Relation between Dynamic Characteristics and Damage Index of RC-MRFs Using Non-Linear Incremental Dynamic Analyses

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SUMMARY:
Assessment of imposed damage to a structure after earthquakes of different intensities is very important. Since a large number of buildings experience seismic loads in their lifetime, determining the usability of a building or a structure and its resistance to future earthquakes is crucial. This paper considers period elongation and calculates damage by Park-Ang damage index and studies the relation between these two parameters. In this paper, a mathematical relation between damage index and the period elongation is proposed. The advantage of this method is the consideration of period elongation for calculation of the damage. The period varies with changes in structure's stiffness which reflects all the elements stiffness. This helps to understand the correlation between the seismic behavior of a structure with distribution and order of plastic hinges and cracks.

Keywords: Damage index, Period elongation, IDARC, IDA

1. INTRODUCTION

Despite significant progress in loading and design codes of seismic resistant structures and technology improvements in building structures the field of civil engineering is still facing critical challenges. An example of those challenges is the assessment of the state of damage that has been imposed to a structure after earthquakes of different intensities. In order to determine the operability of a structure and its resistance to probable future earthquakes, an assessment of the state of damage of the structure is required. Regarding to needs and necessities, in past years various methods for evaluation of structures seismic vulnerability has been made. A number of available response-based damage indices are discussed and critically evaluated for their applicability in seismic damage evaluation. By using different variety of parameters such as base shear, stiffness, drift, rotation of an element, energy, and in some cases dynamic characteristics of the structure (i.e. mode shapes and natural period of vibration) researchers quantified the state of damage by employing different mathematical functions. In general all the methods can be divided into two major types of quantitative and qualitative methods. In this paper, a quantitative method to assess state of the damage of structures is used. As mentioned before base shear, stiffness, drift, rotation of an element, absorbed energy by elements and also Park-Paulay damage indices are used for evaluating damage status of a structure. In 1985 Park-Ang index based on drift, absorbed energy and yielding force of each element has been introduced. In 1987 this method was improved by using computerized methods (Nateghi Elahi & Motamedi, 1998), (Park & Ang, 1985), (Park *et al*., 1987a) In 1989 a damage index based on shear-drift curve has been provided by McCabe and Hall in which ultimate and throughout earthquake drifts are used to evaluate the damage assessment of a structure (Park *et al*., 1987b). Later in 1995 Reinhorn and Valles introduced Damage Evaluation in Inelastic Response of Structures (McCabe & Hall, 1989) (Reinhorn & Valles, 1995).

In this paper, a relation between period elongation in the first three modes and Park-Ang damage index using Non-Linear Incremental Dynamic Analysis (IDA) is shown. Lastly a mathematical relation between period elongation and damage index is introduced. Since it is hard to compute damage index compare to quantity of period after the damage, using this method is much easier and

straightforward. Furthermore, with the observation of the relation between period elongation and damage index, the softening of the structure can be made sense of. Any change in an element stiffness which could be caused by any crack or plastic hinge along the element affects the stiffness matrix of the structure. This change in stiffness matrix causes period elongation.

Due to importance of dissipating energy and deformation of the elements, Park-Ang damage index suits the best. Furthermore it weighs the elements considering their importance:

$$DI = \frac{\delta_m}{\delta_u} + \frac{\beta}{\delta_u P_y} \int dE_h \tag{1.1}$$

in which DI stands for Damage Index, $\int dE_h$ is the hysteretic energy absorbed by the element during the response history, δ_m is the maximum of experienced deformation, P_y is the yield strength of the element, δ_u is the ultimate deformation for an element, and β is the mode constant that is suggested to be 0.1 for nominal strength deterioration (Park $et\ al.$, 1987b).

Considering that in IDARC program the inelastic behavior of reinforced concrete is confined to plastic zones, the above formula can be changed to the following form (Park $et\ al.$, 1987b):

$$DI = \frac{\theta_m - \theta_r}{\theta_u - \theta_r} + \frac{\beta}{M_y \theta_u} E_h \tag{1.2}$$

in which E_h is consumed energy in the section, M_y is the yield moment, θ_u is the ultimate rotation capacity of the section, θ_r is the recoverable rotation through unloading, and θ_m is the maximum rotation during the loading process. Subsequently with calculation of each element's damage index, story and overall damage indices can be obtained:

$$DI_{Story} = \sum (\lambda_i)_{component} (DI_i)_{component}$$
$$(\lambda_i)_{component} = \left(\frac{E_i}{\sum E_i} \right)_{component} \tag{1.3}$$

$$DI_{overall} = \sum (\lambda_i)_{story} (DI_i)_{story}$$
$$(\lambda_i)_{story} = \left(\frac{E_i}{\sum E_i} \right)_{story} \tag{1.4}$$

where E_i is the total absorbed energy in an element or a story and λ_i is the energy weighting factor. For concrete structures Park-Ang damage index is the perfect one among the other damage indices because it considers variety of rupture states. In Table 1.1 the calibration of Park-Ang damage index is shown (Park $et\ al.$, 1987b).

Table 1.1. Overall Damage Index Description

Descriptive Damage State	Damage Index Quantity	Physical State	State of Building Operability
Collapse	More than 1.0	Partial or total collapse of building	Loss of building
Severe	Between 0.4 – 1.0	Extensive crashing of concrete; disclosure of buckled reinforcement	Not repairable
Moderate	Less than 0.4	Extensive large cracks; spall of concrete in weaker elements	Repairable
Minor	-	Minor cracks; partial crushing of concrete in columns	-
Slight	-	Sporadic occurrence of cracking	-

2. VERIFICATION OF ANALYSIS

It is critical to calibrate the analysis with experimental extracted data. Hence a comparison between analytical and experimental data has been done (Massumi, 2004). The experiment was on a one story and one bay reinforced concrete 2D frame. The loading was consisting of cyclic lateral and constant vertical load (Fig. 2.1.). Base shear ratio, C_b, (ratio of base shear to effective weight) versus total drift percentage of roof is shown in Fig. 2.2. Adjustable parameters of the material modeling are defined so that the best calibration with experimental data is achieved. These parameters are used in material modeling of the frames.
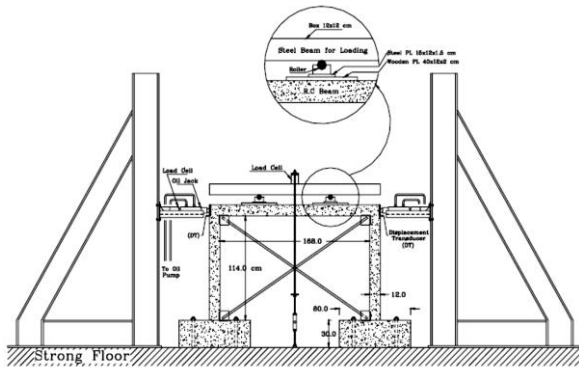
Figure 2. 1. Test setup and loading system
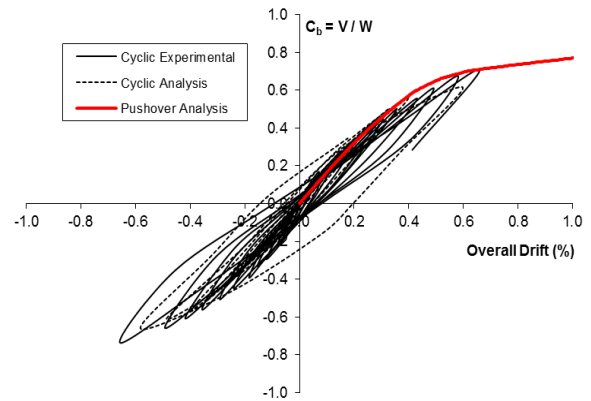
Figure 2. 2. Experimental and analytical data

3. CHOOSING EARTHQUAKE RECORDS FOR ANALYSIS

In this stage, seven ground motion records are elected. All these earthquakes are assumed to be in type B soil and have magnitudes between 6.00 to 7.25 in Richter magnitude scale and their fault mechanisms are strike slip. In Figs. 3.1 through 3.7 time histories of the earthquakes are presented.
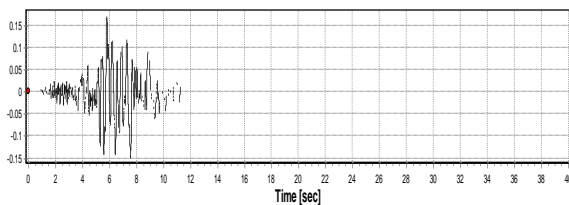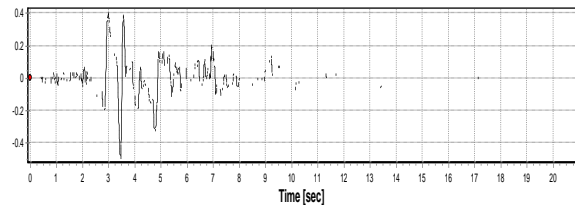

Figure 3.1. Chalfant Valley 1986
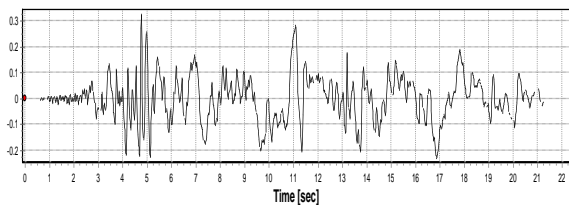

Figure 3.2. Erzincan-Turkey 1992

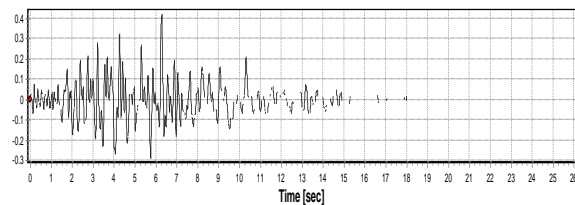

Figure 3.3. Imperial Valley 1972

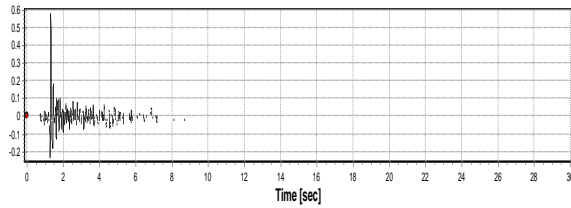**Figure 3.4.** Managua-Nicaragua 1972
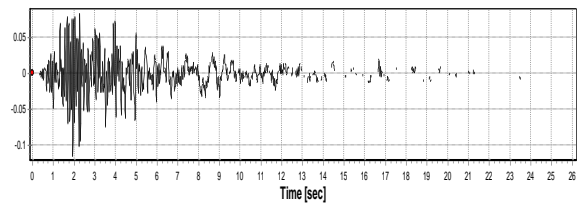
Figure 3. 5. Morgan Hill 1984
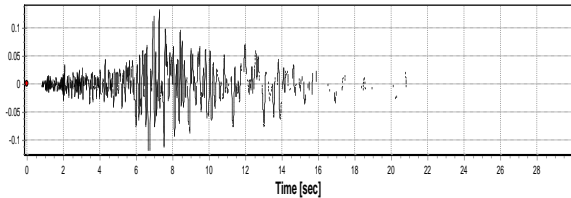

Figure 3.6. Parkfield 1966

Figure 3.7. Superstition Hills1987

Details of earthquake histories which are used in this paper are shown in Table 3.1. Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV) and Peak Ground Displacement (PGD) are listed.

Table 3.1. Characteristics of Selected Earthquakes

Earthquake	Year	Magnitude (Richter)	Fault Mechanism	PGA (g)	PGV (cm/s)	PGD (cm)
Parkfield	1966	6.19	Strike – Slip	0.3588	23.5829	4.0155
Managua-Nicaragua	1972	6.24	Strike – Slip	0.4183	26.6577	6.5459
ImperialValley	1979	6.53	Strike – Slip	0.3188	30.0354	6.6960
Morgan Hill	1984	6.19	Strike – Slip	0.1608	8.1744	1.5218
Chalfant Valley	1986	6.19	Strike – Slip	0.2164	20.0414	6.8207
Superstition Hills	1987	6.22	Strike – Slip	0.1255	12.2078	6.0563
Erzincan-Turkey	1992	6.69	Strike – Slip	0.4518	65.7330	23.0261

4. THEORETICAL MODELS AND ANALYSIS

In this stage, four reinforced concrete moment resistant frames with 2, 4, 6 and 10 stories representing low- and mid-rise frame structures are chosen. These frames are compatible with the third edition of the Iranian Code of Practice for Seismic Resistant Design of Buildings (Building and Housing Research Centre, 2005) and Iranian National Building Codes, Part 9: Design and Construction of Reinforced Concrete Buildings (Ministry of Housing and Urban Development, 2006).

Fig. 4.1 illustrates the elevation views of frames with two, four, six and ten stories, having four bays. All spans of the frames are 4m in width and 3m in height.

IDARC program can identify the location of the plastic hinges considering the capacity of the elements and it implements spread plasticity.

A 2D modeling of the structure is used in this paper. In general, bending, shear and axial deflections are considered in beam and column model. In IDARC Column elements were modeled considering macromodels with inelastic flexural deformations, and elastic shear and axial deformations. Beam elements are modeled using a nonlinear flexural stiffness model with linear elastic shear deformations considered. One of the significant features included in the program, to implement inelastic behavior in

the macromodels, is the distributed plasticity model implemented with the flexibility matrix approach that replaced the generally used hinge model developed for steel frames. The hinge model is not suitable for reinforced concrete elements since the inelastic deformation is distributed along the member rather than being concentrated at critical sections (Park *et al.*, 1987b). However for sake of compatibility with other program and to model plastic characteristics, a concentrated plasticity model was also developed.

In IDARC moments and rotations in two sides of an element, are calculated with Eqn. 4.1:

$$\begin{Bmatrix} M_A' \\ M_B' \end{Bmatrix} = [K'] \begin{Bmatrix} \theta_A' \\ \theta_B' \end{Bmatrix} \tag{4.1}$$

in which M_A', M_B', θ_A' and θ_B' are moments and rotations at the both ends of a beam or column and K is the element's stiffness matrix.
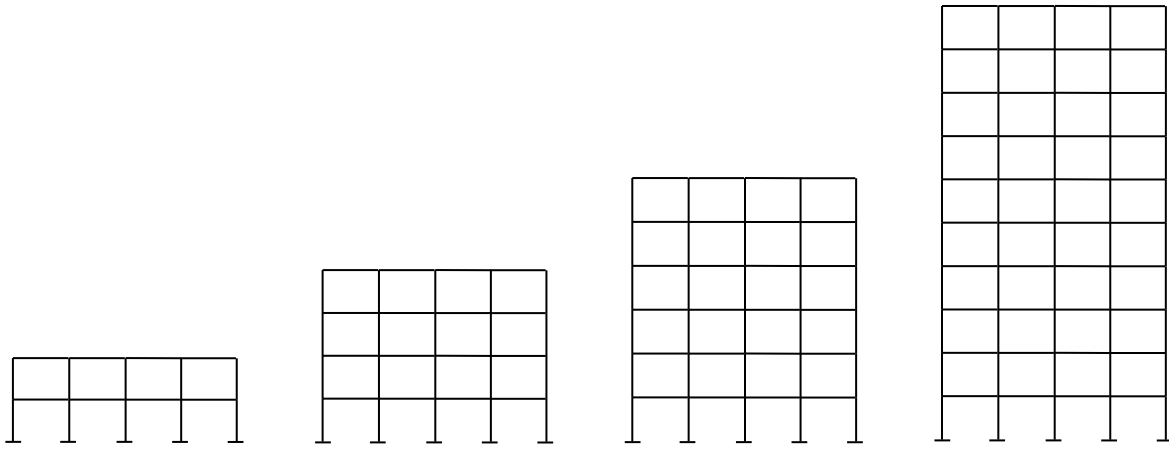

Figure 4. 1. Schematic Representation of the Frames Selected

In analytical modeling the input hysteretic behavior of elements has an essential role in the overall seismic behavior of structures. If incorrect hysteretic is, the results of analytical modeling would not be practical. In this research, a multi-linear hysteretic model is selected for beams and columns, which is compatible with experimental extracted data in the laboratory (Table 4.1) to reflect an actual seismic behavior of structure (Massumi, 1997).

Table 4.1. Hysteretic Parameters Considering Laboratory Results

Stiffness degradation parameter (α)	Ductility-Based strength decay parameter (β_1)	Hysteretic Energy-based strength decay parameter (β_2)	Slip Parameter (γ)
2	0.1	0.1	1

Because this paper is based on IDA method, each earthquake history is implemented through 8 or more steps. In each step the amount of damage index and the first three modal periods are recorded. Afterwards Period elongation is calculated using Eqn. 4.1. (Massumi & Moshtagh, 2010):

$$\text{Period Elongation} = \frac{T_{plastic} - T_{elastic}}{T_{elastic}} \tag{4.2}$$

in which $T_{elastic}$ is elastic period before damage and $T_{plastic}$ is period in the latest damage state.

The results of the analysis are illustrated in Figs. 4.2 to 4.5. The results show that all of the three modes are elongated in each step with the same trend.
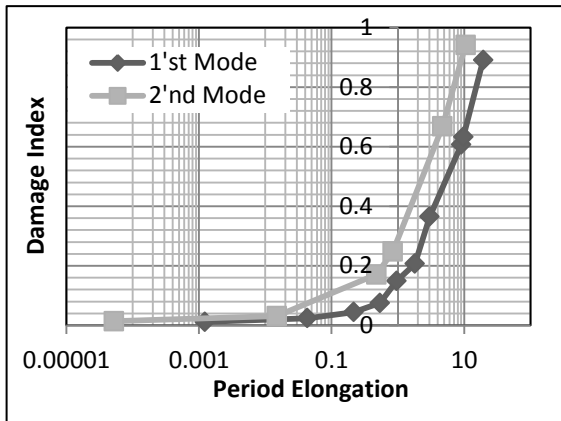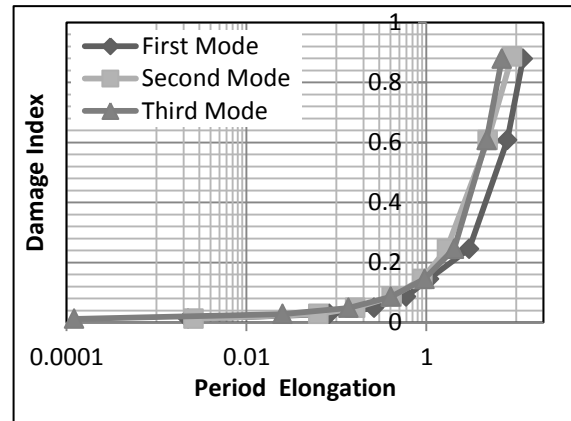
Figure 4.2. Two Story Frame
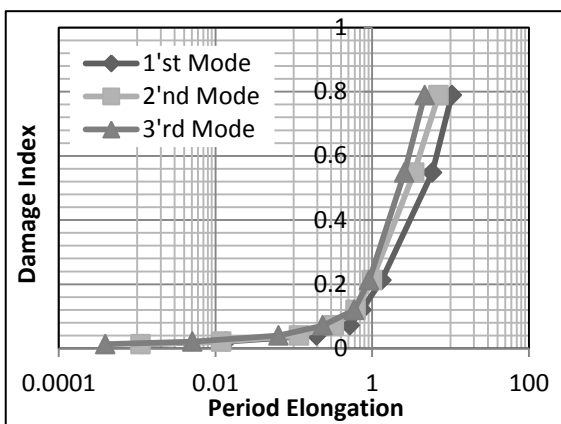

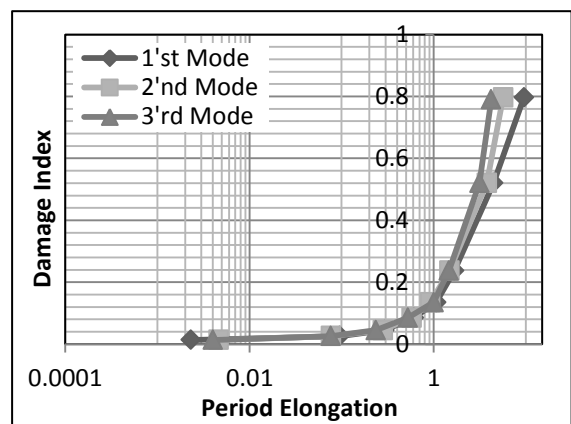**Figure 4.3.** Four Story Frame

Figure 4.4. Six Story Frame

Figure 4.5. Ten Story Frame

The mean results for average period elongation for the first three modes are shown in Figs. 4.6 to 4.8.
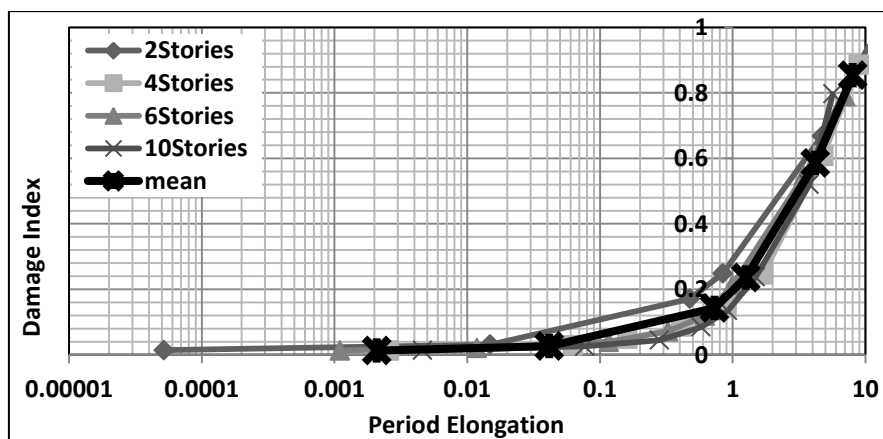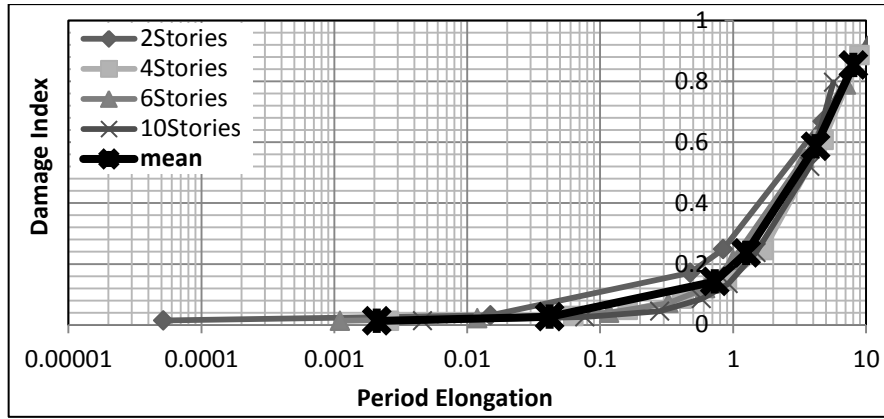

Figure 4.6. First Mode
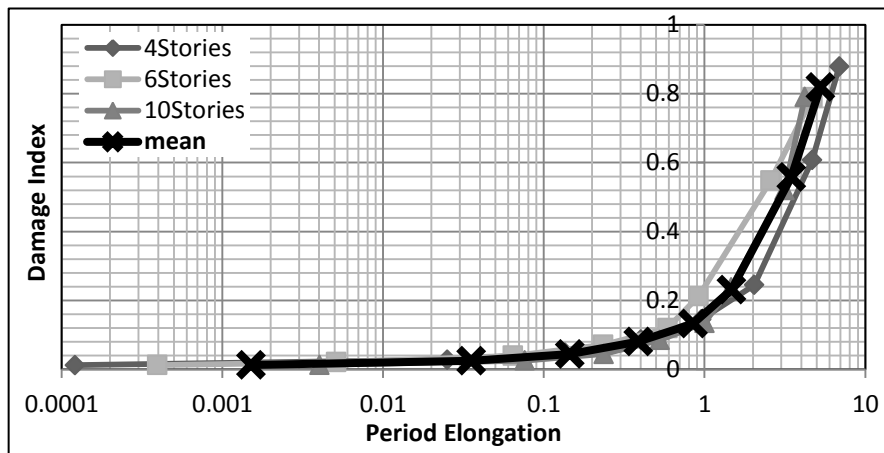
Figure 4.7. Second Mode

Figure 4.8. Third Mode

In the Figs. 4.6 to 4.8 all three modes are changing with a same trend and the period elongated two times of the elastic period around the damage index of 0.15.

5. IDENTIFICATION OF DAMAGE TREND

Above figures show that till the period elongated in the first mode around 4.8 times of the elastic period, the Park-Ang damage index is less than 0.4 which can cause repairable damages like extensive large cracks, spall of concrete in weaker elements. Also in the second mode till the period elongated around 3.2 times of the elastic period, the damages are smaller than 0.4, and for the third mode this rate is around 2.3. With increase of PGA (Peak Ground Acceleration) in the analysis subsequently the amount of the damage increases and damages reach to a stage which is irreparable.

6. PROPOSED DAMAGE PATTERN

In Fig. 6.1 the mean value of period elongation versus damage index of all frames are shown in the first three modes. From set of extracted data a curve can be fitted for each mode and a mathematical relation can be proposed in order to make a logical connection between period elongation and Park-Ang damage index.

In order to decrease the errors, the cubic polynomial is used for first and third modes and quadratic polynomial for the second mode. It should be noted that the curves' equations are reliable when the start point of the curve be considered (0,0).
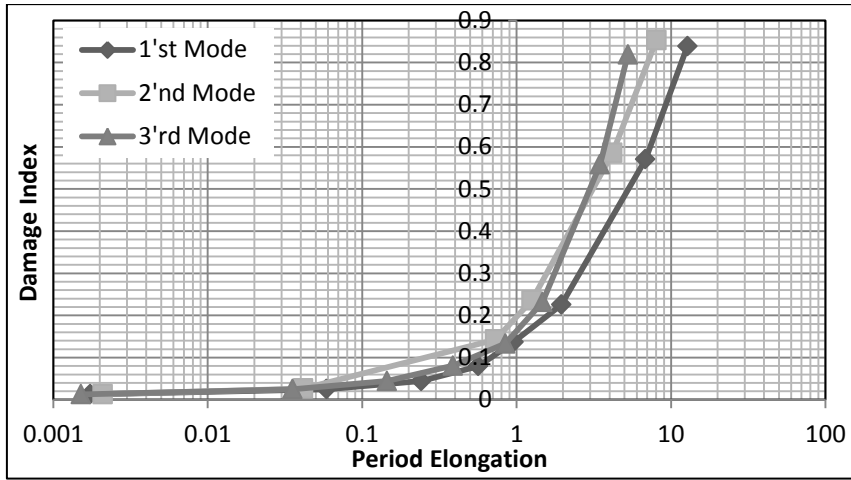
Figure 6.1. Average of all frames' period elongation in first three modes

In the following part proposed formulas and their diagrams for the first three modes are shown.

6.1. First Mode

The cubic for the first mode:

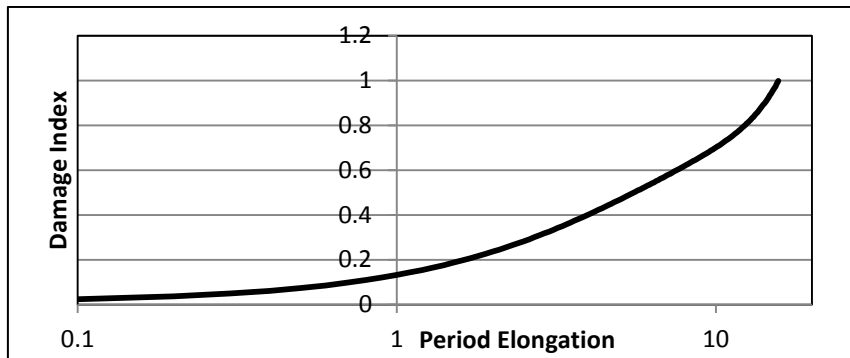$$DI = 0.00032\delta^3 - 0.0092\delta^2 + 0.13\delta + 0.011 \qquad (6.1.1)$$

Figure 6.1.1. First Mode

6.2. Second Mode

The quadratic polynomial is shown for the second mode:

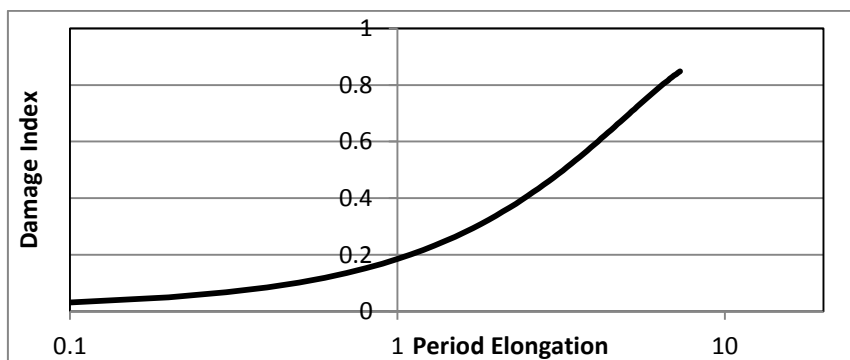$$DI = -0.009\delta^2 + 0.18\delta + 0.014 \qquad (6.2.1)$$

Figure 6.2.1. Second Mode

6.3. Third mode

A cubic formula for the third mode is shown below:

$$DI = -0.0014\delta^3 + 0.01\delta^2 + 0.14\delta + 0.015 \tag{6.3.1}$$
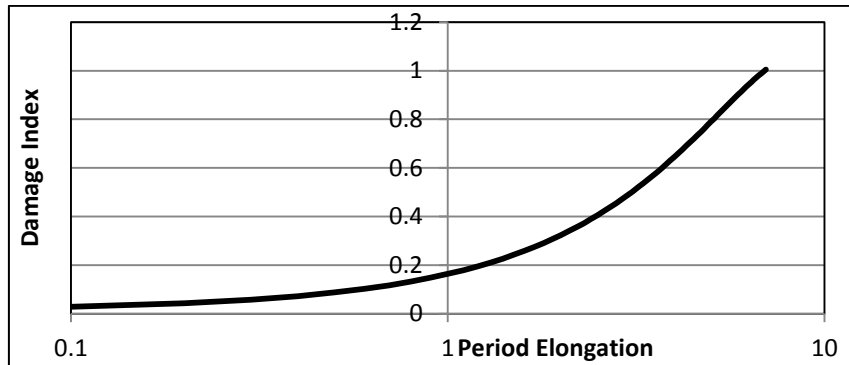

Figure 6.3.1. Third Mode

where DI is the Damage Index and δ is amount of period elongation (Eq. 4.2).

All fitted curves confirm that all three modes follow the same trend. The differences between fitted curves and analytical numbers are less than 10 percent. The error is derived from assumptions made in curve fitting process and the round-off error in analytical phase.

7. CONCLUSION

Seismic behavior of a structure correlates with all of its elements and the configuration of the structure. Computation of the stiffness matrix of structures plays an important role in the definition of its damage state. Because of the hard and time consuming procedure of obtaining the stiffness matrix of a structure, the utilization of period elongation is suggested in this paper in order to overcome this problem. Considering the easy and inexpensive procedure of acquiring a structure's period, utilization of changes in fundamental, second and third period is studied in the accurate assessment of the damage state of structures in this paper. Also dissipating energy and the deformation of an element are important factors to be considered. As a result, Park-Ang damage index has been adopted in this experiment. Moreover, for concrete structures Park-Ang damage index is the perfect one among the other damage indices in that it considers variety of rupture states. All the results unfolded that the all three first modal periods follow the same trend. In first mode when period elongation is around 4.8 Park-Ang damage index is less than 0.4 which can cause repairable damages like extensive large cracks, spall of concrete in weaker elements. Also in the second mode till the period elongated around 3.2 times of the elastic period the damages are smaller than 0.4, and for the third mode this rate is around 2.3. With an increase of PGA in the analysis subsequently the amount of damage increases and damages reach to a stage which is irreparable.

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