Fragility Analysis of Reinforced Concrete Building by Various Modeling Approaches using SAP 2000- A Parametric Study

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Abstract – *Earthquakes are the most destructive and devasting calamities among all the natural disasters since they cause injuries and as well as economic losses. In the present study highlights a very simplified procedure of Non Linear Static Analysis which is nothing but Non Linear Static Pushover Analysis of RC frame structures. In this study by treating uncertainty in strength as a parameter the seismic risk evaluation of RC building has been carried using SAP 2000 version 18 and for the modeling Manders model and Kent and Park model are considered. From the obtained pushover curve the comparison of results of analytical and experimental are carried. The performance level of the structure has been defined. The seismic fragility curves and damage state thresholds are established. Also the comparison of results of Mander model and Kent and Park model is done.*

Key Words: **Pushover analysis, Mander model, Kent and Park model, Capacity curve, Fragility curve, Damage state threshold**

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. To withstand the effects of earthquakes requires special considerations in structural design and evaluation of buildings regarding to their ability. Mainly two random variables are involved in seismic risk assessment namely vulnerability of the structure and the intensity of seismic action. the evaluation of uncertainty plays a vital role in computing the structural response of the structures. This is done by using non-linear static analysis (pushover analysis) by using the finite element program SAP2000. To model the non-linear behaviour of components it provides default or user defined hinge properties options.

Non linear static 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 PRESENT WORK

An attempt is done to study the effect of variation in strength in the structures. For this, 70 models were generated and considered the uncertainty in characteristic strength of concrete (fck) and tensile strength of steel (fy). Using SAP 2000 version 18, for all these models the modelling and analysis of has been done. In this work main focus is done on the performance evaluation of the building for designed earthquake along with the aid of capacity and demand of the building by non linear static analysis. On the basis of obtained performance level, determine the need of structure whether to repair or retrofit or to reconstruct the entire building. From the results obtained by non linear static analysis, based on the performance point different damage state thresholds and fragility curves have been generated.

1.2 OBJECTIVES

The main objectives of present study are, 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. To conduct non-linear static analysis for RC building by adopting different modelling approaches. To establish capacity curves, demand curves, fragility curves and damage thresholds for RC building treating concrete as confined.

1.3 METHODOLOGY

An analytical four storey building model is developed using SAP 2000 version 18 software. Then the non-linear static analysis is conducted by assigning hinge properties by adopting Mander model and Kent and Park model. 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.

1.4 FRAGILITY ANALYSIS

The analysis of seismic loss estimation in built environment is termed as fragility analysis. There are four damage states namely slight, moderate, extensive and complete structural damage. The probability that the expected global damage (d) of a structure exceeds a given damage state as a function of parameter computing the severity of seismic action. By plotting probability of exceedance in the ordinate and S_d in abscissa the fragility curve is defined and it is described by the following lognormal probability density function.

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

where,

$\bar{S}_{d,ds}$ = 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.

1.5 DAMAGE STATE THRESHOLDS

The simplified methods are used to obtain the damage state thresholds in order to analyze the expected damage. The mathematical expression is given below.

$$DI = \frac{1}{n}\sum_{i=1}^{n} x_i p_i$$

where,

DI = mean damage index

x_i = the damage state number which varies from 1 to 4

p_i = the probability of corresponding damage state. The probability of damage is computed from the fragility curves.

2. MODELLING AND ANALYSIS

2.1 PROBABILITY OF VARIATION IN STRENGTH

By considering the uncertainty in strength, an attempt has been made to study the behavior of the structure. Here, two main random variables such as f_{ck} and f_y are considered in this study and the partial safety factor as 1.5 and 1.15 are taken respectively. A wide range of values are taken between 20MPa and 30MPa for concrete and series between 520MPa and 600MPa for steel is considered by taking material uncertainty into account and the specification of IS 456:2000 about the target strength of M20 grade concrete. Thus, 70 models have been developed for the underneath combination of strength of f_{ck} and f_y and the analysis is carried out employing SAP2000 package.

2.2 EXPERIMENTAL REVIEW OF THE BUILDING

Reactor Safety Division (RSD), Bhabha Atomic Research Centre (BARC) led a nationwide application for a four storey RC building with a specific goal to overcome the hardship between the analytical and experimental studies. The structure which is being considered was constructed at Central Power Research institute (CPRI), Bangalore and it was subjected to a lateral monotonically increasing pushover loads till failure. Below Figure1 demonstrates the structure constructed at CPRI Bangalore. The test provided a base shear v/s roof displacement plot.

Figure1: Structure located at the tower testing facility CPRI Bangalore

2.3 EXPERIMENTAL RESULT

The pushover curves as obtained for CL 20 side is shown in fig 2. The drooping parts of the curve could not be obtained since the experiment was conducted under load control condition. Up to a base shear value of around 300kN the structure behaved linearly and at the base of columns the flexural tension cracks are started to generate at this point a reduced stiffness is displayed by the structure. After reaching an approximate value of base shear of 500kN, the stiffness of structure went down due to the cracks at the base of the columns opened wider and the failures at beams and beam-column joints are started to show up. The inter-storey drift also increased rapidly rapid degradation occurs at the joints of the structure after reaching a base shear value of 700kN. Once the lateral load is increased the base shear value also increases. After reaching a base shear value of around 882.90kN at constant load the structure undergoes increasing displacement. Once the structure got stable, the load was removed and unloading curve is obtained from the experiment and is shown in fig.2

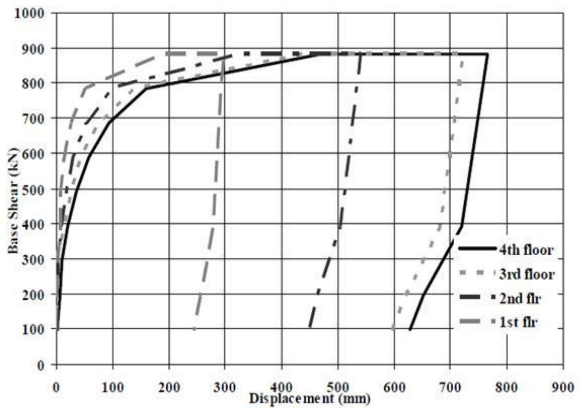
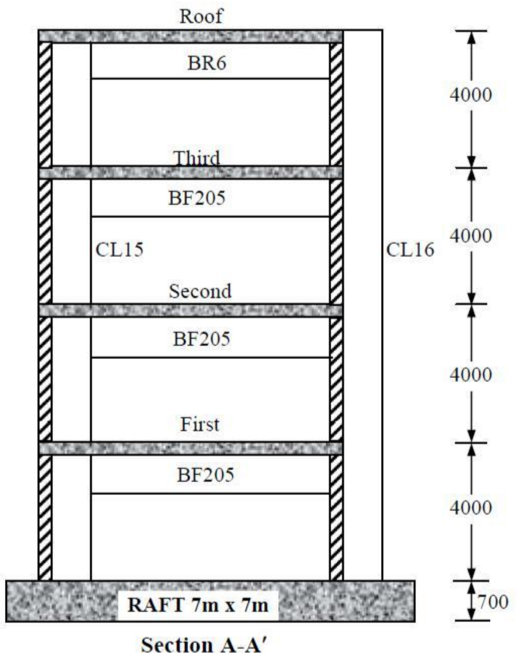


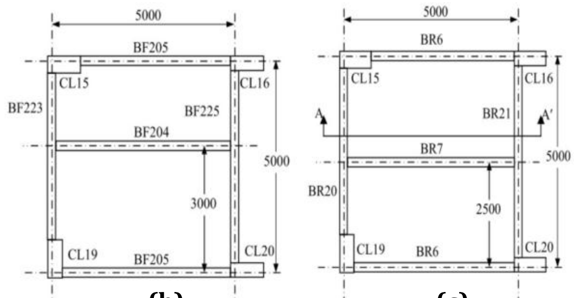
Figure 2: Experimental pushover curve

2.4 OUTLINE OF EXPERIMENTAL BUILDING

A portion of four storied RCC 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 is Ordinary RC moment-resisting frame (OMRF). The total building height is 12m above ground storey and height of each storey is 4m and width of bay in each direction is 5m. The type of foundation used is raft foundation of 700mm thick that is supported on 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 120mm thick concrete slab is provided. The layout of beam at all floors and roof plan and overall geometry of structure is shown in fig.3. The reinforcing details of various structural systems such as floor beams, roof beams and columns are shown in fig.4

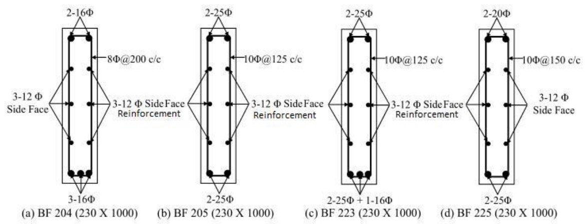


(a)

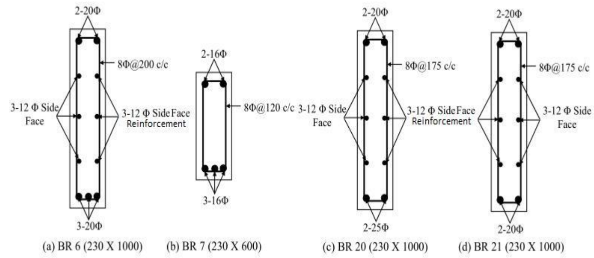


(b) **(c)**

Figure 3: Overall geometry of structure (a) Elevation of the structure (b) Floor plan (c) Roof plan



(a)



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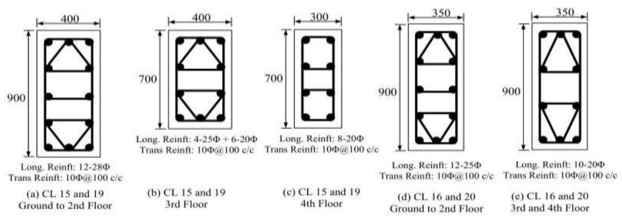


(c)

Figure 4: Details of various structural systems (a) Details of floor beams (b) Details of roof beams (c) Details of columns

2.5 MATERIAL PROPERTIES

For the analysis the following material properties are considered and are represented in table 1.

Material	Characteristic strength (MPa)	Modulus of elasticity (MPa)
Concrete	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	f_y = 520	E_s = 200000
	f_y = 540	E_s = 200000
	f_y = 560	E_s = 200000
	f_y = 580	E_s = 200000
	f_y = 600	E_s = 200000

Table 1: Material properties

2.6 MANDER'S MODEL

Manders model is a widely held model to study the stress–strain relationship of concrete. It was tested on the circular, rectangular and square full scale columns at seismic strain rates in order to examine the impact of different transverse reinforcement to confinement effectiveness. It was observed that the performance over the entire stress-strain range was similar if the ultimate strain and stress directions could be established, regardless of the arrangement of the confinement reinforcement. Manders stress strain model is demonstrated in the below figure 5

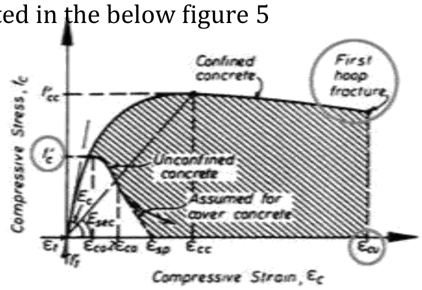


Figure 5: Mander model for stress strain relationship

2.7 KENT AND PARK MODEL

Kent and Park model was developed in the year 1971; it suggested a stress-strain curve for concrete which is confined by rectangular hoops. A second-degree parabola present in the curve represents the ascending part and it is assumed that the confining steel has no consequence on the outline of this ascending part of curve. This means that the ascending curve is exactly the same for both confined as well as unconfined concrete. It was also expected that the topmost stress attained by confined concrete is equal to the cylinder strength f'_c that is reached at a strain of 0.002 .The stress-stress relationship curve developed is as shown in the figure 6.

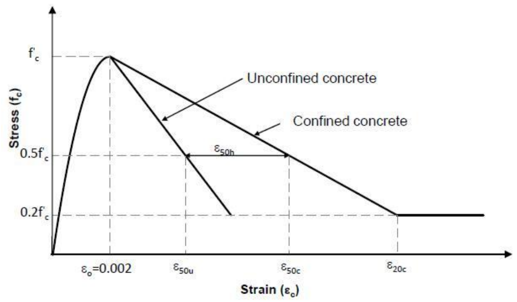


Figure 6: Kent and Park model for stress strain relationship

3. RESULTS AND DISCUSSIONS

3.1 COMPARISON OF EXPERIMENTAL AND ANALYTICAL PUSHOVER CURVE

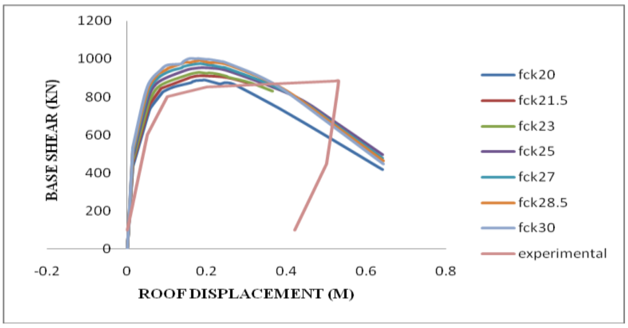


Figure 7: Comparison of experimental and analytical pushover curve for varying f_{ck}(MPa) and for constant f_y=520MPa (Mander model)

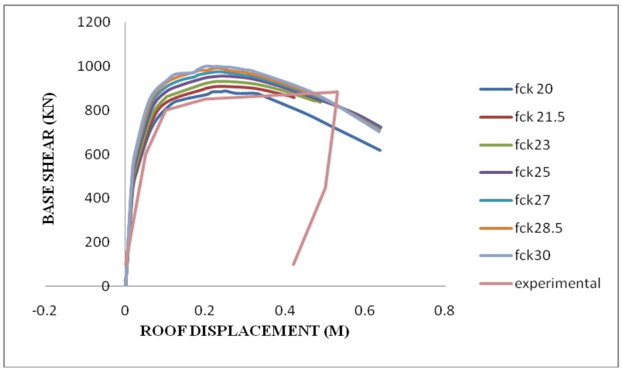


Figure 8: Comparison of experimental and analytical pushover curve for varying f_{ck} (MPa) and for constant f_y=520MPa (Kent and Park model)

The resulting analytical pushover curve for varying f_{ck} (MPa) and for constant f_y=520MPa is compared with the experimental pushover curve for both Mander and Kent and Park model and is represented in fig 7 and fig 8 respectively.

3.2 FRAGILITY CURVES AND DAMAGE STATE THRESHOLD

The damage fragility curves are used to evaluate the seismic risk of the building. From the result of pushover analysis the capacity curve is obtained and table 2 and 3 summarizes the parameters of the damage state thresholds such as ultimate displacement (d_u) and yielding displacement (d_y) of the structure for Mander model and Kent and Park model. Fig 9 and fig 10 shows the capacity curve for f_{ck}=20MPa and f_y=520MPa for Mander and Kent and Park model respectively. The table 2 and table 3 shows the mean damage index for f_{ck}=20MPa and f_y=520MPa for Mander and Kent and Park model respectively.

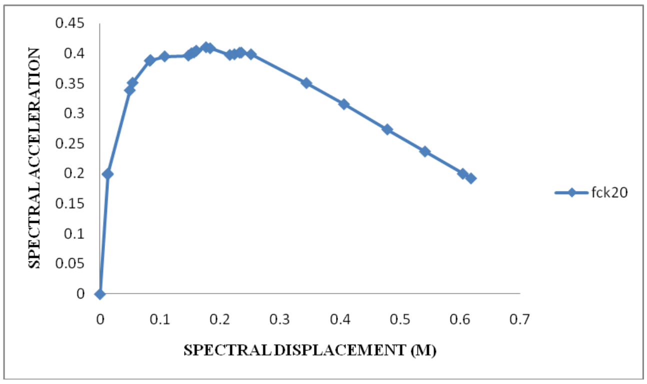


Figure 9: Capacity curve for f_{ck} = 20MPa, f_y = 520MPa in X direction for Mander model

Damage state	Median Spectral Displacement (mm)	
Slight	0.7 d_y	9.107
Moderate	d_y	13.01
Extensive or Severe	$d_y+0.25(d_u- d_y)$	164.286
Collapse	d_u	618.115

Table 2: Mean damage index for f_{ck}=20MPa and f_y=520MPa in X direction for Mander model

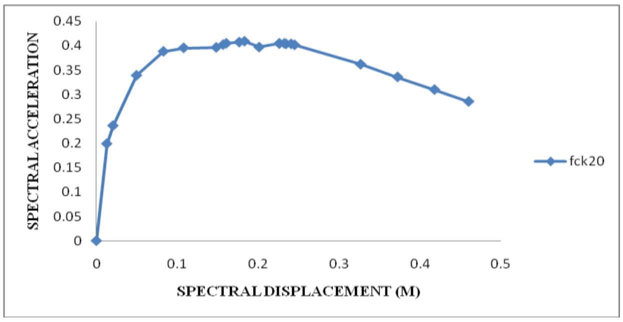


Figure 10: Capacity curve for f_{ck} = 20MPa, f_y = 520MPa in X direction for Kent and Park model

Damage state	Median Spectral Displacement (mm)	
Slight	0.7 d_y	14.35
Moderate	d_y	20.50
Extensive or Severe	$d_y+0.25(d_u- d_y)$	130.375
Collapse	d_u	460

Table 3: Mean damage index for f_{ck}=20MPa and f_y=520MPa in X direction for Kent and Park model

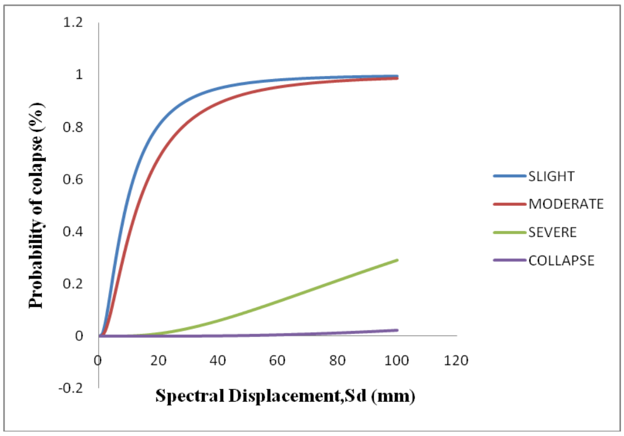


Figure 11: Fragility curve for f_{ck}=20MPa and f_y=520MPa in X direction for Mander model

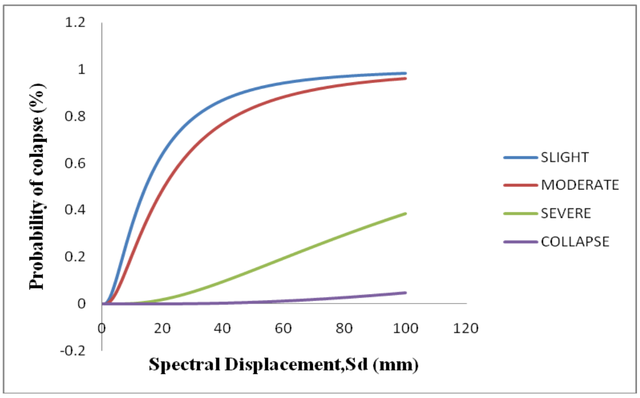
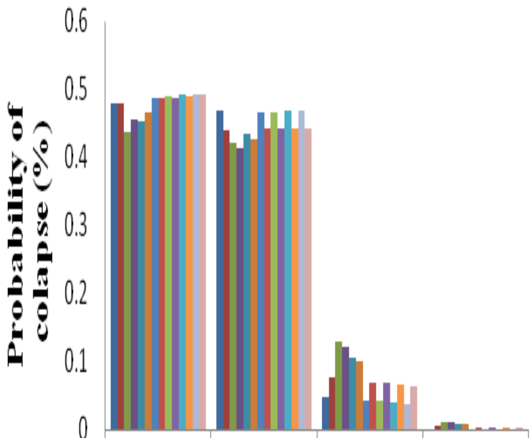


Figure 12: Fragility curve for f_{ck}=20MPa and f_y=520MPa in X direction for Kent and Park model

3.3 COMPARISON OF DAMAGE STATE THRESHOLDS FOR MANDER AND KENT AND PARK MODEL

From the below figure 13 it can be observed that the probability of collapse obtained from both Manders and Kent and Park modal are more or less equal with the variation in the compressive strength of the concrete. It can also be noticed that the slight and moderate structural damage are higher as the compressive strength of the concrete increases and the probabilities of structure lying in the severe and collapse damage state decreases.



	SLIGHT	MODERATE	EXTENSIVE	COLLAPSE
MM Fck20,Fy520	0.48	0.47	0.05	0.002
KNPM Fck20,Fy520	0.48	0.44	0.076	0.004
MM Fck21.5,Fy520	0.438	0.421	0.129	0.011
KNPM Fck21.5,Fy520	0.454	0.414	0.121	0.011
MM Fck23,Fy520	0.453	0.434	0.106	0.007
KNPM Fck23,Fy520	0.467	0.425	0.101	0.007
MM Fck25,Fy520	0.488	0.467	0.044	0.001
KNPM Fck25,Fy520	0.487	0.441	0.069	0.003
MM Fck27,Fy520	0.49	0.467	0.043	0.001
KNPM Fck27,Fy520	0.488	0.441	0.068	0.003
MM Fck28.5,Fy520	0.491	0.468	0.041	0.001
KNPM Fck28.5,Fy520	0.49	0.441	0.066	0.003
MM Fck30,Fy520	0.492	0.468	0.039	0.001
KNPM Fck30,Fy520	0.492	0.442	0.064	0.003

Figure 13: Comparison of Damage state thresholds for Mander and Kent and Park model

4. CONCLUSIONS

From the usage of advanced software package i.e., SAP 2000 version 18, as per the guidelines the pushover analysis is carried out for the seismic risk evaluation of reinforced concrete building where the structure is subjected to an monotonic loading with the inverted load profile.

In the Mander as well as Kent and Park models as the compressive strength of the concrete increases the base shear value also increases. The value of base shear also increases with the increase in the tensile strength of the steel up to a certain extent, later it becomes constant.

The analytical modeling approached base shear values are almost closer to the experimental results having a difference of about plus or minus 5%.

During performance point evaluation, the values of base shear and roof displacement are independent of tensile strength of steel in both Mander and Kent and Park approach. Also, the roof displacement and spectral displacement decreases with the increase in the base shear and spectral acceleration.

The fragility analysis result shows that for the considered buildings with f_{ck}=20MPa and f_y=520MPa, a high probability of slight, moderate, severe and collapse damages is suffered by the building in both Mander and Kent and Park model.

In the damage state threshold, the probability of collapse obtained from both Manders and Kent and Park modal are more or less equal with the variation in the compressive strength of the concrete. It can also be noticed that the slight and moderate structural damage are higher as the compressive strength of the concrete increases and the probabilities of structure lying in the severe and collapse damage state decreases.

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