SEISMIC PERFORMANCE OF STEEL FRAMED BUILDING FROM PUSHOVER ANALYSIS WITH BRACING SYSTEMS

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Abstract

In last decades Steel structure has played an important role in construction Industry. It is important to plan a structure to perform well under seismic loads. The seismic execution of a multi-story steel outline building is planned by the arrangements 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 run of the mill six-story steel outline building is intended for different sorts of whimsical bracings as per the IS 800-2007. D, K, and V are the different types of eccentric bracings considered for the present study. Execution of each casing is considered through nonlinear static examination.

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

INTRODUCTION

There are „n" numbers of possibilities are there to arrange Steel bracings. Such as D, K, and V type eccentric bracings. Outline of such structure ought to have great flexibility property to perform well under seismic burdens. To estimate ductility and other properties for each eccentric bracing Pushover analysis is performed. A straightforward PC based push-over investigation is a method for execution based outline of building systems subject to seismic tremor stacking. Pushover investigation achieves much significance in the previous decades because of its effortlessness and the viability of the outcomes. The present study develops a push-over analysis for different eccentric steel frames designed according to IS-800 (2007) and ductility behavior of each frame.

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 column made with steel and the iron joints utilized as a part of Puri sanctuaries are over 1500 years of age. The modern blast-furnace technology which was developed in AD1350 (Guptha 1998). The substantial scale utilization of iron for basic purposes began in Europe in the last piece 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 altered the iron – making process. In 1784 Henory Cort discovered a method for fashioned iron, which is more grounded, 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 presented in 1740 however was not accessible in expansive amounts until Sir Henry Bessemer of England imagined and licensed the way toward making steel in 1855. In 1865 Siemens and Martin developed the open – hearth process and this was utilized widely for the generation of auxiliary steel. Organizations, for example, Dorman Long began moving steel I-area by 1880.Riveting was used as a fastening method until around 1950when it was superseded by welding. Bessemer steel production in Britain ended in 1974 and last open –hearth furnace closed in 1980.The basic oxygen steelmaking (BOS) process using the CD converter was invented in Austria in 1953.Today we have several varieties of steel.

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.

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.

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

LITERATURE REVIEW

- **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 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 can withstand the presumed seismic force with some significant yielding at all beams and one column.

- **Athanassiadou et al. (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 like 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.

1. A two or three-dimensional model that represents the overall structural behaviour 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 evacuated, and another parallel load augmentation is connected to the adjusted basic model with the end goal that extra member(s) yield. Note that a different investigation with zero introductory conditions is performed on changed auxiliary model under each incremental horizontal load. Therefore, part powers toward the finish of an incremental horizontal load examination are acquired by including the powers from the present investigation to the total of those from the past additions. As such, the consequences of each incremental sidelong load investigation 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.

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 behaviour. 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 Fi lateral force at i-th story

mi: 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 ϕi: 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 Vb : base shear

N: total number of stories

FN: additional earthquake load added to the N-th story when hN >25m

(For hN≤25m. ΔFN=0 otherwise; ΔFN= 0.07T1Vb ≤0.2 Vb where T1 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

Qi = Design lateral force at floor i,

Wi= Seismic weight of floor i,

hi= 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.

'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 T1 ≤0.5 sec and k=2 for T1 >2.5 sec and varies linearly in between)

'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

   Where Γn: modal participation factor for the

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

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

   $F_{in} = \Gamma_n m_i \phi_{in} A_n$
2. Calculate the storey shears, Vin= $\sum_{j=1}^{N} F_{jn}$ 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. 4. Back calculate the lateral storey forces, Fi, at storey levels from the combined storey shears, Vi 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.

The major day and age of the edges are computed by both IS code and model investigation strategies. The qualities are exhibited in the Table 5.1. The basic day and age of the edges, V, K and D are equivalent to 0.742s according to IS code. The era from the model investigation is not as much as that proposed by the code for each situation. This infers the base shear pulled in by the steel outlines demonstrating the firmness of props will be more than that proposed by the code. The base shear increments roughly by 33% of configuration base shear.

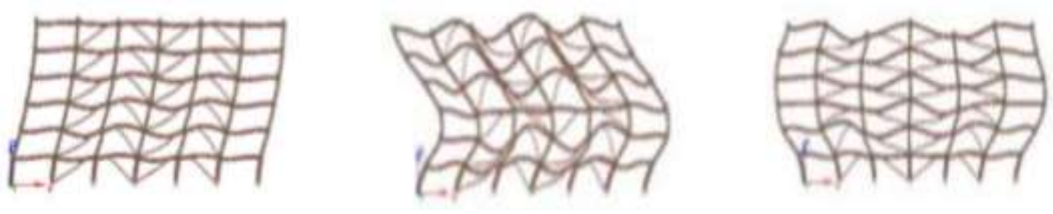
Table 5.1: Fundamental period of vibration

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

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 (b)2 nd mode shape (c)3 rd mode shape

Fig 5.2 Mode shapes for V frames

PUSHOVER CURVE

The sucker bends for all the steel outlines with V sort of propping are appeared in Fig 5.3. The sort of bend is more like a flexible plastic compose. The underlying slants of the sucker bends are possibly 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 underlying slants of the sucker bends are possibly extraordinary. 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

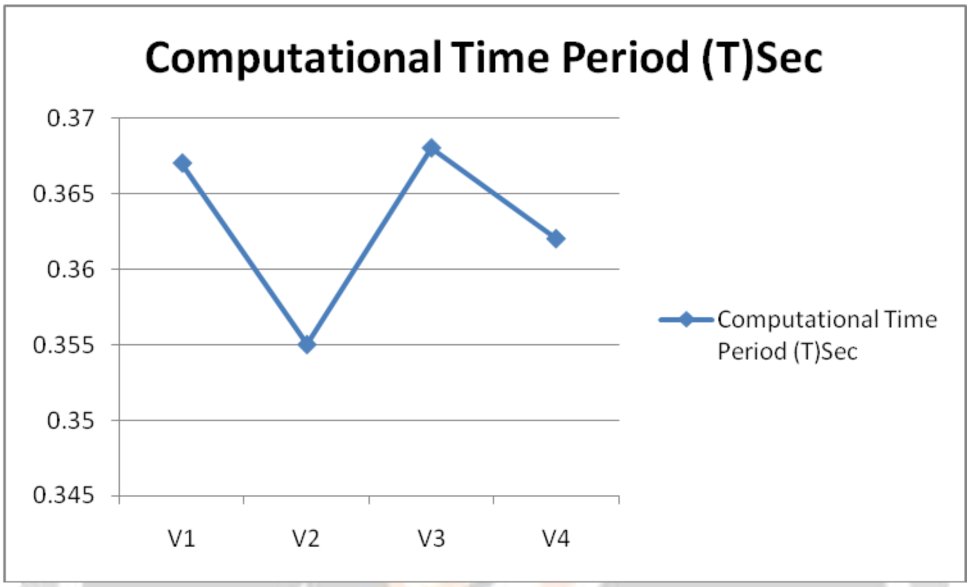


Figure 5.3: Comparison of Push over analysis of V Type Frames

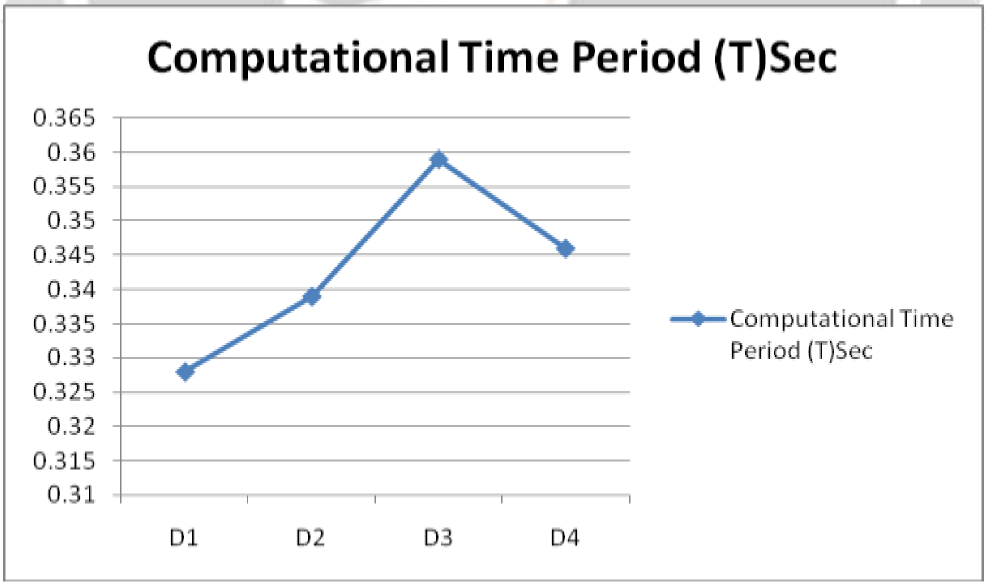


Fig 5.4: Comparison of Push over analysis of D Type Frames

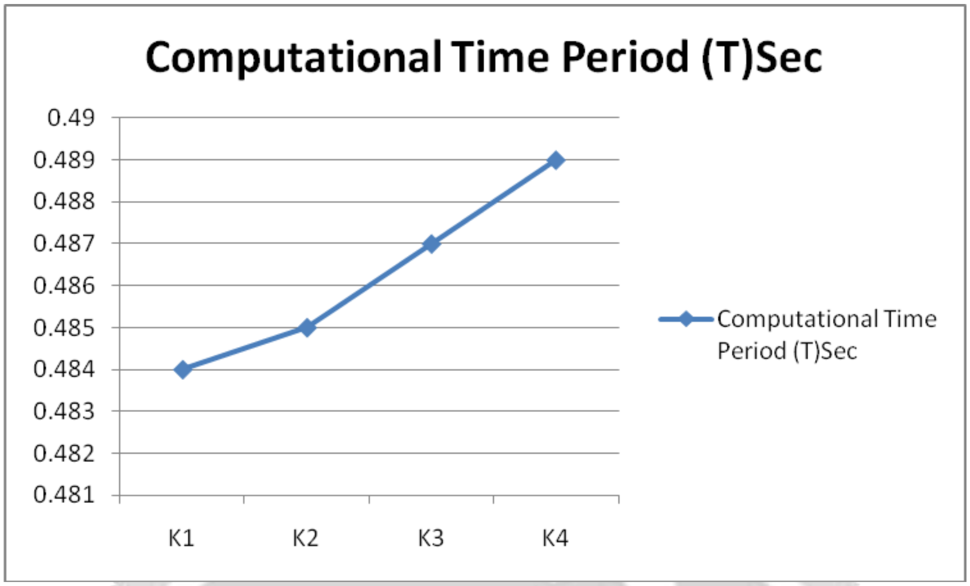


Fig 5.5: Comparison of Push over analysis of K Frame

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.

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