International Research Journal of Engineering and Technology (IRJET) e-ISSN: 2395-0056
Volume: 07 Issue: 05 | May 2020 www.irjet.net p-ISSN: 2395-0072

# Evaluation of Seismic Damage Index of Reinforced Concrete Moment Resisting Frames using Nonlinear Static Procedure

**Ramatai B. Somwanshi[1], Ashish Pawar[2], Anand Singh[3], Nishank Kadukar[4], Rushikesh Kharade[5]**

*[1]Assistant Professor, Civil Engineering Department, Dr.D.Y.Patil Institute of Technology, Pimpri, Pune, Maharashtra, India*
*[2]-5Batchler's Student, Civil Engineering Department, Dr.D.Y.Patil Institute of Technology, Pimpri, Pune, Maharashtra, India*

---------------------------------------------------------------------***---------------------------------------------------------------------

**Abstract -** *The main objective of this study is, evaluation damage index of reinforced concrete moment resisting frames by" NONLINEAR STATIC PROCEDURE" nonlinear static analysis includes the capacity spectrum method (CSM) that uses the intersection of the capacity (pushover) curve and a reduced response spectrum to estimate maximum displacement in terms of damage of building. Nonlinear static procedure is simple and practical method for static damage index. For this purpose, first some functions are derived to estimate damage to the structure using pushover analysis and then designed procedure is proposed. In this study damage function is estimated by using correlation between park-Ang. damage index (NLDD) and nonlinear static damage index (NLSD) which is based on the pushover analysis. For this purpose dynamic and static damage analysis are performed on several concrete frames subjected to various earthquake acceleration records. So the detail explanation is found in this study.*

***Key Words:*** pushover, FEMA-356, ATC-40, Static damage index, Dynamic damage index, *etc...*

## 1. Introduction

Experience learn from past earthquake and increase in design knowledge practicing designing engineers had moved towards predictive methods of design and evaluation. The main aim was to communicate safety related discussion which present seismic design does not clarify. One of such a design procedure is performance base seismic design. This method is generalized design process in which design parameter are expressed in terms of performance objective. These performance objective are statement of acceptable risk due to damage of structural component under specified seismic hazards level. Thought the performance of the structural is addressed but it does not quantify damage associated with performance level. For this damage state is associated with a damage value.

The damage value is expressed with the help of damage index. Damage index is associated with physical measurable parameter known as engineering demand parameter (EDPs). The main ordinary parameter involve in damage assessment are permanent deformation, strength and stiffness degradation and number of hysteresis cycles involve. The damage index can be expressed by nonlinear dynamic analysis as well as nonlinear static analysis. The damage function was defined based on few nonlinear responses which estimate plastic energy dissipated by rotation of beams and columns to verify the damage value it has been compared with Park-Ang. damage value. Park-Ang. expressed seismic structural damage as a linear combination of the damage caused due to more deformation and the effect of repeated cyclic loading. Under elastics response the value of damage index is theoretically zero. And the damage index greater than one means the total collapse or total damage. Therefore the structural damage is a function of response of maximum deformation under earthquake and incremental absorbed hysteretic energy which is depend upon the loading. There are several different nonlinear dynamic analysis procedures are available but it is widely applicable due to its complexity and time consuming.

## 2. Damage Index Based on Nonlinear Static Analysis

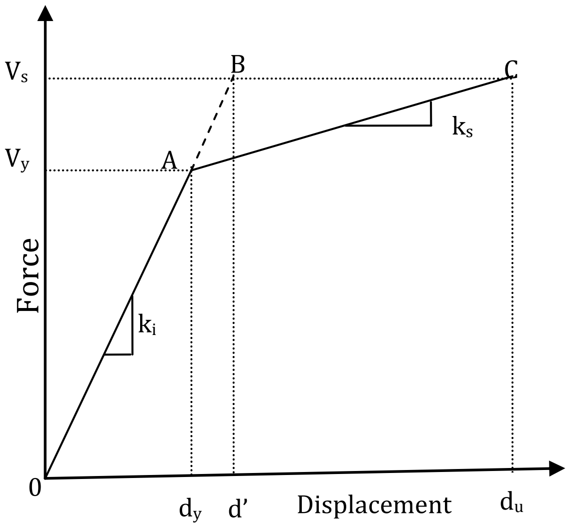



**Fig -1**: Bilinear SDOF system

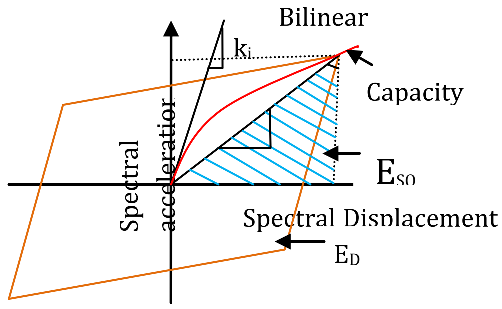



**Fig -2**: Damping for spectral reduction

The existence of plastic energy (PE) in the equation indicates the damage of the structural frame due to earthquake ground motion. It represents the energy that is consumed by the permanent plastic rotation in the beams and columns at the time't'. Larger the value of PE, the more significantly the frame has damaged. Therefore the damage index can be based on the energy stored in the permanent plastic rotation.

The damping that occurs when earthquake ground motion drives a structure in to inelastic energy range is a combination of viscous damping that is inherent in the structure and hysteretic damping. Hysteretic damping is related to the area inside the loops that are formed when the earthquake forces (base shear) is plotted against the structure displacement. Hysteretic damping in the form of equivalent viscous damping as

$$\beta_{eq} = \beta_0 + 0.05 \qquad (1)$$

Where;

$\beta_0$ = Hysteretic damping represented as equivalent viscous damping

0.05 = 5% viscous damping inherent in the structure

$$\beta_0 = \frac{1}{4\pi}\frac{E_D}{E_{s0}} \qquad (2)$$

Where;

$E_D$ = Energy dissipated by damping

$E_{SO}$ = Maximum strain energy

$E_D$ is the energy dissipated by the structure in single cycle of motion that is the area enclosed by a single hysteretic loop. ESO is the maximum strain energy associated with that cycle of motion that is the area the hatched triangle in figure 4[11].

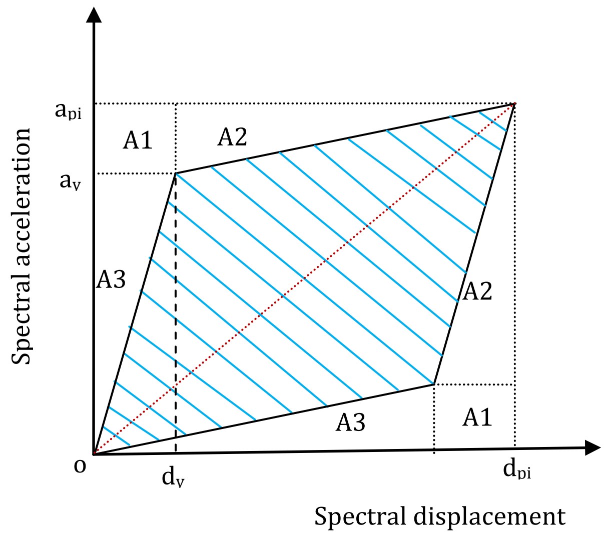



**Fig-3**

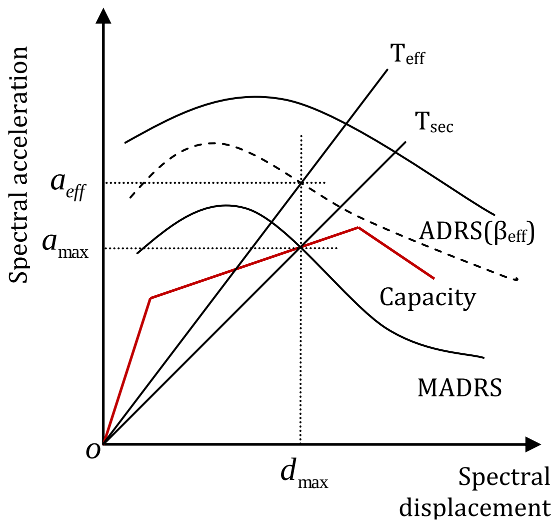



**Fig-4**

$E_D$ = $E_{PP}$ = (shaded area in figure 3)

$$E_{PP} = (a_{pi}d_{pi} - 2A_1 - 2A_2 - 2A_3)$$
$$E_{PP} = (a_{pi}d_{pi} - a_y d_y - (d_{pi} - d_y)(a_{pi} - a_y) - 2d_y(a_{pi} - a_y))$$
$$E_{PP} = (a_y d_{pi} - d_y a_{pi}) \qquad (3)$$

and,

$$E_{SO} = \frac{a_{pi}d_{pi}}{2} \qquad (4)$$

Plastic energy damage index to define structural damage can be formulated based on assumptions;

a) That capacity curve resulting from pushover analysis almost represents the envelopes of the hysteretic loops.

b) The area under the capacity spectrum at the performance point approximately demonstrates the stored energy at the biggest hysteretic loop in which a large portion of energy is dissipated when the structure is subjected to earthquake.

$$a_{max} = a_{pi};\ d_{max} = d_{pi}$$

$$DI_{pp} = \frac{A_p}{A_u} = \frac{E_{pp} - E_{ip}}{E_{fp} - E_{ip}} \quad (5)$$

Where,

$A_p$ = net area of the capacity curve up to the performance point.

$A_u$ = net area of the capacity curve up to the ultimate point

$$E_{pp} = a_y d_{pi} - d_y a_{pi}$$

$$E_{fp} = a_y d_u - d_y a_u$$

$$E_{ip} = 0.5 a_y d_y$$

## 3. Examples building frames

In this study ten reinforced moment resisting frame buildings with 2,4,6,8,10,12,14,16,18,20 stories having three and four bays were designed using seismic force levels obtained from Indian seismic codes ie I.S 456:2000(rev) I.S. 1893:2001(part1) Table 1 and 2 describes the characteristics and preliminary data considered for analysis and design of the considered frames

**Table 1:** Characteristics of example building frames

| **Frame model** | **Height (m)** | **Time Period (sec)** | **Seismic Weight (W)** | **Base Shear (V)** |
|---|---|---|---|---|
| **S2b3** | 6 | 0.28752 | 972 | 87.48 |
| **S4b3** | 12 | 0.4835 | 2052 | 153.9 |
| **S6b4** | 18 | 0.6554 | 5565 | 308.382 |
| **S8b4** | 24 | 0.81328 | 7488 | 332.8 |
| **S10b4** | 30 | 0.9614 | 9408 | 352.8 |
| **S12b4** | 36 | 1.1022 | 11328 | 370.735 |
| **S14b4** | 42 | 1.2373 | 13248 | 387.746 |
| **S16b4** | 48 | 1.3677 | 15168 | 401.506 |
| **S18b4** | 54 | 1.49402 | 17088 | 412.864 |
| **S20b4** | 60 | 1.61687 | 19008 | 425.024 |

**Table 2:** Preliminary data considered for analysis and design

| Sr. No | Particulars | Value | Remarks |
|---|---|---|---|
| 1 | Bay width | 4m | In both direction for all frames |
| 2 | Storey height | 3.0m | In both direction for all frames |
| 3 | Concrete grade (M25) | 25 Mpa | As per I.S 456:2000 |
| 4 | Rebar's | | |
| | a.Main reinforcement | 415 Mpa | |
| | b.Shear reinforcement | 415 Mpa | |
| 5 | Type of exposure | Mild | |
| 6 | Type of soil | Hard soil | As per I.S 1893:2001(part) Z=0.36 Non-ductile Public building |
| 7 | Seismic zone | Zone IV | |
| 8 | Response reduction factor (R) | 5.0 | |
| 9 | Importance factor (I) | 1.0 | |
| 10 | Natural time period (T) | $0.075(h)^{0.75}$ | |
| 11 | Lateral force (V) | $A_h W$ | $A_h = \frac{ZIS_a}{2gR}$ |
| 12 | Storey shear ($Q_i$) | $V\left(\frac{W h_i^2}{\sum_{i=1} W_i h_i^2}\right)$ | W = Seismic weight |

The preliminary design is carried for various load combinations suggested in IS 1893:2000 (Part1) using FEM based software SAP 2000 V 17 for initial values of R =5.0. The gravity design output is tabulated in table No 3.and the result of damage index on the basis of pushovercurve is tabulated in table No 4.

**Table No. 3:** Gravity design results for example buildings

| Model No. | Storey No. | Column design details | | | | |
|---|---|---|---|---|---|---|
| | | Cross section | Main reinf. | Shear reinf. | $P_t$ main | $P_t$ trans. |
| S2B3 | 1 | 400 x 400 mm | 1280 | 65 | 0.8 | Min. |
| S4B3 | 1-4 | 400 x 400 mm | 1280 | 65 | 0.8 | Min |
| S6B4 | 1-6 | 400 x 500 mm | 1600 | 0.433 | 0.8 | Min |
| S8B4 | 1-8 | 500 x 500 mm | 2000 | 0.533 | 0.80 | Min. |
| S10B4 | 1-10 | 600 x 600 mm | 2555 | 65 | 0.80 | Min |
| S12B4 | 1-12 | 650 x 650 mm | 2912 | 65 | 1.4 | Min |
| S14B4 | 1-14 | 700 x 700 mm | 3400 | 65 | 0.9 | Min. |
| S16B4 | 1-16 | 800 x 800 mm | 3600 | 65 | 0.8 | Min |
| S18B4 | 1-18 | 800 x 800 mm | 4000 | 65 | 0.8 | Min |
| S20B4 | 1-20 | 900 x 900 mm | 4600 | 65 | 1.2 | Min |

| Model No. | Storey No. | Beam design details | | | | |
|---|---|---|---|---|---|---|
| | | Cross section | Main reinf. | Shear reinf. | $P_t$ main | $P_t$ trans. |
| S2B3 | 1 | 300 x 300 mm | 303 | 65 | 0.29 | Min |
| S4B3 | 1-4 | 400 x 400 mm | 405 | 65 | 0.30 | Min |
| S6B4 | 1-6 | 400 x 400 mm | 624 | 0.433 | 0.39 | Min |
| S8B4 | 1-8 | 500 x 500 mm | 723 | 0.533 | 0.40 | Min. |
| S10B4 | 1-10 | 600 x 600 mm | 912 | 65 | 0.41 | Min |
| S12B4 | 1-12 | 650 x 650 mm | 1114 | 65 | 0.4 | Min |
| S14B4 | 1-14 | 700 x 700 mm | 1313 | 65 | 0.41 | Min. |
| S16B4 | 1-16 | 700 x 700 mm | 1421 | 65 | 0.4 | Min |
| S18B4 | 1-18 | 800 x 800 mm | 1622 | 65 | 0.4 | Min |
| S20B4 | 1-20 | 900 x 900 mm | 1723 | 65 | 0.29 | Min |

**Table No. 4:** Result of static damage index

| FRAME | Dy | Ay | Dp | Ap | Du |
|---|---|---|---|---|---|
| S2B3 | 0.0078 | 0.1220 | 0.049 | 0.261 | 0.10149 |
| S4B3 | 0.0183 | 0.7903 | 0.018 | 0.767 | 0.086802 |
| S6B4 | 0.0192 | 0.4051 | 0.036 | 0.577 | 0.151189 |
| S8B4 | 0.0097 | 0.2354 | 0.045 | 0.464 | 0.119832 |
| S10B4 | 0.0126 | 0.2615 | 0.038 | 0.421 | 0.115815 |
| S12B4 | 0.0152 | 0.2424 | 0.043 | 0.366 | 0.139597 |
| S14B4 | 0.0184 | 0.2381 | 0.047 | 0.34 | 0.137463 |
| S16B4 | 0.0186 | 0.0628 | 0.135 | 0.092 | 0.171468 |
| S18B4 | 0.0264 | 0.0749 | 0.131 | 0.095 | 0.154805 |
| S20B4 | 0.0267 | 0.0716 | 0.126 | 0.1 | 0.18418 |

| FRAME | Au | Eip | Epp | Efp | DI |
|---|---|---|---|---|---|
| S2B3 | 0.26035 | 0.00048 | -0.0103 | -0.0103 | 0.0353 |
| S4B3 | 1.02082 | 0.00725 | -0.0499 | -0.0545 | 0.0758 |
| S6B4 | 0.65633 | 0.00391 | -0.0486 | -0.0501 | 0.0254 |
| S8B4 | 0.48448 | 0.00115 | -0.0235 | -0.0237 | 0.0239 |
| S10B4 | 0.42139 | 0.00165 | -0.025 | -0.025 | 0.0394 |
| S12B4 | 0.36426 | 0.00185 | -0.0283 | -0.0283 | 0.0352 |
| S14B4 | 0.33796 | 0.0022 | -0.0265 | -0.0265 | 0.0542 |
| S16B4 | 0.09179 | 0.00059 | -0.0091 | -0.009 | 0.0551 |
| S18B4 | 0.09449 | 0.00099 | -0.0091 | -0.0091 | 0.09896 |
| S20B4 | 0.09905 | 0.00096 | -0.0105 | -0.0105 | 0.0797 |

**Table No. 5:** Acceptance Criteria for performance levels [FEMA 356:2000]

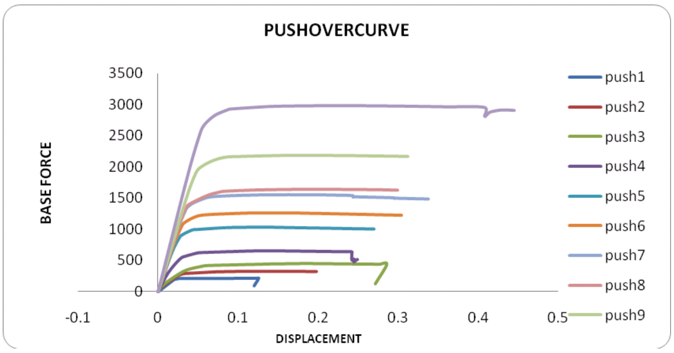
| Type of structure | Performance level | Acceptance criteria (Drift) |
|---|---|---|
| Moment resisting concrete frame | Collapse prevention (S-5) | 4% transient or permanent |
| | Life safety (S-3) | 2% transient ; 1% permanent |
| | Immediate occupancy (S-1) | 1 % transient; negligible permanent |

## 4. Result and Discussion

**Table No. 6**: Result of park-ang damage index and nonlinear static damage index

| STOREY NO | Park-ang damage index DI(IDARC) | Energy base damage index DI(NLSD) |
|---|---|---|
| S2B2 | 0.000 | 0.03535 |
| S4B3 | 0.001 | 0.07583 |
| S6B4 | 0.009 | 0.02541 |
| S8B4 | 0.003 | 0.02395 |
| S10B4 | 0.007 | 0.0394 |
| S12B4 | 0.011 | 0.03525 |
| S14B4 | 0.058 | 0.05427 |
| S16B4 | 0.077 | 0.05516 |
| S18B4 | 0.078 | 0.09896 |
| S20B4 | 0.041 | 0.0797 |

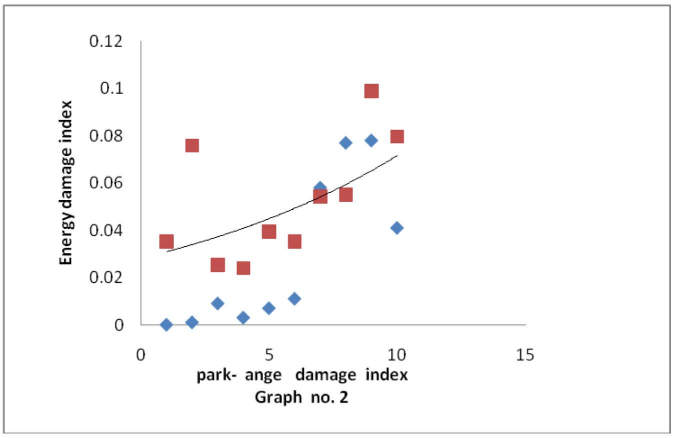
Table no. 6 shows the NLDD and the NLSD ie park ang damage index and energy base damage index, the park-ang damage index is non linear damage index evaluated by using software IDARC. And static damage index is evaluated using pushover curve by sap2000v17.




**Graph No. 1**

### Correlation between the static and dynamic damage index:

Correlation between the energy damage index (static damage index )and the Park–Ang damage index (dynamic Damage index) is determined by comparing damage results of two sets (static and dynamic criteria). Scatter points on graph no. 2 specify this correlation. As seen in graph no. 2 the energy damage index (static) proposed in this research possesses proper dispersal with the Park–Ang damage index (dynamic). Considering this outstanding feature, its simplicity of calculation and the fact that the energy criterion is a global damage index, this static damage index can be introduced as a simple and effective criterion. To develop a relation for estimating the damage index using pushover results, by fitting a curve, according to graph no. 2




**Graph No. 2**

**Table No. 7:** Damage state

| Degree of damage | Damage index | State of building |
|---|---|---|
| collapse | >1.0 | Loss of building |
| Severe | 0.4-1 | Beyond repair |
| Moderate | 0.25-0.4 | Repairable |
| Minor | 0.1-0.25 | Repairable |
| Slight | <0.1 | Repairable |

## 5. Summary and conclusion

1. In this study damage function is estimated by using correlation between park-ang damage index (NLDD) and nonlinear static damage index (NLSD) which is very simple practical method for nonlinear analysis.
2. The nonlinear static damage index ie (Energy damage index) is proposed and implanted to estimate the damage value using nonlinear responses resulting from pushover analysis. The use of dissipated energy by a structure has been implemented to determine damage index
3. The ultimate deformation capacity of the structure is found by using nonlinear static pushover analysis and for that deformation the energy capacity of the structure is calculated.

| Abbreviations | |
|---|---|
| **ATC** | Applied Technical Council |
| **FEMA** | Federal Emergency Management Authority |
| **DI** | Damage index |
| **NLSD** | Nonlinear Static Damage index |
| | |
| **NLDD** | Nonlinear Dynamic Damage index |
| | |

## References

1. A.R. Habibi a, M. Izadpanahb, A.R. Habibi a, M. Izadpanahb, New method for the design of reinforced concrete moment resisting frames with damage control. proceeding of Scientia Iranica A (2012) 19 (2), 234–241.
2. Anthugari Vimala, Pradeep Kumar Ramancharla, Expended energy based damage assessment of RC bare frame using nonlinear pushover analysis. in Urban Safety of Mega Cities in Asia 2014 (USMCA 2014)
3. FEMA-356., 2000. Prestandard and commentary for the seismic rehabilitation of buildings. Federal Emergency Management Agency, Washington.
4. Ghobarah, A., Abou-Elfath, H. and Buddha, A., 1999 , Response-based damage assessment of structures Earthquake Engineering and Structural Dynamics 28, 79-104.

5. Kappos, A. J., 1997,Seismic damage indices for RC buildings: evaluation of concepts and procedures Progress in Structural Engineering and Materials 1, 78-87.
6. Kunnath, S. K., Reinhorn, A. M. and Park, Y. J., 1990 ,Analytical modeling of inelastic seismic response of r/c structures. Journal of Structural Engin eering ASCE 116, 996-1017.
7. Phaiboon Panyakapo ,Seismic capacity digram for damage based design. The 14 Th World Conference on Earthquake Engineering October 12-17, 2008, Beijing, China
8. Comartin, C. D., Niewiarowski, R. W., Rojahn, C. and California Seismic Safety Commission. 1996. ATC-40 Seismic evaluation and retrofit of concrete buildings, Applied Technology Council, Applied Technology Council, Redwood City, California.
9. Cosenza, E. and Manfredi, G., 2000. Damage indices and damage measures. Progress in Structural Engineering and Materials 2, 50-59.
10. FEMA-356., 2000. Prestandard and commentary for the seismic rehabilitation of buildings, Federal Emergency Management Agency, Washington.
11. Ghobarah, A., Abou-Elfath, H. and Buddha, A., 1999. Response-based damage assessment of structures. Earthquake Engineering and Structural Dynamics 28, 79-104.
12. Yu-chi Sung, Chin-Ku Su, Chuan Wei Wu, I-Chau Tsai, Journal of the Chinese institute of engineers, 29(1), 2006,p. 51-58.