An Investigation on the Influence of Modeling Approach and Load Pattern on Seismic Performance of RC Structures

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ABSTRACT

Non-linear Static Analysis serves as a suitable measure to evaluate the performance of a structural system. The careful selection of modelling approach and the load pattern is critical to arrive at an adequate performance evaluation. The present study seeks to evaluate and compare the response of an existing eight story reinforced concrete structure, through the application of different modeling approaches and load patterns prescribed by FEMA 356. The results indicates that, with extreme clarity, that in all cases, the shape of the lateral load distribution is what the response of the buildings is finely accustomed to. This is especially true when different patterns of load are considered. It can also be observed that there is a very small difference between various load patterns.

Nomenclature

UD	Uniformly Distributed Load pattern	Ω	Over strength
ELF	Equivalent Lateral Force	μ	Ductility
MM	Mander's Stress–Strain Model	V_d & V_u	Design and Ultimate Base Shear
KPM	Kent and Park's Stress–Strain Model	W	Seismic weight
ISD	Interstorey Drift	A_h	Sesimic Coefficient
Δ_u & Δ_y	Ultimate Yield Displacement		

1. Introduction

Extreme shaking due to earthquakes is a relatively rare phenomena and the current earthquake design philosophy allows for some structural damage caused by shaking during strong earthquakes, in normal buildings, ensuring that they do not collapse. Linear analysis has to be augmented by nonlinear analysis. Nonlinear static analysis permits the inelasticity of structures and hence, is rightly regarded as an advancement made over linear static analysis. The height of the building, in the method of non-linear static analysis, is assumed with a set of static incremental lateral loads over it. It has a comparatively simplistic implementation process and reveals details about several aspects of the structure like its strength, deformation and ductility and even delineates the distribution of demands. This provides the information necessary to identify the liabilities of the structure, such as those key components that could plausibly reach their limit states in the course of an earthquake and consequently, require due consideration at the stage of designing and detailing of the structure. The present study attempts to evaluate the structure by adopting the non-linear static analysis by considering the available different modeling approaches pertaining to the stress–strain relationship of confined concrete

and loading patterns. Among the various stress-strain modeling approaches proposed by researchers Mander et al. [1] and Kent and Park's [2] modeling approaches are adopted to account for the uncertainty in modeling. Performance based analysis and design requires the pattern of loading to be applied laterally in an incremental order. For this purpose, the uncertainty in the loading pattern is examined by considering uniformly distributed load and equivalent lateral load [3] and IS: 1893(2016) [4] loading patterns. In this study performance based analysis and design is accomplished through non linear static analysis (push over) of tall RC bare framed structure, considering the uncertainties in modelling approaches and the lateral loading patterns

1.1. Structural Performance Limits

From the past few decades, the field of Earthquake Engineering has witnessed the gradual advent of concepts relating to the performance based seismic design philosophy, which clearly defines multiple performance limit states. Several factors such as gravity force level, expected plastic hinging mechanism and so on, affect the deformational capacity of an individual structure, thereby necessitating the definition of limit states for each such structure [5]. Limit states which are also referred to as structural performance points combined with seismic hazard levels define a Structure's Performance Objective. ATC 40 [6] and FEMA 356 [7] determine the varied performance levels on the basis of the intended function and type of the structure. ATC 40 specifies these performance levels as per the maximum inter-storey drift ratio, which is used across the globe as a measure to assess damage. Among various performance levels specified in ATC 40, the immediate occupancy level associated with an earthquake with 2%/50Yr probability is adopted in this study. There should not be any exceedance in the maximum inter-storey drift ratio beyond 0.01 for this performance level. The performance levels and objectives in this study are used as targets for performance of structure and serves as verification criteria.

1.2. Structural Model

In this study an existing eight storied structure is considered for the analysis and located in the seismic zone V. The structural model is of 3 and 2 bays in X and Y direction respectively, the overall plan dimension is 20 m × 6 m as shown in the Fig. 1, there are eight storey with each storey height of 3m. The beam dimensions are 300×600 mm with 8 numbers of 16 mm diameter and two legged 8mm diameter ties vertical stirrups with 130 mm c/c spacing. The column dimensions are 300×550 mm with 14 numbers of 20mm diameter with

five legged 8mm diameter ties at 150 mm c/c. The slab thickness is 150 mm. The material used is M_{26} grade concrete and Fe_{500} grade reinforcement. The beam and column dimensions are same throughout the height of the building. Table 1 presents the details of study frame.
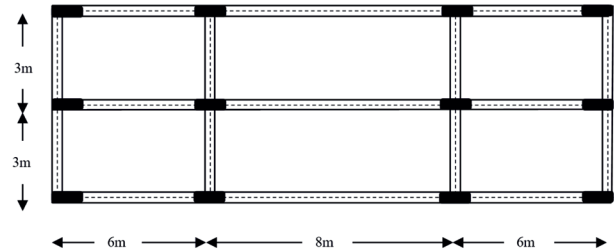

Figure 1: Beam-Column layout plan of the structural Model

Table 1: Details of the Study RC frame

Frame	Height(m)	W(kN)	A_h	V_d(kN)
8 Storey	24	16941	0.09	1525

1.3. Modelling of RC Members

1.3.1. Model Generation

SAP 2000 is a finite element software that is commercially available, was utilized to develop a 3D finite element model [8]. Beams and Columns are modeled as frame elements, the concrete slabs on each floor as shell elements and the base of the structure as fixed. Equal plane displacements were facilitated through the modelling of slabs as stiff diaphragms, in order to restrain all the nodes on each floor. Cracked sectional properties is accounted for by modifying the flexural stiffness for beams and columns using modification factors of 0.5 and 0.7 respectively [6, 7].

1.3.2. Concrete Confinement Models

A variety of stress strain models for confined concrete have been suggested by various researchers. Scott et al., reformed the Kent and Park model in order to take into account the enhancement in strength and ductility brought about by confinement and strain rate [9]. The concept of effectively confined concrete within the concrete core was incorporated by Sheikh and Uzumeri [10]. Adjustments to the model for flexural behavior of a column were later made by Sheikh and Yeh [11]. All the above concrete models are based on the column testing.

The modelling approaches involves in the derivation of the stress strain curve for the sectional member. Here, combined effect of concrete and steel is taken into consideration and suitable stress strain curves are formed. Modelling approaches takes the consideration of the effects of the monotonic and cyclic loading conditions

and simulates the hysteresis behavior of the structural elements are derived. In concrete sections confinement of reinforcements are made for the resistance against lateral forces. Modelling approaches also gives the information about the strength degradation of the structural elements and corresponding stress strain results plotted in the smooth curves. Here in this study the Mander's [1, 12] and Kent & Parks [2, 12] modelling approach have been considered.

1.3.2.1. Mander's Model (MM)

Mander's model [1, 12] is an extremely accepted model since it is simple and effective in considering the effects of confinement. It was first utilized for investigating the effect that various transverse reinforcement arrangements had on the confinement effectiveness and comprehensive performance, by testing the circular, rectangular and square full scale columns at seismic strain rates. The subsequent observations revealed that finding the peak strain and stress coordinates (ε_{cc}, f'_{cc}) would result in similarity in the behaviour over the entire stress-strain range, despite the arrangement of the confinement reinforcement used. The details of the
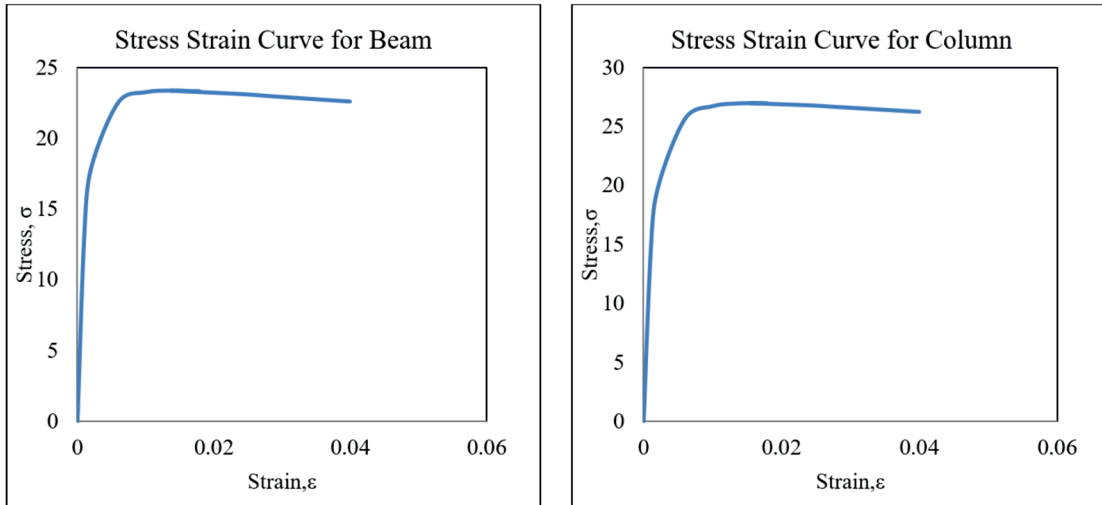
formulations considered for the derivation is presented in Table 2 and 3. Fig. 2 represents the derived stress strain curve for MM.

1.3.2.2. Kent and Park Model (KPM)

Kent and Park model [2, 12] was proposed in the year 1971.This model analysis gives a clear picture on the effect of confinement of concrete on stress strain curve. Rising branch is rendered by altering the Hognestad second degree parabola with strain value of 0.002. Stress strain curve has an ascending parabola and then assumed that confinement of concrete has no role with respect to ascending shape of parabola curve, correspondingly the maximum strain at the maximum stress. After the iteration and plotting the graph, this model analysis says that the ascending curve is same for the confined and unconfined concrete. It is assumed that the corresponding strain at 0.002 has the stress value equal to the cylinder strength of the concrete. The essential details of formulations considered for the development of the stress strain models by both Mander's model [1] and Kent and Park model [2] approaches are presented in Table 2.

Table 2: Description of Mander's Model [1, 12] and Kent and Park Model [2, 12]

S. No.	Model	Standard Stress Strain Curve	Envelope Curve
1	Mander's Model (MM)		$f_c = \dfrac{f'_{cc}\,xr}{r - 1 + x^r}$ $r = \dfrac{E_c}{E_c - E_{sec}}$
2	Kent & Park Model(KPM)		$f_c = f'_c\left[\dfrac{\varepsilon_c}{\varepsilon_{co}} - \left(\dfrac{\varepsilon_c}{\varepsilon_{co}}\right)^2\right]$ $f_c = f'_c\left[1 - Z\left(\varepsilon_c - \varepsilon_{co}\right)\right]$ $f_c = 0.2 f'_c$

Figure 2: Stress strain curve for Mander's Model

The various independent parameters and derived parameters required to generate the stress strain curves for beam and column elements of the study frame are computed and listed out in Table 3. The profile of the stress strain curves obtained for KPM is presented in Fig. 3. The parameters considered for deriving the stress strain relationship for beam and column elements by Kent and Park's approach is presented in Table 4.

Table 3: Required parameters for beam and column to derive stress strain relationship for mander's model

Parameter		Beam	Column
Concrete Cylinder Strength (f'_c)		20.8	20.8
Ratio 0f Volume of Reinforcement to Concrete Core (f)		0.05024	0.12941
Maximum Lateral Passive Confining Pressures	(σ_{1x})	0.69499	1.73747
	(σ_{1y})	0.34749	0.94771
Effective Area (A_e)		95259.3	71672.7
Confinement Effect Coefficient (K_e)		0.55721	0.49895
Effective Lateral Confining Pressure	(σ'_{1x})	0.38726	0.8669
	(σ'_{1y})	0.19363	0.47286
Magnification Factor (σ_{cc}/σ_{co})		1.12494	1.29695
Strength Of Confined Concrete (σ_{cc})		23.3987	26.9766
Axial Strain Corresponding to σ_{cc} (ε_{cc})		0.01382	0.01562
Initial Tangent Elasticity Modulus (E_{co})		22803.5	22803.5
Secant Modulus (E_{sec})		1693.41	1726.55
Constant (r)		1.08022	1.08192

Table 4: Tequired parameters for beam and column to derive stress strain relationship for kent and park model

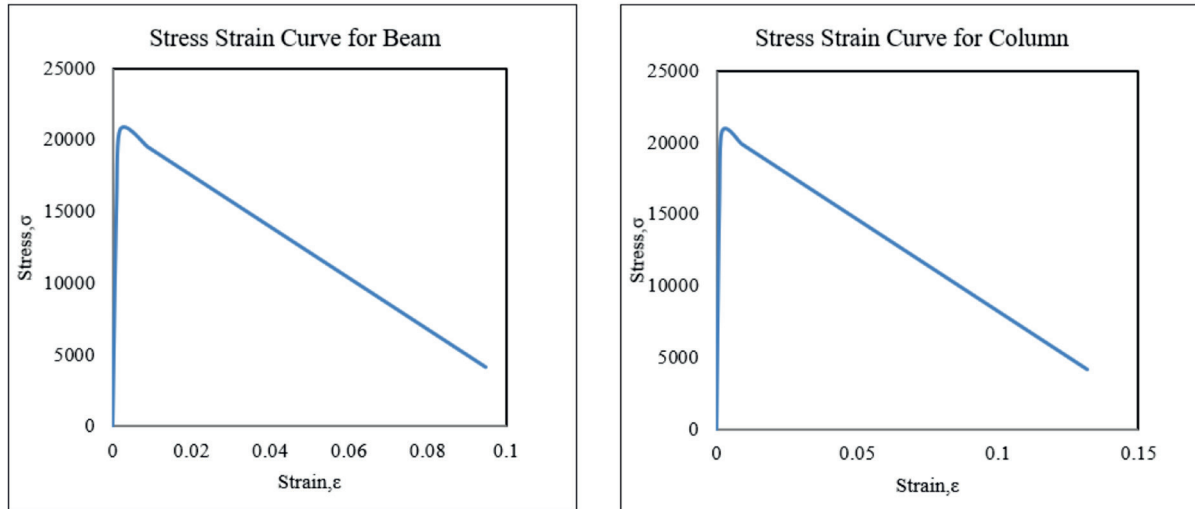| Parameter	Beam	Column
Concrete Cylinder Strength (f'_c)	20800	20800
Ratio of volume of reinforcement to concrete core (f)	0.1256	0.2156
ε_{50h}	0.0576	0.0808
ε_{50u}	0.0022	0.0022
Constant (Z)	8.6299	6.1628

Figure 3: Stress strain curve for Kent & Park Model

1.3.2.3. Modal Analysis

SAP 2000 was used to perform a computational modal analysis, an integral division of the study to recognize the structural dynamic characteristics for the 3D model. During this free vibration, there was a determination of periods and corresponding mode shapes for the structure. The description of the modes of vibration for a multistorey structure has numerous degrees of freedom and mode shapes. For the structure taken for the present study, the mode shapes were first derived before conducting its seismic analysis [13]. The first three mode shapes of the structures are identified with their frequencies as 0.684 Hz, 0.88176 Hz and 0.90 Hz for 1st, 2nd and 3rd mode shape respectively.

1.3.2.4. Plastic Hinge Characteristics

As recommended by the earlier researchers, for the frame type buildings, the lumped plasticity model with prospect of hinge formation at ends of a member is used for Pushover analysis. The default properties presented in FEMA 356 [7] and ATC 40 [6] are mostly favoured for practical use. Knowledge about the proficiency of the program and its underlying assumptions can help the user capitalize on the default hinge. Here in this study, the default hinges have been adopted as the focus of the study is modeling approach and variations in load pattern.

2. Methodology

The future of seismic design codes is inclined towards the idea of performance based seismic design. Nonlinear analysis procedures assume importance in this approach, especially in ascertaining the range and patterns of damage, which in turn are crucial in gauging the inelastic behaviour and failure pattern in structures brought about by seismic events of some severity [14]. To procure the reaction of a particular structure that has been treated with a lateral load pattern that is monotonically increasing, an incremental iterative solution of the static equilibrium equations has been undertaken. This process facilitates the observation of the series of yielding and failure on the member and structural level [15]. Pushover analysis can help discern a characteristic nonlinear force displacement relationship of the MDOF system. Selecting a suitable load pattern is then a decisive step in pushover analysis. Using two different load patterns and enveloping the outcome can serve as one practical possibility [16]. Diverse patterns ranging from simplistic rectangular and inverted triangular ones to more advanced modal and modal adaptive ones have been suggested [17]. A thorough discussion by Krawinkler and Seneviratna provides an elaborated discussion of Pushover analysis [18].

2.1. Load Patterns

The performance of push over analysis requires an equal load pattern and earthquake load, the former of which is laterally applied to the structure in increments.

2.1.1. Lateral load Distribution as per FEMA-356

Here in this study, Uniform Distribution (UD) and Equivalent Lateral Force Distribution (ELF) load patterns are considered.

(a) The uniform distribution can be calculated by the equation

$$F_i = \frac{m_i}{\sum mj}$$

Where, m_i is the storey mass

(b)   The equivalent lateral force can be calculated as

$$F_i = \frac{m_i h_i^k}{\sum m_i h_j^k}$$

Where, k = 1.0; T ≤ 0.5s and k= 2.0; 2.5 ≤ T

2.1.2. IS: 1893 (2016) Response Spectrum Load

The design base shear is computed as per the equation provided below and is distributed along the height of the structure.

$$Q_i = V_b \frac{W_i (h_i)^2}{\sum_{j=1}^{n} W_j (h_i)^2}$$

Using the above expressions and computations, the profile of the load patterns obtained is as shown in the Fig. 4.
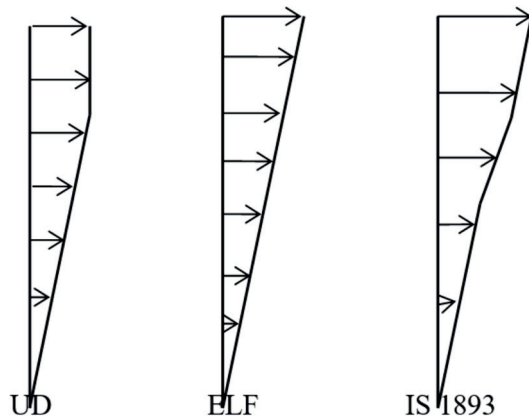

Figure 4: Lateral Load Pattern

The three load patterns computed are applied to the study frame under two modelling approaches i.e., Mander's Model (MM) and Kent and Park's Model (KPM) and a parametric study is conducted.

3.   Results and Discussions

3.1. On Pushover Curves

When the inelastic behavior is present, the structure dissipates more amount of seismic energy. The capacity curve serves as an effective index of the inelastic behavior of structure. The maximum roof displacement and base shear of the building in displacement controlled analysis can be obtained. Maximum displacement in all the models did not exceed 20 cm, due to the imposed limit.

The base shear are found in UD is higher when compared to the ELF and IS1893 load distribution. The uniform load produced larger base shear forces for the displacement of 111mm for Mander's model and 110mm for Kent and Park's model. The differences between the IS1893 load and ELF are minimum. The displacement value for UD versus ELF and IS 1893 load distribution is found to have differences 2mm in Mander's modelling approach.

The results indicate a higher base shear capacity as compared to the design base shear for the structure, assuring its safety for the designed level of earthquake. The elastic displacements are smaller than the inelastic displacements when the period of the structure is short. The Time period during transition of a structure from elastic to inelastic phase also increases. Decreased stiffness is introduced through cumulative material straining, consequently leading to the elongation of the periods of vibration. This reveals significant transformations in the response characteristics of the building.

A difference of less than 1% is showcased by comparing the UD results with the ELF results. This can be attributed to the sharing of inertia forces in the elastic range. As a result, consideration for higher mode effects is not completely given in the post elastic phase, in Mander's model approach.

In Kent and Park's modeling approach, there is reduction in the base shear by 7.8% at the performance level in the uniform force distribution when compared to Mander's model, however the differences between UD and ELF pattern base shear results is less than 1%, but there is reduction in 8% of base shear in Kent and Park's approach in comparison with Mander's approach.

The above discussion on pushover curves is made in references to the Table 5 and Table 6. The variations in the spectral acceleration, spectral demand, time period and damping is very minimal at the performance point for all the load patterns for Mander's model, however, the Kent and park model produces slighltly lesser values.
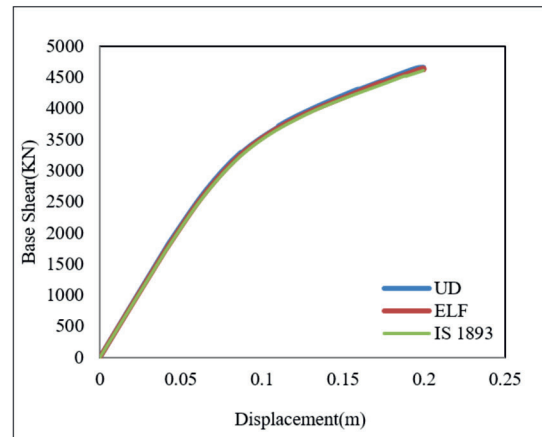
Fig. 5 shows the pushover curves for Mander's model and Kent and Park's model. It can be understood from the curve that the variation of base shear with respect to displacement is almost same for all the three load pattern. The structures moves into inelastic phase at the base shear value of 2163kN for all the load patterns for the displacement of 5mm in the Mander's model where as in the Kent and Park model, for the base shear value of 1700kN the structures enters in to the inelastic phase with displacement of 4mm, however the ultimate base shear is found be almost near in both the modeling approaches.

TABLE 5: Pushover parameters at performance point for Mander's Model

Parameter	UD	ELF	IS 1893
V (kN)	4013.343	3979.695	3979.695
Δ(m)	0.111	0.113	0.113
S_a	0.294	0.292	0.292
S_d	0.086	0.087	0.087
T_{eff} (s)	1.086	1.095	1.095
B_{eff}	0.133	0.113	0.113

TABLE 6: Pushover parameters at performance point for Kent and Parks's Model

Parameter	UD	ELF	IS 1893
V (kN)	3720.051	3683.734	3683.734
Δ (m)	0.11	0.112	0.112
S_a	0.27	0.268	0.268
S_d	0.086	0.087	0.087
T_{eff} (s)	1.13	1.139	1.139
B_{eff}	0.13	0.129	0.129
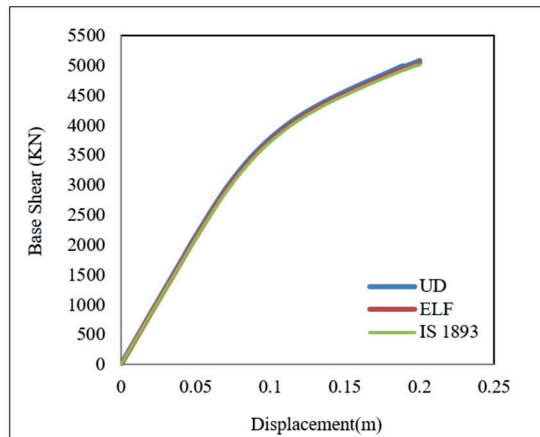
Pushover Curve for Mander's Model　　　Pushover Curve for Kent & Park Model

Figure 5: Pushover Curves

3.2. Collapse Mechanism

Local nonlinear effects i.e flexural hinges, which are opined to take place at the ends of the members should be modeled. Progressive plastic hinge formation results in the simultaneous occurrence of the redistribution of internal forces. The eventuality of local collapse taking place at some hinge locations is a result of the exceedance of the limit on plastic deformation. A global collapse of the structure is triggered either when adequate number of plastic hinges forms in structure or, when the mobilization of load paths in order to sustain force equilibrium (i.e., after local hinges break down) cannot be undertaken by the structure. When the structure is completely in elastic phase, all the hinges are formed in the elastic limit. For UD with Mander's Confinement approach, the structure enters into the inelastic phase of immediate occupancy when the base shear is in the range of 2163kN to 4364kN with 16% of the hinges formed in IO level 84% hinges are at elastic level. Also when the base shear increase beyond 4364kN till its maximum value, 4.7% of the hinges have been formed at life safety level, 15.5% of the hinges in IO Level and 79.8% in elastic level. The time period of the structure in elastic phase is

0.973s and in the inelastic phase it is 1.327s.

The structure enters into the inelastic phase of immediate occupancy when the base shear is in the range of 2163kN to 4312.71kN with 16% of the hinges formed in IO level 84% hinges are at elastic level. Also when the base shear increase beyond 4364kN till its maximum value, 4.3% of the hinges have been formed at life safety level, 15.9% of the hinges in IO Level and 79% in elastic level. The time period of the structure in elastic phase 0.98315s and in the inelastic phase it is 1.33249s for ELF and IS 1893 load pattern of Mander's Modelling Approach.

For Kent & Park Model of UD, the structure moves into the inelastic phase of immediate occupancy when the base shear is in the range of 1716.18 kN to 3723.79 kN with 16% of the hinges formed in IO level 84% hinges are at elastic level. Also when the base shear increases beyond 3723.79 kN till its maximum value, 6.4% of the hinges have been formed at life safety level, 14.8% of the hinges in IO Level and 78.8% in elastic level. The time period of the structure in elastic phase is 0.97332s and in the inelastic phase it is 1.40132s.

For Kent & Park Model with ELF and IS 1893 load

pattern the base shear values in the inelastic phase of immediate occupancy is in the range of 1718.54 kN to 3550.84 kN with 13.7% of the hinges formed in IO level 86.3% hinges are at elastic level. Also when the base shear increase beyond 3901.38 kN till its maximum value, 5.3% of the hinges have been formed at life safety level, 19.6% of the hinges in IO Level and 75.1% in elastic level. The time period of the structure in elastic phase is 0.98315s and in the inelastic phase it is 1.39959s.

For all the three different types of loading, the performance points are close, however the base shear values in Mander's model is higher in all the three load patterns in comparison with Kent & Park Model.

3.3. Interstorey Drift (ISD)

Contemporary developments witnessed in performance based engineering have posited inter storey drifts as crucial parameters in assessing structural response, owing to their affinity to the damage endured by a building. Providing drift values, as close to the predictions of a more meticulous dynamic analysis as possible, are then essentialised for static methods which often tend to focus only on providing correct capacity curves. Therefore the drift profiles are obtained in each of the models. At the level of performance, all the models under the considered earthquake satisfies the permissible limit of 1% specified in ATC 40. The storey drifts are closer to the immediate occupancy limit in all the models, which is in concurrence with the defined performance objective. Table 7 gives the details of ISD for various combinations of load patterns. Fig. 6 depicts the variations of Interstory drift along the elevation for Mander's Model and Kent and Park Model. ISD appears to be more at middle lower stories

and gradual decrease in the ISD is noticed in the upper stories . A Similar observations could be made in both modeling approaches.

**TABLE 7**: Inter-storey Drift for various combinations of load patterns

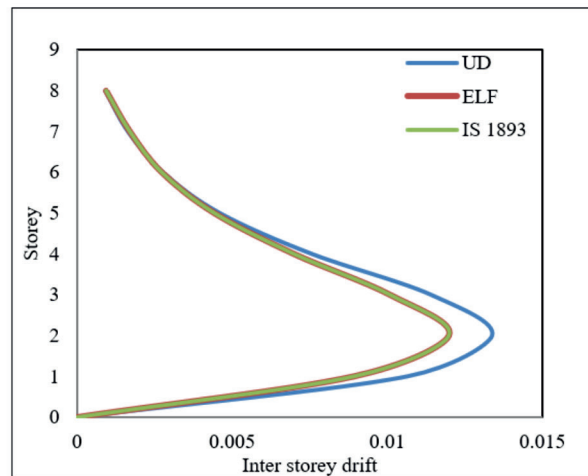| Model	ISD(%)		
	UD	ELF	IS 1893
MM	0.46	0.47	0.47
KPM	0.46	0.5	0.5

3.4. Over Strength (Ω)

The calculated over strength for the all the models is tabulated in Table 8. When the structure is subjected to earthquake forces, the over strength for all the models is found be in the range of 3.05 to 3.33. The over strength in the structures is found to have variations due to the modeling approach and load pattern adopted. In the Mander's modeling approach, the UD is found to exhibit greater over strength of 3.33 where as the over strength in ELF and IS1893 load pattern the over strength is similar. In the KPM, the over strength factor in all the models is found to have 3.05.

3.5. Ductility (μ)

Based on the results and its observation the ductility in the KPM is found to greater in UD with a value of 5.33 in comparison with EFL and IS 1893 with a value of 5.09 (Table 8), there is reduction in the ductility value of all the load patterns in MM approach to 4. KPM produces better ductility.
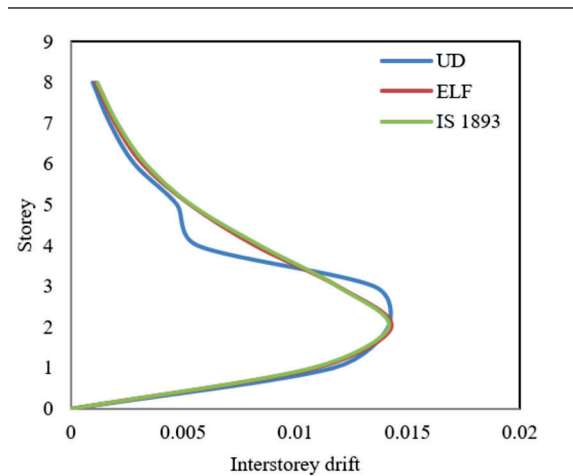
ISD for Mander's Model   ISD for Kent and Park Model

Figure 6: Inter-storey Drift

Table 8: Pushover Parameters

Frame	Load Pattern	V_d(kN)	V_u(kN)	Δ_y	Δ_u	$\mu = \Delta_u / \Delta_y$	$\Omega = V_u / V_d$
MM	UD	1525	5087	0.04	0.16	4	3.33
	ELF	1525	5017	0.04	0.16	4	3.33
	IS 1893	1525	5017	0.04	0.16	4	3.33
KPM	UD	1525	4655	0.03	0.16	5.33	3.05
	ELF	1525	4655	0.031	0.158	5.09	3.05
	IS 1893	1525	4655	0.031	0.158	5.09	3.05

4. Conclusions

1. Performance based analysis of RC bare framed eight storied structure was investigated using non linear static (pushover) analysis. This method is based on the assumption that the fundamental mode has a control over the structural performance even in the non linear stage.
2. In Mander's modeling approach, the base shear is greater by 1.40% for UD load pattern in comparison with ELF and IS 1893 load patterns. Kent and Park's modeling approach does not show any variation in the base shear between all the three loading patterns. Due to the fact that, the load pattern illustrates the distribution of inertia forces in the elastic range only. Meanwhile, it is only in the post-elastic phase that the amplication of higher mode effects can be noted.
3. Both Mander's model and Kent and Park's model exhibit higher value of base shear for UD loading pattern.
4. In Mander's stress strain modeling approach the base shear is 10% higher than the base shear obtained in Kent and park's model. However, both approaches exhibit a higher value of base shear than the design base shear (1525 kN) at the performance level.
5. It is shown with extreme clarity, that in all cases, the shape of the lateral load distribution is what the response of the buildings is finely attuned to. This rings especially true when different patterns of load considered. It can also be observed that there is a very small difference between various load pattern.
6. Kent and Park model shows increased ductility with UD in contrast to ELF and IS 1893 load pattern where as in Mander's model the ductility capacity decreases.
7. Mander's modeling approach along with UD exhibits higher overstrength in comparison with other two load patterns and also with the Kent and Park modeling approach, as well as that, the Interstory drift variation is insignificant for all the loading patterns.
8. By modeling material stress strain relationship with Mander's approach and UD loading pattern, seismic performance evaluation of bare framed moderately tall structures using pushover analysis may be reliable.
9. With the increase in the height of the structure, inelasticity becomes more pronounced and so does the higher modal participation. The effect of higher modes and its control over the structural performance is to be clarified.
10. To arrive at an appropriate loading pattern, performance of buildings with varying geometrical parameters is to be rigorously investigated and compared with dynamic analysis.
11. In order to procure useful insights into the elastic and inelastic responses elicited by structures which have experienced earthquake ground motions, a suitable modelling of the building, vigilant choice of the lateral load distribution and a lucid understanding of the results brought about by pushover analysis must be undertaken and compared with dynamic analysis.

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