PUSHOVER ANALYSIS OF STEEL FRAMES

Thesis submitted in partial fulfilment of the requirements for the degree of

MASTER OF TECHNOLOGY

in

STRUCTURAL ENGINEERING

by

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DEPARTMENT OF CIVIL ENGINEERING

NATIONAL INSTITUTE OF TECHNOLOGY

ROURKELA, ORISSA-769008

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CERTIFICATE

This is to certify that the thesis entitled **"PUSHOVER ANALYSIS OF STEEL FRAMES"** submitted by **Mr. PADMAKAR MADDALA** in partial fulfilment of the requirements for the award of Master of Technology Degree in Civil Engineering with specialization in Structural Engineering at the National Institute of Technology Rourkela is an authentic work carried out by him under my supervision.

To the best of my knowledge, the matter embodied in the thesis has not been submitted to any Other University/Institute for the award of any degree or diploma.

Place: NIT Rourkela
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ABSTRACT

Keywords: Pushover analysis, Steel frames,Bracings, Behaviour factor

In last decades Steel structure has played an important role in construction Industry. It is necessary to design a structure to perform well under seismic loads. The seismic performance of a multi-story steel frame building is designed according to the provisions of the current Indian code (IS 800 -2007). The shear capacity of the structure can be increased by introducing Steel bracings in the structural system. Bracings can be used as retrofit as well. There are 'n' numbers of possibilities to arrange Steel bracings such as D, K, and V type eccentric bracings. A typical six-story steel frame building is designed for various types of eccentric bracings as per the IS 800- 2007. D, K, and V are the different types of eccentric bracings considered for the present study. Performance of each frame is studied through nonlinear static analysis.

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NOTATIONS

F_i - Lateral force at i-th story

m_i - Mass of i-th story

ϕ_i - Amplitude of the elastic first mode at i-th story

V_b - Base shear

h - Height of i-th story above the base

N - Total number of stories

ΔF_N - Additional earthquake load added to the N-th story when h_N>25m

Q_i - Design lateral force at floor i,

W_i - Seismic weight of floor i,

h_i - Height of floor i measured from base

n - Number of stories in the building is the number of levels at which the masses are located

Γ_n - Modal participation factor for the n-th mode

ϕ_{in} - Amplitude of n-th mode at i-th story

A_n - Pseudo-acceleration of the n-th mode SDOF elastic system

CHAPTER -1

INTRODUCTION

Steel

Historical development of steel

Types of structural steel

Objectives

Methodology

Scope of the present study

Organization of the thesis

1. INTRODUCTION

In last decades Steel structure plays an important role in the construction industry. It is necessary to design a structure to perform well under seismic loads. Shear capacity of the structure can be increased by introducing Steel bracings in the structural system. Bracings can be used as retrofit as well. There are ‘n’ numbers of possibilities are there to arrange Steel bracings. Such as D, K, and V type eccentric bracings. Design of such structure should have good ductility property to perform well under seismic loads. To estimate ductility and other properties for each eccentric bracing Push over analysis is performed.

A simple computer-based push-over analysis is a technique for performance-based design of building frameworks subject to earthquake loading. Push over analysis attains much importance in the past decades due to its simplicity and the effectiveness of the results. The present study develops a push-over analysis for different eccentric steel frames designed according to IS-800 (2007) and ductility behaviour of each frame.

1.1 STEEL

Steel is by far most useful material for building construction in the world. Today steel industry is the basic or key industry in any country. It strength of approximately ten times that of concrete, steel is the ideal material of modern construction. It’s mainly advantages are strength, speed of erection, prefabrication, and demountability. Structural steel is used in load-bearing frames in buildings, and as members in trusses, bridges, and space frames. Steel, however, requires fire and corrosion protection. In steel buildings, claddings and dividing walls are made up of masonry or other materials, and often a concrete foundation is provided. Steel is also used in conjunction frame and shear wall construction. Due to its large strength to weight ratio, steel structures tend to be more economical than concrete structures for tall buildings and large span buildings and bridges. Steel structures can be constructed very fast and this enables the structure to be used early thereby leading to overall economy steel offers much better compressive and tensile strength than concrete and enables lighter constructions.

To get the most benefit out of steel, steel structures should be designed and protected to resist corrosion and fire. They should be designed and detailed for easy fabrication and erection. Good quality control is essential to ensure proper fitting of the various structural elements. The effects of temperature should be considered in design. Steel structures are ductile and robust and can withstand severe loadings such as earthquakes. Steel structures can be easily repaired and retrofitted to carry higher loads. Steel is one of the friendliest environmental building materials – steel is 100% recyclable.

To get the most benefit out of steel, steel structures should be designed and protected to resist corrosion and fire. They should be designed and detailed for easy fabrication and erection. Good quality control is essential to ensure proper fitting of the various structural elements. The effects of temperature should be considered in design. To prevent development of cracks under fatigue and earthquake loads the connections and in particular the welds should be designed and detailed properly. Special steels and protective measures for corrosion and fire are available and the designer should be familiar with the options available. Since steel is produced in the factory under better quality control, steel structures have higher reliability and safety.

1.2 HISTORICAL DEVELOPMENT OF STEEL

Steel has been known from 3000 BC steel was used during 500-400 BC in china and then in Europe. In India the Ashoakan pillar made with steel and the iron joints used in Puri temples are more than 1500 years old. The modern blast-furnace technology which was developed in AD1350 (Guptha 1998).

The large-scale use of iron for structural purposes started in Europe in the latter part of the eighteen century.The first major application of cast iron was in the 30.4 –m-span Coalbroakadale Arch Bridge by Darby in England, constructed in 1779 over the river Severn.The use of cast iron was continued up to about 1840. In 1740, Abraham Darby found a way of converting coal into coke, which revolutionized the iron –making process. In 1784 Henory Cort found way of wrought iron, which is stronger, flexible, and had a higher tensile strength than cast iron. During 1829 wrought iron chains were used in Menai Straits suspension bridge designed by Thomas Telford and Robert Stephenson’s Britannia Bridge was the first box girder wrought iron bridge. Steel was first introduced in 1740,but was not available in large quantities until Sir Henry Bessemer of England invented and patented the process of making steel in 1855 .In 1865,Siemens and Martin invented the open –hearth process and this was used extensively for the production of structural steel. Companies such as Dorman Long started rolling steel I-section by 1880.Riveting was used as a fastening method until around 1950when it was superseded by welding.Bessemers steel production in Britain ended in 1974 and last open –hearth furnace closed in 1980.The basic oxygen steel making (BOS) process using the CD converter was invented in Austria in 1953.Today we have several varieties of steel.

1.3 TYPES OF STRUCTURAL STEEL

The structural designer is now in a position to select structural steel for a particular application from the following general categories.

a) Carbon steel (IS 2062)

Carbon and manganese are the main strengthening elements. The specified minimum ultimate tensile strength for these varies about 380 to 450 MPa and their specified minimum yield strength from about 230 to 300MPa(IS 800:2007)

b) High –strength carbon steel

This steel specified for structures such as transmission lines and microwaves towers. The specified ultimate tensile strength, ranging from about 480-550 MPa, and a minimum yield strength of about 350-400 MPa.

c) Medium-and-high strength micro alloyed steel(IS 85000)

This steel has low carbon content but achieves high strength due to the addition of alloys such as niobium, vanadium, titanium, or boron. The specified ultimate tensile strength, ranging from about 440-590 MPa,and a minimum yield strength of about 300-450 MPa.

d) High –strength quenched and temperature steels(IS 2003)

This steel is heat treated to develop high strength. The specified ultimate tensile strength, ranging from about 700-950 MPa,and a minimum yield strength of about 550-700 MPa.

e) Weathering steels

This steel low-alloy atmospheric corrosion –resistant .They have an ultimate tensile strength of about 480 MPa and a yielded strength of about 350 MPa.

f) Stainless steels

This steel is essential low-carbon steel to which a minimum of 10.5% (max 20%) chromium and 0.5% nickel is added.

g) Fire-resistant steels

Also called thermo-mechanically treated steels, they perform better than ordinary steel under fire.

1.4 OBJECTIVES

Following are the main objectives of the present study:

a) To investigate the seismic performance of a multi-story steel frame building with different bracing arrangements such as D, K and V, using Nonlinear Static Pushover analysis method.

b) To evaluate the performance factors for steel frames with various bracing arrangements designed according to Indian Code.

1.5 METHODOLOGY

a) A thorough literature review to understand the seismic evaluation of building structures and application of pushover analysis.

b) Seismic behavior of steel frames with various eccentric bracings geometrical and structural details

c) Model the selected in seismic behavior of steel frames with various eccentric bracings computer software Seismostruct (2007).

d) Carry out pushover analysis of seismic behavior of steel frames with various eccentric bracings and arrive at a conclusion.

1.6 SCOPE OF THE PRESENT STUDY

In the present study, modeling of the steel frame under the push over analysis using Seismostruct (2007) software and the results so obtained have been compared. Conclusions are drawn based on the ductility and energy dissipation of pushover curves obtained.

1.7 ORGANIZATION OF THE THESIS

The thesis is organized as per detail given below:

Chapter 1: Introduces to the topic of thesis in brief.

Chapter 2: Discusses the literature review i.e. the work done by various researchers in the

Field of modeling of structural members by pushover analysis.

Chapter 3: In this chapter pushover analysis has been discussed in detail. The theory related

To pushover analysis also discussed in brief.

Chapter 4: Deals with the details of seismic behavior of steel frames with various eccentric bracing using Seismostruct (2007)

Chapter 5: The results from push over analysis, comparison between the steel frames with various eccentric bracing, all are discussed in this chapter.

Chapter 6: Finally, salient conclusions and recommendations of the present study are given in this chapter followed by the references.

CHAPTER -2

LITERATURE REVIEW

General

Literature Review on Pushover analysis

Limitations of existing studies

Closure

LITERATURE REVIEW

2.1 GENERAL

To provide a detailed review of the literature related to modeling of structures in its entirety would be difficult to address in this chapter. A brief review of previous studies on the application of the pushover analysis of steel frames is presented is this section. This literature review focuses on recent contributions related to pushover analysis of steel frames and past efforts most closely related to the needs of the present work.

2.2 LITERATURE REVIEW ON PUSHOVER ANALYSIS

- **Humar and Wright (1977)** studied the dynamic behaviour of multi-storeyed steel frame buildings with setbacks. The observations made based on a detailed parametric study are as follows. The fundamental period decreased by 35% for a setback of 90% (i.e., tower occupying 10% of the base area). The higher mode vibration of setback buildings made substantial contribution to their seismic response; these contributions increased with the slenderness of the tower. The contribution of the higher modes increased to 40% for a setback of 90%. For very slender towers the transition region between the tower and the base was, in some cases, subjected to very large storey shears. This increase in shear force was found to be as high as 300% to 400% for a setback of 90%. Storey drift ratios and storey shears for tower portions of setback buildings were substantially larger than for building without setbacks. For the tower portion, the increase in inter-storey drift was found to be four times compared to that of a regular structure. This increase was influenced by the extent of the setback. It was also observed that beam ductility demand in the tower portion showed a large increase with increase in the slenderness of the tower. The column ductility demands in the tower portion also showed a similar trend.

- **Shahrooz and Moehle (1990)** studied the effects of setbacks on the earthquake response of multi-storeyed buildings. In an effort to improve design methods for setback structures, an experimental and analytical study was undertaken. A six-storey moment-resisting reinforced concrete space frame with 50% setback in one direction at mid-height was selected. The analytical study focused on the test structure. The displacement profiles were relatively smooth over the height. Relatively large inter-storey drifts at the tower-base junction were accompanied by a moderate increase in damage at that level. Overall, the predominance of the fundamental mode on the global translational response in the direction parallel to the setback was clear from the displacement and inertia force profiles. The distribution of lateral forces was almost always similar to the distribution specified by the UBC code; no significant peculiarities in dynamic response were detected. To investigate further, an analytical study was also carried out on six generic reinforced concrete setback frames.
- **Wood (1992)** investigated the seismic behaviour of reinforced concrete frames with steps and setbacks. Two small-scale reinforced concrete 9-storeyed test framed structures (one -with steps and the other with setbacks) were constructed and subjected to simulated ground motion. The displacement, acceleration and the shear force responses of these frames were compared with those of seven previously tested regular frames. The setback structure comprises two-storey base with seven additional storeys in the tower portions. The stepped structure includes a three storey tower, a three storey middle section and a

three storey base. The displacement and shear force responses of these two frames were governed primarily by the first mode. Acceleration response at all levels exhibited the contribution of higher modes. The mode shapes for both the frames indicated kinks at the step locations. However, distributions of maximum storey shear were well represented by the equivalent lateral force distributions for all frames as given in UBC for regular frames. The differences between the linear dynamic analyses of regular, stepped and setback frames were not significant.

- **Ghobarah A. et al., (1997)** the control of inter story drift can also be considered as a means to provide uniform ductility over the stories of the building. A story drift may result in the occurrence of a weak story that may cause catastrophic building collapse in a seismic event. Uniform story ductility over all stories for a building is usually desired in seismic design.
- **Foley CM. (2002)** a review of current state-of-the-art seismic performance-based design procedures and presented the vision for the development of PBD optimization. It is recognized that there is a pressing need for developing optimized PBD procedures for seismic engineering of structures.
- **R. Hasan and L. Xu, D.E. Grierson (2002)** conducted a simple computer-based push-over analysis technique for performance-based design of building frameworks subject to earthquake loading. And found that rigidity-factor for elastic analysis of semi-rigid frames, and the stiffness properties for semi-rigid analysis are directly adopted for push-over analysis.

- **B. AKBAS.*et.al*.(2003**) conducted a push over analysis on steel frames to estimate the seismic demands at different performance levels, which requires the consideration of inelastic behaviour of the structure.
- **X.-K. Zou et al., (2005)** presented an effective technique that incorporates Pushover Analysis together with numerical optimisation procedures to automate the Pushover drift performance design of reinforced concrete buildings. PBD using nonlinear pushover analysis, which generally involves tedious computational effort, is highly iterative process needed to meet code requirements.
- **Oğuz, Sermin (2005)** Ascertained the effects and the accuracy of invariant lateral load patterns Utilized in pushover analysis to predict the behaviour imposed on the structure due to randomly Selected individual ground motions causing elastic deformation by studying various levels of Nonlinear response. For this purpose, pushover analyses using various invariant lateral load Patterns and Modal Pushover Analysis were performed on reinforced concrete and steel moment resisting frames covering a broad range of fundamental periods. The accuracy of approximate Procedures utilized to estimate target displacement was also studied on frame structures. Pushover analyses were performed by both DRAIN-2DX and SAP2000. The primary observations from the study showed that the accuracy of the pushover results depended strongly On the load path, the characteristics of the ground motion and the properties of the structure.
- **Mehmet et al., (2006),** explained that due to its simplicity of Push over analysis, the structural engineering profession has been using the nonlinear static procedure or pushover analysis. Pushover analysis is carried out for different nonlinear hinge

properties available in some programs based on the FEMA-356 and ATC-40 guidelines and he pointed out that Plastic hinge length (Lp) has considerable effects on the displacement capacity of the frames. The orientation and the axial load level of the columns cannot be taken into account properly by the default-hinge properties (Programme Default).

- **Shuraim et al., (2007)** summarized the nonlinear static analytical procedure (Pushover) as introduced by ATC-40 has been utilized for the evaluation of existing design of a building, in order to examine its applicability. He conducted nonlinear pushover analysis shows that the frame is capable of withstanding the pre-assumed seismic force with some significant yielding at all beams and columns.
- **Girgin. et.,(2007**),Pushover analysis has been the preferred method for seismic performance evaluation of structures by the major rehabilitation guidelines and codes because it is computationally and conceptually simple. Pushover analysis allows tracing the sequence of yielding and failure on member and structural level as well as the progress of overall capacity curve of the structure.
- **A. Shuraim et al., (2007)** the nonlinear static analytical procedure (Pushover) as introduced by ATC-40 has been utilized for the evaluation of existing design of a new reinforced concrete frame. Potential structural deficiencies in reinforced concrete frame, when subjected to a moderate seismic loading, were estimated by the pushover approaches. In this method the design was evaluated by redesigning under selected seismic combination in order to show which members would require additional reinforcement. Most columns required significant additional reinforcement, indicating

their vulnerability when subjected to seismic forces. The nonlinear pushover procedure shows that the frame is capable of withstanding the presumed seismic force with some significant yielding at all beams and one column.

- **Athanassiadou (2008)** analysed two ten-storeyed two-dimensional plane stepped frames and one ten-storeyed regular frame designed, as per Euro code 8 (2004) for the high and medium ductility classes. This research validates the design methodology requiring linear dynamic analysis recommended in Euro code 8 for irregular buildings. The stepped buildings, designed to Euro code 8 (2004) were found to behave satisfactorily under the design basis earthquake and also under the maximum considered earthquake (involving ground motion twice as strong as the design basis earthquake). Inter-storey drift ratios of irregular frames were found to remain quite low even in the case of the 'collapse prevention' earthquake. This fact, combined with the limited plastic hinge formation in columns, exclude the possibility of formation of a collapse mechanism at the neighbourhood of the irregularities. Plastic hinge formation in columns is seen to be very limited during the design basis earthquake, taking place only at locations not prohibited by the code, i.e. at the building base and top. It has been concluded that the capacity design procedure provided by Euro code 8 is completely successful and can be characterized by conservatism, mainly in the case of the design of high-ductility columns. The over-strength of the irregular frames is found to be similar to that of the regular ones, with the over-strength ratio values being 1.50 to 2.00 for medium – high ductility levels. The author presented the results of pushover analysis using 'uniform' load pattern as well as a 'modal' load pattern that account the results of multimodal elastic analysis.

- **Karavasilis et. al. (2008)** presented a parametric study of the inelastic seismic response of plane steel moment resisting frames with steps and setbacks. A family of 120 such frames, designed according to the European seismic and structural codes, were subjected to 30 earthquake ground motions, scaled to different intensities. The main findings of this paper are as follows. Inelastic deformation and geometrical configuration play an important role on the height-wise distribution of deformation demands. In general, the maximum deformation demands are concentrated in the tower-base junction in the case of setback frame and in all the step locations in the case of stepped frames. This concentration of forces at the locations of height discontinuity, however, is not observed in the elastic range of the seismic response.
- **A.Kadid and A. Boumrkik (2008),** proposed use of Pushover Analysis as a viable method to assess damage vulnerability of a building designed according to Algerian code. Pushover analysis was a Series of incremental static analysis carried out to develop a capacity curve for the building. Based on capacity curve, a target displacement which was an estimate of the displacement that the design earthquake would produce on the building was determined. The extent of damage Experienced by the structure at this target displacement is considered representative of the Damage experienced by the building when subjected to design level ground shaking. Since the Behaviour of reinforced concrete structures might be highly inelastic under seismic loads, the global inelastic performance of RC structures would be dominated by plastic yielding effects and consequently the accuracy of the pushover analysis would be influenced by the ability of the Analytical models to capture these effects.

- **Kala.Pet. al. (2010),** conducted study on steel water tanks designed as per recent and past I. S codes and they found Compression members are more critical than tension members. And he pointed out that, in Limit state method the partial safety factors on load and material have been derived using the probability concept which is more rational and realistic
- **P.Poluraju and P.V.S.N.Rao (2011),** has studied the behaviour of framed building by conducting Push over Analysis, most of buildings collapsed were found deficient to meet out the requirements of the present day codes. Then G+3 building was modelled and analysed, results obtained from the study shows that properly designed frame will perform well under seismic loads.
- **Haroon Rasheed Tamboli & Umesh N. Karadi (2012),** performed seismic analysis using Equivalent Lateral Force Method for different reinforced concrete (RC) frame building models that included bare frame, in filled frame and open first story frame. In modelling of the masonry Infill panels the Equivalent diagonal Strut method was used and the software ETABS was used for the analysis of all the frame models. In filled frames should be preferred in seismic regions than the open first story frame, because the story drift of first story of open first story frame is Very large than the upper stories, which might probably cause the collapse of structure. The infill Wall increases the strength and stiffness of the structure. The seismic analysis of RC (Bare frame) structure lead to under estimation of base shear. Therefore other response quantities such as time period, natural frequency, and story drift were not significant. The underestimation of base shear might lead to the collapse of structure during earthquake shaking.

- **Narender Bodige, Pradeep Kumar Ramancharla (2012),** modelled a 1 x 1 bay 2D four storied building using AEM (applied element method). AEM is a discrete method in which the elements are connected by pair of normal and shear springs which are distributed around the elements edges and each pair of springs totally represents stresses and deformation and plastic hinges location are formed automatically. Gravity loads and laterals loads as per IS 1893-2002 were applied on the structure and designed using IS 456 and IS 13920. Displacement control pushover analysis was carried out in both cases and the pushover curves were compared. As an observation it was found that AEM gave good representation capacity curve. From the case studies it was found that capacity of the building significantly increased when ductile detailing was adopted. Also, it was found that effect on concrete grade and steel were not highly significant.

2.3 LIMITATIONS OF EXISTING STUDIES

Many experimental and analytical works has been done by many researchers in the area of the pushover analysis of the steel frames. The concept of pushover analysis is rapidly growing nowadays.

This research is concerned with the pushover analysis of the steel frames. The uses of pushover analysis of the steel frames have been studied extensively in previous studies. However, many researchers performed experimentally and analytically on the pushover analysis but limited work is done on the study of pushover analysis. Push over analysis is carried out using Seismostruct (2012) software.

2.4 CLOSURE

The literature review has suggested that use of a pushover analysis of the steel frame is feasible. So it has been decided to use Seismostruct for the modeling and analysis. With the help of this software study of steel frame has been done.

CHAPTER -3

PUSH OVER ANALYSIS AND BEHAVIOUR FACTORS

Pushover Analysis– An Over View

Pushover Analysis Procedure

Lateral Load Profile

Use of Pushover Results

Limitations of Pushover Analysis

Behaviour factor

PUSH OVER ANALYSIS AND BEHAVIOUR FACTORS

3.1 PUSHOVER ANALYSIS – AN OVERVIEW

The use of the nonlinear static analysis (pushover analysis) came in to practice in 1970's but the potential of the pushover analysis has been recognized for last two decades years. This procedure is mainly used to estimate the strength and drift capacity of existing structure and the seismic demand for this structure subjected to selected earthquake. This procedure can be used for checking the adequacy of new structural design as well. The effectiveness of pushover analysis and its computational simplicity brought this procedure in to several seismic guidelines (ATC 40 and FEMA 356) and design codes (Euro code 8 and PCM 3274) in last few years.

Pushover analysis is defined as an analysis wherein a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components and elements of the building shall be subjected to monotonically increasing lateral loads representing inertia forces in an earthquake until a 'target displacement' is exceeded. Target displacement is the maximum displacement (elastic plus inelastic) of the building at roof expected under selected earthquake ground motion. The structural Pushover analysis assesses performance by estimating the force and deformation capacity and seismic demand using a nonlinear static analysis algorithm. The seismic demand parameters are storey drifts,global displacement(at roof or any other reference point), storey forces, and component deformation and component forces. The analysis accounts for material inelasticity, geometrical nonlinearity and the redistribution of internal forces. Response characteristics that can be obtained from the pushover analysis are summarized as follows:

a) Estimates of force and displacement capacities of the structure. Sequence of the member yielding and the progress of the overall capacity curve.

b) Estimates of force (axial, shear and moment) demands on potentially brittle elements and deformation demands on ductile elements.

c) Estimates of global displacement demand, corresponding inter-storey drifts and damages on structural and non-structural elements expected under the 20 earthquake ground motion considered.

d) Sequences of the failure of elements and the consequent effect on the overall structural stability.

e) Identification of the critical regions, when the inelastic deformations are expected to be high and identification of strength irregularities (in plan or in elevation) of the building. Pushover analysis delivers all these benefits for an additional computational effort (modeling nonlinearity and change in analysis algorithm) over the linear static analysis. Step by step procedure of pushover analysis is discussed next.

3.2 PUSHOVER ANALYSIS PROCEDURE

Pushover analysis can be performed as either force-controlled or displacement controlled depending on the physical nature of the load and the behavior expected from the structure. Force-controlled option is useful when the load is known (such as gravity loading) and the structure is expected to be able to support the load. Displacement controlled procedure should be used when specified drifts are sought (such as in seismic 2 1 loading), where the magnitude of the applied load is not known in advance, or where the structure can be expected to lose strength or become unstable.

Some computer programs (e.g. Seismostruct , DRAIN-2DX [44], Nonlinear version of SAP2000 [14], ANSYS [2]) can model nonlinear behavior and perform pushover analysis directly to obtain capacity curve for two and/or three dimensional models of the structure. When such programs are not available or the available computer programs could not perform pushover analysis directly (e.g. ETABS [13], RISA [45], SAP90 [12]), a series of sequential elastic analyses are performed and superimposed to determine a force displacement curve of the overall structure. A displacement-controlled pushover analysis is basically composed of the following steps:

1. A two or three dimensional model that represents the overall structural behavior is created.

2. Bilinear or tri-linear load-deformation diagrams of all important members that affect lateral response are defined.

3. Gravity loads composed of dead loads and a specified portion of live loads are applied to the structural model initially.

4. A pre -defined lateral load pattern which is distributed along the building height is then applied.

5. Lateral loads are increased until some member(s) yield under the combined effects of gravity and lateral loads.

6. Base shear and roof displacement are recorded at first yielding.

7. The structural model is modified to account for the reduced stiffness of yielded member(s).

8. Gravity loads are removed and a new lateral load increment is applied to the modified structural model such that additional member(s) yield. Note that a separate analysis with zero initial conditions is performed on modified structural model under each incremental lateral load. Thus, member forces at the end of an incremental lateral load analysis are obtained by adding the forces from the current analysis to the sum of those from the previous increments. In other words, the results of each incremental lateral load analysis are superimposed.

9. Similarly, the lateral load increment and the roof displacement increment are added to the corresponding previous total values to obtain the accumulated values of the base shear and the roof displacement.

10. Steps 7, 8 and 9 are repeated until the roof displacement reaches a certain level of deformation or the structure becomes unstable.

11. The roof displacement is plotted with the base shear to get the global capacity(pushover) curve of the structure (Figure 3.1).

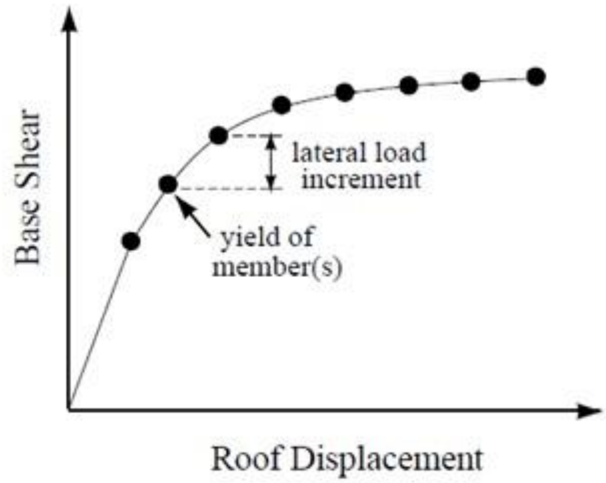


Figure 3.1: Global Capacity (Pushover) Curve of Structure

3.3 Lateral Load Profile

The analysis results are sensitive to the selection of the control node and selection of lateral load pattern. In general case, the centre of mass location at the roof of the building is considered as control node. In pushover analysis selecting lateral load pattern, a set of guidelines as per FEMA 356 is explained in Section 2.5.2. The lateral load generally applied in both positive and negative directions in combination with gravity load (dead load and a portion of live load) to study the actual behavior. Different types of lateral load used in past decades are as follows.

- ‘Uniform’ Lateral Load Pattern

The lateral fore at any story is proportional to the mass at that story.

$$F_i = m_i / \sum m_i$$

Where F_i : lateral force at i-th story

m_i : mass of i-th story

- 'First Elastic Mode' Lateral Load Pattern

  The lateral force at any story is proportional to the product of the amplitude of the elastic first mode and mass at that story,

  $F_i = m_i \phi_i / \sum m_i \phi_i$

  Where ϕ_i : amplitude of the elastic first mode at i-th story

- 'Code' Lateral Load Pattern

  The lateral load pattern is defined in Turkish Earthquake Code (1998) [53] and the lateral force at any storey is calculated from the following formula:

  $F_i = (V_b - \Delta F_N) \frac{m_i h_i}{\sum_{j=1}^{N} (m_j h_j)}$

  Where V_b : base shear

  h : height of i-th story above the base

  N : total number of stories

  ΔF_N : additional earthquake load added to the N-th story when h_N>25m

  (For $h_N \leq$25m. ΔF_N=0 otherwise; ΔF_N = $0.07 T_1 V_b \leq 0.2 V_b$ where T_1 is the fundamental period of the structure)

$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2}$

Where Q_i = Design lateral force at floor i,

W_i = Seismic weight of floor i,

h_i = Height of floor i measured from base, and

n = Number of stories in the building is the number of levels at which the masses are located.

3.3.1 'FEMA-273' Lateral Load Pattern

The lateral load pattern defined in FEMA_273 [18] is given by the following formula that is used to calculate the internal force at any story:

Where h: height of the i-th story above the base

K: a factor to account for the higher mode effects (k=1 for $T_1 \leq 0.5$ sec and k=2 for $T_1 > 2.5$ sec and varies linearly in between)

3.3.2 'Multi-Modal (or SRSS)' Lateral Load Pattern

The lateral load pattern considers the effects of elastic higher modes of vibration for long period and irregular structures and the lateral force at any story is calculated Square Root of Sum of Squares (SRSS) combinations of the load distributions obtained from the modal analysis of the structures as follows:

1. Calculate the lateral force at i-th storey for n-th mode from equations

   $F_{in} = \Gamma_n m_i \phi_{in} A_n$

   Where Γ_n : modal participation factor for the n-th mode

ϕ_{in} : Amplitude of n-th mode at i-th story

A_n : Pseudo-acceleration of the n-th mode SDOF elastic system

2. Calculate the storey shears, $V_{in} = \sum_{j=1}^{N} F_{jn}$ where N is the total number of storeys
3. Combine the modal storey shears using SRSS rule, $V_i = \sqrt{\sum_n (V_{in})^2}$.
4. Back calculate the lateral storey forces, F_i, at storey levels from the combined storey shears, V_i starting from the top storey.
5. Normalize the lateral storey forces by base shear for convenience such that

$$F_i' = F_i / \sum F_i.$$

The first three elastic modes of vibration of contribution was considered to calculate the 'Multi-Modal (or SRSS)' lateral load pattern in this study.

3.4 Use of Pushover Results

Pushover analysis has been the preferred method for seismic performance evaluation of structures by the major rehabilitation guidelines and codes because it is computationally and conceptually. Pushover analysis allows tracing the sequence of yielding and failure on member and structural level as well as the progress of overall capacity curve of the structure.

The expectation from pushover analysis is to estimate critical response parameters imposed on structural system and its components as close as possible to those predicted by nonlinear dynamic analysis. Pushover analysis provides information on many response characteristics that can't be obtained from an elastic static or elastic dynamic analysis.

These are

• Interstory drifts are estimates and its distribution along the height

• Determination of force demands on brittle members, are axial force demands on columns, beam-column connections are moment demands

• Deformation demands of determination for ductile members

• In location of weak points identification in the structure (or potential failure modes)

• Effort of an action strength deterioration of individual members on the behavior of structural system

• In plan or elevation identification of strength discontinuities that will lead to changes in dynamic characteristics in the inelastic range

• Verification of the completeness and adequacy of load path. Pushover analysis also exposes design weaknesses that may remain hidden in an elastic analysis. They are story mechanisms, excessive deformation demands, irregularities strength and overloads on potentially brittle members.

3.5 Limitations of Pushover Analysis

Although pushover analysis has advantages over elastic analysis procedures, underlying assumptions, pushover predictions are accuracy and limitations of current pushover procedures must be identified. Selection of lateral load patterns and identification of failure mechanisms for estimate of target displacement due to higher modes of vibration are important issues that affect the accuracy of pushover results. In a design earthquake target displacement are global displacement are expected. The mass center of roof displacement structure is used as target displacement. The estimation of target displacement accurate associated with specific performance objective affect the accuracy of seismic demand predictions of pushover analysis. Target displacement is the global displacement expected in a design earthquake. The estimate of target displacement, identification of failure mechanisms due to higher modes of vibration are important issues that affect, selection of lateral load patterns the accuracy of pushover results.

3.6 Behavior factor (*R*)

The behavior factor (*R*) is the ratio of the strength required to maintain the structure elastic to the inelastic design strength of the structure, Reza Akbari and Mahmoud R.Maheri (2001). In other words, it is a force reduction factor used to reduce the linear elastic response spectra to the inelastic response spectra. It is found through Push over analysis. The behavior factor, *R*, accounts for the inherent ductility, over strength of a structure and difference in the level of stresses considered in its design. FEMA (1997), UBC (1997) suggests the R factor in force-based seismic design procedures. It is generally expressed in the following form taking into account the above three components,

$$R = R_{\mu} \bullet R_{s} \bullet Y$$

$$R_{\mu} = \frac{V_e}{V_y}, R_s = \frac{V_y}{V_s}, Y = \frac{V_s}{V_w}$$

Where, R_{μ} is the ductility dependent component also known as the ductility reduction factor, R_S is the over-strength factor and Y is termed the allowable stress factor. With reference to *figure 3*, in which the actual force–displacement response curve is idealized by a bilinear elastic–perfectly plastic response curve, the behavior factor parameters may be defined as

$$R(R_w) = \left(\frac{V_e}{V_y}\right)\left(\frac{V_y}{V_s}\right)\left(\frac{V_s}{V_w}\right) = \frac{V_e}{V_w}$$

Where, V_e, V_y, V_s and V_w correspond to the structure's elastic response strength, the idealized yield strength, the first significant yield strength and the allowable stress design strength, respectively as shown in the Fig. 3.2.

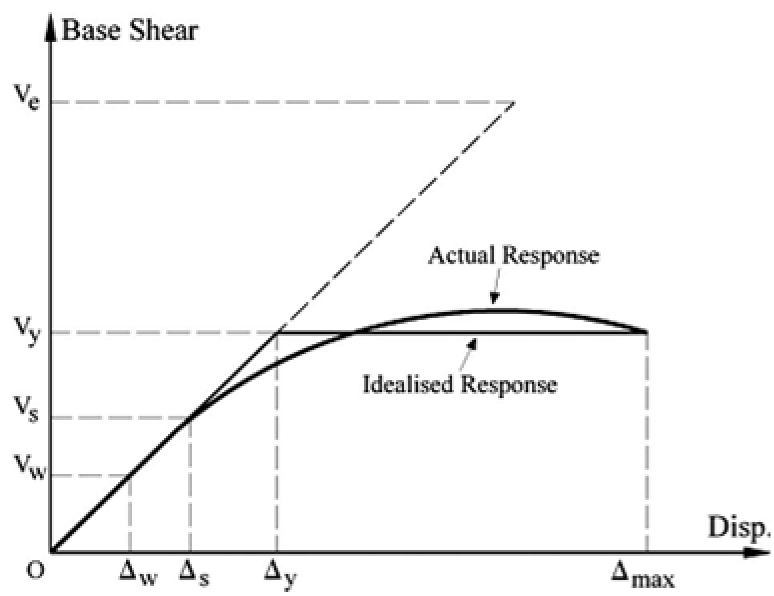


Figure 3.2: Typical Pushover response curve for evaluation of behavior factor, R

The structure ductility, μ, is defined in as maximum structural drift (Δ_{max}) and the displacement corresponding to the idealized yield strength (Δ_y) as:

$$\mu = \frac{\Delta_{max}}{\Delta_y}$$

CHAPTER -4

STRUCTURAL MODELLING

Introduction

Frame geometry

Frame designs

Modeling For Nonlinear Analysis

STRUCTURAL MODELLING

4.1 INTRODUCTION

The study in this thesis is based on nonlinear analysis of steel frames with eccentric bracings models. Different configurations of frames are selected such as D, K and V frames by keeping total weight of building is same. This chapter presents a summary of various parameters defining the computational models, the basic assumptions and the steel frame geometry considered for this study .Accurate modelling of the nonlinear properties of various structural elements is very important in nonlinear analysis. In the present study, beams and columns were modelled with inelastic flexural deformations using fibre based element using the software Seismostruct.

4.2 FRAME GEOMETRY

The details of frames are obtained from literature (AdilEmreÖzel, Esra Mete Güneyisi,2010).The buildings are assumed to be symmetric in plan, and hence a single plane frame may be considered to be representative of the building along one direction. Typical bay width and column height in this study are selected as 6m and 3m respectively. A configuration of 6 stories and 6 bays (G+5) is considered in this study. Different arrangements of steel frames such as K, D and V frames are considered as shown in fig. 4.1

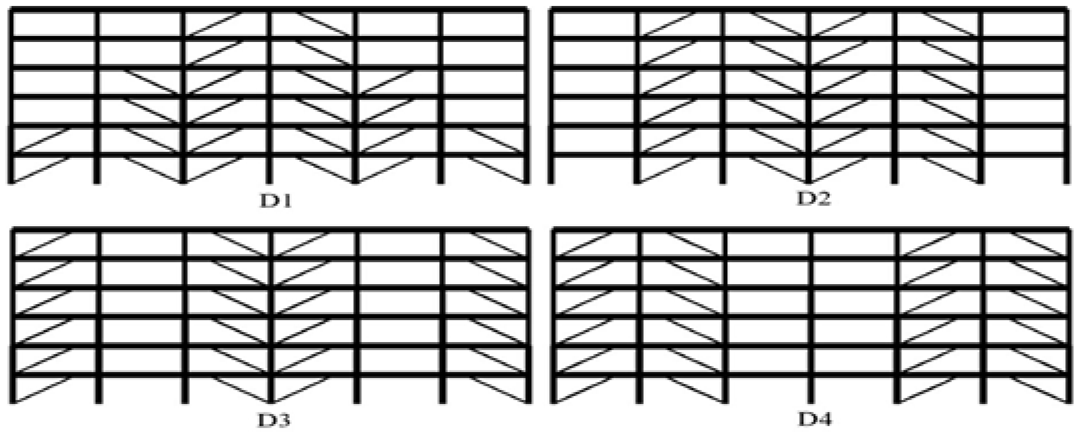


Fig 4.1(a): D-frames

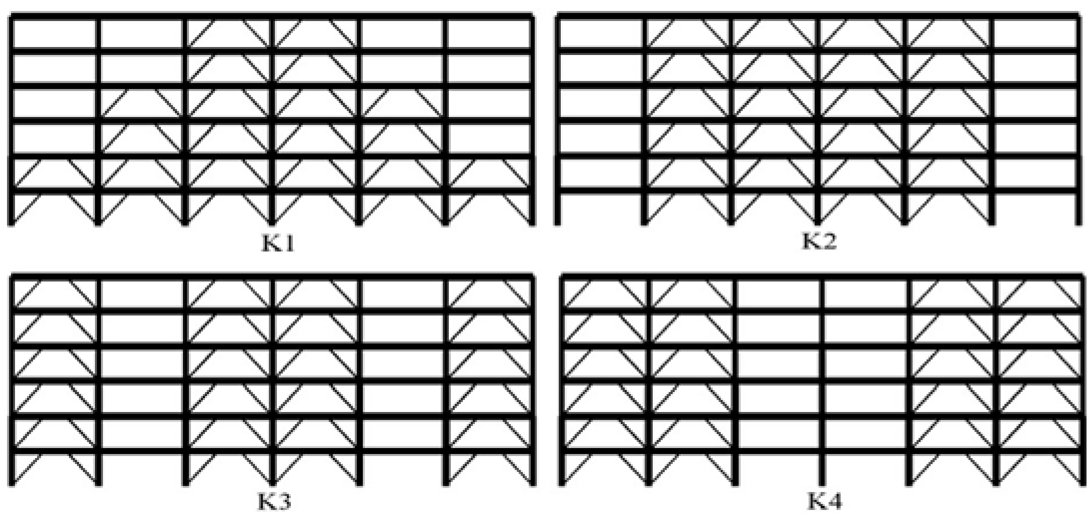


Fig 4.1(b): K-frames

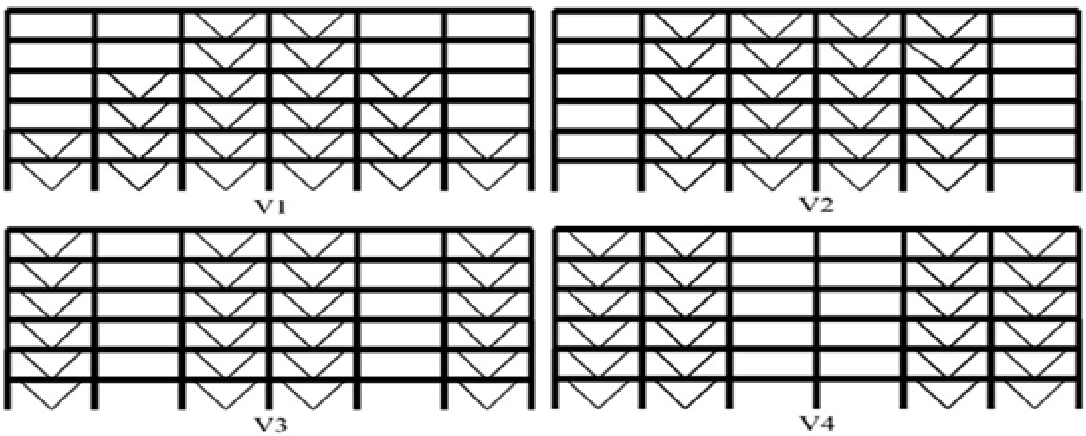


Fig 4.1(c): V-frames

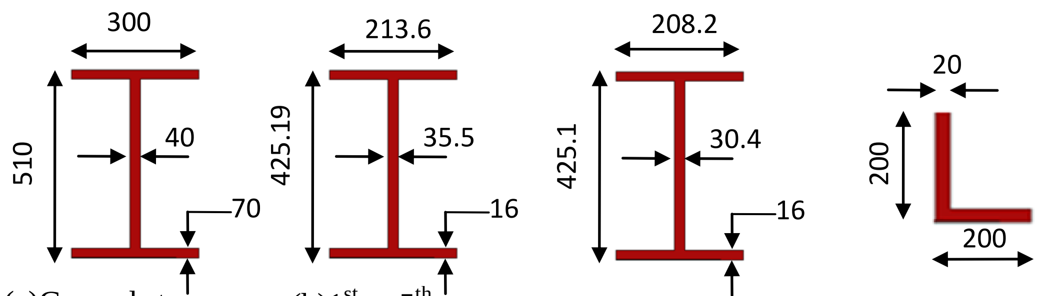
Fig 4.1. Braced frame Layout (a) D-frames, (b) K-frames and (c) V-frames

4.3 FRAME DESIGNS

The building frame considered in this study is assumed to be located in Indian seismic zone V with medium soil conditions. The design peak ground acceleration (PGA) of this zone is specified as 0.36*g*. The frame is designed as per prevailing practice in India. Seismic loads are estimated as per IS 1893 (2002) and the design of the steel elements are carried out as per IS 800 (2007) standards. The characteristic strength of steel is considered 415MPa. The dead load of the slab (6 m x 6 m panel), including floor finishes, is taken as 2.5 kN/m^{2}and live load as 3 kN/m^2. The design base shear (*VB*) is calculated as per IS 1893 (2002).

$$V = \left(\frac{Z}{2} \frac{I}{R} \frac{S_a}{g} \right) W$$

Where, seismic zone factor, Z = 0.36, Importance factor I = 1.0, Response reduction factor
R = 4.0. Estimated Base shear from above formula is found to be 357 kN. Figure 4.2, Shows the designed cross section details of steel columns, beams and bracings.



(a)Ground storey Column (b)1st to 5th storey Column (c)Beam (d)Bracings

Fig 4.2: *Cross sectional details of the frames*

4.4 MODELLING FOR NONLINEAR ANALYSIS

Non-linear properties of Steel frames are modeled in the program Seismostruct (2007). Seismostruct uses fiber based spread plasticity elements for frame elements. Bilinear stress-strain curve is used to model nonlinear property of steel. Eigen value analysis is performed to find its computational time period '*T*' and the corresponding mode shapes. Table 5.1 shows the Time period in sec for each type of frames along with its designations.

Nonlinear static pushover analyses of the frames are conducted. The first mode shape is taken as the load profile for the pushover analysis. The base shear versus roof displacement for each frames are generated and presented in the Fig.5.6. The seismic weight and design base shear levels of the selected buildings are almost in same range. The design base shear level is shown in the Fig. 5.6. In terms of the capacities of base shear and the displacements, each of the frames behaves differently. The difference between the frames considered is mainly in the arrangement of the bracings. To study the effectiveness of the arrangement of the bracings, the behavior factors of each frame are found out as per an accepted methodology Reza Akbariet. al. (2013).

CHAPTER -5

SEISMIC BEHAVIOUR OF STEEL FRAMES WITH VARIOUS ECCENTRIC BRACING ARRANGEMENTS

Introduction

Pushover analysis

Fundamental Period

Mode Shapes

Pushover Curve

Performance Parameters

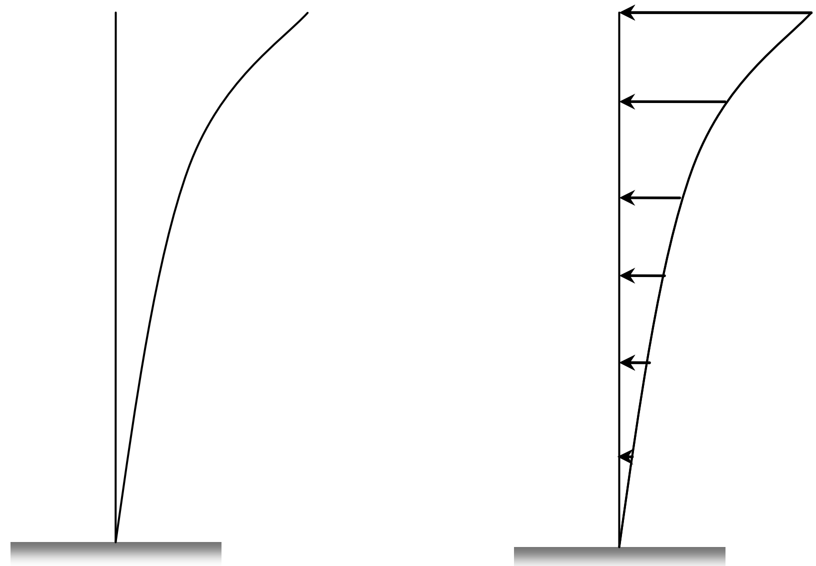
SEISMIC BEHAVIOUR OF STEEL FRAMES WITH VARIOUS ECCENTRIC BRACING ARRANGEMENTS

5.1 INTRODUCTION

The selected frame model is analyzed using pushover analysis. This chapter presents behavior factors for the different eccentric steel frames using pushover curves obtained from push over analysis. First natural time period Building and corresponding mode shape is calculated. Load for push over analysis is selected according to first mode shape. The results obtained from these analyses are compared in terms of behavior factors.

5.2 PUSHOVER ANALYSIS

Pushover analyses carried out using FEMA 356 displacement coefficient method. Building first natural time period and corresponding mode shape is found for all the building frames. A First mode shape load pattern was used for standard pushover analysis. Fig. 5.1 shows the load pattern used for standard pushover analysis and the typical 1st mode shape of the steel frames.



Typical Ist mode shape   Lateral Load profile

Fig 5.1 Ist mode shape and Lateral Load profile

5.3 FUNDAMENTAL PERIOD

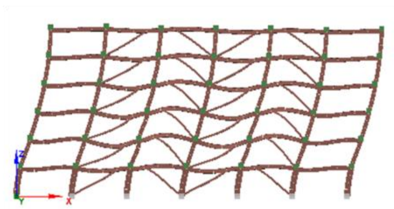
The fundamental time period of the frames are calculated by both IS code and model analysis methods. The values are presented in the Table 5.1. The fundamental time period of the frames, V, K and D are equal to 0.742s as per IS code. The time period from the model analysis is less than that suggested by the code in each case. This implies that the base shear attracted by the steel frames modelling the stiffness of braces will be more than that suggested by the code. The base shear increases approximately by 33% of design base shear.

Table 5.1: *Fundamental period of vibration*

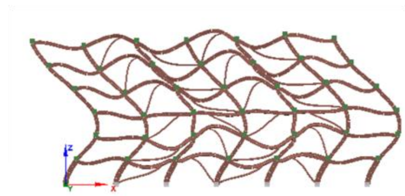
Frame	IS Code Time Period (T) sec	Computational Time Period (T) sec
V1	0.742	0.367
V2	0.742	0.355
V3	0.742	0.368
V4	0.742	0.362
D1	0.742	0.328
D2	0.742	0.339
D3	0.742	0.359
D4	0.742	0.346
K1	0.742	0.484
K2	0.742	0.485
K3	0.742	0.487
K4	0.742	0.489

5.4 MODE SHAPES

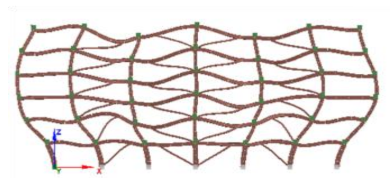
The mode shapes obtained for the frame V is shown in the Figure 5.1. The same types of mode shapes are obtained for other types of frames.



(a) 1 st mode shape



2 nd mode shape



3 rd mode shape

Fig 5.2 Mode shapes for V frames

5.5 PUSHOVER CURVE

The pushover curves for all the steel frames with V type of bracing are shown in Fig 5.3.. The type of curve is more close to an elastic plastic type. The initial slopes of the pushover curves are marginally same. The base shear capacity of steel frame V1 is marginally more than that of other frames. It is observed that over strength is high for V1 frames and ductility is more for V4 frames among the V family type.

The pushover curves for all the steel frames with D type of bracings are shown in Fig 5.4. The initial slopes of the pushover curves are marginally different. The base shear capacity of steel frame D3 is marginally more than that of other frames. It is observed that over strength is high for D1 frames and ductility is more for D1 frame among the D family type.

The pushover curves for all the steel frames with K type of bracing are shown in Fig 5.5. The initial slopes of the pushover curves are marginally same. The base shear capacity of steel frame K3 is marginally more than that of other frames. It is observed that over strength is high for K1 frames and ductility is more for K4 frame among the K family type.

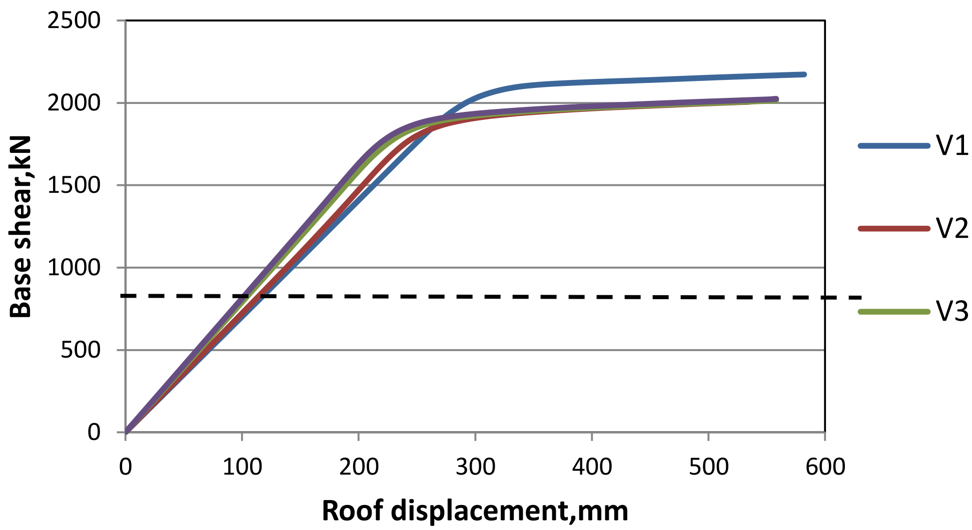


Figure 5.3: Comparison of Push over analysis of V Type Frames

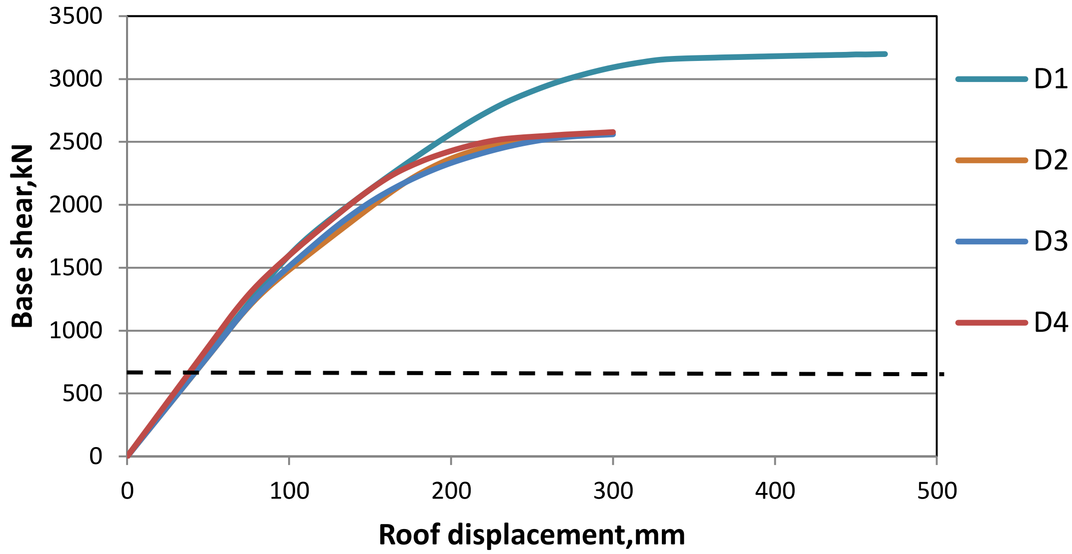


Fig 5.4: Comparison of Push over analysis of D Type Frames

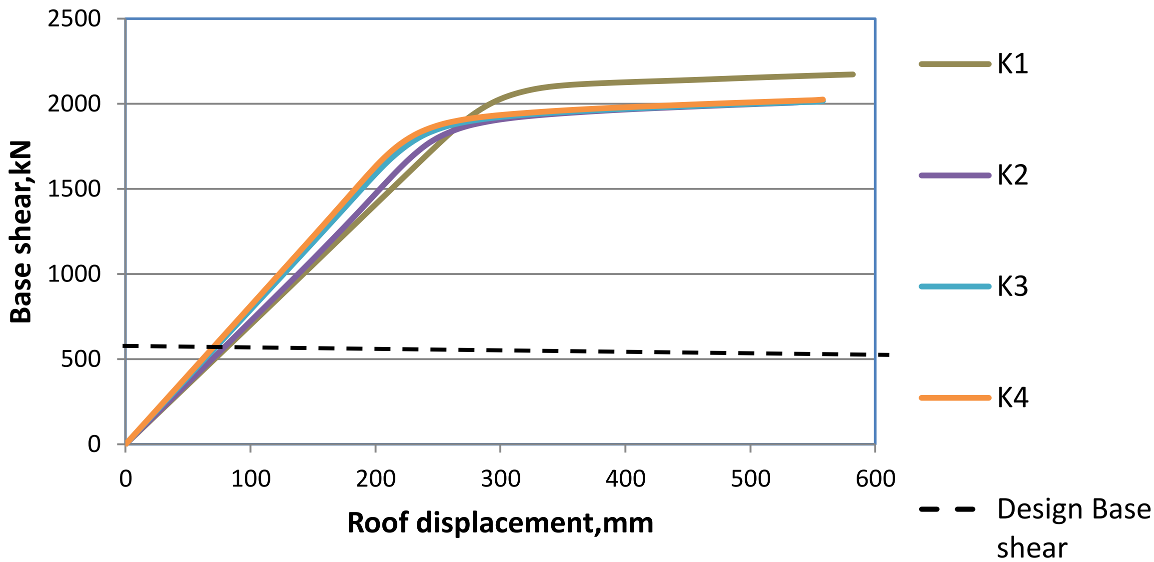


Fig 5.5: Comparison of Push over analysis of K Frames

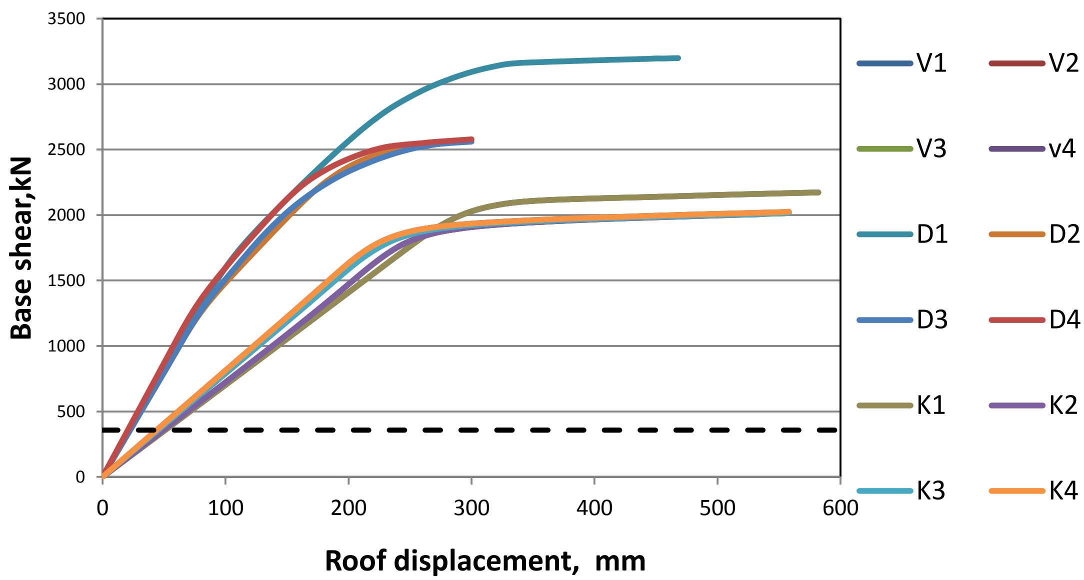


Fig. 5.6: Comparison of Push over analysis of all types of Frames selected

5.6 PERFORMANCE PARAMETERS

Table 5. 2 illustrates various frames considered for the study, time periods and response reduction factors considered for the design. The parameters, displacement ductility (Reza Akbariet. al.(2013)),R_μ, R_s, and Y are calculated and depicted in the Table 5.2 for all the frames. From table 5.2. In order to find the effectiveness of each bracing arrangement, the frames with same weights are considered. It is seen that V4 and D4 have more ductility when compare with other frames. Ductility reduction factor is more for D1 type of frame marginally. V4 and D1 give more over-strength factor. K3 gives more allowable stress factor. It can be seen that with regard to the frames V1, V2, V3 and V4 the total weight is same, and the behavior factor, R are different. For the frame V4, the R factor is marginally more than that of others. Hence the bracing arrangement of the frame V4 can be treated as relatively efficient. But overall D1 frame shows more reduction factor as shown in table1. Similarly it is found that the bracing

arrangement in D and K family, D1 & K4 respectively are found to be performing better compared to that of others.

Table 5.2: *R factors parameters of the frames.*

Frame	***Design R value***	***Ductility μ***	***Rμ***	***Rs***	***Y***	***Over strength***	***R***	***Total weight ton-force***
V1	4	1.94	1.73	1.59	3.57	5.67	9.83	884
V2	4	2.20	1.80	2.58	2.06	5.31	9.56	884
V3	4	2.33	1.88	1.65	3.30	5.44	10.29	884
V4	4	2.43	1.92	2.67	2.04	5.44	10.50	884
D1	4	2.42	2.02	2.84	2.85	8.09	16.41	884
D2	4	1.90	1.69	2.35	2.68	6.29	10.69	884
D3	4	1.92	1.75	1.60	4.00	6.40	11.31	884
D4	4	2	1.78	2.09	3.07	6.41	11.47	884
K1	4	2.03	1.72	1.34	4.38	5.86	10.14	884
K2	4	2.11	1.81	1.40	3.68	5.15	9.38	884
K3	4	2.42	1.91	1.14	4.65	5.30	10.21	884
K4	4	2.43	1.95	1.34	3.96	5.30	10.36	884

CHAPTER -6

SUMMARY AND CONCLUSIONS

Summary

Conclusions

SUMMARY AND CONCLUSIONS

6.1 SUMMARY

The selected frame models are analyzed using pushover analysis. The seismic performance of a multi-story steel frame building is designed according to the provisions of the current Indian code (IS 800 -2007). Shear capacity of the structure can be increased by introducing Steel bracings in the structural system. Bracings can be used as retrofit as well. There are ‘n’ numbers of possibilities to arrange Steel bracings such as D, K, and V type eccentric bracings. A typical six-story steel frame building is designed for various types of eccentric bracings as per the IS 800- 2007. D, K, and V are the different types of eccentric bracings considered for the present study. Performance of each frame is studied through nonlinear static analysis. Fundamental period of the Building frames and corresponding mode shapes are calculated. Pushover curves and behavior factors for the different eccentric steel frames are compared to find the relative performances of various frames considered.

6.2 CONCLUSIONS

Following are the major conclusions obtained from the present study.

- Modal analysis of a 2D steel frame models reveals that, there is huge difference between Computational Time periods and IS code Time period.
- Ductility of a moment-resisting steel frame is to some extent affected by its height. When bracing systems are included, the height dependency of ductility is greatly magnified. Shorter
- Steel-braced dual systems exhibit higher ductility and therefore higher R factors.
- Considering the range of ductility capacities shown by different systems discussed, it is found that the bracing arrangement in D and K family, D1 & K4 respectively are found to be performing better compared to that of others.

Bibliography

Bibliography

1. Akbas, B., Kara, F.I., and Tugsal, U.M. (2007), “Comparison of Pushover Analysis and Nonlinear Dynamic Time History Analysis on Low-, Medium-, and High-Rise Steel Frames”, Project No. 02-A-02-01-03, Scientific Research ProjectFund, Gebze Institute of Technology
2. Applied Technology Council, ATC-40. Seismic evaluation and retrofit of concrete Buildings, California, 1996; Vols. 1 and 2.
3. American Institute of Steel Construction Inc.– Steel Construction Manual (13th Edition)
4. AS 4100 ; 1998 – Australian Code for Design of Steel Structures
5. Bandyopadhyay T K and GuhaArijit, “Steel Structures” – Seminar on Revision of National Building Code(NBC) organized by Indian Institute of Engineers, Delhi Centre, 2002.
6. Bandyopadhyay T K and GuhaArijit, “Code Stipulations – Indian & International Codes and Revision of IS800” – Course at S V University College of Engineering, Tirupati, 2003
7. Bandyopadhyay T K and GuhaArijit, (2004), “Structural Member Design Based on Draft IS: 800 (Limit States Method)” - published in INSDAG journal, Steel In Construction
8. Bandyopadhyay T K and GuhaArijit, (2005), "Structural Member Design Based on Draft IS: 800 (Limit StatesMethod), Members subjected to Combined Bending and Axial Forces” - published in INSDAG journal, Steel In Construction

9. Chopra AK, Goel RK (2001), “A modal pushover analysis procedure to estimate seismic Demands for buildings: Theory and preliminary evaluation”, Pacific Earthquake Engineering Research Centre, University of California, Berkeley.

10. Duggal.S.K. ‘Limit State Design of Steel Structrues,Tata McGraw Hill Education

11. Federal Emergency Management Agency (FEMA),(1997).NEHRP provisions for the seismic rehabilitation of buildings. Rep FEMA 273 and 274. Washington, DC: FEMA.

12. IS 1893 Part 1 (2002),”Indian Standard Criteria for Earthquake Resistant Design of Structures”, Bureau of Indian Standards, New Delhi

13. IS: 800 (1984), General Construction in Steel – Code of Practice, Bureau of Indian Standards, New Delhi, 2007.

14. IS: 800 (2007), General Construction in Steel – Code of Practice, Bureau of Indian Standards, New Delhi, 2007

15. Ohtori, Y., Christenson, R.E., Spencer, Jr, Dyke, S.J. (2000). Benchmark Control Problems for Seismically Excited Nonlinear Buildings, http:/www.nd.edu/~quake/, Notre Dame University, Indiana.

16. Prakash, V., Powell, G.H., and Campbell, S. (1993). DRAIN-2DX User Guide V.1.10, Department of Civil Engineering, University of California at Berkeley.

17. Reza Akbari and Mahmoud R. Maheri (2013), “Analytical investigation of response modification (behavior) factor, R, for reinforced concrete frames rehabilitated by steel chevron bracing” , Structure and Infrastructure Engineering, Vol. 9, No. 6, 507–515.

18. Seismostruct (2012), “SeismoStruct - A computer program for static and dynamic nonlinear analysis of framed structures”, [online]. < http://www.seismosoft.com/ > (July 20,2012)

19. Structural steel design course material, NITTTR, Taramani, Chennai, Sep2010, pp2097.

20. Uniform Building Code (UBC), (1997). International conference of building officials. Whittier, CA: International Conference of Building Officials (ICBO).

21. Zou XK, Chan CM. (2001) ,”Optimal drift performance design for nonlinear pushover response of concrete structures”. In: WCSMO - 4: Proceedings of the fourth world congress of structural and multidisciplinary optimization

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