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A parametric study on seismic fragility analysis of RC buildings

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Abstract. Among all the natural disasters, earthquakes are the most destructive calamities since they cause a plenty of injuries and economic losses leaving behind a series of signs of panic. The present study highlights the moment-curvature relationships for the structural elements such as beam and column elements and Non-Linear Static Pushover Analysis of RC frame structures since it is a very simplified procedure of non-linear static analysis.

The highly popular model namely Mander's model and Kent and Park model are considered and then, seismic risk evaluation of RC building has been conducted using SAP 2000 version 17 treating uncertainty in strength as a parameter. From the obtained capacity and demand curves, the performance level of the structure has been defined. The seismic fragility curves were developed for the variations in the material strength and damage state threshold are calculated. Also the comparison of experimental and analytical results has been conducted.

Keywords: pushover analysis; moment-curvature relationship; Mander model; Kent and Park model; fragility analysis; damage state thresholds

1. Introduction

Of all the natural disasters, earthquakes are one of the most devastating and unpredictable phenomena that have influenced on mankind from the immemorial time. In 2001, after the Bhuj earthquake a significant involvement in this country has been focused towards the destructive impact of earthquakes and has enhanced the awareness of the hazard regarding seismic risk events. In order to resist the effects of earthquakes needs special considerations in structural design and evaluation of buildings regarding to their ability.

Mainly, two random variables are involved in seismic risk assessments which are vulnerability of the structure and the intensity of seismic action. The uncertainty related to the former one depends on mechanical properties of the materials, the participation of the structures among others

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and weight supported by the structure. The uncertainty related to the second one depends on soil conditions and fault mechanism. Hence the evaluation of uncertainty plays a vital role in computing the structural response of the structures. This is done by assessing non-linear static analysis using pushover procedure.

The finite element program SAP2000 version 17 is used to perform this analysis. To model the non-linear behavior of components it provides default or user defined hinge properties options.

Pushover analysis is an approximate method in which the structure is subjected to monotonically increasing lateral forces with an invariant height wise distribution is done until target displacement is reached.

1.1 Scope of the study

An attempt is done to study the effect of variation in strength in the structures. For this 35 models were generated and considered the uncertainty in characteristic strength of concrete (f_{ck}) and tensile strength of steel (f_y). The modelling and analysis of all these models has been carried out using SAP 2000 version 17. The focus of attention is to find the performance level of the building with the help of capacity and demand of the building for designed earthquake using nonlinear static pushover analysis. On the basis of obtained performance level, one can determine the need of structure whether to repair or retrofit or to demolish and reconstruct the entire building. From the results obtained by pushover analysis, based on the performance point different damage state thresholds and fragility curves have been generated.

The main objective of this paper is to conduct vulnerability derivation process for an RC building assumed to be located in Zone-IV of IS: 1893(Part1)-2002 treating mechanical properties as variation of strength and to perform non-linear static analysis by adopting different modelling approaches.

2. Stress-strain models pertaining to confined concrete

Various modelling approaches pertaining to stress-strain relation of confined concrete are available. Such as,

- Mander model
- Kent and Park model

Mander model is simple and the most used model since it is effective in considering the effect of confinement (Mander *et al*. 1998). Regardless of the arrangement of the confinement reinforcement used, the performance over the stress-strain range is similar and its peak stress and strain co-ordinates can be found.

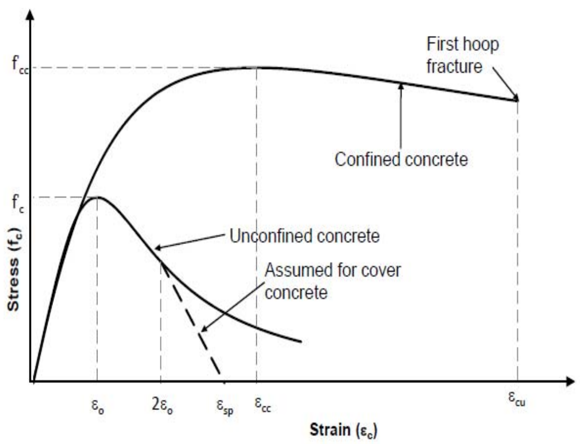
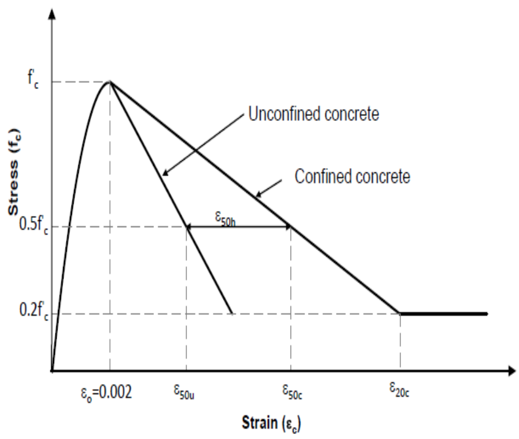
In Kent and Park model, a second degree parabola represents the ascending part of curve and assumes that the confined steel has no effect on shape of this part of the curve or strain at maximum stress (Madhu 2009).

Brief descriptions of both models are represented in Table 1.

3. Fragility analysis

The analysis of seismic loss estimation in built environment is termed as fragility analysis. The

Table 1 Description for Mander model and Kent and Park model (Mander *et al*. 1998, Madhu 2009)

MANDER MODEL	KENT AND PARK MODEL
	
$f_c = \frac{f'_{cc} x r}{r - 1 + x^r}$	$If, \varepsilon_c \leq 0.002,$
$where, x = \frac{\varepsilon_c}{\varepsilon_{cc}}$	$f_c = f'_c \left[\frac{2\varepsilon_c}{0.002} - \left(\frac{\varepsilon_c}{0.002} \right)^2 \right]$
$\varepsilon_c = 0.002 \left[1 + 5 \left(\frac{f'_{cc}}{f'_{co}} - 1 \right) \right]$	$If, 0.002 \leq \varepsilon_c \leq \varepsilon_{20,c}$
$r = \frac{E_c}{E_c - E_{\sec}}$	$f_c = f'_c [1 - Z(\varepsilon_c - 0.002)]$
$E_c = 5000 \sqrt{f'_{co}}$	$where,$
$E_{\sec} = \frac{f'_{cc}}{\varepsilon_{cc}}$	$Z = \frac{0.5}{\varepsilon_{50u} + \varepsilon_{50h} - 0.002}$
$f'_{cc} = f'_{co} \left(-1.254 + 2.254 \sqrt{1 + \frac{7.94 f'_l}{f'_{co}}} - 2 \frac{f'_l}{f'_{co}} \right)$	$\varepsilon_{50u} = \frac{3 + 0.002 f'_c}{f'_c - 1000}, \varepsilon_{50h} = \frac{3}{4} \rho_s \sqrt{\frac{b''}{s_h}}$
$k_e = \frac{\left(1 - \sum_{i=1}^{n} \frac{(w'_i)^2}{6 b_c d_c} \right) \left(1 - \frac{s'}{2b_c} \right) \left(1 - \frac{s'}{2d_c} \right)}{1 - \rho_{cc}}$	$\rho_s = \frac{2(b'' + d'') A_s}{b'' d'' s_h}, \varepsilon_{50c} = \varepsilon_{50u} + \varepsilon_{50h}$
$f'_{lx} = k_e \rho_x f_{yh}$	$If, \varepsilon_c \geq \varepsilon_{20,c}$
$f'_{ly} = k_e \rho_y f_{yh}$	$f_c = 0.2 f'_c$
$f'_l = f'_{lx} + f'_{ly}$	$\varepsilon_{20c} = \frac{0.8}{Z} + 0.002$

probability that the expected global damage (d) of a structure exceeds a given damage state as a function of parameter quantifying the severity of seismic action.

By plotting probability of exceedance in the ordinate and spectral displacement in abscissa the fragility curve is defined and it is described by the following lognormal probability density function (Yeudy *et al*. 2013)

$$P\left[\frac{d_s}{S_d}\right]=\phi\left[\frac{1}{\beta_{ds}}\ln\left(\frac{S_d}{\overline{S}_{d,ds}}\right)\right]$$

where,

$\overline{S}_{d,ds}$ =Median value of the spectral displacement at which the building reaches the threshold of damage state, *ds*.

β_{ds} =Standard deviation of the natural logarithm of spectral displacement for damage state, *ds*.

ϕ =Standard normal cumulative distribution function.

S_d=Given peak spectral displacement.

Four damage states (Barbat *et al*. 2008, Yeudy *et al*. 2013) are considered in this study namely slight, moderate, extensive and collapse where, slight damage indicates shear or flexure type hairline cracks within joints or in some beams and columns near joints. Moderate damage considers that some structural elements in ductile frames reached yield capacity indicated by

Table 2 Damage state thresholds (Barbat *et al*. 2008 and Yeudy *et al*. 2013)

Damage state	**Median spectral displacement,** $\overline{S}_{d,dsi}$
Slight	$0.7d_y$
Moderate	d_y
Extensive	$d_y+0.25(d_u-d_y)$
Collapse	d_u

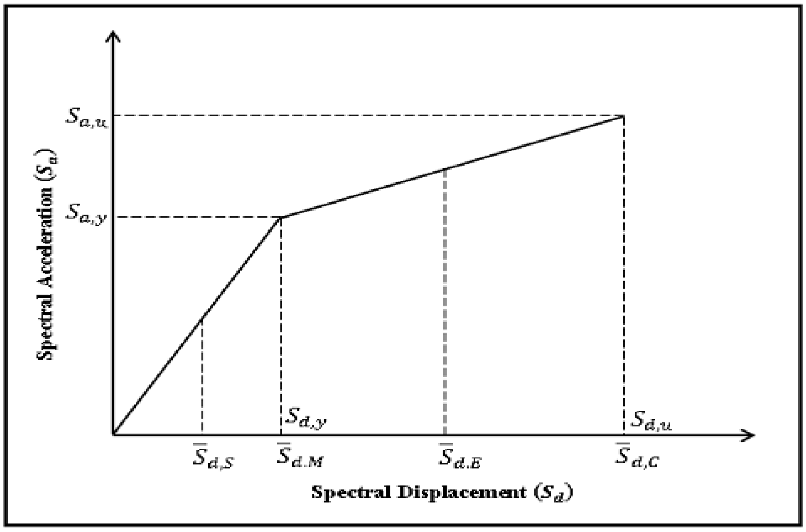


Fig. 1 Bilinear capacity spectrum representing damage state thresholds

flexure cracks and some spalling of concrete. Extensive damage indicates large flexure cracks, buckled main reinforcement resulting in partial collapse of non-ductile frame elements due to shear failures or bond failures at reinforcement. Collapse is complete structural damage the structure will collapse or loss of frame stability occurs. Table 2 resumes the main values for each damage state and are defined by using the bilinear capacity spectrum as a function of ultimate displacement (d_u) and yielding displacement (d_y) of the structure as shown in Fig. 1.

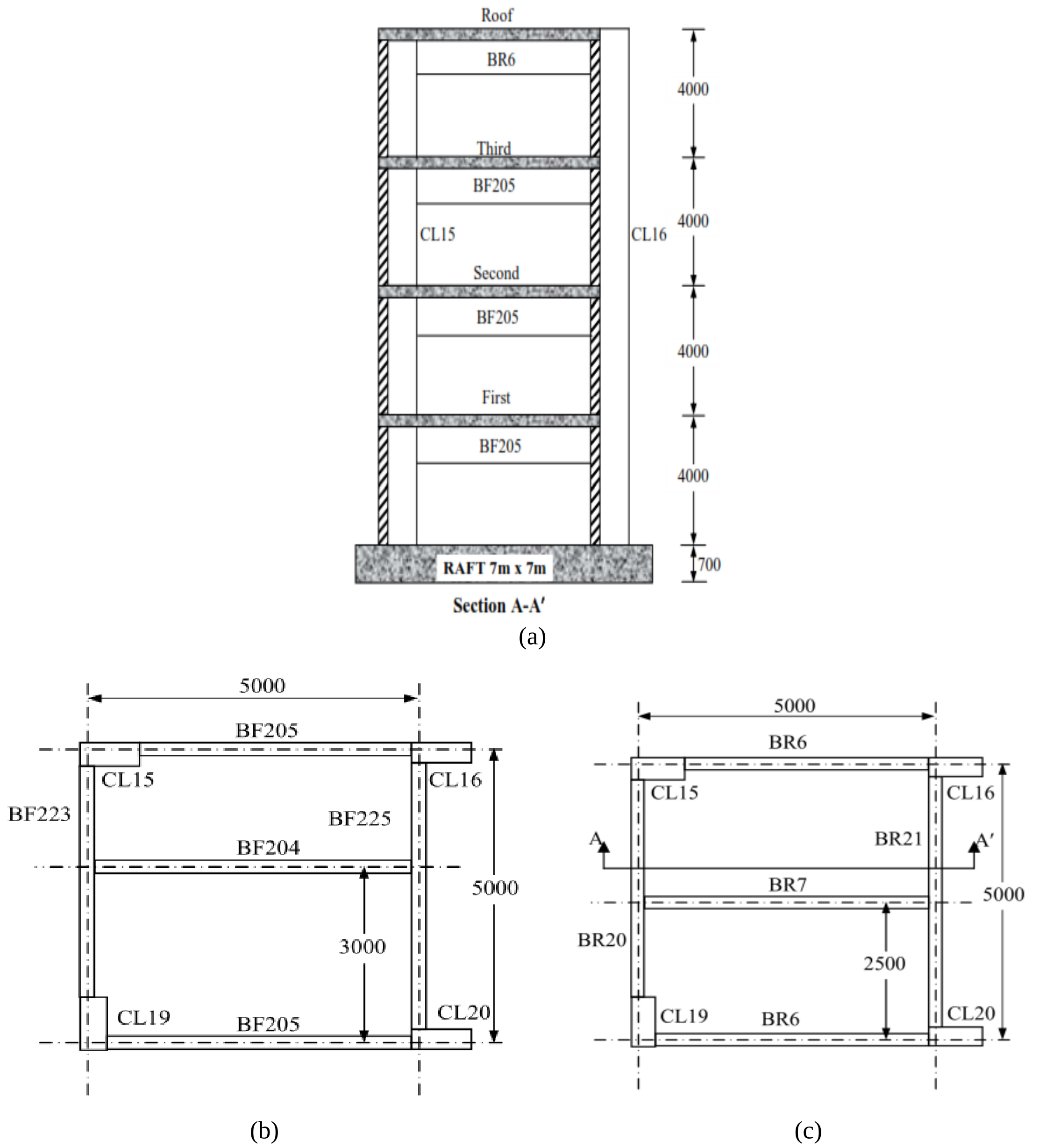
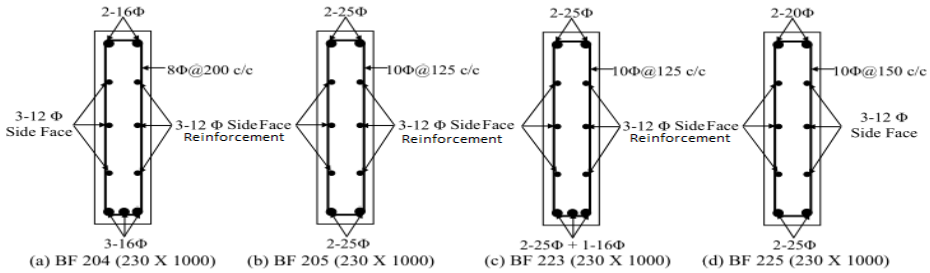


Fig. 2 Overall geometry of structure (a) Elevation of the structure (b) Floor plan (c) Roof plan (Robin exercise BARC and CPRI and Ravi *et al*. 2014)

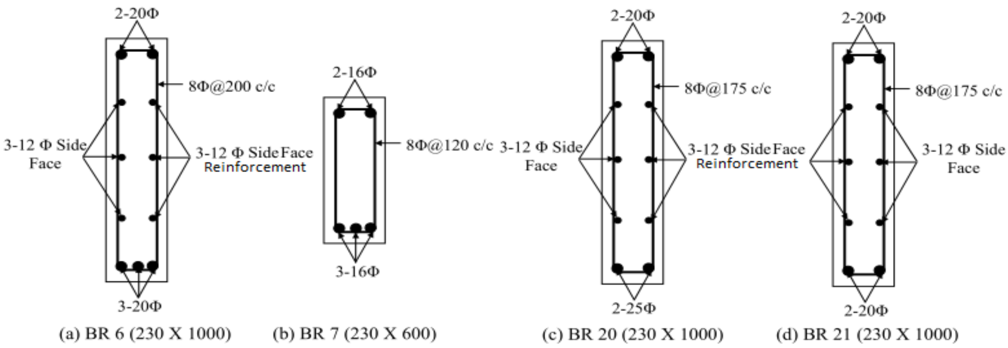
4. Analytical review

4.1 Outline of experimental building

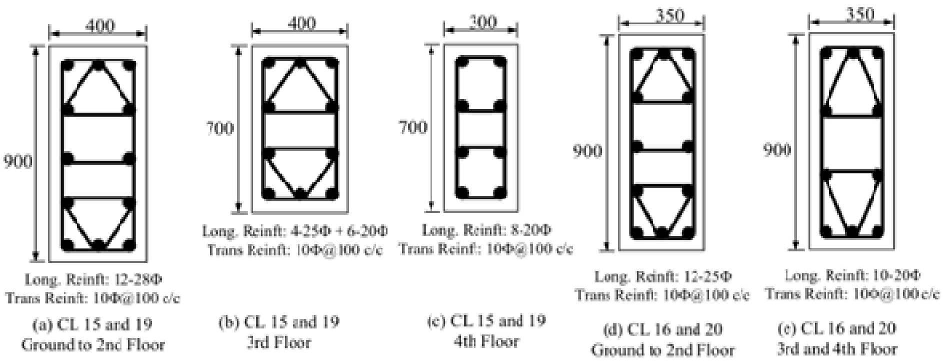
In order to bridge the gap between experimental and analytical data, Reactor Safety Division (RSD), Bhabha Atomic Research Centre (BARC) conducted a national round robin exercise in which a full-scale four storied RCC structure was tested under lateral monotonically increasing Pushover loads at the tower testing facility at Central Power Research Institute (CPRI), Bangalore. The test was conducted under gradually increasing monotonic lateral load in an inverted triangular pattern till failure.



(a) Details of floor beams



(b) Details of roof beams



(c) Details of columns

Fig. 3 Details of various structural systems (Robin exercise BARC and CPRI and Ravi *et al*. 2014)

A brief summary of the building (Ravi *et al*. 2014 and Robin exercise BARC and CPRI) is as follows.

• A portion of four storied RC beam column framed system structure of single bay assumed to be located in seismic zone IV is tested and analyzed.

• Type of the building frame system considered is Ordinary RC moment-resisting frame (OMRF).

• Grade of the concrete and reinforcing steel is M20 and Fe415 respectively.

• The total building height is 12 m above ground storey and height of each storey is 4 m. Width of bay in each direction is 5 m.

• The type of foundation used is raft foundation of 700 mm thick which is supported on a rock bed using rock grouting. In this model, at the column ends fixed supports are assumed and in the analysis the effect of soil structure interaction is ignored.

• At each floor level 120 mm thick concrete slab is provided.

The layout of beam at all floors, roof plan and overall geometry of structure are shown in Fig. 2. The reinforcing details of various structural systems such as floor beams, roof beams and columns are shown in Fig. 3.

5. Methodology

Steps of the methodology (Ravi *et al*. 2014) are described below.

• An analytical four storey building model is developed using SAP 2000 version 17 software.

• Then the non-linear static pushover analysis is conducted by assigning hinge properties (capacity curve).

• Then the damage state indicator levels are defined to evaluate the performance level of the building.

• An analytical fragility estimates are developed to quantify the seismic vulnerability of RC frame buildings.

6. Probability strength matrix

In this study, an attempt has been made to study the behavior of the structure considering the uncertainty in strength. Here, the characteristic strength of concrete (f_{ck}) as well as tensile strength of steel (f_y) has been taken as random variables. Since the IS 456:2000 specifies the target strength for concrete of grade M20 to lie between 27 MPa and 28.5 MPa, therefore, taking into account of material uncertainty and partial safety factor as 1.5, the upper limit and lower limit was obtained as 20 MPa and 30 MPa respectively, and a series of characteristic strength between these values are chosen and similarly a wide range values for tensile strength of steel were considered between the ranges of 520 MPa and 600 MPa.

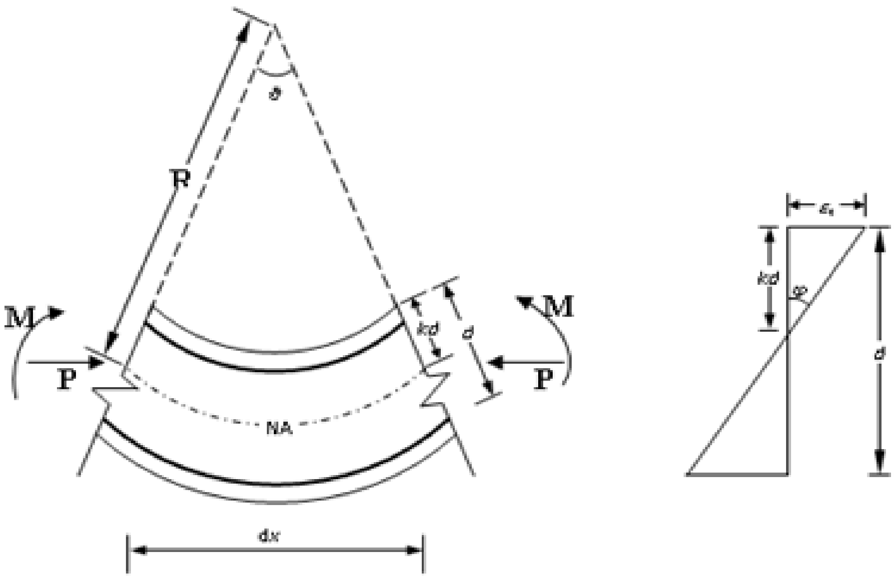
The material properties considered for the analysis are given in Table 3.

7. Moment curvature relationship

It is the representation of strength and deformation of the section in terms of moment and

Table 3 Material properties

Material	Characteristic strength (MPa)	Modulus of elasticity (MPa)
Concrete (M20)	f_{ck}=20	E_c=22360
	f_{ck}=21.5	E_c=23184
	f_{ck}=23	E_c=23979
	f_{ck}=25	E_c=25000
	f_{ck}=27	E_c=25980
	f_{ck}=28.5	E_c=26692
	f_{ck}=30	E_c=27386
Reinforcing steel (Torsteel)	f_y=520	E_c=200000
	f_y=540	E_c=200000
	f_y=560	E_c=200000
	f_y=580	E_c=200000
	f_y=600	E_c=200000



(a) (b)

Fig. 4 Deformation of flexural member (a) Element of member (b) Strain distribution

corresponding curvature of the structure.

Consider a small element of length ‘*dx*’ of a flexural member acted upon by equal moments ‘*M*’ and axial force ‘*P*’ as shown in Fig. 4.

The radius of curvature ‘*R*’ is measured up to the neutral axis and the rotation ‘*θ*’ between the ends of the element is given by

$$\theta = \frac{dx}{R} = \frac{\varepsilon_c dx}{kd}$$

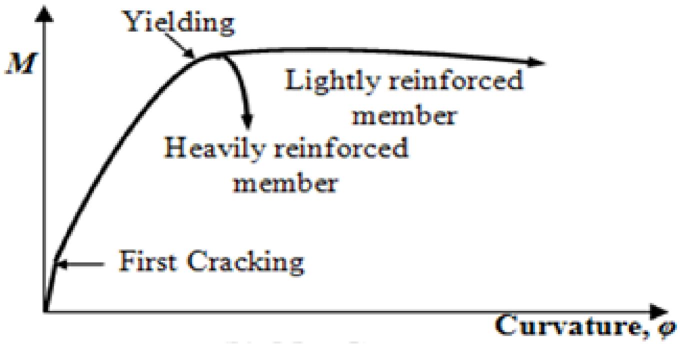


Fig. 5 Typical moment curvature curve

$$\frac{1}{R}=\frac{\varepsilon_c}{kd}$$

where, $1/R$ is the curvature at the element

$$\phi=\frac{\varepsilon_c}{kd}$$

The relationship between moment '*M*' and curvature 'ϕ' is given as follows

$$\frac{M}{I}=\frac{f}{y}=\frac{E}{R}$$

Considering the first and last terms of above equation we get

$$\phi=\frac{1}{R}=\frac{M}{EI}$$

Where, EI is the flexural rigidity of the section and is equal to the slope of M-ϕ curve.

From the Fig. 5 it can be observed that the M-ϕ curve is perfectly linear up to the first crack, after that crack flexural rigidity and slope of the curve reduces to some extent. Since the cracking is not extensive the curve is still almost linear. When the excessive cracking occurs, the member will yields the flexural rigidity and slope of curve reduces to extremely low value.

8. Results

8.1 Experimental result

The following Fig. 6 represents the pushover curve obtained from the experiment done at CPRI, Bangalore.

8.2 Moment-curvature relationship for Mander model and Kent and Park model

It is the representation of strength and deformation of the section in terms of moment and corresponding curvature of the section. Main results are shown in Figs. 7-8.

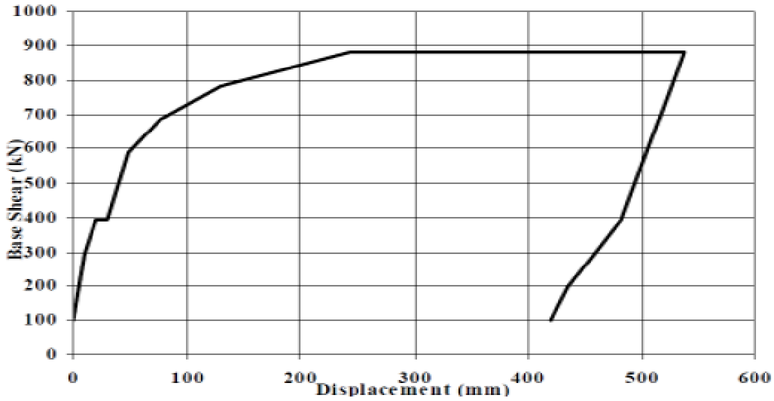
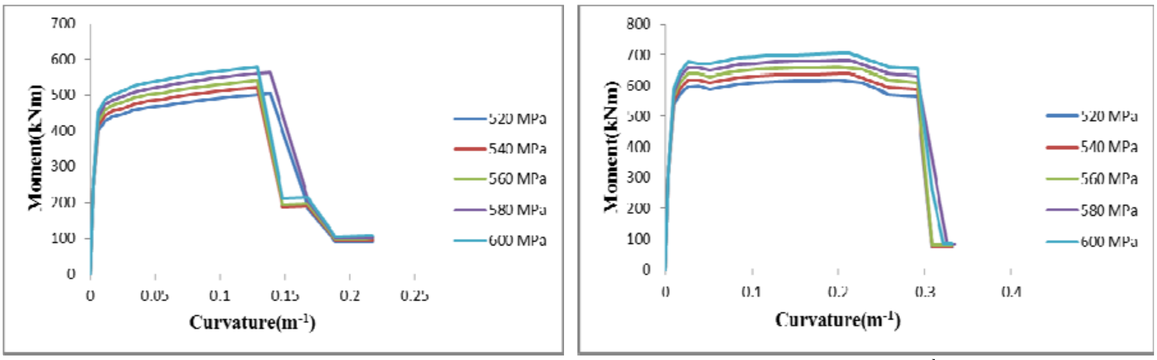
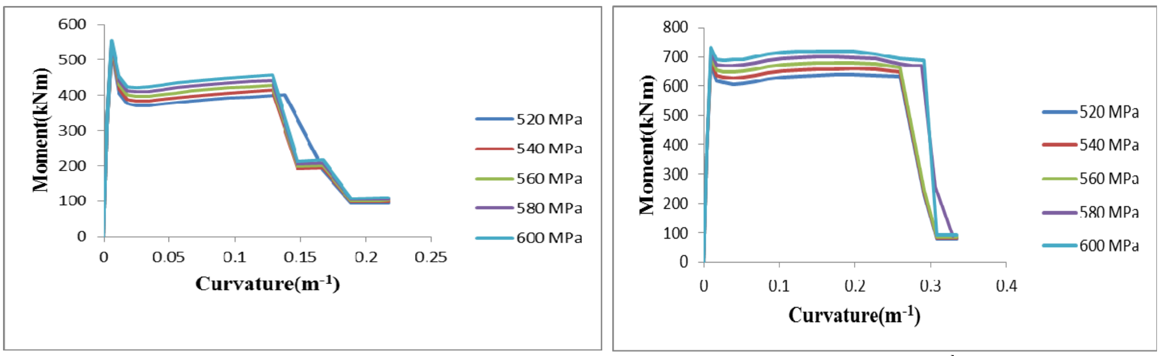


Fig. 6 Pushover curve obtained from experiment done at CPRI, Bangalore



(a) BF204 (b) CL 15 & 19 3rd floor

Fig. 7 Moment curvature relation for BF204 and CL 15 & 19 3rd floor for f_{ck}=20 MPa and varying f_y (Mander model)



(a) BF204 (b) CL 15 & 19 3rd floor

Fig. 8 Moment curvature relation for BF204 and CL 15 & 19 3rd floor for f_{ck}=20 MPa and varying f_y (Kent and Park model)

In Mander and Kent and Park models, it can be noted that from Figs. 7-8 for both beam and column with the increase in tensile strength and characteristic strength of concrete the moment carrying capacity of the structure will also increases.

8.3 Comparison of resulting analytical pushover curves with experimental pushover curves

The resulting analytical pushover curve for varying f_{ck}(MPa) and for constant f_y=520 MPa is compared with the experimental pushover curve for both Mander and Kent and Park model. Main results are represented in Figs. 9-10 respectively.

One can observed through capacity curves for the derived values of f_{ck} and f_y, that the analytical base shear values are more higher for Kent and Park model than for Mander model and those obtained by experimental test.

8.4 Fragility curves and damage state thresholds

From the pushover analysis results the capacity curves are obtained and are represented as bilinear capacity spectrum as shown in Figs. 11-12 for both Mander model and Kent and Park model respectively and then the mean damage index are calculated. Based on the mean damage index obtained from the Table 2 and proceeding with the probabilistic approach, 100 random

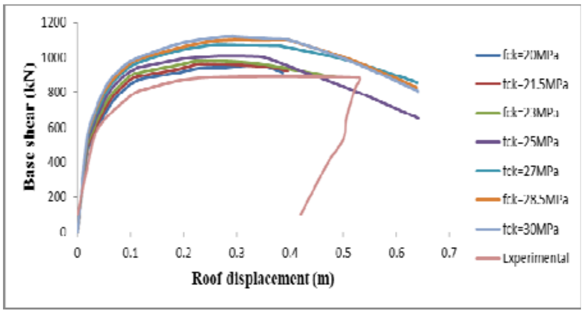


Fig. 9 Combined analytical and experimental pushover curve for Mander's model

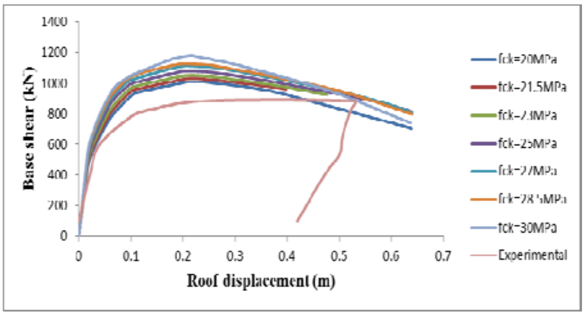


Fig. 10 Combined analytical and experimental pushover curve for Kent and Park model

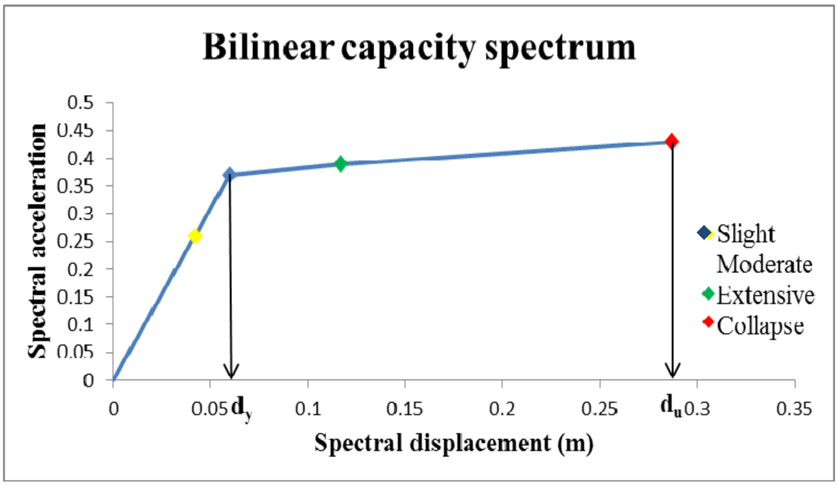


Fig. 11 Bilinear capacity spectrum for f_{ck}=20 MPa and f_y=520 MPa in X direction for Mander model

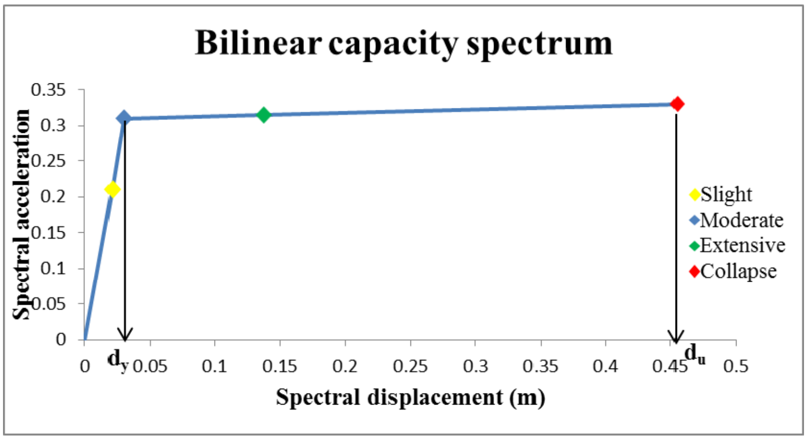


Fig. 12 Bilinear capacity spectrum for f_{ck}=20 MPa and f_y=520 MPa in X direction for Kent and Park model

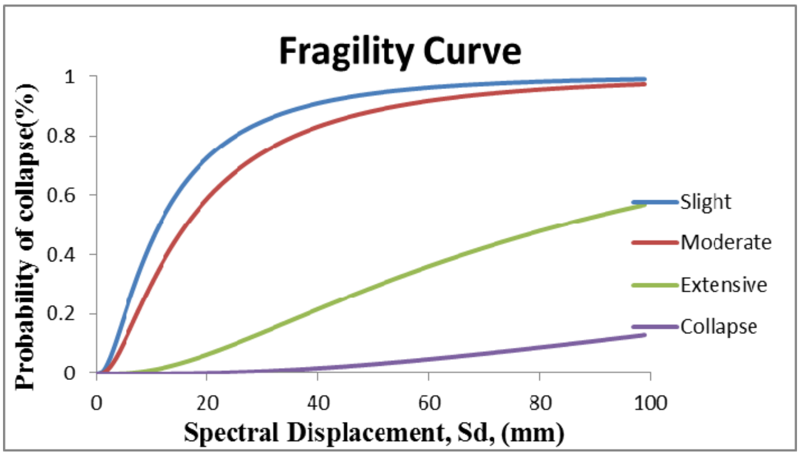


Fig. 13 Fragility curve for f_{ck}=20 MPa and f_y=520 MPa in X direction for Mander model

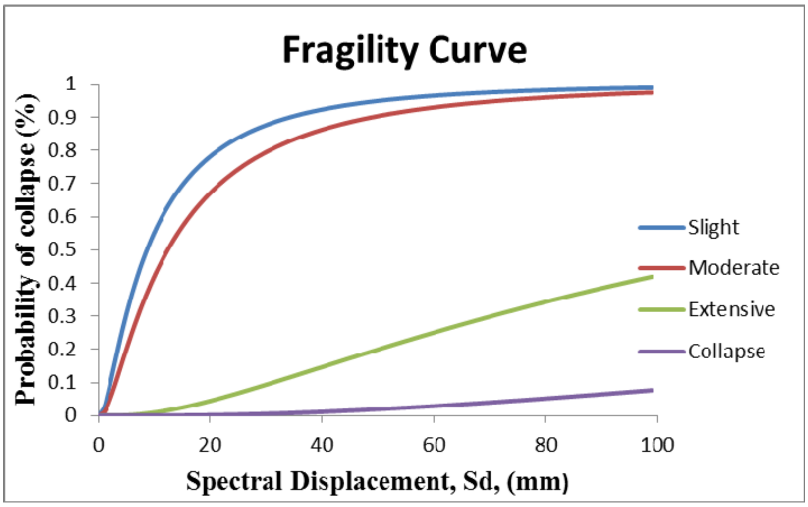
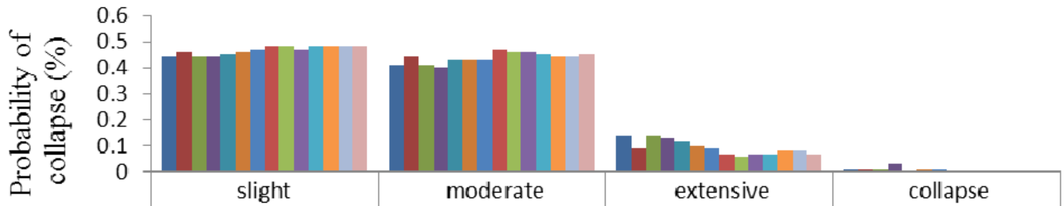


Fig. 14 Fragility curve for f_{ck}=20 MPa and f_y=520 MPa in X direction for Kent and Park model



	slight	moderate	extensive	collapse
MDRS20,520	0.44	0.41	0.14	0.01
KNP20,520	0.46	0.44	0.09	0.01
MDRS21.5,520	0.44	0.41	0.14	0.01
KNP21.5,520	0.44	0.4	0.13	0.03
MDRS23,520	0.45	0.43	0.12	0
KNP23,520	0.46	0.43	0.1	0.01
MDRS25,520	0.47	0.43	0.09	0.01
KNP25,520	0.48	0.47	0.07	0
MDRS27,520	0.48	0.46	0.06	0
KNP27,520	0.47	0.46	0.07	0
MDRS28.5,520	0.48	0.45	0.07	0
KNP28.5,520	0.48	0.44	0.08	0
MDRS30,520	0.48	0.44	0.08	0
KNP30,520	0.48	0.45	0.07	0

Fig. 15 Combined damage state thresholds for Mander model and Kent and Park model
MDRS: Mander’s model; KNP: Kent and Park model

samples were generated for the ultimate spectral displacement (d_u) of the capacity curve because of the lack of enough ground motion records. The standard deviation, σ is computed for these 100 random samples and then the fragility curves are computed and the obtained fragility curves are shown in Figs. 13-14 for Mander model and Kent and Park model respectively.

Also the analytical combined damage state thresholds are represented in Fig. 15 for all models i.e., for both Mander model and Kent and Park model of different f_{ck} and f_y combinations.

9. Conclusions

• By using any advanced software and also the suggested methodology by various researchers for the assessment of seismic risk of buildings, the pushover analysis can be used as a guideline for the analysis of RC structures subjected to seismic loading.

• To simulate the moment curvature behavior of four storied reinforced concrete, an analytical model is presented and on the basis of proposed stress-strain Mander, Kent and Park models enables the determination of moment-curvature stress-strain relationship and material properties up to the maximum capacity of the section. The moment curvature relations play a vital part in the study of limit state analysis of two dimensional RC frames.

• For the derived values of f_{ck} and f_y, with the increase in f_{ck} and f_y values, there is an increase in both moment carrying capacity and curvature values.

• Also it can be observed that for the derived values of f_{ck} and f_y and by taking factor of safety into account, the analytical base shear values are higher for Kent and Park model when compared to Mander model and experimental values in capacity curve of pushover analysis. The assessment of fragility and damage states has been performed for the building located in zone 1V.

• From the experimental pushover curve it can be noted that the roof displacement is 530 mm for the corresponding base shear of 882.9 kN, whereas from the analytical curve for f_{ck}=20 MPa and f_y=520 MPa it was found that V_B=995.29 kN which linearly increases to a base shear value of 1118.001 kN in Mander model whereas in Kent and Park model it was found to be V_B=1009.397 kN and is linearly increased to a value of 1179.752 kN.

• The fragility analysis result shows that for the considered buildings with varying f_{ck}(MPa) and for constant f_y=520 MPa, a high probability of slight, moderate damages can be observed and a low probability of severe and collapse damage states can be noted and is almost found to be similar in both Mander and Kent and Park models.

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