Damage Index of Reinforced Concrete Structures in India

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ABSTRACT

In today's world, there has been a lot of discussion in vulnerability of the structures. The major problem faced in world as well for the developing countries like India is proper quantification of damage. In spite of making signification progress in the design codes, there has been significant lack in the progress of quantification of damage. Thus Damage Index is the response parameter used for quantification of degree of damage. A land where major earthquakes occurred over last two decades causing devastation both social and economic, an urgent concern for damage quantification arises. The present study is done to carry out quantification of damage for a four-storey RC Moment resisting frame by modified Park and Ang damage model which is then compared by different Indian Standard code for Earthquake resistant design.

Key Words: Damage Index, Energy, Park and Ang, Pushover

INTRODUCTION

Seismic Vulnerability can be defined as the degree of damage to the structure or of a region when it is subjected to earthquake action. Vulnerability results can be obtained from the performance characteristics of the structure. The main problem which is faced today's world is a proper quantification of damage. Inspite of making significant progress in loading and design codes of seismic resistant structure and technology improvement in building structures the field of civil engineering is facing critical challenges. Very clear example of those challenges is the assessment of the damage state which is imposed on the structure after earthquakes of different intensity. A reinforced concrete structure is damaged by the combination of stress reversals and high stress excursions. So the damage criterion should not on include the damage criterion but also the effect of repeated cyclic loading. In order to determine the operability of the structure and its resistance to earthquake, a thorough knowledge of the damage state is required. For reliable economic loss quantification for a structure or of a region, the view and information of structural damage is critically important. Many studies and researches have been performed for estimating the risk level of structures in the past few years. For characterizing the damage in a structure or of a region, the relationship between earthquake ground motion severity and structural damage are very well used. The motion damage relationships at specific ground motion are usually expressed in the form of probability distribution of damage to form fragility curve. Damage Index is a response quantity or a dimensionless ratio or structural response to earthquake simulation used for measuring imposed damage of imposed structure based on performance ranges. Damage Index measures the amount of damage as well as the degradation that takes place in the structure. Damage Index is a dimensionless quantity usually ranges from 0 to 1. 0 represents undamaged state where 1 represents collapse state. The intermediate value gives some degree of damage.

Extensive studies were carried out by Williams and Sexsmith [1995], Ghobarah et. Al [1999] and Padilla et. al [2009] for evaluating Damage Index. DiPasquale [1990] presented Damage Index based on Global Damage Index and Stiffness degradation. Mihai [2012] provided classification of Damage Indices taking into effect the use of different parameters for local and global Damage Indices. Vimala and Ramcharla [2012] considered displacement damage estimation on four-storey structure. Damage Index may also be considered based on SDOF approximation, dynamic characteristics of building and micro-level modeling of element. Massumi and Mostagh [2013] proposed Damage Index which shows elongation between fundamental periods. Few recent works have been done across the globe for quantification of damage based on different parameters. The damage model which is used to quantify the damage is the Park and Ang [1985] damage model. The damage model is used for expressing the potential damage of reinforced structure as a function of maximum deformation and dissipated energy.

$$I_D = \frac{d_m}{d_u} + \beta_e \frac{\int dE}{F_y d_u} \quad (1)$$

Where, d_m =Maximum displacement due to point of maximum capacity, d_u = Ultimate displacement due to monotonic loading, β_e =Parameter representing the cyclic loading-strength degradation factor, dE =incremental dissipated energy, F_y = longitudinal reinforcement yielding force.

Kunnath [1992] modified the Park and Ang Damage Index which is used to compute the structural damage for the earthquakes. The modified Park and Ang damage index is given as

$$I_D = \frac{d_m - d_y}{d_u - d_y} + \beta_e \frac{\int dE}{F_y d_u}$$

Where, d_m =Maximum displacement obtained from Time history analysis.

STRUCTURAL MODELLING

The example building is a regular four-storey reinforced concrete structure. The building is one which addresses strong column-weak beam theory. Strong column-weak beam buildings cause the structure to exhibit intermediate damage state and to avoid the formation of collapse mechanism. The damage index is used to quantify the damage caused in beams and columns or due to the local collapses that occur in the structure. The building has a rectangular plan of 12m x 16m which is shown in plan of Fig 1 and 2. The lateral load resisting element in X-direction consists consist of 3 bays each of 4m width and in Y-direction 4 bays of 4m width. The height of each storey is taken as 3m. Space frame model is used (Fig 2). The characteristic compressive strength which is used in design is M25 and that for steel it is Fe415. Dynamic analysis of the structure is determined by free vibration. The first three frequencies obtained are

1.412Hz, 4.831Hz, and 9.556Hz which is obtained in X-direction. The structure is designed as per IS-456:2000. The dynamic analysis which is considered are the Indian Standard, IS-1893(Part I):2002, IS-1893:1984 and IS-1893:1970. Comparisons are done for this structure against the different Indian standard codes. Table 1 shows the Base Shear for different IS codes.

Table 1: - Evaluation of Base Shear

IS CODE	BASE SHEAR (KN)
IS-1893:2002	392
IS-1893:1984	359
IS-1893:1970	300

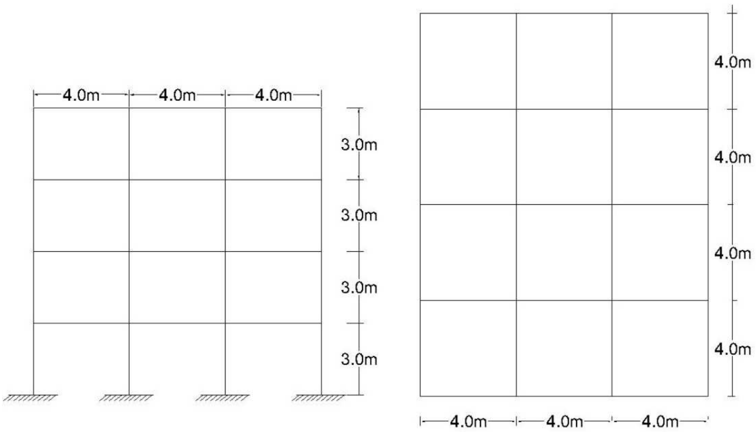

Fig 1: - Plan and Elevation of the structural model

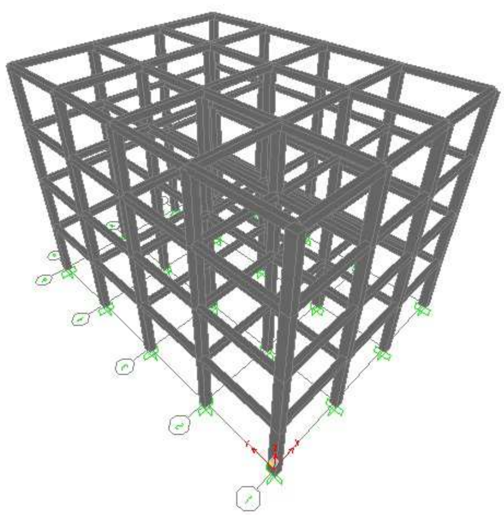

Fig 3: - 3-D view of the structural model showing beams and columns

GROUND MOTION SELECTION

Seismic provision in today's construction has become must for the safety of the structure which includes standards and model building codes. This rule is basically to be followed specially for non-linear response history analysis. Ground motions are selected and have to be scaled before put into analyzing the structure. There must be a clear objective on the necessity of scaling the ground motion data. The appropriate method depends on structural parameter on which the responses are to be calculated. Ground motions can be selected from previous earthquake records or by creating self-defined models where there is a lag in appropriate data. Ground motion plays a great role in assessing the response due to the non-linear analysis of the structure. Large set of ground motion can be used for the analysis. For realistic results, the ground motions are scaled with respect to the design spectrum for which the structure is modeled. It is thus useful to have generated the scaled spectrum to match the design response spectrum [C.B. Haselton et.al, 2012]. In this analysis, Indian Standard code for Earthquake Resistant Design is used. Zone V Type II soil is taken. For comparison and to assess the ductility and Damage Index, IS-1893(Part I):2002, IS-1893:1984 and IS-1893:1970 has been used. Five earthquakes are considered in this example model to perform the time history analysis. Table 1 shows the dynamic characteristics of the selected ground motion.

Table 2: - Characteristics of the Ground Motion

Event	Station	PGA (g)	Mechanism	Magnitude	Vs30 (m/s)
Bigbear- 1992	North Shore	0.043	Strike-Slip	6.46	338.5
Imperial Valley-1979	El Centro E10	0.176	Strike-Slip	6.53	202.8
Kobe- 1995	Kakogawa	0.058	Strike-Slip	6.9	312.0
Landers- 1992	Mission Creek Fault	0.122	Strike-Slip	7.28	345.4
Northridge- 1994	Arleta	0.237	Reverse	6.69	297.7

The analyses are carried out in the example building using the above mentioned accelerograms time history scaled to 0.108g, 0.216g, 0.324g, 0.432g, 0.54g and the time each ground motion has its own time step ranging from 0.005sec to 0.02sec.

NON-LINEAR STATIC PUSHOVER ANALYSIS

The recent major earthquakes occurred across the world leaves a question on the seismic rehabilitation of the older concrete structures. It's a matter of growing concern to assess the vulnerability of the structures. So it is not possible to carry out the analysis up to linear range. A pushover analysis is a process of incremental static analysis which is used to carry out to develop a capacity curve for the building. Based on the capacity curve, a target displacement will be obtained. When the load is increased incrementally various structural elements may yield sequentially. Consequently, at each step, there is a loss in stiffness. Using a pushover analysis, a characteristic non linear force-displacement relationship can be determined. Pushover analysis is performed on the example building in SAP2000, and the capacity curve is plotted from the Base Shear and displacement as shown in Fig 4.

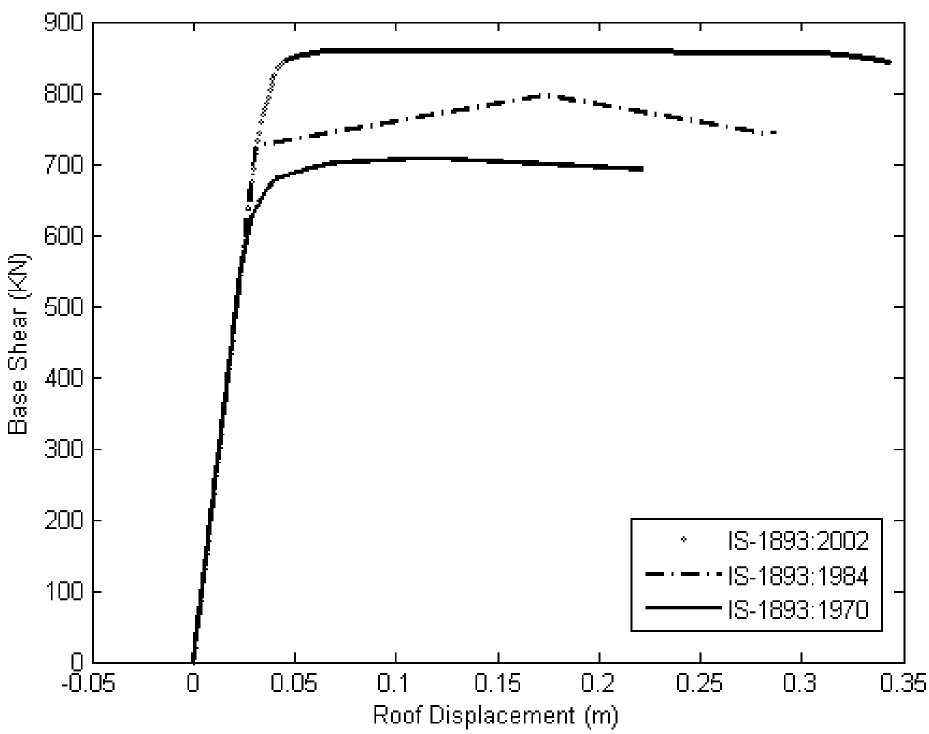

Fig 4: - Pushover curve by different Indian Standard codes

The Capacity curve which is obtained from the Pushover analysis has to be converted into an equivalent bilinear curve. The Capacity curves are based on two points – Yield and Ultimate which characterizes the capacity curve. The Capacity curve is assumed to remain in plastic state after the yield point. The bilinearisation is done with the help of BILIN. Table 3 shows the yield and the ultimate points of the different pushover curves obtained after bilinearisation.

Table 3: - Bilinear result of Pushover Analysis

IS CODE	Base Shear (KN)		Roof Displacement (m)	
	Yield	Ultimate	Yield	Ultimate
IS-1893:2002	855.875	855.875	0.03649	0.34298
IS-1893:1984	749.945	778.866	0.03196	0.28720
IS-1893:1970	690.473	702.660	0.02888	0.22130

NON-LINEAR TIME-HISTORY ANALYSIS

The use of Non-Linear Response History Analysis is practiced now-a-days rigorously to demonstrate the performance of the structures. This method requires a selection and a scaling procedure as described in the earlier sections. This is generally used to design the hazard levels. The seismic demands are generally determined by Response history method for several ground motions. Current procedure involves in scaling of the ground motions to meet the spectral response of design or target design. The non-linear response history analysis is performed for the FIVE earthquakes which are scaled up to visualize the structural response due to these scaled earthquake motions. The non-linear response, i.e., the hysteresis curve is of utmost importance, which is the plot between Force and Roof Displacement. As the maximum displacement occurs in the top storey, so the response of the top storey is plotted. The results from the dynamic analysis by different Indian standard codes are evaluated in order to assess the damage of the structure. The parameters which are taken into account are – Maximum Displacement (m), Roof Drift (%) and Inter-Storey Drift (%) The building is subjected to spectrum scaled ground motion of different Peak Ground Acceleration. Damage Index is calculated from Equation (2) which is the combination of damage assessed for maximum displacement and energy dissipated. The mean results are given in Table 4.

As the maximum displacement occurs in the top storey, so the response of the top storey is plotted. The plots among different responses are given as under in Fig 5, Fig 6 and Fig

Table 4: - Plot of responses obtained from Time-History Analysis

PGA (g)		0.108	0.216	0.324	0.432	0.54
Max. roof displacement (m)	IS-1893:2002	0.02952	0.05280	0.09672	0.11796	0.20976
	IS-1893:1984	0.02712	0.05208	0.08616	0.13284	0.22308
	IS-1893:1970	0.02628	0.04836	0.09572	0.13776	0.20064
Inter-storey Drift (%)	IS-1893:2002	0.29%	0.52%	0.77%	1.36%	2.96%
	IS-1893:1984	0.30%	0.56%	1.33%	1.87%	2.86%
	IS-1893:1970	0.33%	0.66%	1.74%	2.24%	3.27%
Damage Index	IS-1893:2002	0	0.070	0.201	0.400	0.862
	IS-1893:1984	0.009	0.116	0.280	0.459	0.996
	IS-1893:1970	0.036	0.188	0.466	0.694	1.008

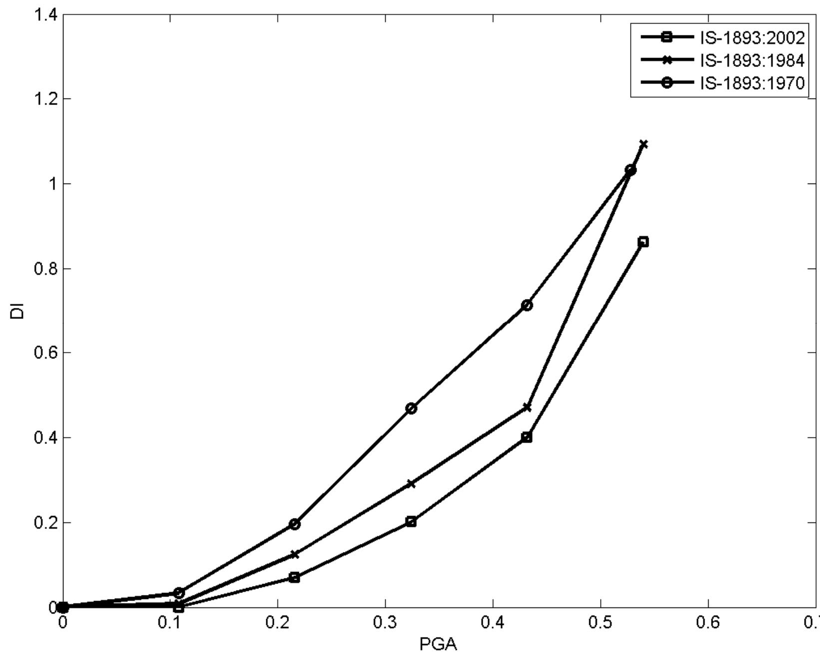

Fig 5: - Plot of Damage Index and PGA

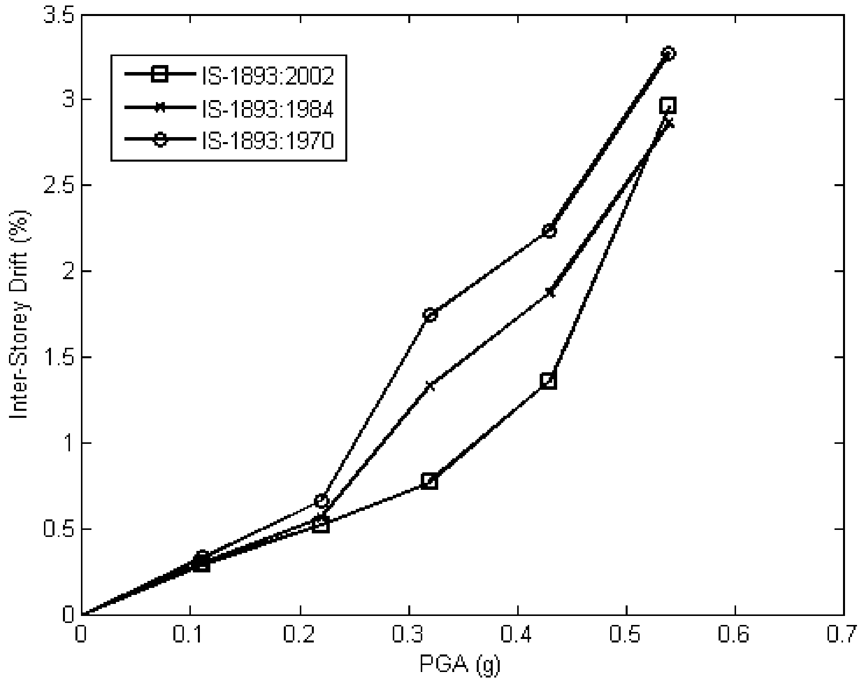

Fig 5: - Plot of Inter-storey Drift and PGA

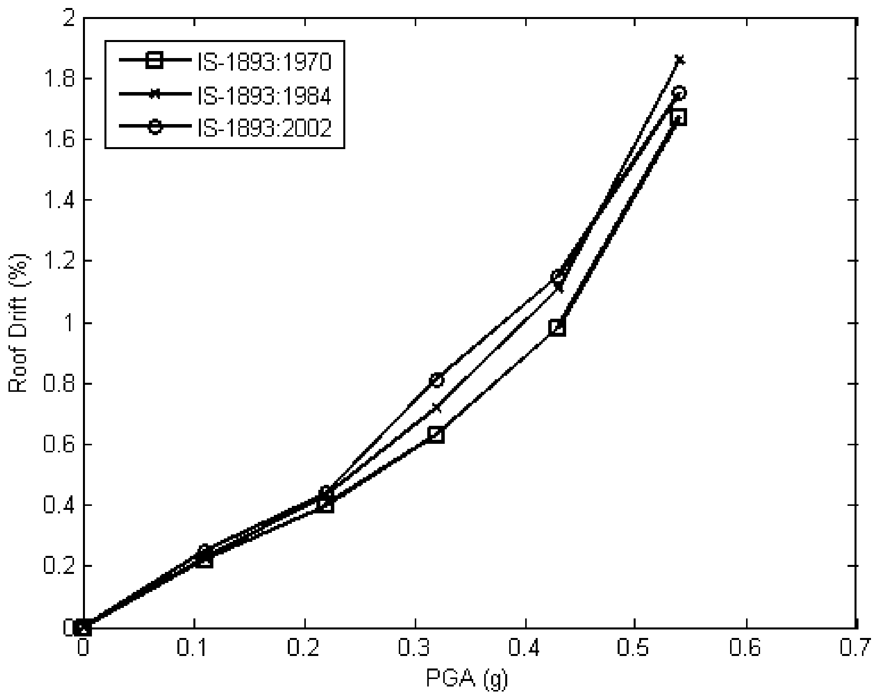

Fig 7: - Plot of Roof Drift and PGA

DAMAGE STATES

The building damage prediction allows us to study the expected damage patterns in a given region for different earthquake scenarios. The damage predication allows us to glean the nature and extent of physical damage of building type from damage predication. The result of damage estimation enables us to predict the risk involved in defining a damage model. It is also used to determine the casualties due to structural damage, monetary losses due to building damage and other social and economic impacts. The damage states which is taken into account are – Slight, Moderate, Extensive and Collapse. Slight Damage state extends from the threshold of slight damage to the threshold of moderate damage. It is same for the other damage state as well. Thus the Damage States are defined on the basis of the response parameters calculated above which is shown in Table 5.

Table 5: - Damage States showing limits

Damage States		Slight	Moderate	Extensive	Collapse
Peak Ground Acceleration (g)	IS-1893:2002	0.17	0.3	0.44	0.5
	IS-1893:1984	0.10	0.19	0.32	0.46
	IS-1893:1970	0.10	0.16	0.28	0.38
Damage Index	IS-1893:2002	0.06	0.18	0.44	0.7
	IS-1893:1984	0.03	0.1	0.3	0.7
	IS-1893:1970	0.04	0.16	0.38	0.6
Inter-Storey Drift (%)	IS-1893:2002	0.4	0.7	1.5	2.5
	IS-1893:1984	0.3	0.5	1.4	2.2
	IS-1893:1970	0.3	0.5	1.3	2

CONCLUSIONS

1. The Base Shear is compared. Design by recent code or high code ensures proper ductility as the capacity is maximum by this code. More the ductility more is the energy dissipation before collapse.
2. For IS-1893(Part I):2002, design PGA of 0.36g brings the structure in Moderate Damage State with Damage Index of 0.27, while that

for IS-1893:1984, it is in Extensive Damage State with Damage Index of 0.35 and for IS-1893:1970 it is in Extensive Damage State with Damage Index 0.55

3. Higher the PGA value more is the damage occurred in the structure. For a given value of PGA, the damage is highest for IS-1893:1970 while it is lowest for IS-1893:2002.
4. Conventionally it lightens up that the Inter-storey Drift would increase as capacity goes on reducing, but for PGA 0.54g the Inter-storey drift obtained by IS-1893:2002 is more than that obtained by IS-1893:1984
5. Due to high ductility obtained by the most recent code, the pattern would follow that the maximum roof displacement will occur for IS-1893:2002, but at 0.54g PGA the maximum roof displacement for IS-1893:1984 comes out to be higher than IS-1893:2002.
6. The further scope of work does include the vulnerability models for different soil conditions and capacity variations. This also includes the models for other building classes too.

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