researchers to estimate ultimate displacement of buildings due to severe earthquake from linear analysis of building for different regions of Bangladesh.

5.3 Recommendations for Future Research

A. Earthquake data base for Bangladesh is not available. This database could be available in any engineering institution after equalizing data of other country.

B. Nonlinear time history analysis is necessary to varify structutes' stability. But detail procedure is not available in BNBC. Which can be incorporated in subsequet version of BNBC.

REFERENCES

ACI 318 (2008) *Building Code Requirements for Structural Concrete (ACI 318M-08) and Commentary*. ACI Committee 318, American Concrete Institute, Michigan.

Amin, M. R., (2011) *Nonlinear Seismic Analysis of Multi-Storied Reinforced Concrete Buildings with Masonry Infill*, M.Sc. Engg. Thesis, Department of Civil Engineering, Bangladesh University of Engineering and Technology.

Astrid, R. F., Julian, C., and Hugo, H.B., "Analysis of the Earthquake Resistant Design Approach for Buildings in Mexico," Engineering, Research and Technology., vol. 15 (Issue 1), pp 151-162, 2014.

BNBC (2020) *Bangladesh National Building Code*. Housing and Building Research Institute, Dhaka.

Chopra,A.K., (2012), Dynamics of Structures – Theory and Application to Earthquake Engineering, Fourth Edition, Prentice Hall Inc, USA

Clough, R. W., (1966) "Effect of Stiffness Degradation on Earthquake Ductility Requirements," Structural and Materials Research, Structural Engineering Laboratory, Univ. of California, Berkeley, Report 66-16.

CSi (2015) *ETABS: User Interface Reference Manual*. Computers and Structures, Inc., Berkeley.

Kim, J., and Choi, H., (2005) "Response Modification Factors of Chevron-braced Frames," *Engineering Structures*, vol. 27(2), pp. 285-300.

Elfath, H. A., (2019) "Evaluating the Inelastic Displacement Ratios of Moment-Resisting Steel Frames Designed According to the Egyptian Code," *Earthquake Engineering and Engineering Vibration*, vol. 18, pp. 159-170.

Elnashai, A. S., and Mwafy, A. M., (2002) "Overstrength and Force Reduction Factors of Multistorey Reinforced-concrete Buildings," *The Structural Design of Tall Buildings*, vol. 11(5), pp. 329–351, 2002.

FEMA 695 (2009) *Quantification of Building Seismic Performance Factors*. Federal Emergency Management Agency, Washington, D.C.

Gunay, M. S., (2011) *A Practical Guide to Nonlinear Static Analysis of Reinforced Concrete Building with Masonry Infill Walls*. Computers and Structures, Inc., Berkeley.

Ilki, A., (2000) *The Nonlinear Behavior of Reinforced Concrete Members Subjected to Reversed Cyclic Loads*, PhD Thesis supervised by Prof. Dr. N. Kumbasar and submitted to Istanbul Technical University.

Ilki, A., Yuksel, E., Darilmaz, K., Bakan, I., Zorbozan, M., and Karadogan, F., (1998) "Prefabricated Columns Subjected to Displacement Reversals," Seismic Safety of Big Cities, Earthquake Prognostics World Forum, Istanbul.

Jirsa, J. O., and Feghali, H. L., (1999) "Simulation of Seismic Response of Reinforced Concrete Structures," in *Uğur Ersoy Symposium on Structural Engineering*, pp. 261-274.

Khan, M. N., (2013) *Evaluation of Seismic Performance of RC Frame Structures by Pushover and Time History Analyses*, M.Sc. Engg. Thesis, Department of Civil Engineering, Bangladesh University of Engineering and Technology.

Medina, R.A., and Krawinkler, H.,(2004) "Influence of hysteretic behaviour on the nonlinear response of frame structures," 13th World Conference on Earthquake Engineering at Vancouver, Canada.

Mohammadi, R. K., (2002) "Approximate Evaluation of Deflection Amplification Factor," *Journal of Structural Engineering*, vol. 128(2), pp. 179-187.

Mollaioli, F., Mura, A., and Decanini, L. D., (2007) "Assessment of the Deformation Demand in Multi-Storey Frames," *Journal of Seismology and Earthquake Engineering*, vol. 8(4), pp. 203-219.

Ottosen,N.S., and Ristinmaa,M.,(2005), *The Mechanics of Constitutive Modeling,* First Edition, Division of Solid Mechanics, Lund University, Sweden

Samimifar, M., Oskouei, A.V., and Rofooei, F. R., (2015) "Deflection Amplification Factor for Estimating Seismic Lateral Deformations of RC Frames," *Earthquake Engineering and Engineering Vibration*, vol. 14, pp. 373-384.

Uang, C. M., and Maarouf, A., (1994) "Deflection Amplification Factor for Seismic Design Provisions," *Journal of Structural Engineering*, vol. 120(8), pp. 2423-2436.

Uang, C. M., (1991) "Establishing R (or R_w) and C_d Factor for Building Seismic Provision," *Journal of Structural Engineering*, vol. 117(1), pp. 19–28.

www.researchgate.net/publication/328571852_Seismic_Evaluation_and_Retrofi_t_of _Existing_Tall_Buildings_in_California_Case_Study_of_a_35-Story_Steel_Moment-Resisting_Frame_Building_in_San_Francisco/figures?lo=1

APPENDIX-A

Non-Linear Dynamic Time History Analysis (Non-Linear Direct Integration)

Analysis of a Sample Building

Step 1: Grid-Define Concrete Properties-Column etc.

Step 2: Define Beam as Shown

Step 3: Define Supports as Fixed. The Define Slab as Shown

Step 4: Define Frame Hinge Properties as Shown (Concrete/Used Defined)

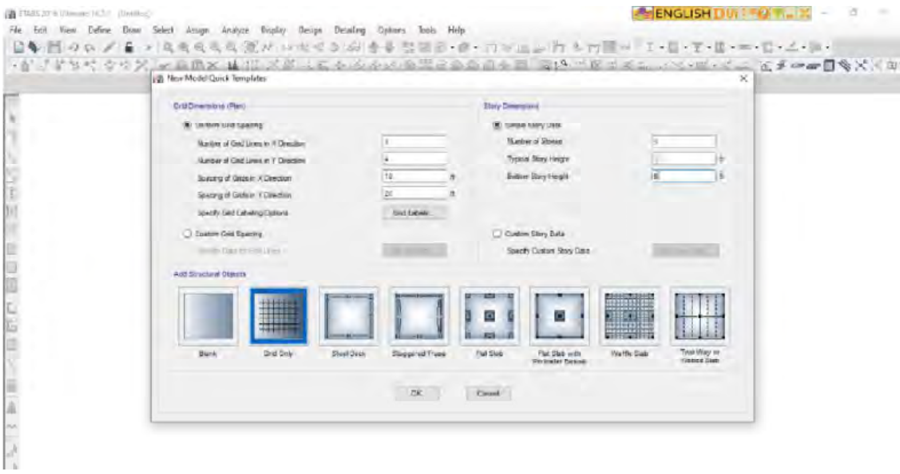
Step-5: Assign Frame Hinge Properties to Beams and Columns as Shown

Step-6: Define Time History Function as Shown

Step-7: Define Load case Data as Shown

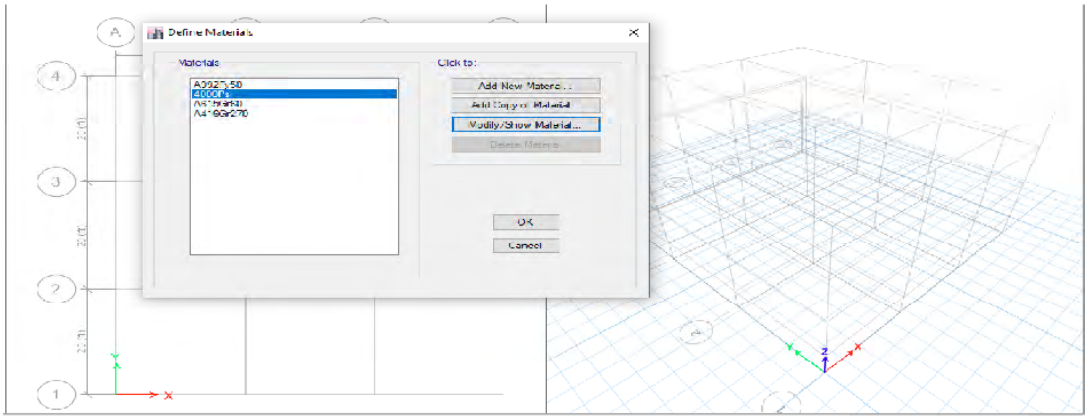
Step-8: Apply Load Mesh Slab as Shown

Step 1: Grid- Define Concrete Properties- Column etc

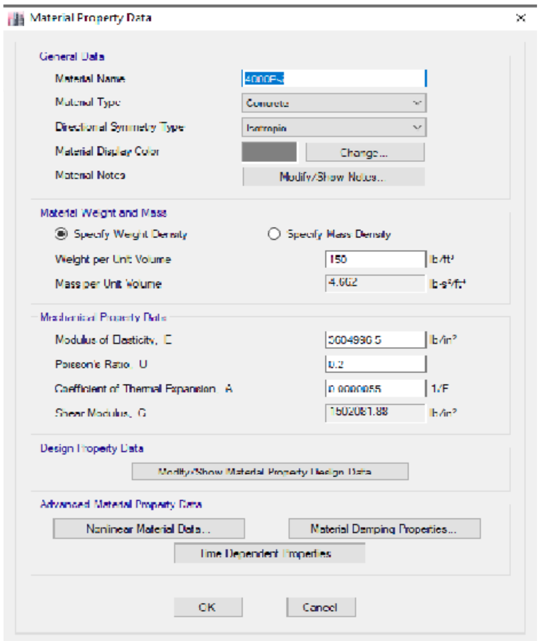


Grid Only Option (ACI 318-11)

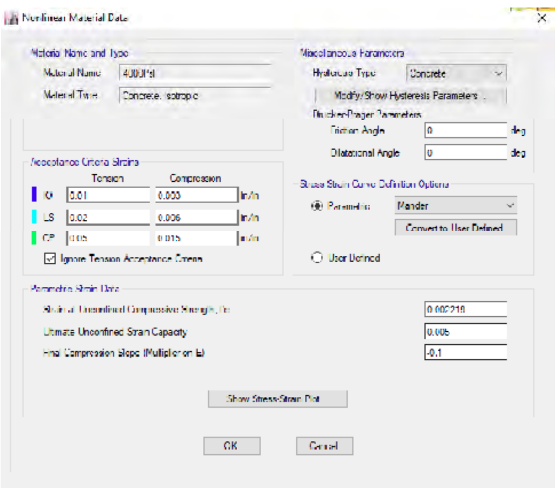
Define Material Properties



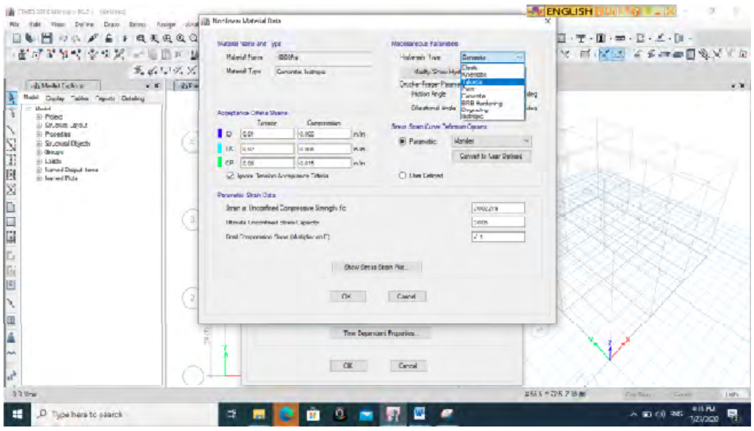
4000 psi to be Modified



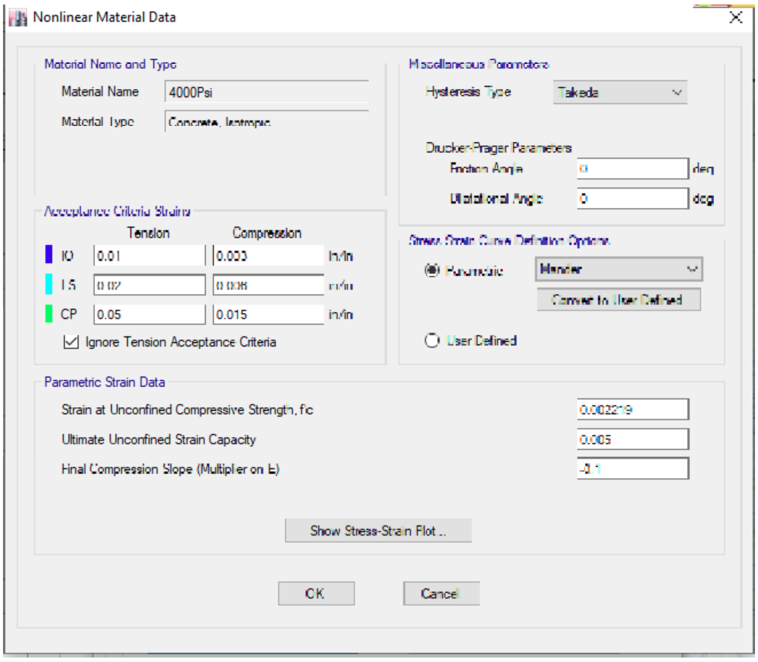
Clicking on Nonlinear Material Data



Hysteresis Type-Takeda

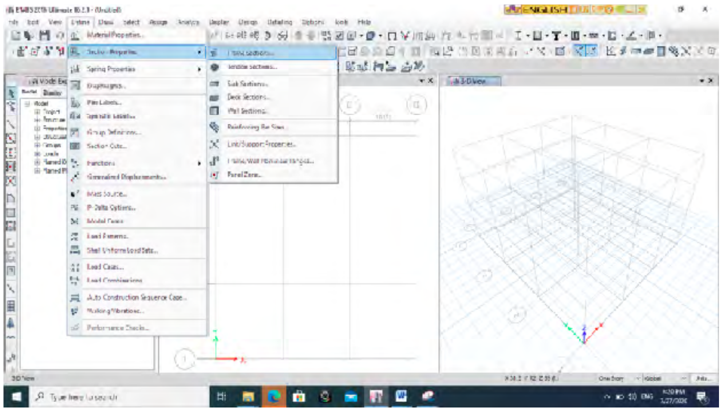


Hysteresis Type Takeda is Selected

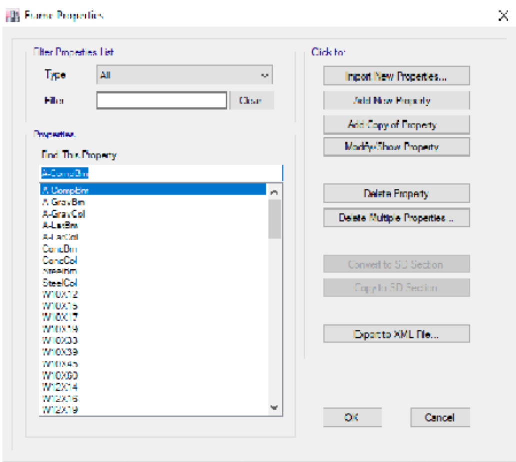


Define-Section Properties-Frame sections

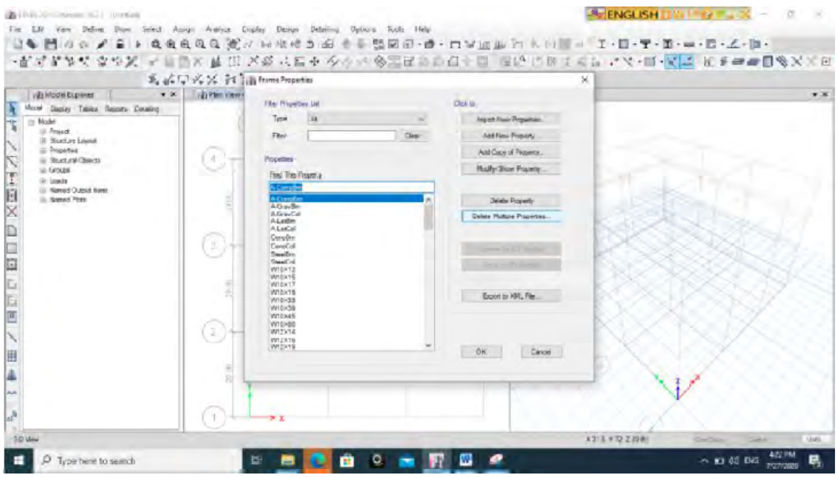
Frame Section is Selected



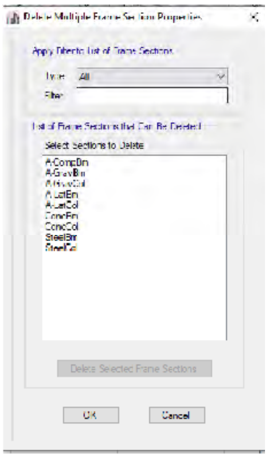
Previous all Frames are Deleted



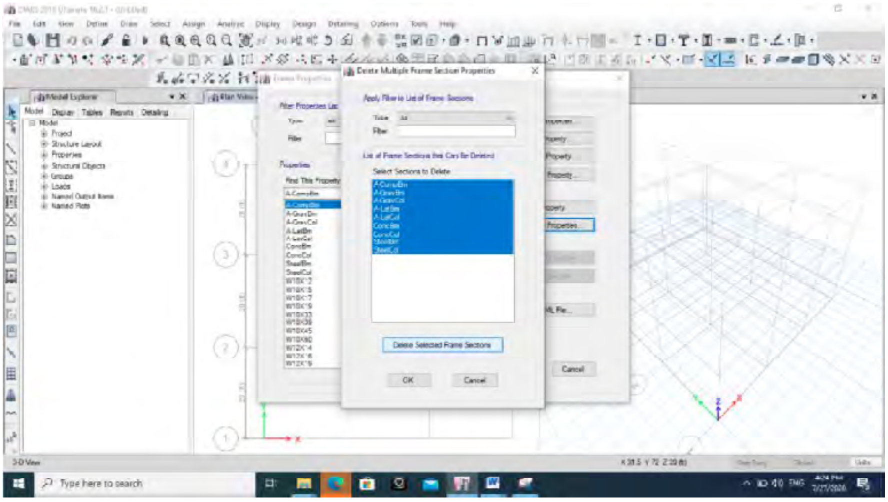
Previous Frames are Selected



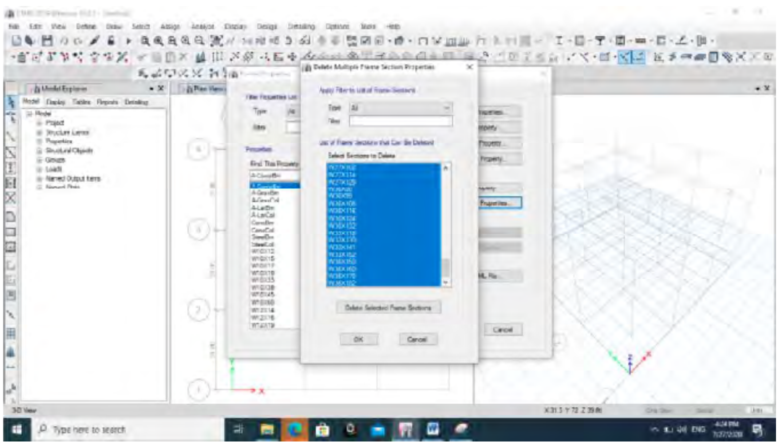
Multiple Frame Appears to be Selected



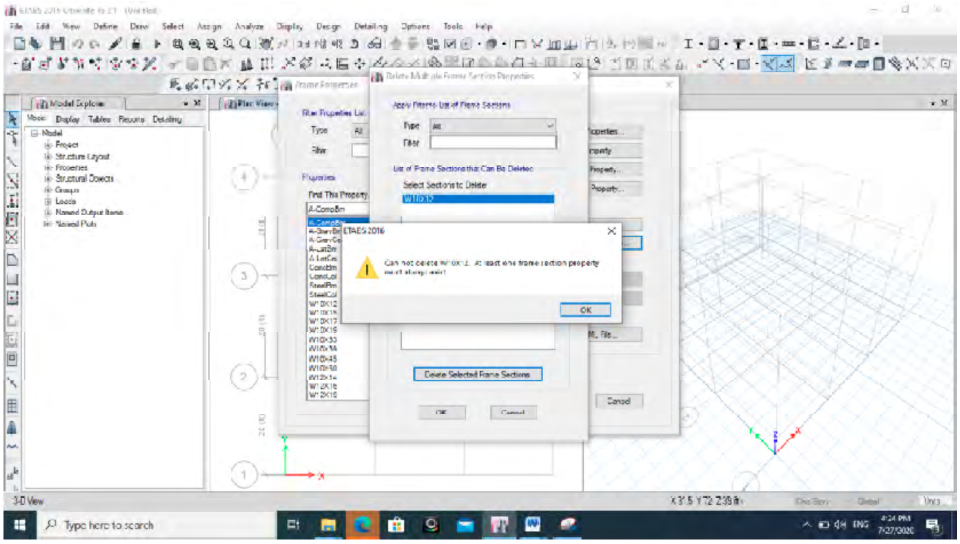
Frames to be Deleted are Selected



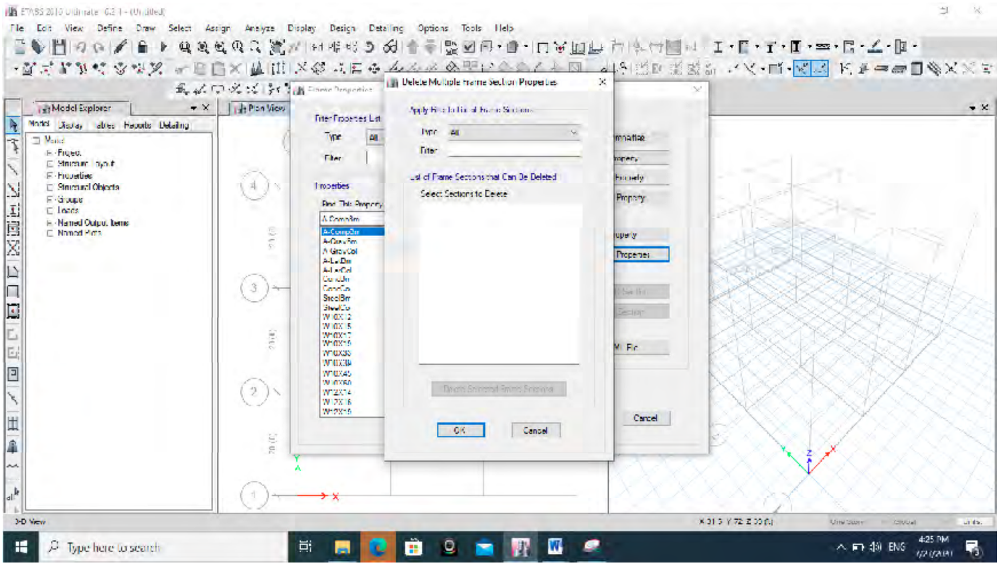
Frames to Deleted



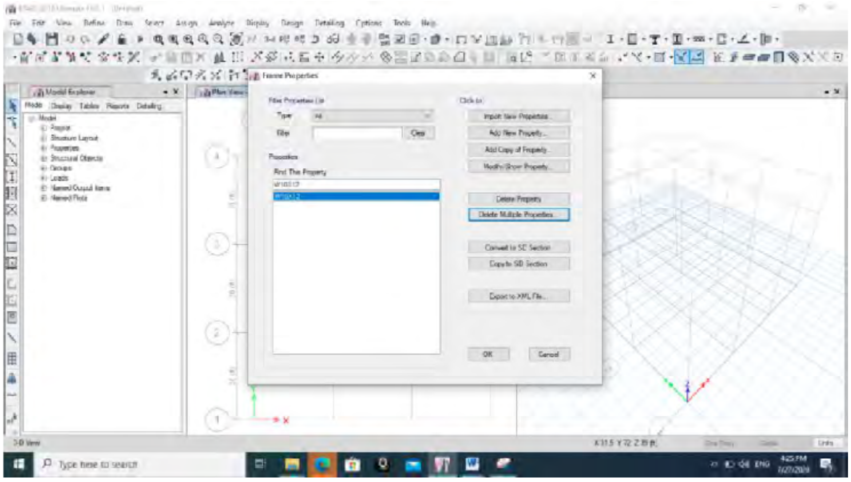
One Frame cannot be Deleted



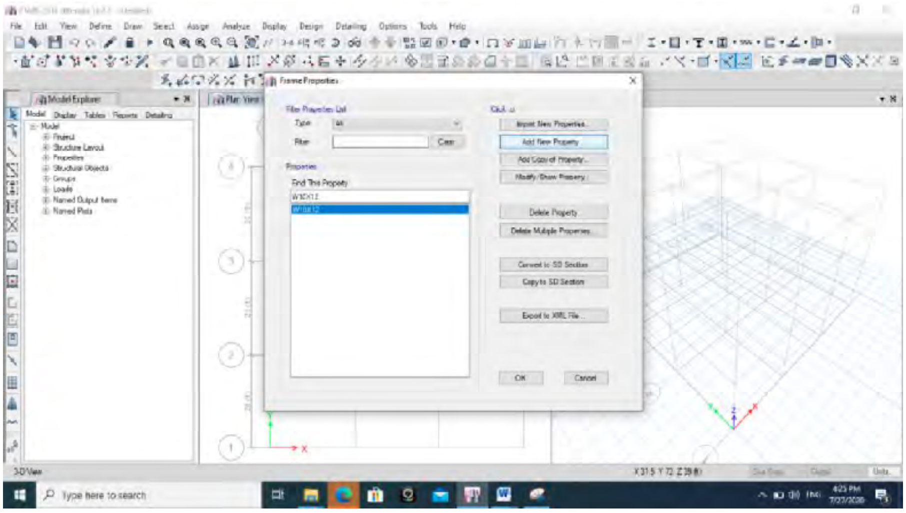
Frames are Deleted



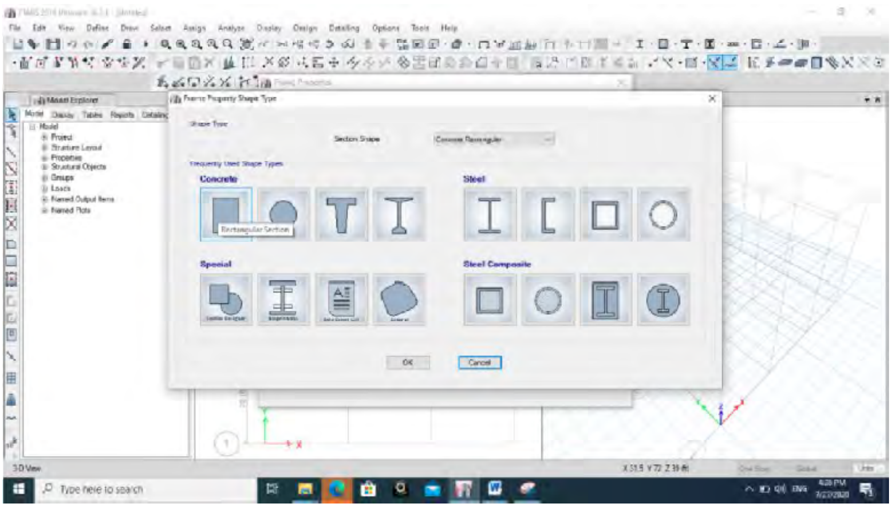
One Frame Remains



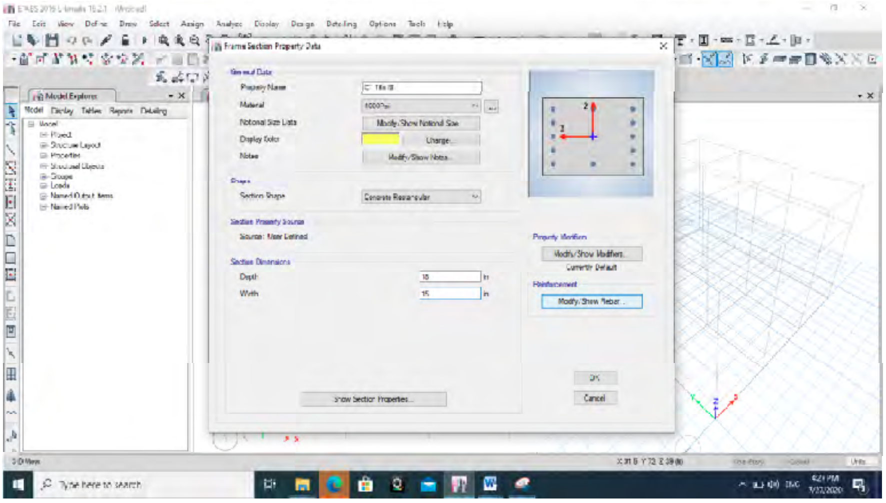
Add New Property



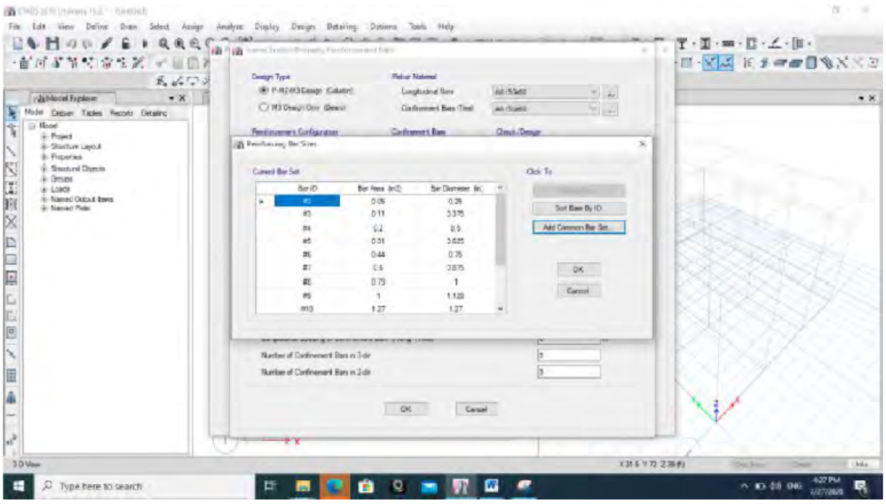
Frame to be Selected



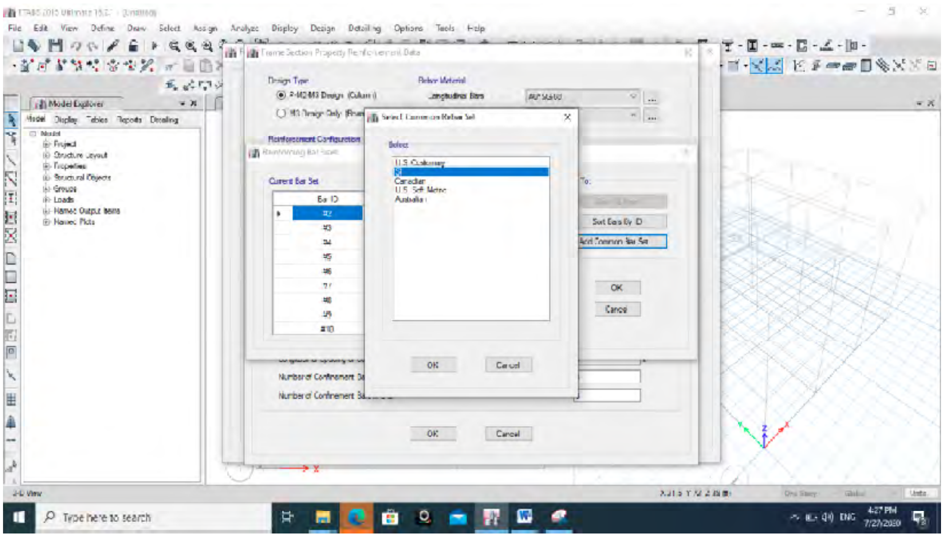
Column Size Selected



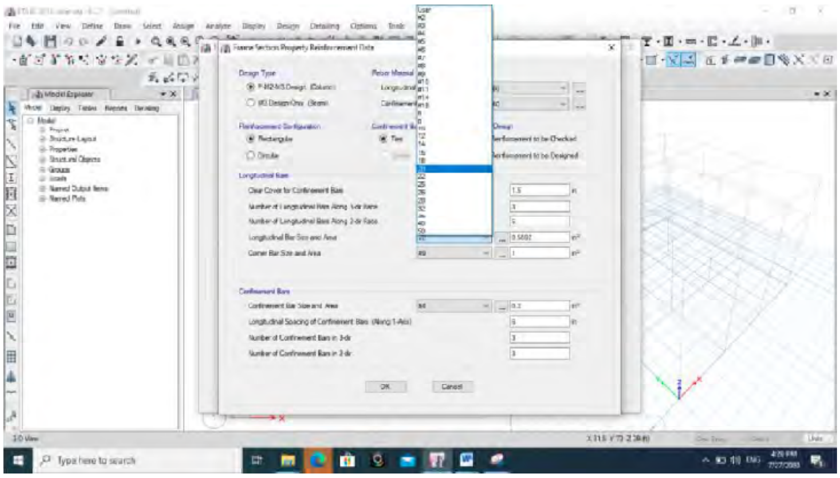
Reinforcement selected



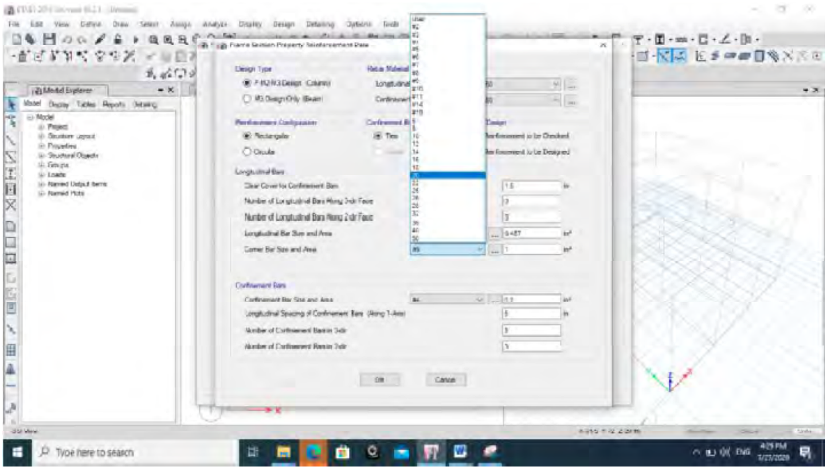
SI unit



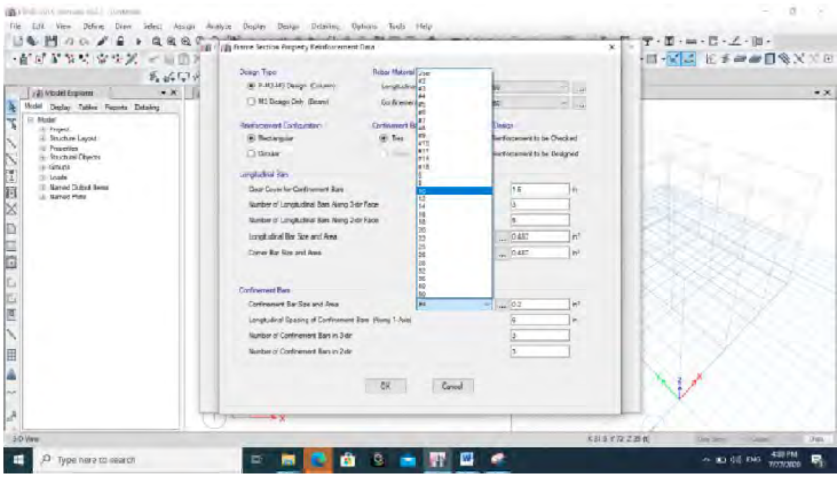
No 20 bar



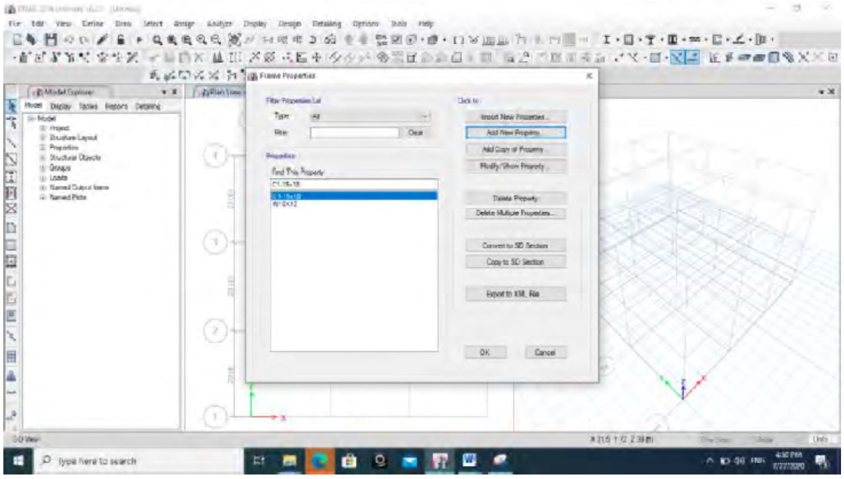
No 20 bar



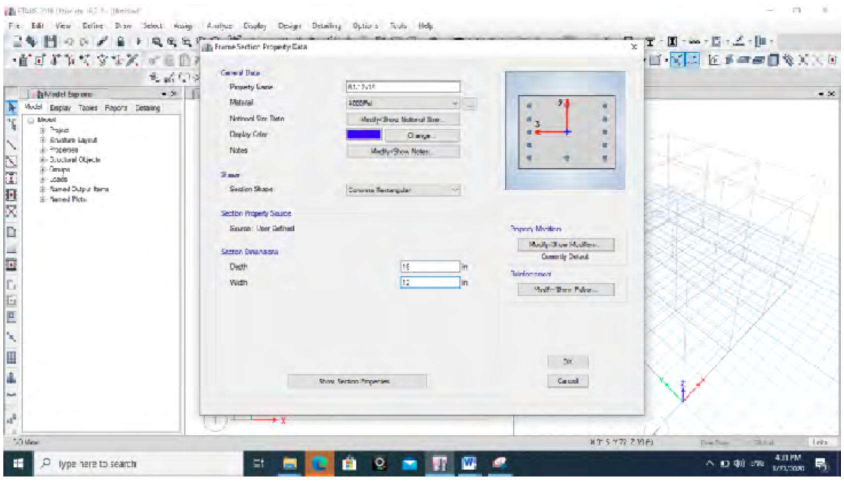
No 10 bar



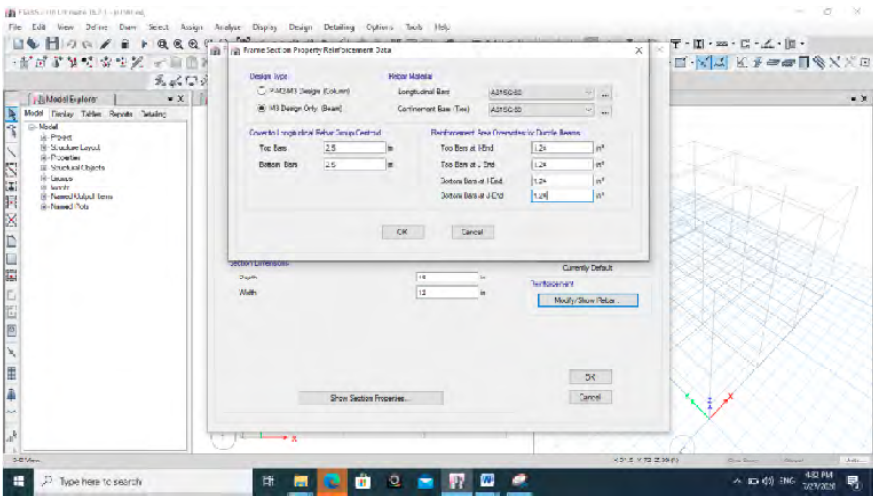
Previous Frames are Deleted



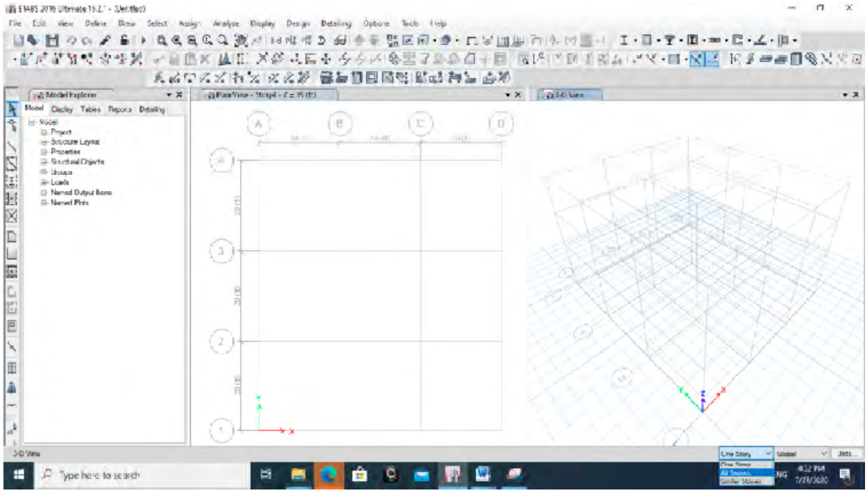
Step 2: Define Beam



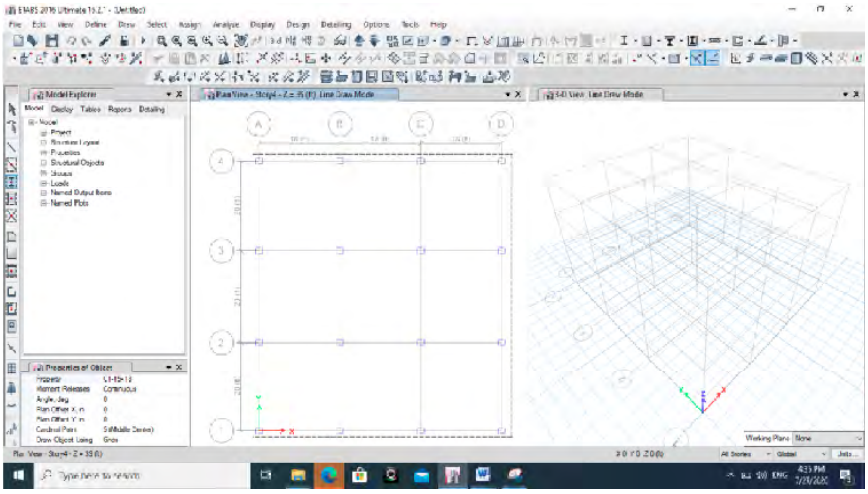
M3 Design only for Beam



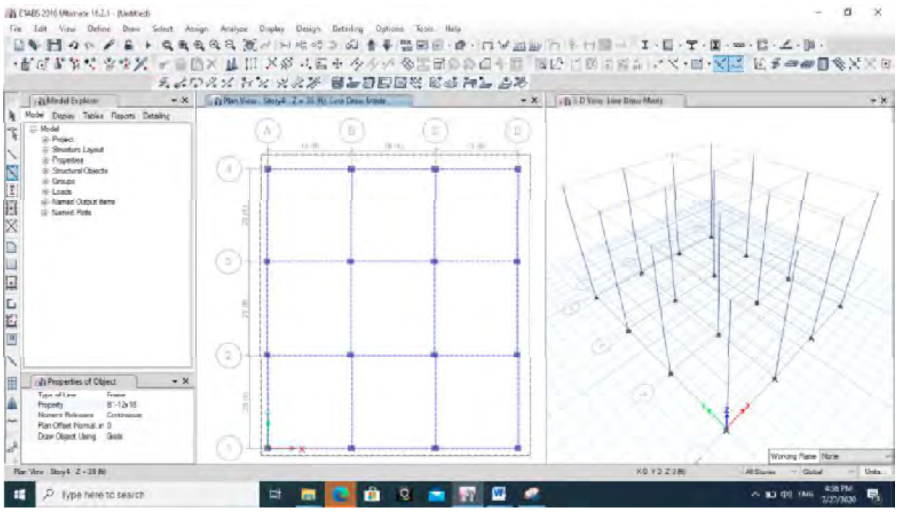
Grid only Selected



Drawing of Column

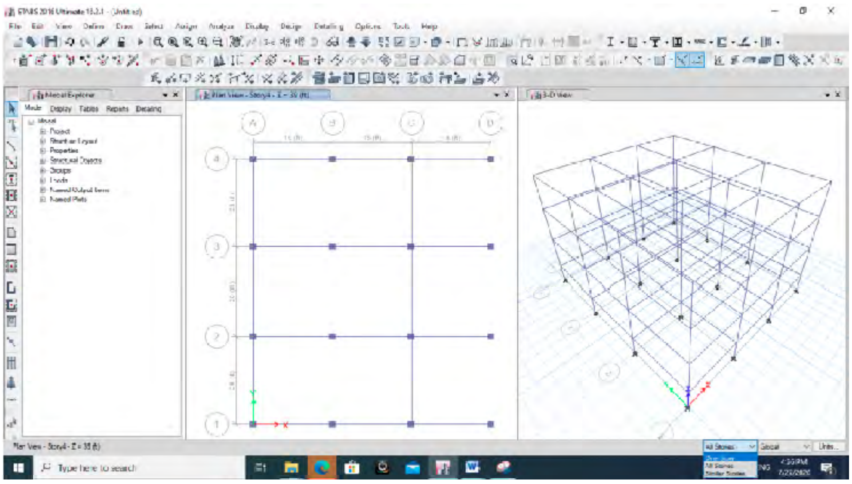


Drawing of Beam

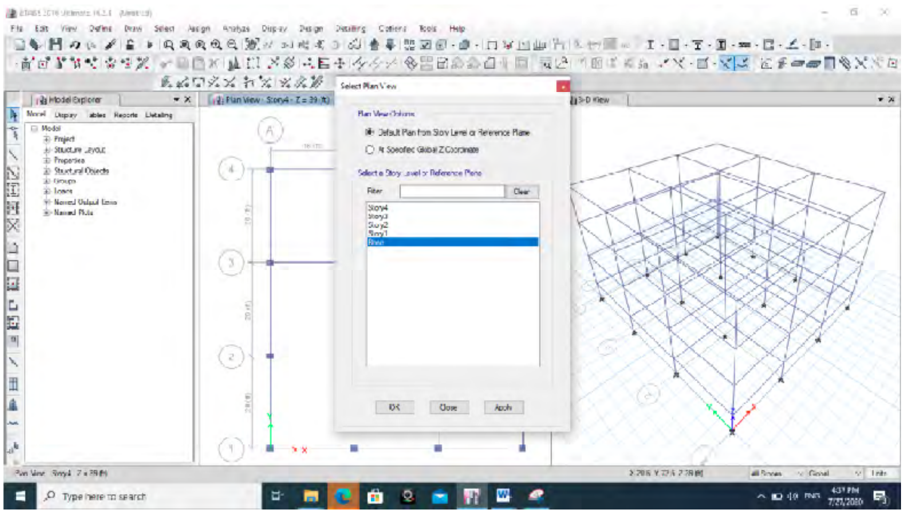


Step 3: Define Supports as Fixed. Then Define Slab

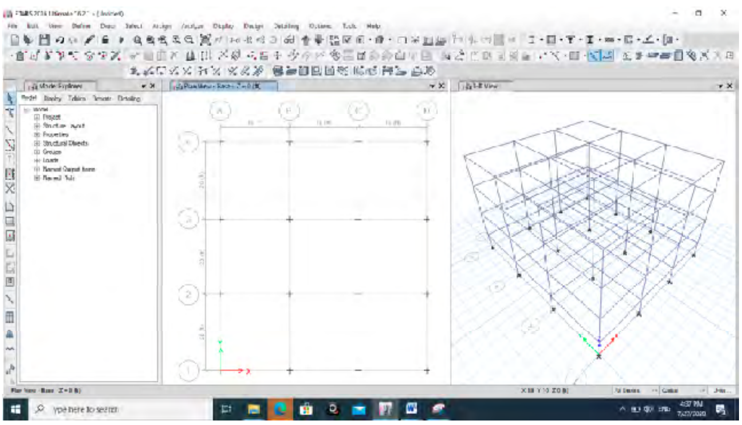
Base to be Fixed



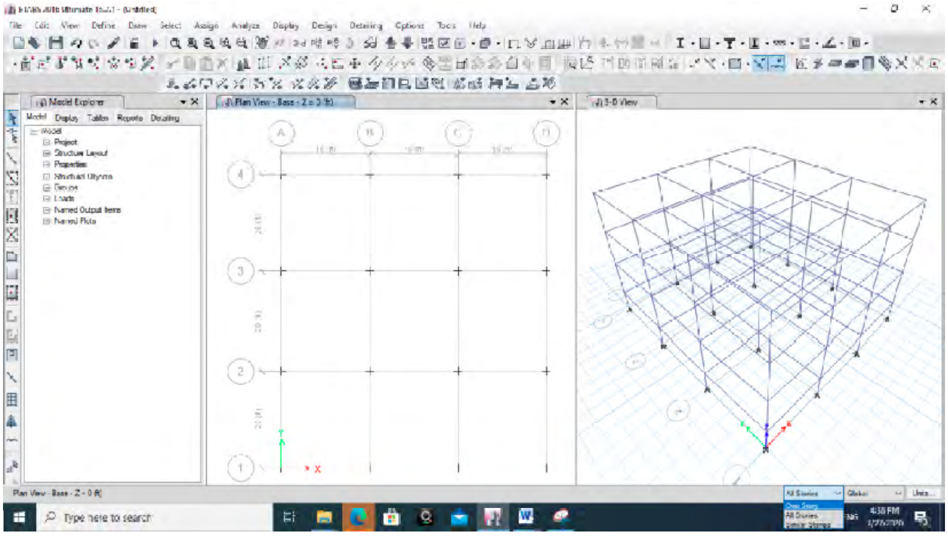
Base Selected



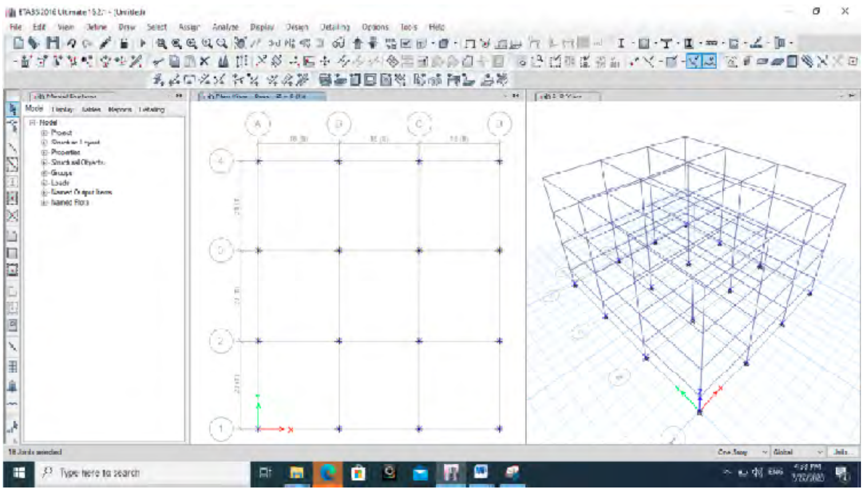
Pointer



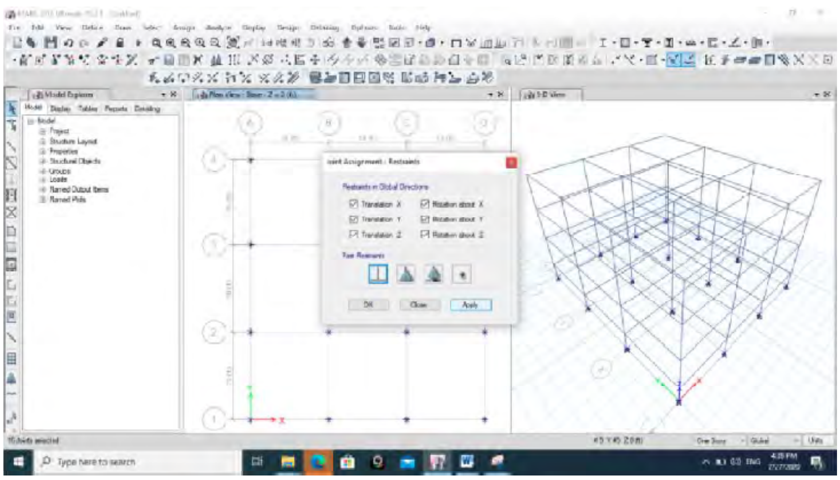
Base



Base selected

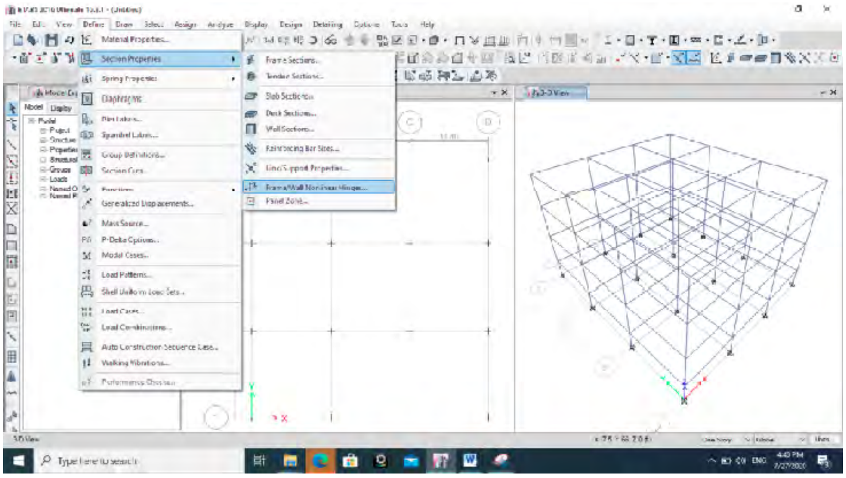


Joint Assignment

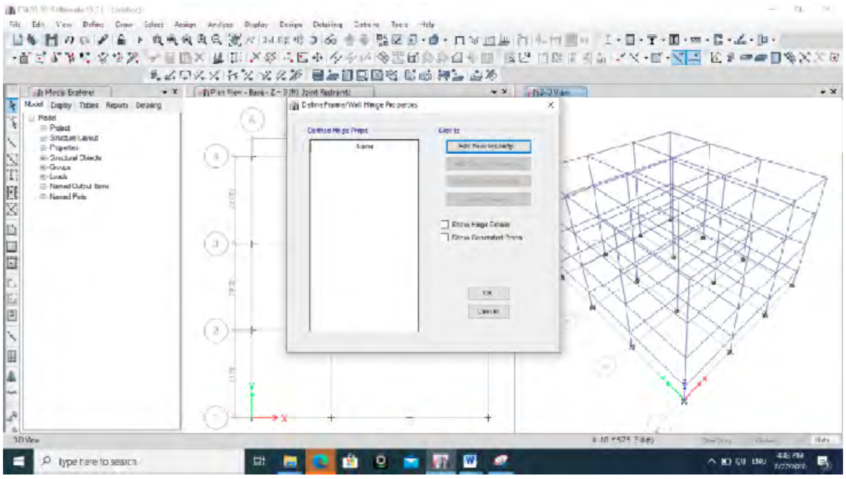


Step 4: Define Frame Hinge Properties

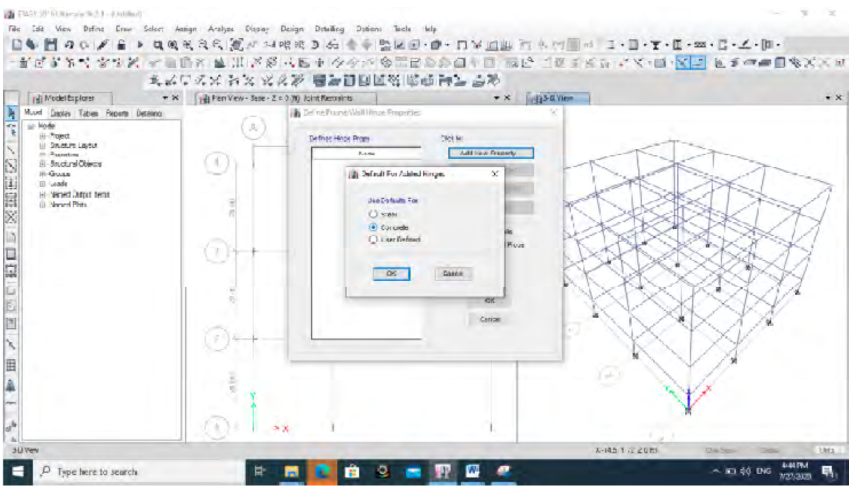
Define-Section Properties-Frame/Wall Nonlinear Hinges



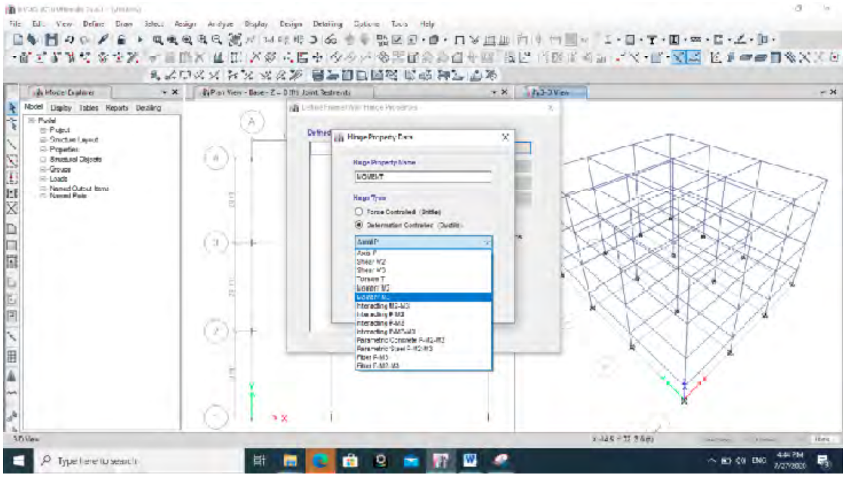
Nonlinear Hinge



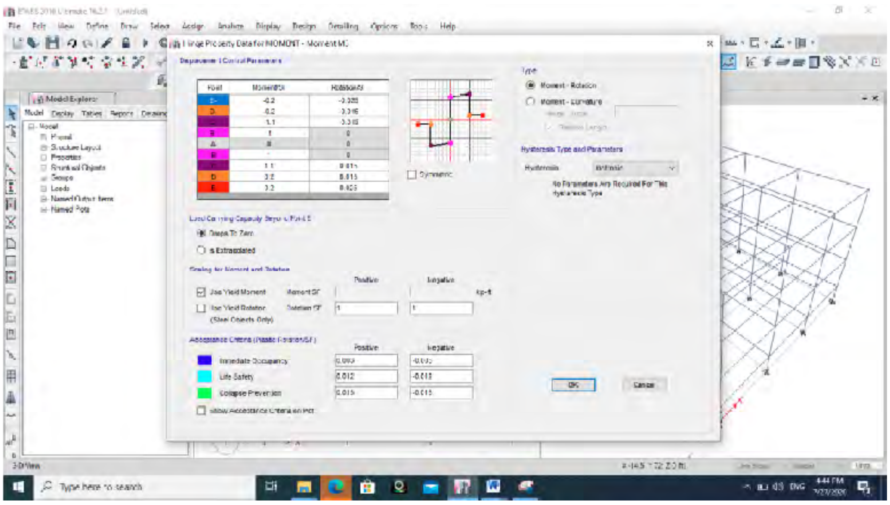
Concrete



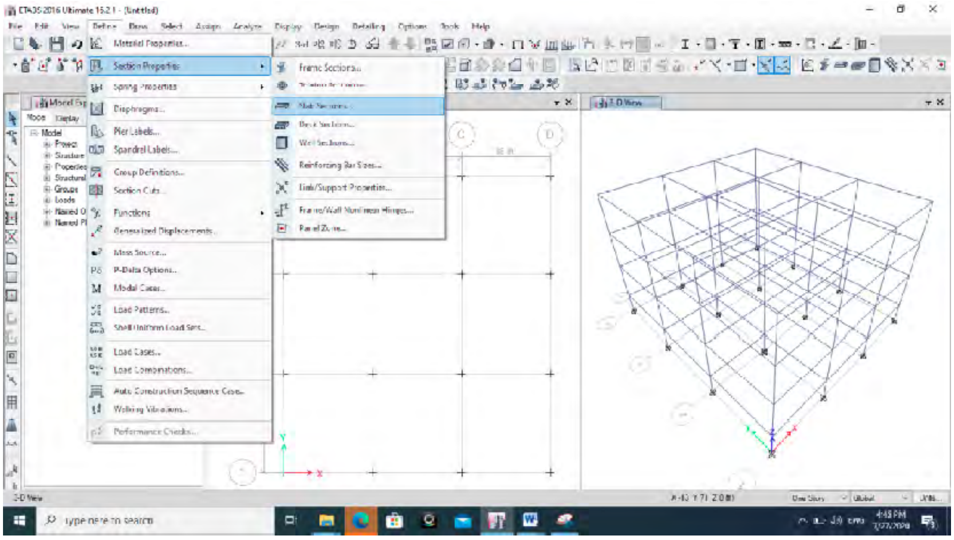
Moment M3



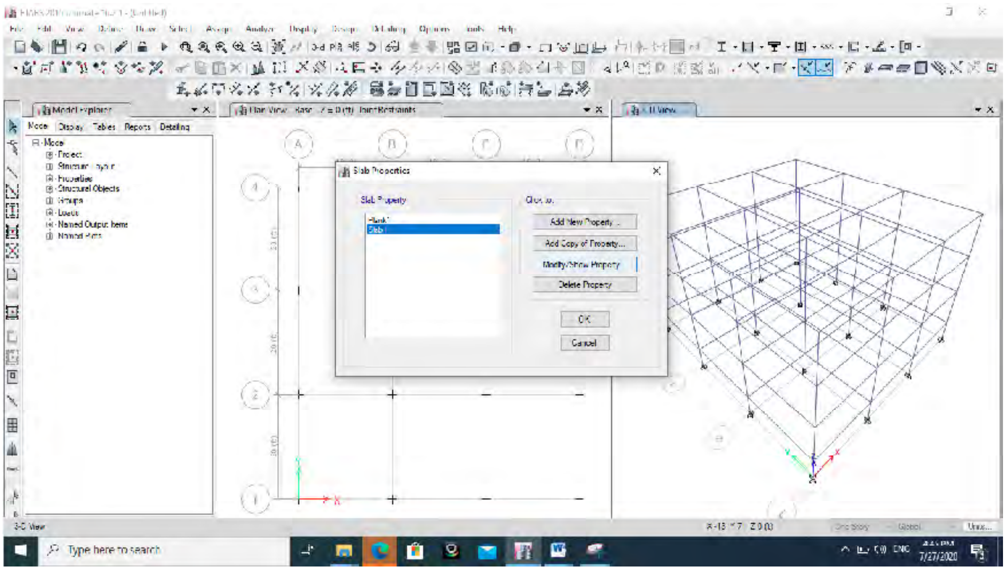
Moment M3



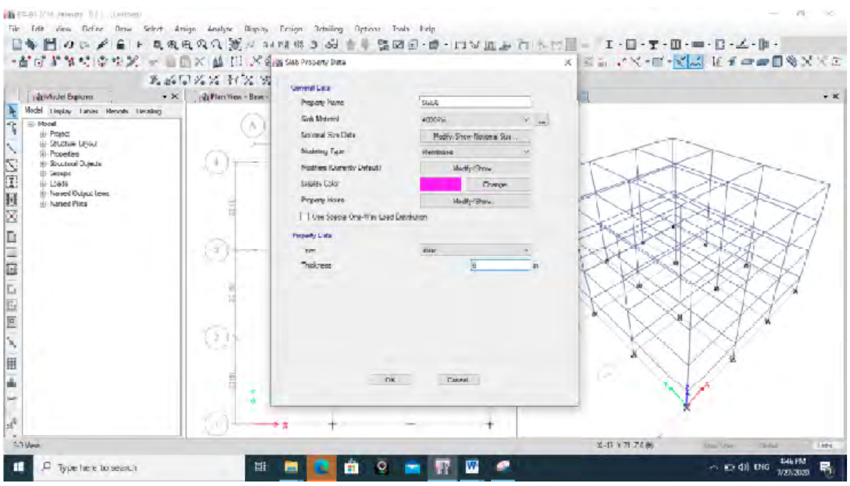
Slab Section



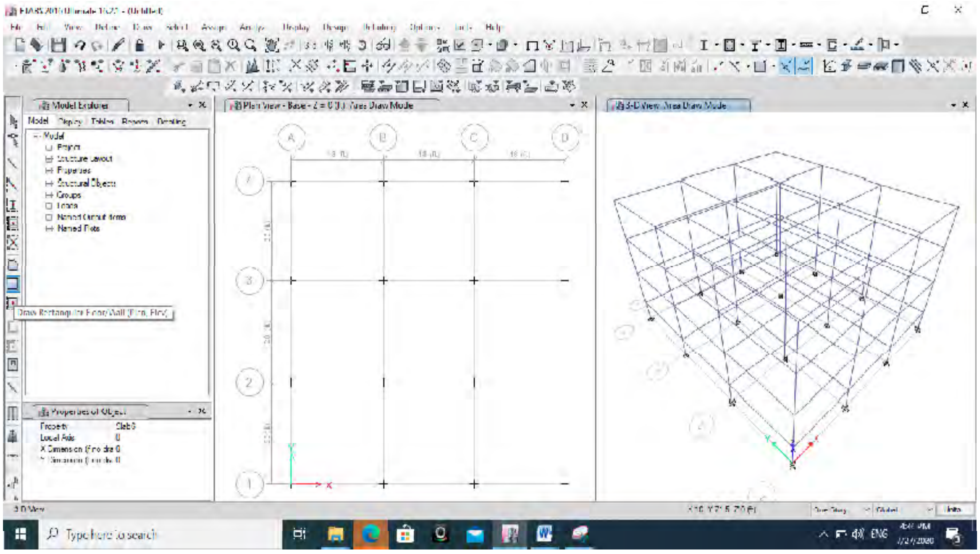
Slab to be Modified



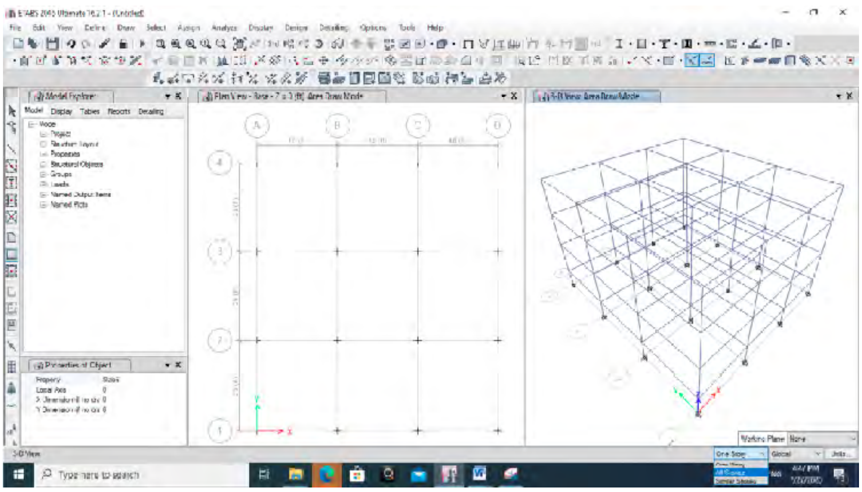
Slab 6 inch



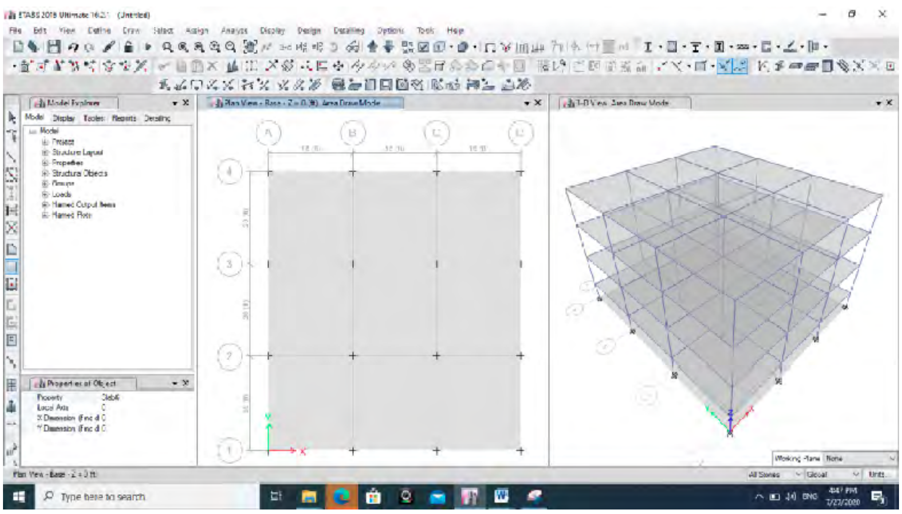
Drawing of Slab



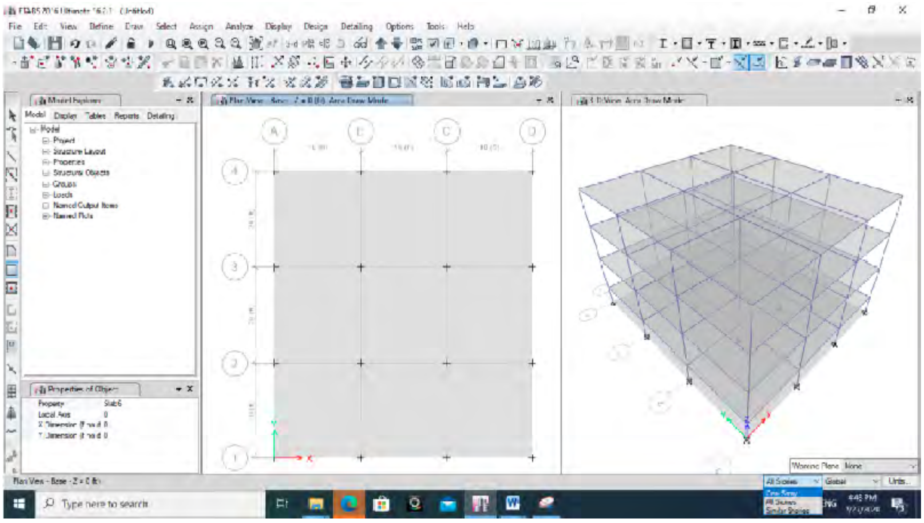
Base



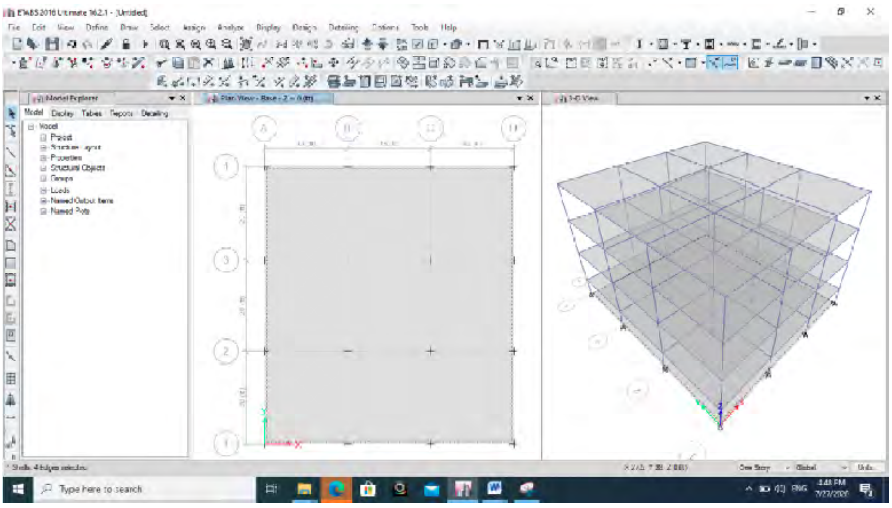
Slab Drawn



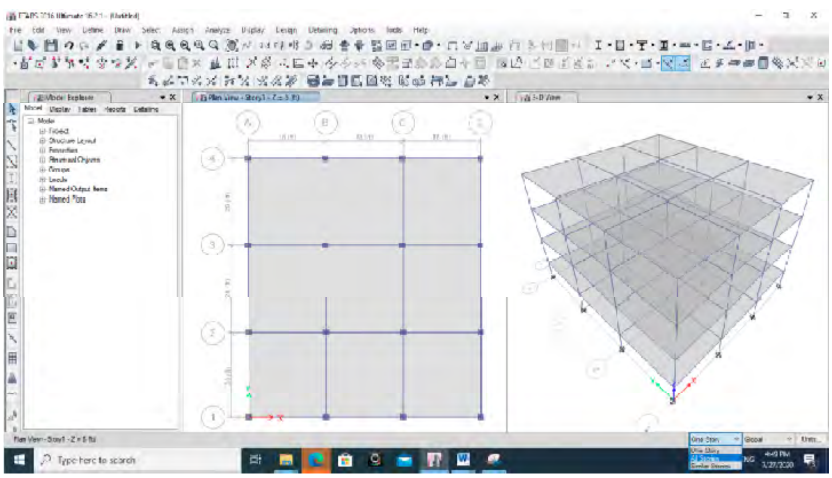
Base Slab Deleted



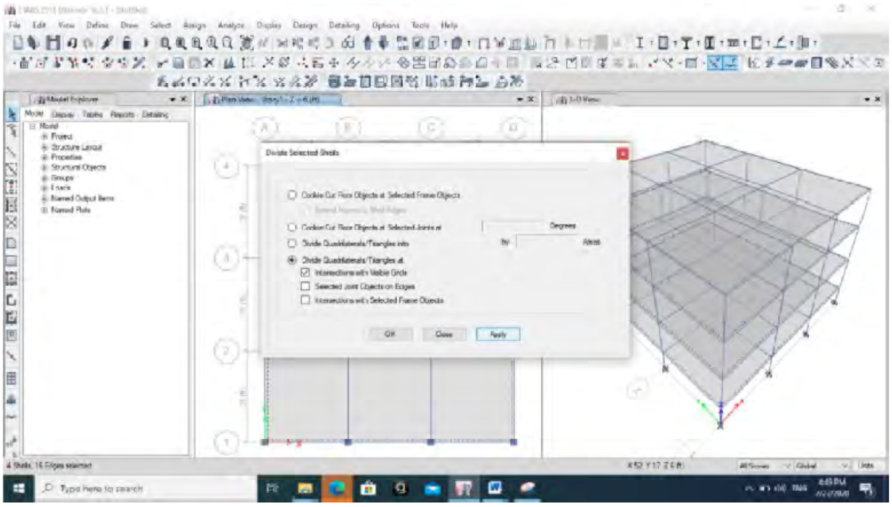
Base Slab Deleted



Story 1 slab

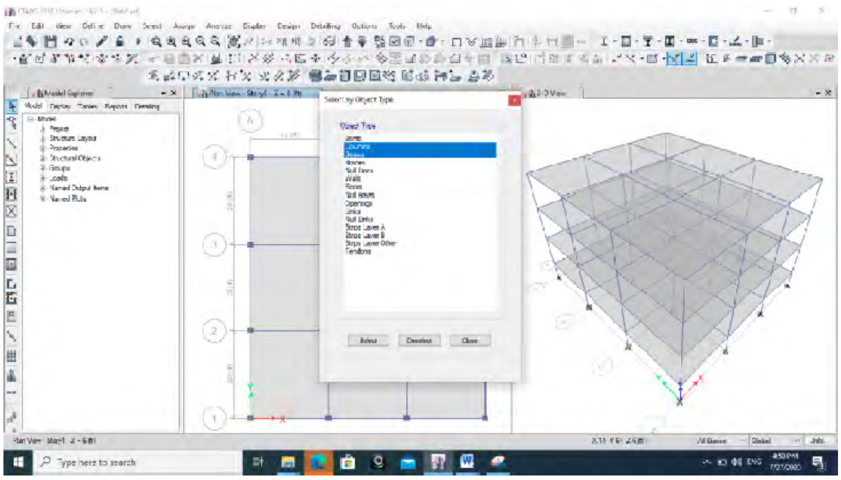


Visible grid

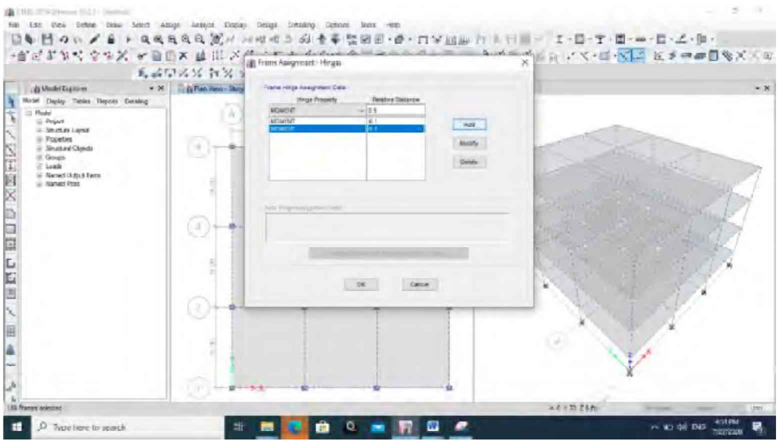


Step 5: Assign Frame Hinge Properties

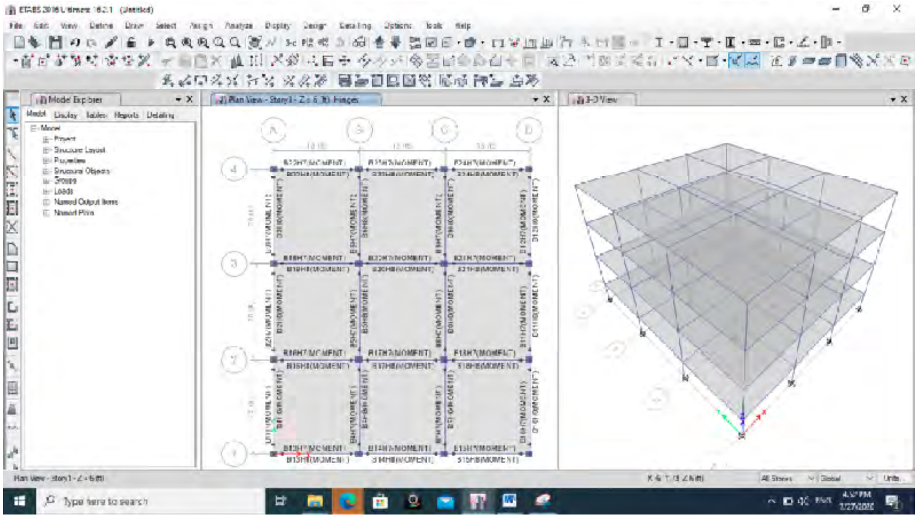
Hinge Location



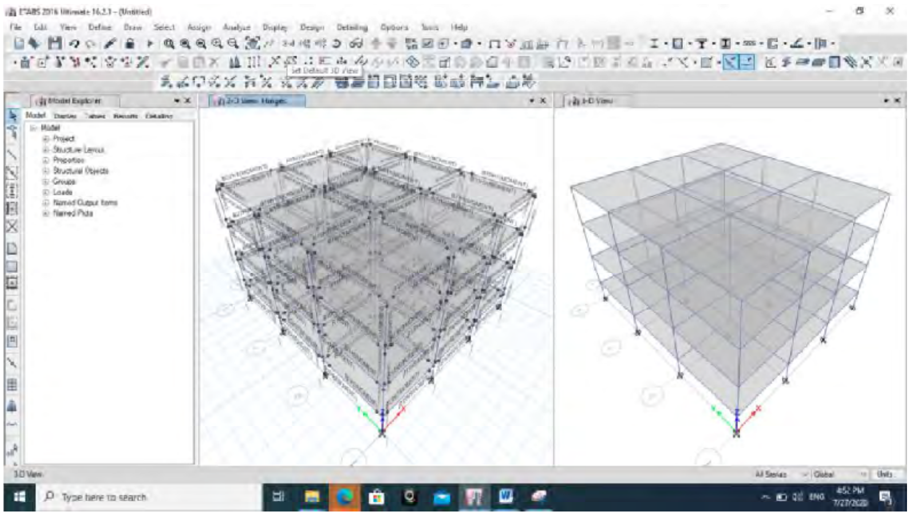
Hinge Assignment



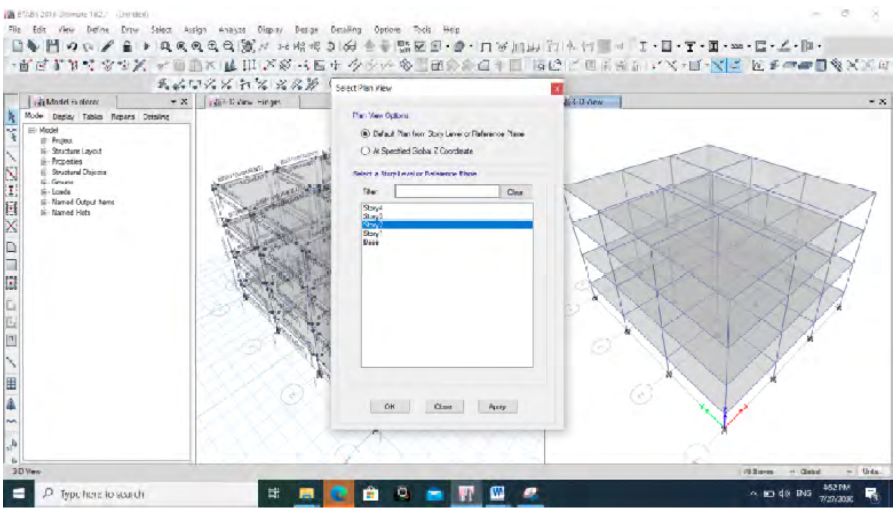
Hinge



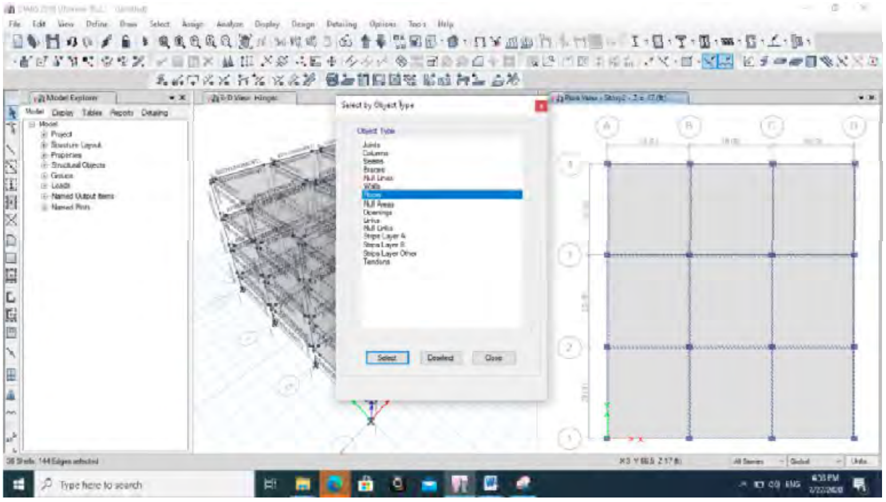
Hinge 3D



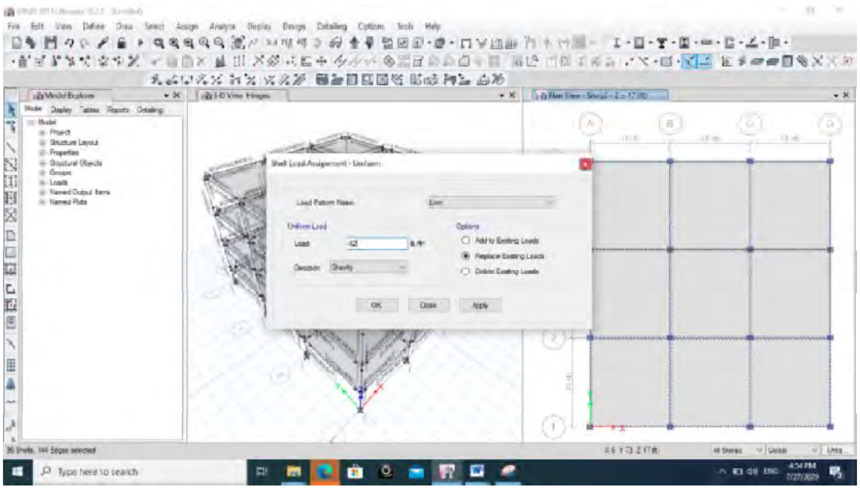
Plan View for Story 2



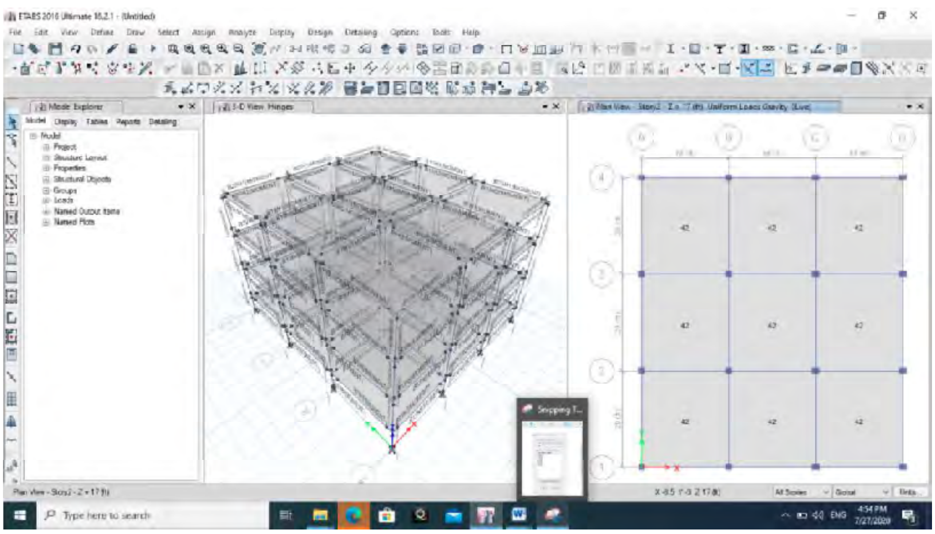
Select Floor



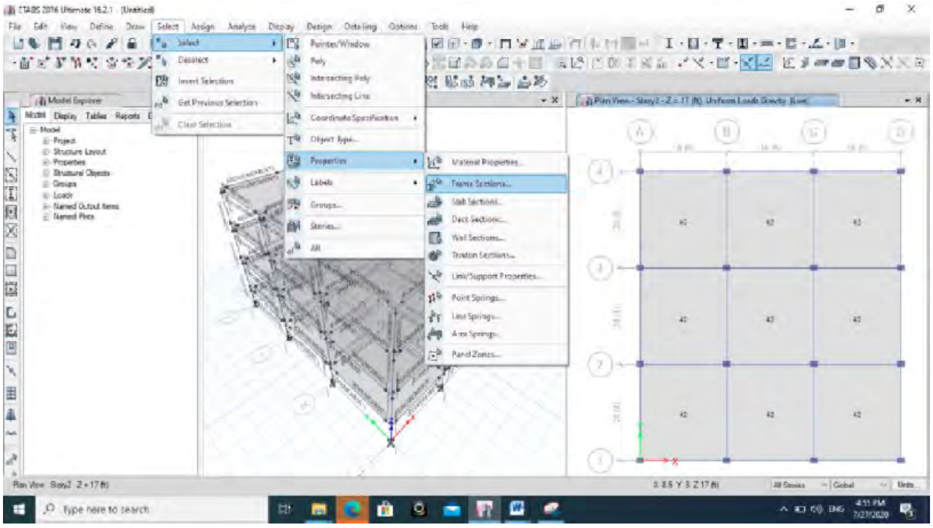
Load in Floor



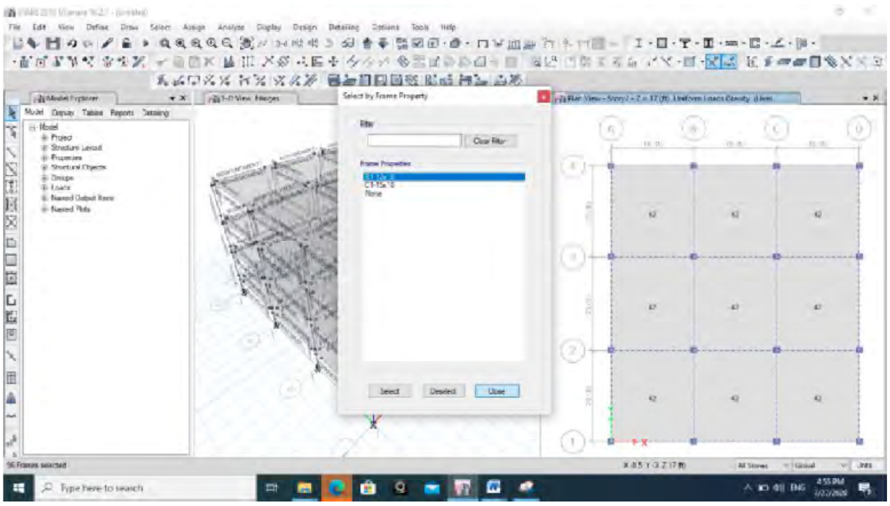
Live Load in floor



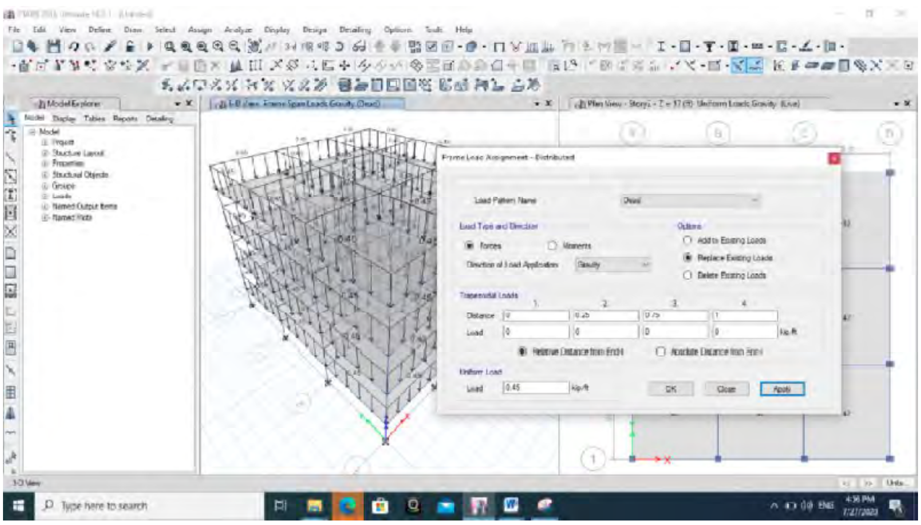
Frame selected



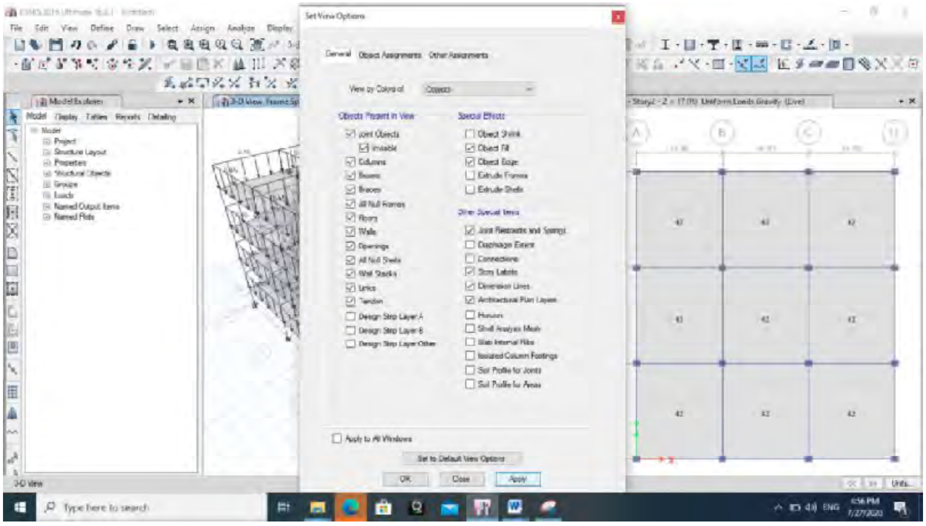
Frame



Uniform load

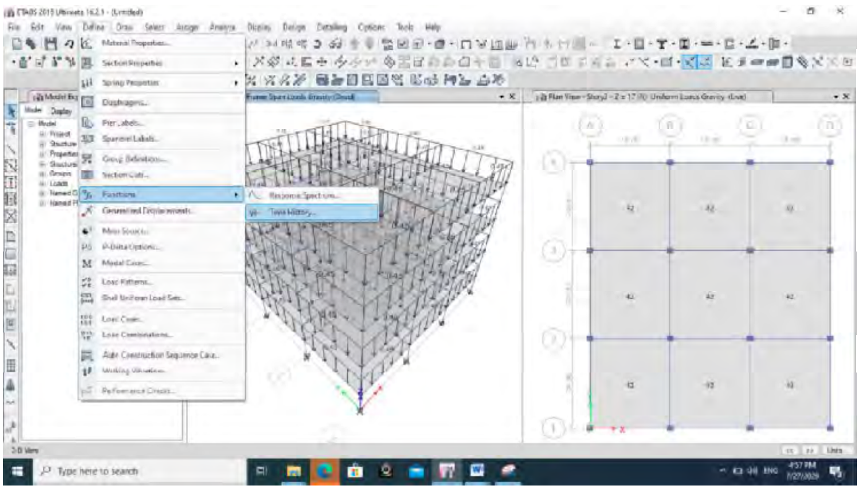


Object

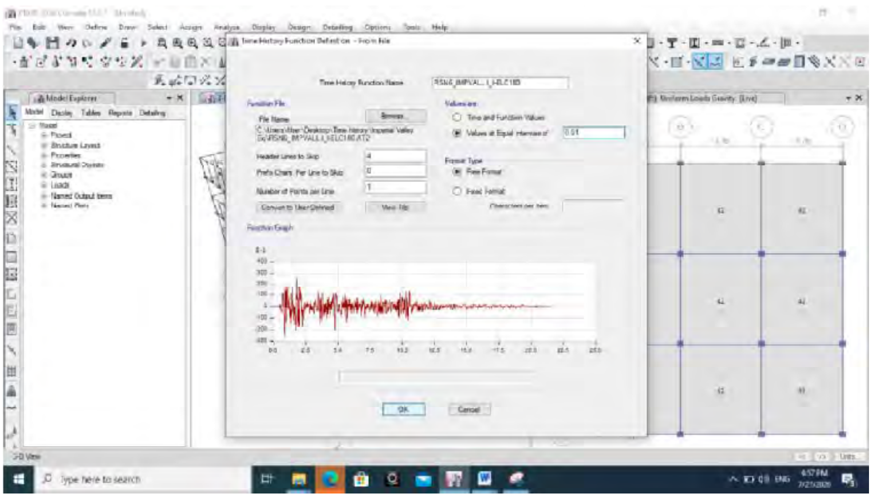


Step 6: Define Time History Function

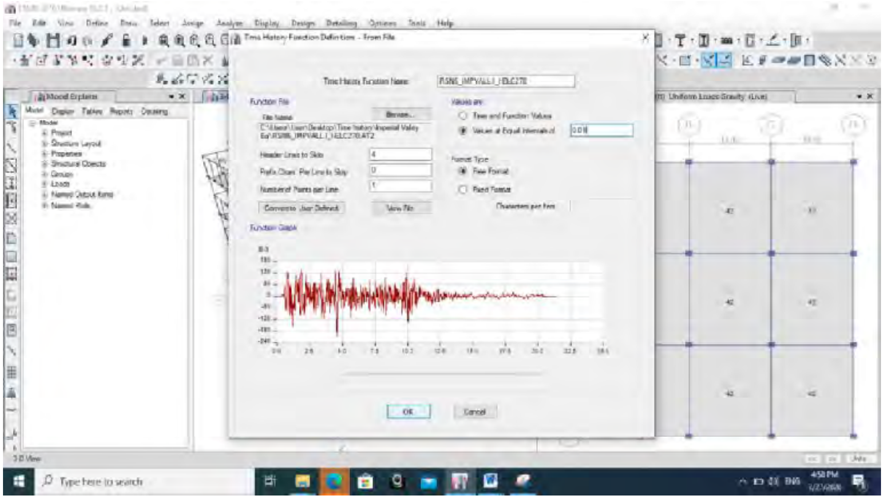
Time History function



Time History in X direction

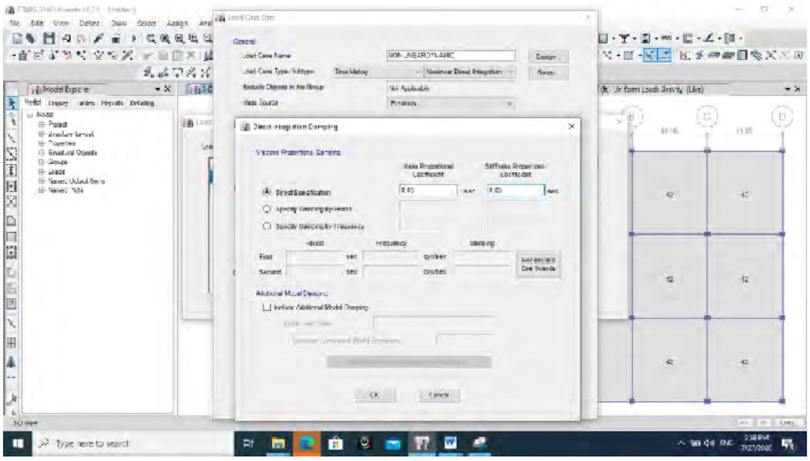


Time History in Y direction

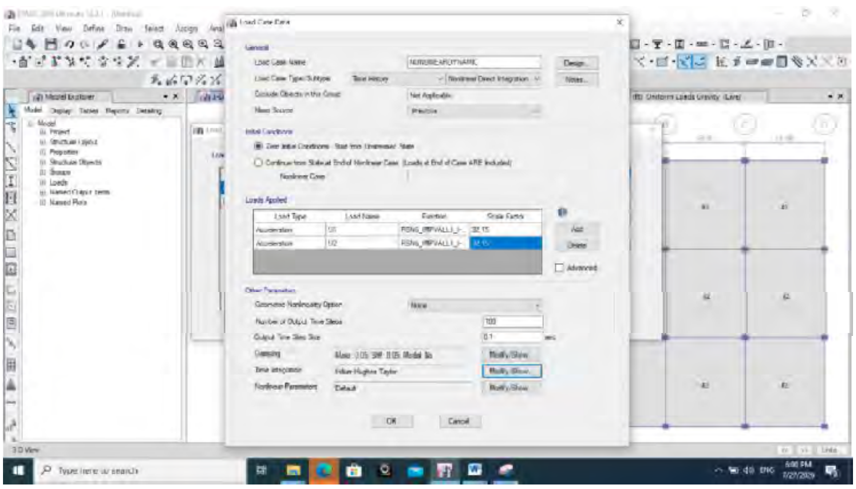


Step 7: Define Load Case

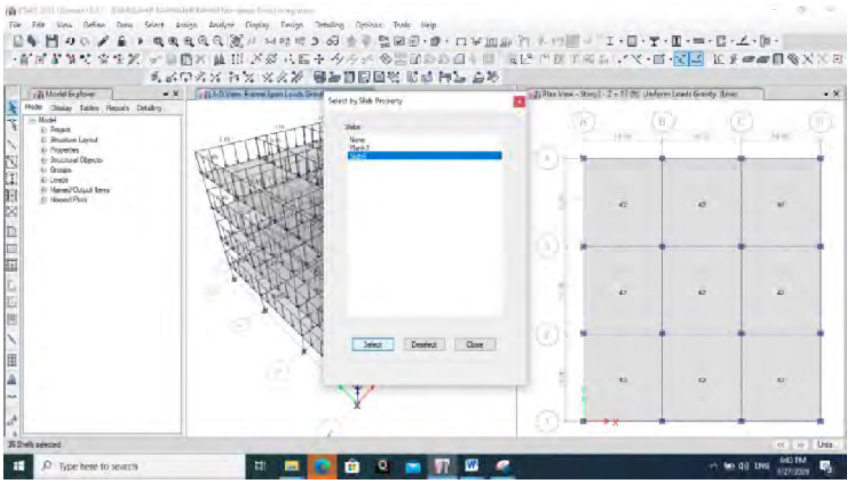
Load Case



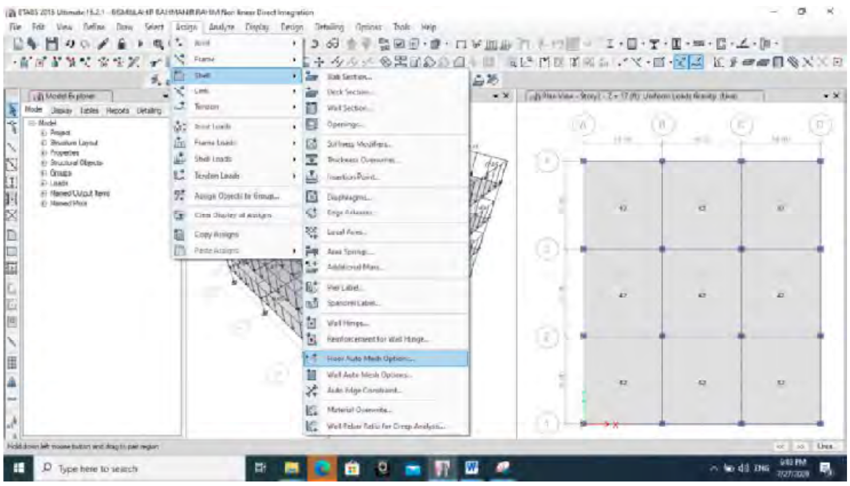
Load Case Data



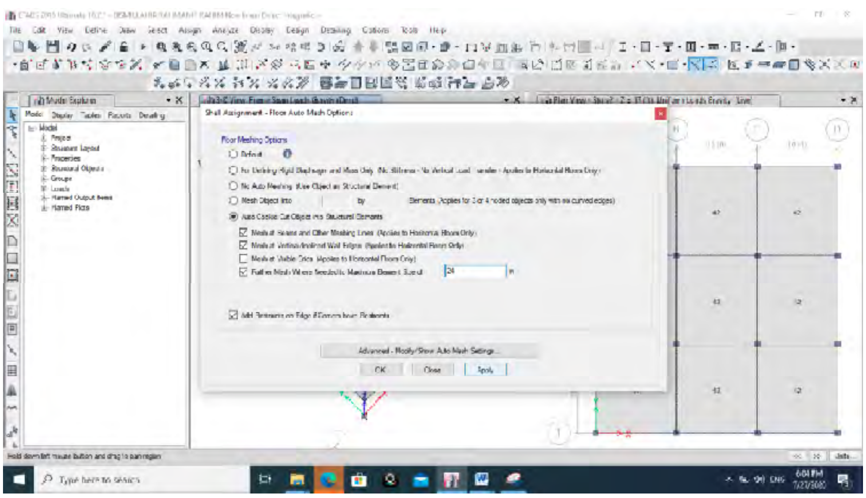
Step 8: Apply Load Mesh Slab



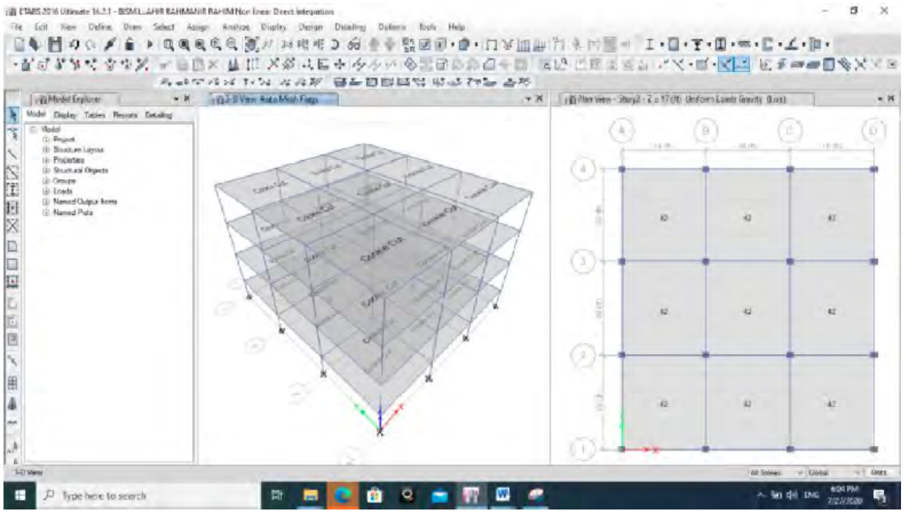
Select Shell for Cookie Cut



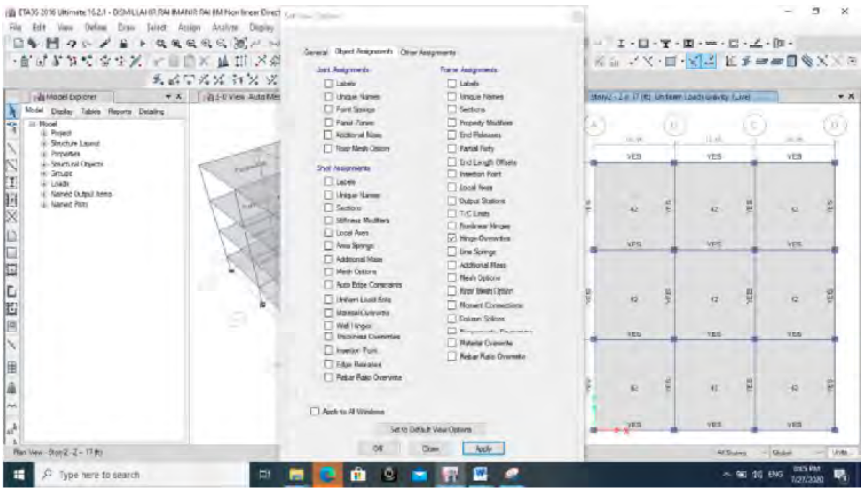
Assign for Cookie Cut



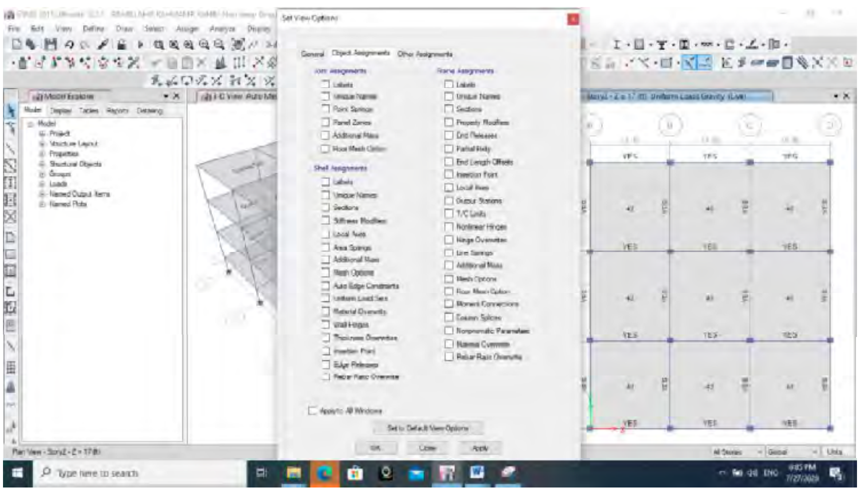
Cookie cut



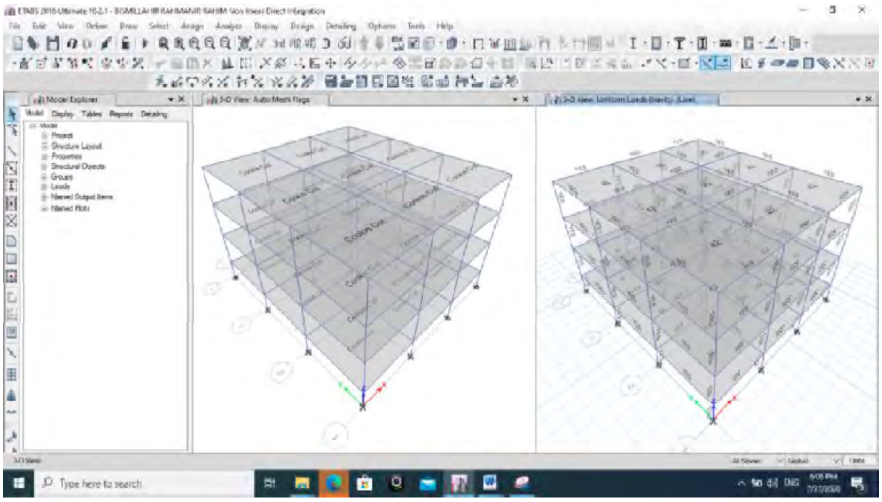
Object Assignment



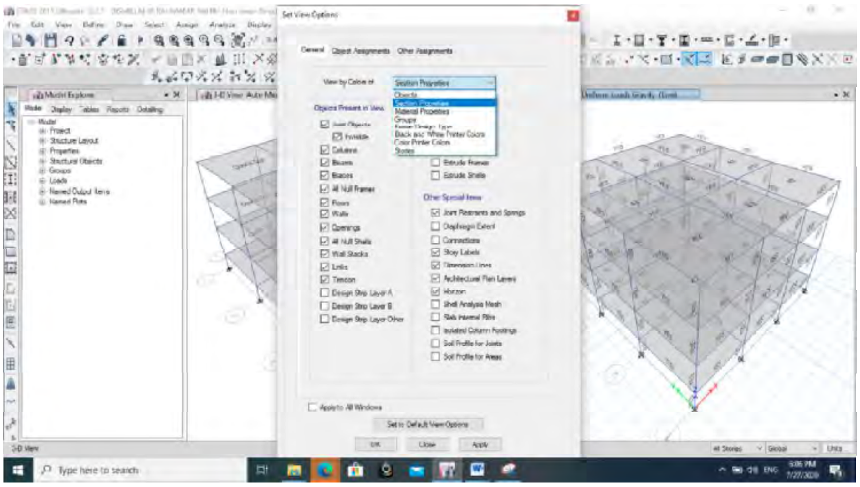
Hinge Overwrite



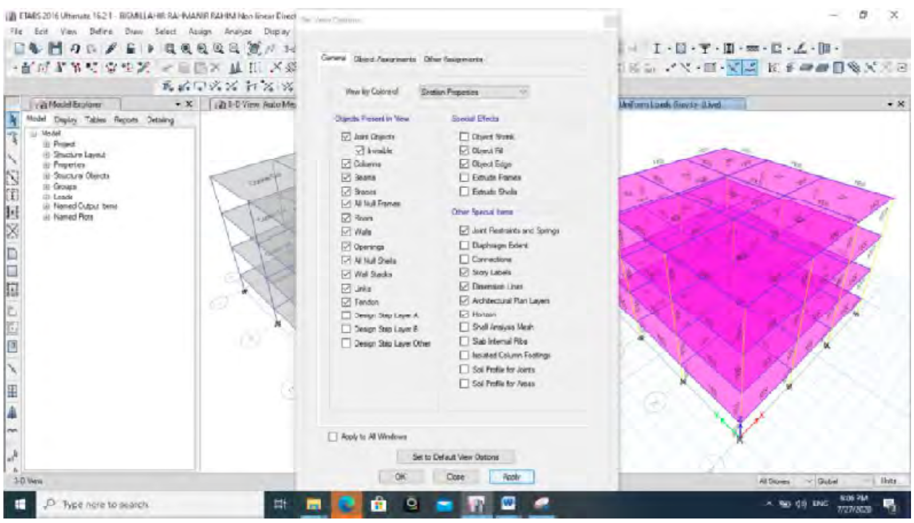
Cookie Cut



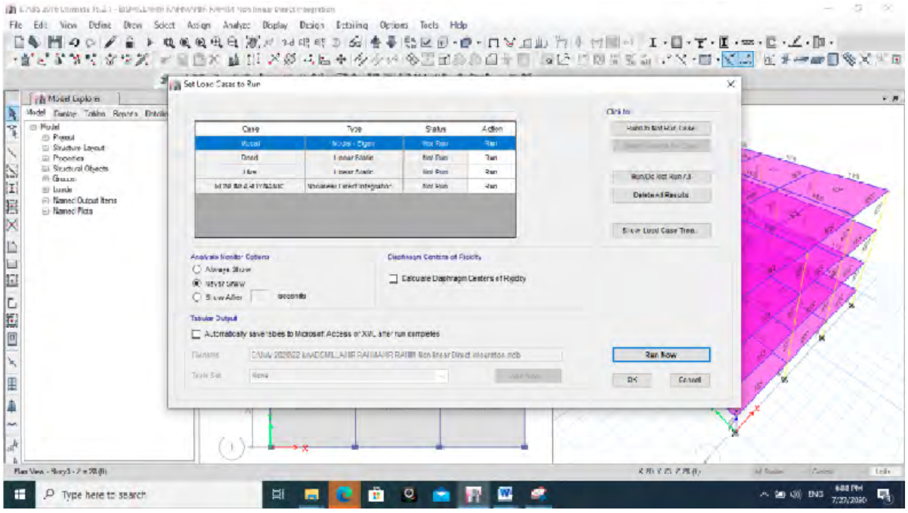
Section Properties



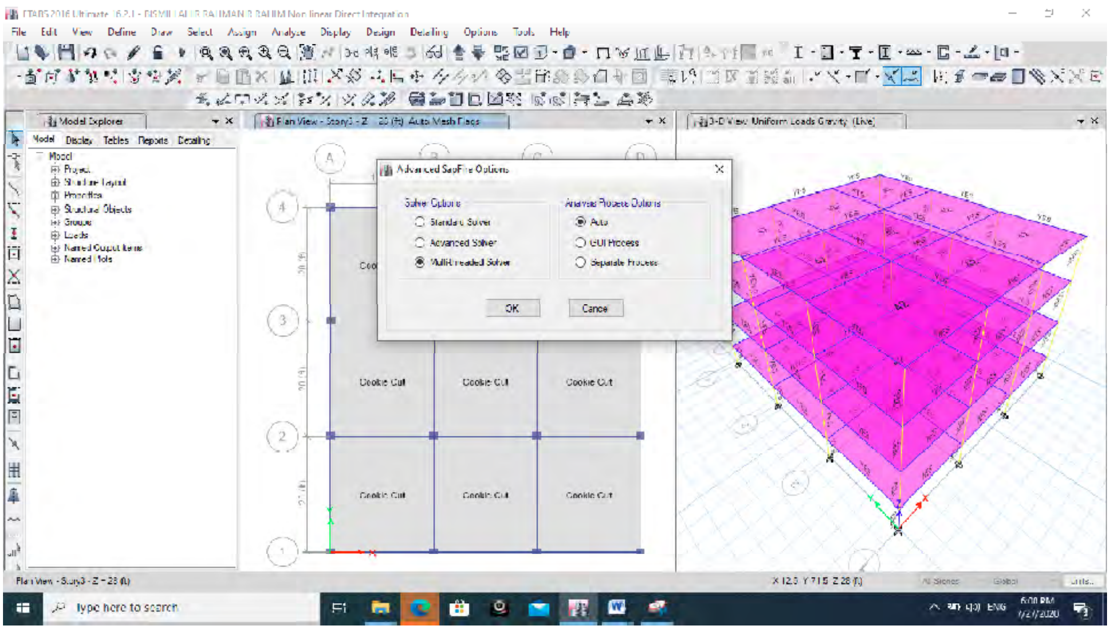
Section Properties Selected



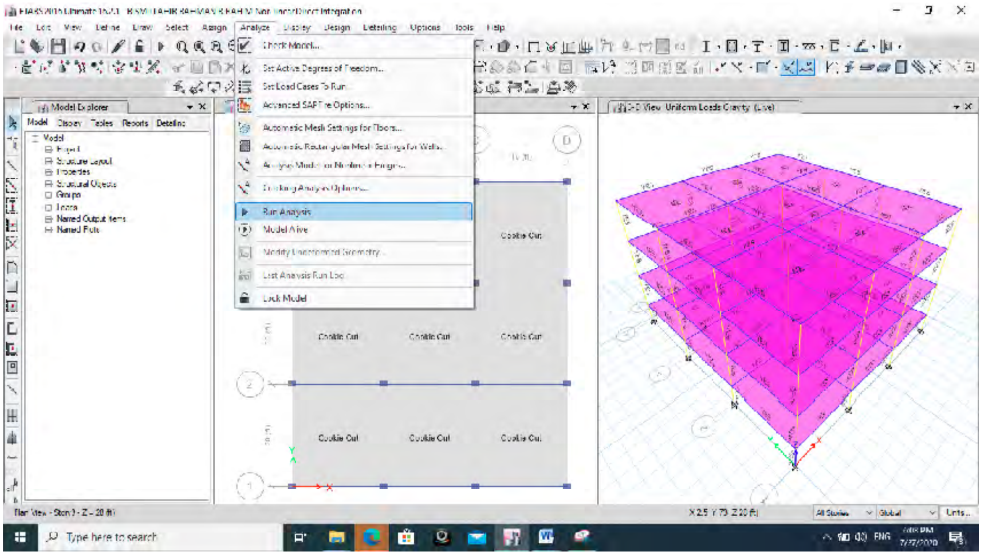
Run



Multithreaded Solver



Run Analysis



APPENDIX-B

Imperial Valley Earthquake Data

Earthquake Database-Time History Function Definition

Imperial Valley Data

Imperial earthquake database is downloaded from the Pacific Earthquake Engineering Research Center data base. Modern approaches to assessing seismic performance of infrastructure rely on good information about likely ground shaking at a site. Historically, access to earthquake ground motion data has been hampered by difficult access to the large body of data, as well as by the inconsistency in how the data are gathered and stored.

In the late 1990s Pacific Earthquake Engineering Research Center recognized the need to improve access to earthquake ground motion data and thus embarked on an effort to create a web-based searchable database of strong ground motion data. The first step was to collect the most important ground motion records worldwide. The second step was to ensure that all the data had been processed consistently and reliably. The following step was to gather related metadata such as earthquake magnitude, various site-to-source distance measures, style of faulting, local site conditions at the recording stations, and other relevant engineering parameters. Finally, Pacific Earthquake Engineering Research created the online database to make all the information available to the public

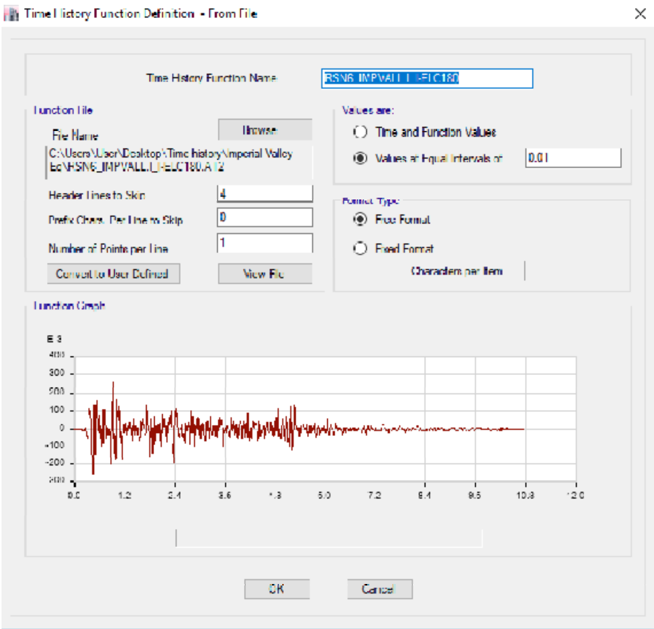
Imperial valley earthquake data is browsed from file**.** A portion of earthquake data file is shown below:

<u>Imperial Valley Strong Motion Database Record-Part</u>

```
PEER NGA STRONG MOTION DATABASE RECORD
Imperial Valley-02, 5/19/1940, El Centro Array #9, 180
ACCELERATION TIME SERIES IN UNITS OF G
NPTS-   5372, DT-   .0100 SEC,
   .9904052E 03   .9991426E 03   .9997266E 03   .1000260E 02   .1
000757E-02
   .1001207E-02   .1001612E-02   .1001966E-02   .1002269E-02   .1
002537E-02
   .1002757E-02   .1002925E-02   .1003053E-02   .1003140E-02   .1
003195E-02
   .1003243E-02   .1003288E-02   .1003316E-02   .1003334E-02   .1
003311E-02
   .1003195E-02   .1002974E-02   .1002630E-02   .1002125E-02   .1
001546E-02
   .1001034E-02   .1000775E-02   .1000968E-02   .1001736E-02   .1
003126E-02
   .1004969E-02   .1006837E-02   .1008275E-02   .1008834E-02   .1
008467E-02
   .1007064E-02   .1004826E-02   .1001962E-02   .9986503E-03   .9
949472E-03
```

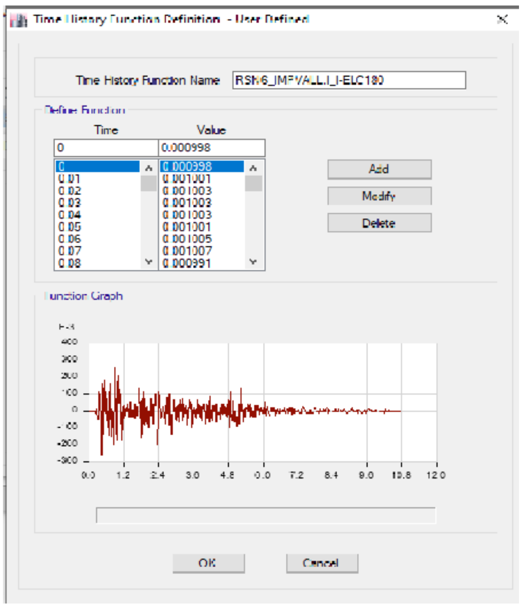
In earthquake data there are 4 lines above acceleration value. So, Header Lines to Skip is 4. Number of points per line is 1. Values at equal intervals of 0.01 sec. As shown below:

Time History Function Defined/Incorporated



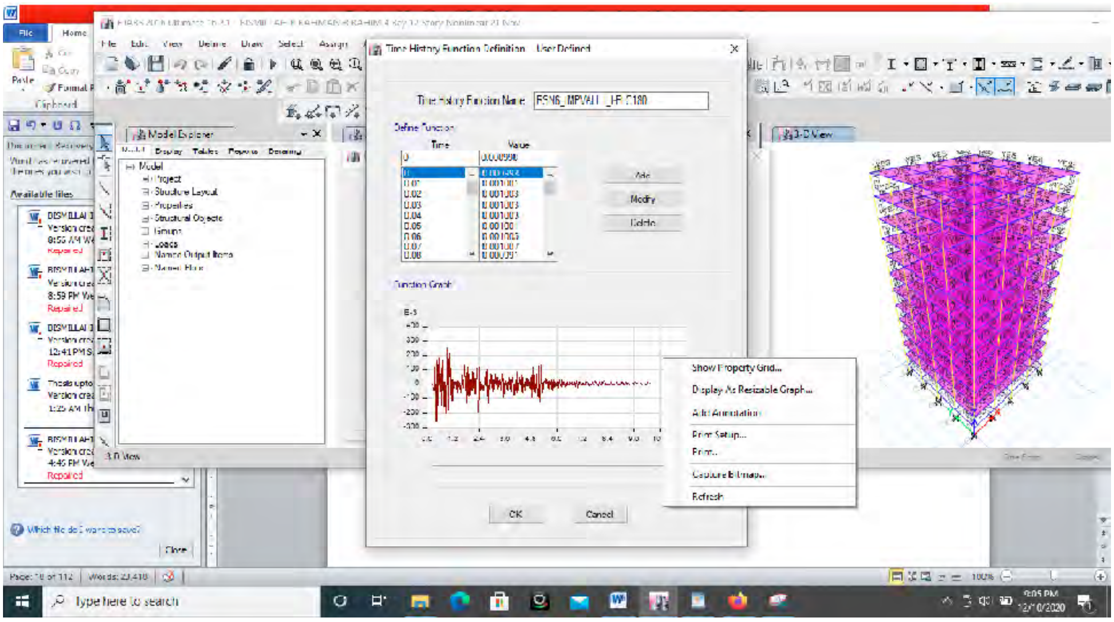
Earthquake data once incorporated from file to ETABS. Then data can be converted to user defined format. As shown below:

Time History Function Definition- User Defined



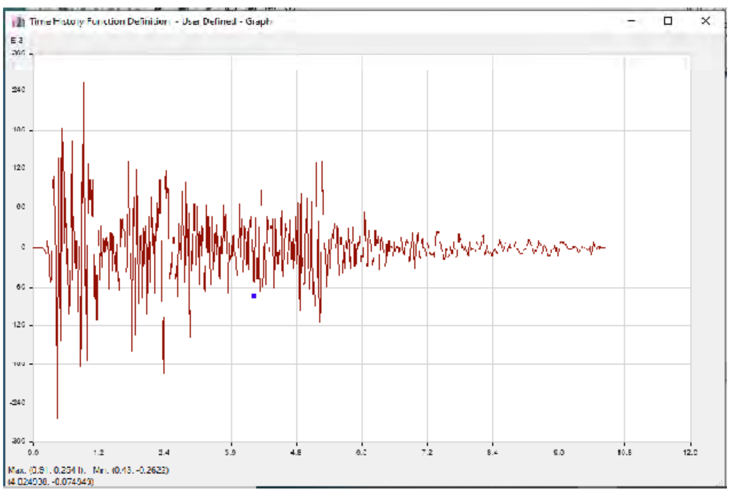
Time history function graph once found is converted to user defined graph by right clicking above/below the acceleration graph. Thereby the user defined graph can be found by clicking on Display as Resizable Graph.

<u>Pictorial Presentation to Display As Resizable Graph</u>

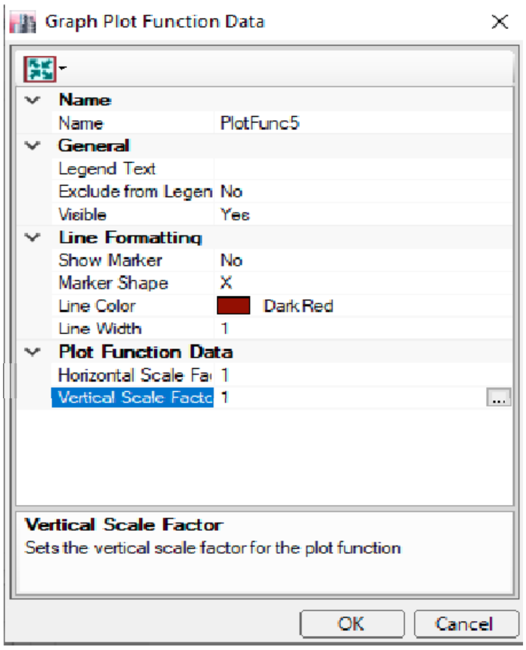


Thereby the User Defined Graph is Displayed. From this graph it is found that Peak Ground Acceleration is 0.2622g at 0.43 Sec.

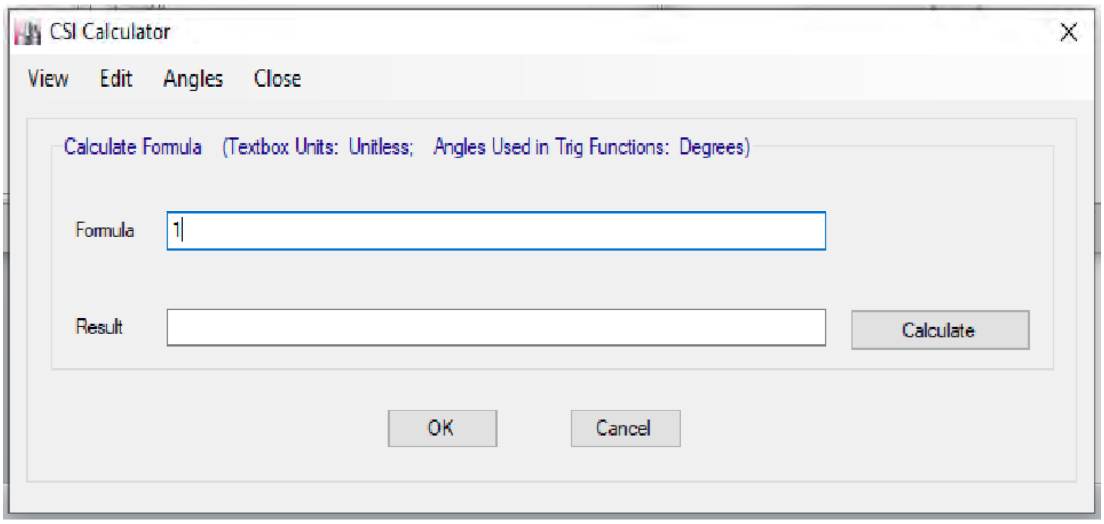
<u>User Defined Graph</u>



Once clicking on User Defined Graph, Graph Plot Function Data is displayed. In this box Horizontal scale factor and vertical scale factor is 1. For scaling up or down, the scale factor can be multiplied by the coefficient or value.

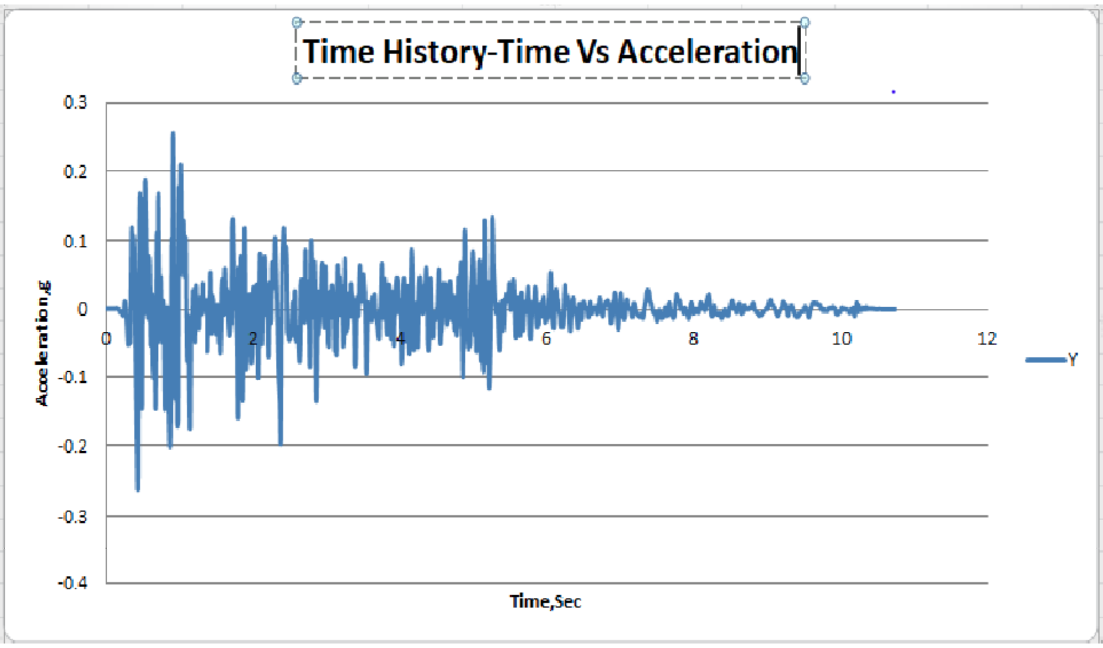


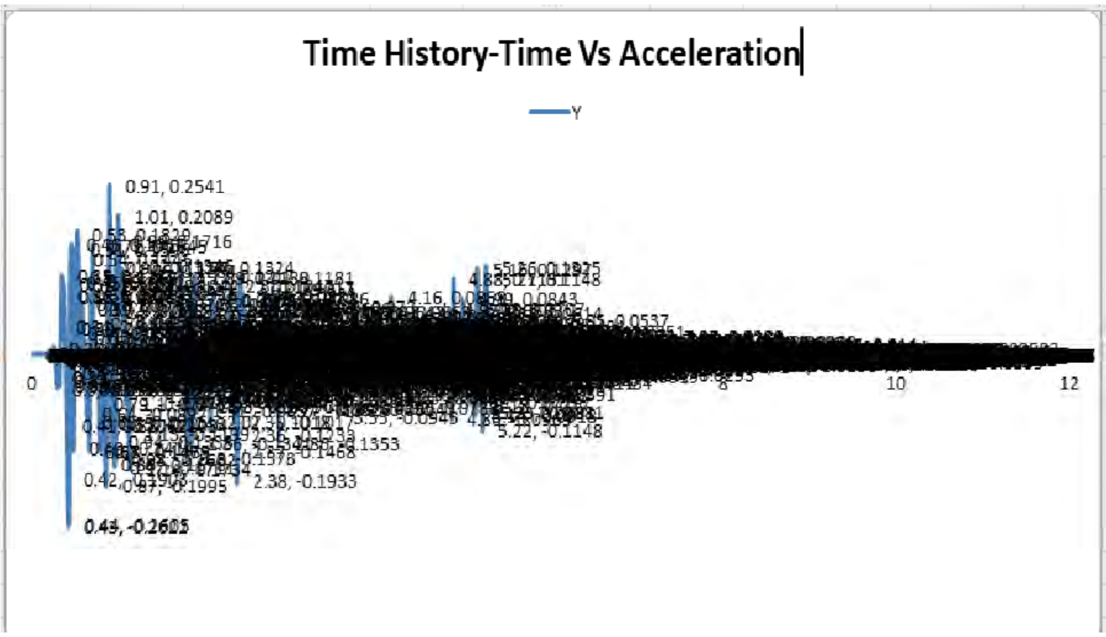
The CSI calculator appears by clicking right side rectangular box of vertical scale factor. Putting the required coefficient in formula box peak ground acceleration can be scaled down or up.



User Defined graph (Above/below) once clicked show property grid appears. By Clicking on show property grid, plot functions appear. On right side of plot function, once clicked plot function data (points 1075 defined) appears. On right side of 1075 once clicked Time history data appears. Then this data can be copied to Excel to have graphs.

Peak Ground Acceleration of Imperial Valley Earthquake

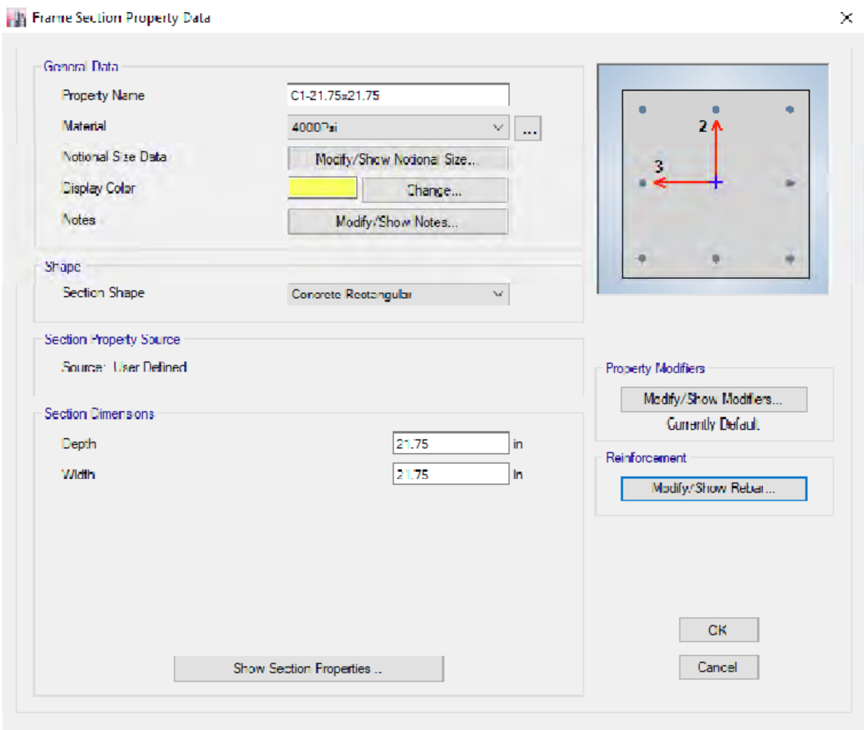




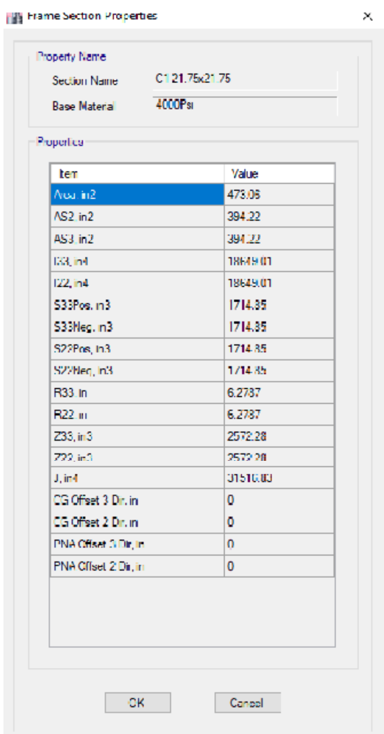
APPENDIX-C

Structural Element

Column



Column Section Properties


Frame Section Properties

Property Name

Section Name C1 21.75x21.75

Base Material 4000Psi

Properties

Item	Value
Area, in2	473.06
AS2, in2	394.22
AS3, in2	394.22
I33, in4	18649.01
I22, in4	18649.01
S33Pos, in3	1714.35
S33Neg, in3	1714.35
S22Pos, in3	1714.35
S22Neg, in3	1714.35
R33, in	6.2787
R22, in	6.2787
Z33, in3	2572.28
Z22, in3	2572.28
J, in4	31510.03
CG Offset 3 Dir, in	0
CG Offset 2 Dir, in	0
PNA Offset 3 Dir, in	0
PNA Offset 2 Dir, in	0

OK Cancel

Column Longitudinal Bars and Confinement Bars