A procedure for damage-based seismic design of moment frame structures

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Abstract
A new method for seismic design of structures with the aim of controlling earthquake damage to a prescribed level is presented in this paper. The method is specified for special moment frame buildings with vertical and horizontal regularity. The main idea is determining a design story drift in correlation to a selected damage index value. For this purpose, the Park-Ang damage index is correlated with the maximum story drift. Through performing several nonlinear dynamic analyses of selected moment frame steel structures and regression analysis, it is shown that such a correlation is possible for the studied building type by a simple linear relation. Beginning from a desired value of the damage index, the design story drift is calculated using the developed linear equation and the same buildings are designed using a displacement-base procedure. Results of the nonlinear dynamic analysis of the buildings show that the maximum story damage index under a suit of spectrum compatible ground motions is consistent with the selected initial damage index using the proposed procedure.

Keywords Seismic design · Damage based · Moment frame · Park-Ang · Drift

1 Introduction

The ideal goal of a seismic design procedure is limiting the seismic damage of a building system under the design earthquake motion to a repairable level. In the last 100 years, this goal has been pursued by determining an appropriate lateral strength, based on seismicity and building system, and then containing the maximum story drift. This design approach is known as the force-based procedure of seismic design. It is the basis of the current earthquake resistant design codes of practice. The existing buildings represent the outcome of such a practice and they have shown an improving trend of seismic behavior during years as the design codes gradually evolved. In more recent years, there has been a tendency to give the displacement-checking part of the procedure more importance by exchanging it with the strength-design part in the sequence of seismic design. In other words, the required lateral strength is defined from the maximum lateral displacement amongst all

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stories. Such an approach is the so-called displacement-based seismic design procedure which has evolved to design code proposals DBD09 and DBD12 (Sullivan et al. 2012). The newer idea, followed in this research, is to use this procedure to achieve directly a certain seismic damage level such that the user can decide on the expected intensity of earthquake damage based on the function, value, and anticipated final age of the building among other factors. Quantifying the seismic damage by a damage index was developed much sooner than emerge of the damage-based design idea. A damage index represents the intensity of damage in a structural member, at a story, or overall in a building. It is based on maximum lateral deformations, cyclically-stored plastic energy, or a combination of both. There has been several damage indices proposed by different researchers, such as those of Powell and Allahabadi based on maximum deformation (Powell and Allahabadi 1988), and the combined indices proposed by Krawinkler and Zohrei (1983), Bozorgnia and Bertero (2001), and Park and Ang (1987). The later damage index, modified by Kunnath et al. (1992), has gained more acceptance because of it is versatility and consistency with the results of experiments. As it is the basic tool in the current research, it will be explained in more detail in the next sections.

Developing practical procedures for seismic damage-based design has been tried by a number of researchers in recent years. Panyakapo (2008) presented graphs of the global damage index as a function of demand spectral displacements and accelerations. The developed diagrams were called the demand-capacity curves based on seismic damage. They used the pushover analysis for determining the lateral capacity and the Park-Ang damage index for the mentioned curves as the main output of their study. Jiang et al. (2013) proposed a damage control procedure by presenting a seismic design path in which the maximum drift of an equivalent single-degree-of-freedom (SDOF) system was checked first using the results of analysis of the main building. Kamaris et al. (2014) developed a damage-based seismic design procedure for moment frame steel structures including effects of stiffness and strength degradation in higher cycles of deformation. For the basis of their procedure, they used a damage index developed by the same researchers (Kamaris et al. 2013). Although their method was shown to be successful in predicting the maximum damage under spectrum-compatible ground motions, necessity for use of time consuming nonlinear dynamic analysis put the procedure practically out of hand. Ke and Michael (2018) expressed the performance design method on the basis of damage control in moment frames combined with energy dissipation bay frames (EDBs). The results exhibited that all prototypes of steel moment frames combined with EDBs were capable to reach the damage control behavior based upon the considered drift value.

It has been shown that a good match exists between what is predicted by some of the damage indices and the story drift (Estekanchi and Arjmandi 2007). The correlation relations were presented as conversion curves between the damage index and the maximum story drift. Such a relation makes it possible to devise a practical procedure for damage-based design. Behnamfar and Kazemi (2020) showed that developing a linear relation for conversion of the Park-Ang damage index to the maximum story drift is possible at least for moment frames, simply based on the number of stories. This is the main idea that is developed in the present study. In this paper, a conversion equation is presented for low rise special steel moment frames regular in plan and elevation by which it is possible to calculate the maximum drift based on the design value of the Park-Ang damage index. Regression analysis of the results of several nonlinear dynamic analysis is used for the same purpose. Then a simple damage-based design procedure is presented within the context of the displacement-design approach. Finally, it is shown that the designed buildings follow the maximum damage index laid down for their design.
2 Developing the damage-drift relation

2.1 General

This section presents the development of an equation relating the maximum story drift to the maximum damage index of the story. The Park-Ang damage index as modified by Kunnath et al. (1992), is selected as the basis of calculations. It is an index combined of the effects of maximum drift ratio and the cumulative stored plastic energy. It is shown as follows:

\[ DI = \frac{\theta_m - \theta_y}{\theta_u - \theta_y} + \beta \frac{\int dE}{M_y \theta_u} \]  

(1)

in which \( \theta_m \) is the maximum rotation of the section, \( \theta_y \) is the rotational capacity, \( \theta_u \) is the yield rotation, \( M_y \) is the yield moment, \( \int dE \) is the absorbed hysteretic energy, and \( \beta \) is a calibration factor representing the effect of the dissipated energy on the limit damage. \( \beta \) is equal to 0.4 for steel structures. Then the story damage index \( DI_{\text{Story}} \) is calculated as the weighted average of the member damage indices using Eq. (2):

\[ DI_{\text{Story}} = \sum (\lambda_i)_{\text{component}} (DI_i)_{\text{component}} \]

\[ (\lambda_i)_{\text{component}} = \left( \frac{E_i}{\sum E_i} \right) \]

(2)

in which \( \lambda_i \) is an energy weight factor and \( E_i \) is the absorbed energy by the \( i \)-th member or story.

To prepare the necessary data for developing the conversion equation, use is made of several nonlinear dynamic analyses of selected buildings and regression analysis. The mentioned buildings are introduced in the following.

2.2 The studied buildings

For developing the damage-based methodology, three buildings composed of special steel moment frames are selected. Number of stories is 4, 8, and 12. Their common plan consists of three bays each way with each bay spanning at 5 m. The story height is 3.2 m. The steel material is A36 (American standard) or St-37 (European standard) having a yield stress of 240 MPa. Box and IPE sections are used for the columns and beams, respectively. Design of the buildings is performed based on requirements of ASCE 7–16 (2016) and AISC-LRFD 360–10 (2010). The dead load is calculated to be 5.7 KN/m² for the story floors and 6.4 KN/m² for the highest roof. The equivalent partition load is 1 KN/m². Weight of the perimeter walls is imposed as a uniform linear load. The live load is 2 KN/m² for the stories and 1.5 KN/m² for the roof.

Design seismic load is calculated based on the design spectrum. It is assumed that the buildings are located in a highly seismic area (such as Los Angeles, USA) on a firm soil [Type D, according to ASCE 7–16 (2016)]. The design spectrum for such conditions is shown in Fig. 1 Since the building plans and loadings are completely symmetric, only the accidental torsion is taken into account for design, by displacing the mass center in each direction by 5% of the plan dimension (that is identical in both directions).
As a result, the beam sections emerge to be IPE240 to IPE330 for different bays and stories of the 4-story building, IPE240 to IPE360 for the 8-story building, and IPE270 to IPE400 for the 12-story building. For the same order of buildings, the column box sections are 160×20 to 300×20 mm, 180×20 to 340×20 mm, and 180×20 to 440×30 mm. Common story plan of the buildings and distribution of the member sections along height of the buildings are shown in Fig. 2 and Tables 1, 2 and 3.

Important dynamic characteristics of the buildings are shown in Table 4.
2.3 Nonlinear modelling

The three-dimensional nonlinear response history analysis of the buildings was performed using the OpenSees software. There are several options for nonlinear modelling of the one-dimensional (1D) members, i.e. beams and columns, in this software. In the concentrated plasticity modelling approach, use can be made of nonlinear rotational (M-θ) springs, or of nonlinear fibers. Each fiber is represented by a bilinear stress–strain relation that, in contrast with the M-θ spring, bears no degradation in stiffness and strength along its positive-slope second branch. Use of nonlinear M-θ springs is appropriate enough if large deformations are not expected. Since in special moment frames columns are designed to be stronger than beams, it is not anticipated for columns of the studied buildings to develop large plastic actions. Then it is decided to use the concentrated plasticity approach throughout.

The M-θ path of the nonlinear springs under positive and negative moments is shown in Fig. 3.

Characteristics of the M-θ curve are taken from ASCE 41–17 (2017). Besides, the cyclic deterioration rate follows the rule developed by Rahnama and Krawinkler (1993). According to this rule, the rate of strength and stiffness cyclic deterioration depends on the hysteretic energy of the component dissipated in excursions. It is considered that each component has a reference energy hysteretic dissipation capacity \( E_t \), which is an inherent property of the components and it is independent of the applied loading history. The reference hysteretic energy dissipation capacity is determined by multiplying the reference cumulative rotation capacity, \( \Lambda \), and the effective yield strength, \( M_y \). Values of \( \Lambda \) are calculated according to the relationships developed by Lignos and Krawinkler (2011), as follows, respectively for beam and column sections:
in which, $h$ and $t_w$ are the web depth and thickness, respectively, $b_f$ and $t_f$ are respectively width and thickness of the flange, $D$ and $t$ are the side dimension and thickness of the box section, $N$ and $N_y$ are the existing and yield axial force, $F_y$ is the yield strength, and $c$ is a conversion factor that equals unity when using MPa and mm dimensions.

For the M-θ springs of beams and columns, the expected bending capacity is the fully plastic moment including strain hardening effect, $M_{pe}$, which is equal to $zF_{ye}$ where $z$ is the plastic section modulus and $F_{ye}$ is the expected yield stress. The yield rotation, $\theta_y$, is calculated using the following equations:

$$\Lambda = \frac{E_t}{M_y} = 495 \left( \frac{h}{t_w} \right)^{-1.34} \left( \frac{b_f}{2t_f} \right)^{-0.595} \left( \frac{c^2 \text{unit} \cdot F_y}{355} \right)^{-0.360}$$

(3)

$$\Lambda = \frac{E_t}{M_y} = 3012 \left( \frac{D}{t} \right)^{-2.49} \left( 1 - \frac{N}{N_y} \right)^{3.51} \left( \frac{c \cdot F_y}{380} \right)^{-0.20}$$

(4)
Table 3  Structural member sections for the 12-story building: (a) beams, (b) columns

(a)

| Story | Interior beams in the X direction | Interior beams in the Y direction | Perimeter beams in X and Y directions |
|-------|----------------------------------|----------------------------------|-------------------------------------|
|       | Interior | Edge | Interior | Edge | A-1–2 | A-2–3 | A-3–4 | 1-A-B | 1-B-C | 1-C-D | D-1–2 | D-2–3 | D-3–4 | 4-A-B | 4-B-C | 4-C-D |
| 1     | IPE300   | IPE400 | IPE270 | IPE270 | IPE450 | IPE160 | IPE450 | IPE180 | IPE450 | IPE180 |
| 2     | IPE360   | IPE450 | IPE360 | IPE400 | IPE550 | IPE160 | IPE550 | IPE200 | IPE600 | IPE200 |
| 3     | IPE360   | IPE400 | IPE360 | IPE450 | IPE500 | IPE180 | IPE500 | IPE200 | IPE600 | IPE200 |
| 4     | IPE400   | IPE400 | IPE360 | IPE500 | IPE450 | IPE180 | IPE450 | IPE200 | IPE600 | IPE200 |
| 5     | IPE400   | IPE400 | IPE360 | IPE500 | IPE400 | IPE180 | IPE400 | IPE200 | IPE500 | IPE200 |
| 6     | IPE360   | IPE500 | IPE360 | IPE500 | IPE400 | IPE180 | IPE400 | IPE200 | IPE500 | IPE200 |
| 7     | IPE450   | IPE450 | IPE360 | IPE500 | IPE360 | IPE220 | IPE360 | IPE220 | IPE450 | IPE220 |
| 8     | IPE400   | IPE400 | IPE360 | IPE450 | IPE360 | IPE220 | IPE360 | IPE220 | IPE450 | IPE220 |
| 9     | IPE400   | IPE400 | IPE360 | IPE400 | IPE360 | IPE220 | IPE360 | IPE220 | IPE450 | IPE220 |
| 10    | IPE400   | IPE400 | IPE360 | IPE330 | IPE360 | IPE220 | IPE360 | IPE220 | IPE450 | IPE240 |
| 11    | IPE330   | IPE330 | IPE330 | IPE330 | IPE330 | IPE200 | IPE330 | IPE200 | IPE360 | IPE200 |
| 12    | IPE240   | IPE240 | IPE270 | IPE270 | IPE270 | IPE160 | IPE270 | IPE160 | IPE300 | IPE160 |

(b)

| Story | Column locations |
|-------|------------------|
|       | Interior | Edge | Corner |
| 1     | Box380 × 25 | Box440 × 30 | Box360 × 25 |
| 2     | Box380 × 25 | Box440 × 30 | Box360 × 25 |
| 3     | Box380 × 25 | Box440 × 30 | Box340 × 20 |
| 4     | Box380 × 25 | Box440 × 30 | Box340 × 20 |
| 5     | Box380 × 25 | Box440 × 30 | Box340 × 20 |
| 6     | Box380 × 25 | Box360 × 25 | Box280 × 20 |
| 7     | Box380 × 25 | Box360 × 25 | Box260 × 20 |
| 8     | Box340 × 20 | Box320 × 20 | Box240 × 20 |
| 9     | Box340 × 20 | Box320 × 20 | Box240 × 20 |
| 10    | Box320 × 20 | Box240 × 20 | Box240 × 20 |
| 11    | Box320 × 20 | Box240 × 20 | Box220 × 20 |
| 12    | Box180 × 20 | Box220 × 20 | Box160 × 20 |

Table 4  Dynamic characteristics of the buildings (MPF = Mass Participation Factor)

| No. of stories | First mode |  | Second mode |  | Third mode |  |
|---------------|------------|---|-------------|---|------------|---|
|               | Period, sec | MPF | Period, sec | MPF | Period, sec | MPF |
| 4             | 1.02        | 0.82 | 0.98        | 0.79 | 0.38        | 0.13 |
| 8             | 1.82        | 0.76 | 1.77        | 0.75 | 0.69        | 0.12 |
| 12            | 2.24        | 0.75 | 2.14        | 0.70 | 0.86        | 0.12 |
where

\[ \theta_y = \frac{M_{pe}L(1 + \eta)}{6EI} \]  

(5)

\[ \eta = \frac{12EI}{L^2GA_s} \]  

(6)

in which \( L \) is the free member length, \( E \) is the modulus of elasticity, \( I \) is the moment of inertia about the axis of rotation, \( \eta \) is the adjustment factor for the effect of shear deformation and \( G \) and \( A_s \) are the shear modulus and shear area, respectively. The other parameters needed for defining the M-\( \theta \) curve are \( a \), \( b \), and \( c \) (see Fig. 3). They represent the ductility and residual strength that are functions of compactness of the section and condition of its lateral bracing. They are taken from ASCE 41–17 (2017) as listed in Table 5, along with values of \( \theta_y \).

Other than the beams and columns, the panel zone, encompassing part of the steel columns located at their junctions with beams are under large shear forces subjected to lateral earthquake motions. Therefore, the resulting shear deformations should be appropriately included. Since the studied buildings are composed of special moment frames, it is assumed that the panel zones are perfect equipped with continuity plates and adequate shear strength of the column webs. Panel zone flexibility leads to intensification of the deformations and demands in the elements. For this purpose, elastic elements at the ends of beams through the column depth are used to model the panel zone deformations as shown in Fig. 4.

Characteristics of the panel zone elements are according to a 3D joint in which for interior joints seven nodes (six nodes at the center of the panel zone faces and one node in the center of the panel zone), and six elastic elements are used. Obviously, for exterior joints number of nodes decreases accordingly. To these elastic elements, a section including the continuity plates and the column web is assigned. Also, the distance between face of column and plastic hinge is assumed to be zero to model the direct connection of the beam to
the column. Finally, damping is presumed to be of the Rayleigh type in this study, using a damping ratio of 5% and the periods of the first and third modes that are the important modes according to Table 4. This presumed value will be used to construct the damping matrix for nonlinear dynamic analysis. Value of the damping ratio being 5% is consistent with the fact that the selected ground motions will be scaled to the design spectrum, as discussed in Sect. 2.4. The numerical integration of the equations of motion is performed using the Newmark’s constant acceleration scheme. Moreover, the diaphragms are assumed to be rigid and the P-Delta effects were taken into account.

Adequacy of the nonlinear modelling of this study is verified by repeating the dynamic analysis of the 4-story building of Farahani et al. (2019), being similar to the 4-story building of this study, under the Loma Prieta earthquake. Results of analysis are compared in Fig. 5 for time history of the roof displacement. A very good match is observed between the outputs of the two analyses.

### 2.4 The selected ground motions

According to ASCE 7–16 (2016), at least 11 pairs of ground motions should be selected for nonlinear dynamic analysis of each building. These earthquake motions are extracted from

| Beam section | Modeling parameters | $a$ | $b$ | $c$ | $\theta_y$ | $\theta_a$ | $\Lambda$ |
|--------------|---------------------|-----|-----|-----|-----------|----------|----------|
| IPE180       | 0.094               | 0.115 | 0.600 | 0.010 | 0.125 | 2.052 |
| IPE240       | 0.068               | 0.083 | 0.600 | 0.007 | 0.091 | 1.631 |
| IPE300       | 0.055               | 0.067 | 0.600 | 0.006 | 0.074 | 1.319 |
| IPE330       | 0.050               | 0.061 | 0.600 | 0.005 | 0.066 | 1.252 |
| IPE360       | 0.046               | 0.056 | 0.600 | 0.005 | 0.061 | 1.244 |
| IPE400       | 0.041               | 0.050 | 0.600 | 0.005 | 0.055 | 1.188 |
| IPE450       | 0.037               | 0.045 | 0.600 | 0.004 | 0.050 | 1.155 |
| IPE500       | 0.033               | 0.041 | 0.600 | 0.003 | 0.044 | 1.144 |
| IPE550       | 0.030               | 0.037 | 0.600 | 0.003 | 0.040 | 1.141 |
PEER NGA. They are selected to be consistent with the characteristics of the site for which the buildings were designed, i.e. motions recorded on the soil type D in a highly seismic area in which the design magnitude M is between 6.5 and 7.5 for an epicentral distance of 20–50 km. Although, for this limited number of constraints, a too large number of earthquakes is reported within the search engine. In a two-stage procedure, first an affordable number of ground motions is selected, and then, the selected records are modified to be consistent with the design spectrum, as explained below.

In the first stage, the response spectra of both horizontal components of all of the ground motions retrieved from the database are calculated and the square root of the sum of the squares (SRSS) of the response spectra of the horizontal components of each ground motion is calculated. For each SRSS spectrum, a scale factor is determined. The scale factor is defined as a number that is multiplied by a spectrum and modifies it such that nowhere between 0.2 and 2 T periods, it falls below 90% of the design spectrum (Fig. 1), where T is the fundamental period of the building. For each building, 11 earthquakes with
their scale factors being closer to unity are retrieved. The selected set of ground motions for a building, are used for the nonlinear dynamic analysis of the same building.

In the second stage, a unique scale factor is determined for each set of the selected records by repeating the above procedure this time for the average spectrum of the 11 SRSS spectra. This way, the scale factors appear to be 1.27, 1.25, and 1.24 for the ground motion sets belonging to the 4, 8, and 12-story buildings, respectively.

The selected earthquake ground motions and their average SRSS spectra before and after scaling are shown in Table 6 and Fig. 6, respectively.

### 2.5 Nonlinear dynamic analysis results

The structures introduced in Sect. 2.2 are analyzed under the ground motions of Sect. 2.4. As a result, time history of the damage index is calculated at each story using Eqs. (1) and (2) and its maximum value is determined for each story in each earthquake. For example, distribution of the plastic hinges is shown for an interior frame of the 4-story building under the earthquake having NGA No. 20 at the time instant 13.50 s in Fig. 7. As seen, for instance, there are 34 plastic hinges developed in the first story. The analysis shows that their plastic rotations change between 0.001 and 0.005 rad at this moment. Using Eqs. (1) and (2), damage index of the first story at this time instant is calculated as follows:

\[
(DI_i)_{\text{Component}} = \frac{\theta_{iu} - \theta_{is}}{\theta_{iu} - \theta_{is}} + \beta \int dE \frac{M_{yi} \theta_{iu}}{0.074 - 0.006 + 0.4 \times \frac{4204}{127540 \times 0.074}} = 0.145
\]

\[
(\lambda_i)_{\text{Component}} = \frac{E_i}{\sum E_i} = \frac{4204}{269892} = 0.024
\]

\[
DI_{\text{Story}} = \sum (\lambda_i)_{\text{Component}} (DI_i)_{\text{Component}} = 0.162
\]

Similarly, maximum story drifts are determined. Table 7 shows the maximum story drift ratios. The maximum story damage index in each earthquake is shown only for the 4-story building to save space.

Obviously, since the P-Delta effect is considered in the analysis, part of the observed story drifts and the corresponding damage indices is due to the P-Delta second order responses. Because the buildings are designed according to the current building codes and since they are not extremely tall, the P-delta contribution to the story drifts cannot be large.

### 2.6 Derivation of the conversion equation

The peak values of the maximum drifts and maximum damage indices in each earthquake are picked up for each building. However, the values retrieved from Table 7 are too few to be appropriate for regression analysis because they cover only a small part of the 0–1 range for the damage index. The maximum damage index observed in Table 7 for the 4-story building is about 0.27 that is in the repairable range. It is expected because the studied buildings have been designed based on the current building codes. To cover a wider range, it is decided to utilize the incremental dynamic analysis (IDA), i.e., to calculate the maximum damage index for the same ground motion when scaled
up to have progressively larger first mode spectral accelerations. This procedure is performed by augmenting the same response from its original value each time according to the Hunt & Fill algorithm. The mentioned algorithm uses variable incremental steps to
increase the convergence speed compared to the fixed stepping approach. Performing this way results in much more populated (Damage index, Drift ratio) points as observed in Fig. 8.

A curve fitting process is then followed for the data of Fig. 8 for the 4-story building, and similarly for the other ones, using different curves. This results in Fig. 9 for example for the 4-story building, where linear, quadratic, cubic, and power regressions are tested. Results for the 8 and 12-story buildings are very similar.

As observed, the linear regression is both accurate and simple enough to be picked up from within the regression curves. Values of the $R^2$ parameter are 0.890, 0.912, and 0.875 for the 4, 8, and 12-story buildings, respectively. Then, the following relation emerges as the conversion equation:

$$ U = a \cdot DI + b $$

where $U$ is the design story drift, $DI$ is the desired damage index, and $a$ and $b$ are constants. Table 8 shows values of $a$ and $b$ for the studied buildings.

Suitably based on Table 8, again linear regression equations between other options, are derived for $a$ and $b$ as functions of $n$:
Equation (8) is valid at least for the buildings studied, consisting of low rise special steel moment frames with vertical and in-plan regularity. Coefficients of Eq. (8) may change to some extent if more building cases are analyzed. However, it is anticipated that the variation will not be significant if the extra buildings are regular and the number of stories does not exceed that of the tallest building of this study. Because of inherent direct correlation between the drifts and plastic hinge rotations, it is anticipated that the linear form of the equation would not change when considering buildings comprising of different lateral load bearing systems, but the coefficients would vary between various systems. Moreover, value of the $R$-square parameter would improve for each group of systems when taking more cases into account.

$$a = -0.003 n + 0.081$$

$$b = -0.0006 n + 0.015$$

Equation (8) is valid at least for the buildings studied, consisting of low rise special steel moment frames with vertical and in-plan regularity. Coefficients of Eq. (8) may change to some extent if more building cases are analyzed. However, it is anticipated that the variation will not be significant if the extra buildings are regular and the number of stories does not exceed that of the tallest building of this study. Because of inherent direct correlation between the drifts and plastic hinge rotations, it is anticipated that the linear form of the equation would not change when considering buildings comprising of different lateral load bearing systems, but the coefficients would vary between various systems. Moreover, value of the $R$-square parameter would improve for each group of systems when taking more cases into account.

### 3 Design of the buildings based on damage

The damage-based seismic design procedure proposed in this study consists of selecting a design damage index that is not to be exceeded in any story of the building under design and then calculating its associated maximum drift ratio using Eq. (7). The latter quantity is input to the displacement-based design procedure outlined in DBD 12 (AISC 2010).

As a guide to selecting the desired damage index, Table 9 is presented. In this table, each range of the damage index is described with its corresponding damage extent.

Steps of the procedure are presented for instance for $DI = 0.4$ by following the equations of DBD 12 (AISC 2010), as follows:

**Step 1.** $DI = 0.4 \rightarrow U = 0.040, 0.033$, and $0.026$ for 4, 8, and 12 story buildings, respectively, using Eq. 7.
Table 7 Maximum drift ratios of each story under each earthquake. (a) 4-story, (b) 8-story, (c) 12-story buildings

(a) NGA no | Max drift ratio | Max damage index
|---|---|---|---|---|---|---|---|---|
| Story 1 | Story 2 | Story 3 | Story 4 | Story 1 | Story 2 | Story 3 | Story 4 |
| 20 | 0.015 | 0.018 | 0.017 | 0.015 | 0.1619 | 0.1292 | 0.0836 | 0.0178 |
| 68 | 0.009 | 0.012 | 0.013 | 0.011 | 0.0158 | 0.0143 | 0.0213 | 0.0005 |
| 169 | 0.017 | 0.018 | 0.014 | 0.013 | 0.2195 | 0.1454 | 0.0736 | 0.0064 |
| 778 | 0.012 | 0.014 | 0.014 | 0.015 | 0.0957 | 0.0774 | 0.0769 | 0.0234 |
| 826 | 0.009 | 0.013 | 0.013 | 0.010 | 0.0376 | 0.0597 | 0.0371 | 0.0003 |
| 900 | 0.011 | 0.014 | 0.013 | 0.013 | 0.0823 | 0.0881 | 0.0549 | 0.0060 |
| 987 | 0.013 | 0.017 | 0.015 | 0.011 | 0.1228 | 0.1142 | 0.0550 | 0.0005 |
| 1107 | 0.011 | 0.015 | 0.015 | 0.014 | 0.0980 | 0.0808 | 0.0589 | 0.0117 |
| 4853 | 0.012 | 0.014 | 0.012 | 0.013 | 0.1316 | 0.1095 | 0.0523 | 0.0087 |
| 5814 | 0.022 | 0.027 | 0.027 | 0.024 | 0.3059 | 0.2687 | 0.2090 | 0.0634 |
| 6923 | 0.010 | 0.014 | 0.014 | 0.018 | 0.0675 | 0.0408 | 0.0671 | 0.0428 |

(b) NGA no | Max drift ratio
|---|---|---|---|---|---|---|---|---|---|
| Story 1 | Story 2 | Story 3 | Story 4 | Story 5 | Story 6 | Story 7 | Story 8 |
| 20 | 0.0099 | 0.0165 | 0.0214 | 0.0238 | 0.0233 | 0.0189 | 0.0124 | 0.0119 |
| 68 | 0.0033 | 0.0060 | 0.0068 | 0.0064 | 0.0063 | 0.0068 | 0.0067 | 0.0076 |
| 169 | 0.0083 | 0.0136 | 0.0162 | 0.0156 | 0.0157 | 0.0144 | 0.0160 | 0.0156 |
| 778 | 0.0136 | 0.0181 | 0.0188 | 0.0178 | 0.0197 | 0.0197 | 0.0159 | 0.0134 |
| 900 | 0.0047 | 0.0098 | 0.0113 | 0.0148 | 0.0205 | 0.0228 | 0.0187 | 0.0166 |
| 987 | 0.0037 | 0.0070 | 0.0096 | 0.0110 | 0.0112 | 0.0125 | 0.0119 | 0.0113 |
| 1110 | 0.0056 | 0.0103 | 0.0124 | 0.0128 | 0.0121 | 0.0126 | 0.0116 | 0.0138 |
| 4849 | 0.0096 | 0.0156 | 0.0195 | 0.0216 | 0.0212 | 0.0238 | 0.0198 | 0.0147 |
| 5814 | 0.0087 | 0.0141 | 0.0161 | 0.0163 | 0.0157 | 0.0201 | 0.0228 | 0.0235 |
| 5988 | 0.0055 | 0.0113 | 0.0150 | 0.0186 | 0.0224 | 0.0228 | 0.0176 | 0.0145 |
| 6923 | 0.0106 | 0.0159 | 0.0183 | 0.0182 | 0.0158 | 0.0141 | 0.0177 | 0.0191 |

(c) NGA no | Max drift ratio
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Story 1 | Story 2 | Story 3 | Story 4 | Story 5 | Story 6 | Story 7 | Story 8 | Story 9 | Story 10 | Story 11 | Story 12 |
| 20 | 0.0036 | 0.0081 | 0.0107 | 0.0134 | 0.0152 | 0.0164 | 0.0166 | 0.0164 | 0.0138 | 0.0117 | 0.0101 | 0.0091 |
| 68 | 0.0027 | 0.0054 | 0.0061 | 0.0061 | 0.0059 | 0.0061 | 0.0060 | 0.0061 | 0.0060 | 0.0063 | 0.0065 | 0.0071 |
| 169 | 0.0038 | 0.0081 | 0.0097 | 0.0098 | 0.0103 | 0.0115 | 0.0129 | 0.0147 | 0.0140 | 0.0105 | 0.0128 | 0.0118 |
| 777 | 0.0106 | 0.0146 | 0.0152 | 0.0151 | 0.0165 | 0.0189 | 0.0196 | 0.0181 | 0.0165 | 0.0179 | 0.0158 | 0.0136 |
| 900 | 0.0034 | 0.0075 | 0.0094 | 0.0103 | 0.0106 | 0.0116 | 0.0137 | 0.0165 | 0.0166 | 0.0135 | 0.0117 | 0.0112 |
| 987 | 0.0027 | 0.0052 | 0.0057 | 0.0056 | 0.0055 | 0.0061 | 0.0072 | 0.0093 | 0.0108 | 0.0114 | 0.0108 | 0.0099 |
| 1110 | 0.0035 | 0.0081 | 0.0104 | 0.0112 | 0.0106 | 0.0093 | 0.0085 | 0.0088 | 0.0123 | 0.0148 | 0.0147 | 0.0132 |
| 4849 | 0.0043 | 0.0096 | 0.0127 | 0.0145 | 0.0156 | 0.0167 | 0.0171 | 0.0173 | 0.0195 | 0.0181 | 0.0141 | 0.0108 |
| 5814 | 0.0058 | 0.0112 | 0.0137 | 0.0144 | 0.0136 | 0.0120 | 0.0109 | 0.0104 | 0.0142 | 0.0182 | 0.0188 | 0.0178 |
| 5988 | 0.0033 | 0.0070 | 0.0084 | 0.0087 | 0.0083 | 0.0088 | 0.0100 | 0.0122 | 0.0128 | 0.0109 | 0.0089 | 0.0094 |
Table 7 (continued)

(c)

| NGA no | Max drift ratio |
|--------|-----------------|
|        | Story 1 | Story 2 | Story 3 | Story 4 | Story 5 | Story 6 | Story 7 | Story 8 | Story 9 | Story 10 | Story 11 | Story 12 |
| 6923   | 0.0037  | 0.0078  | 0.0091  | 0.0092  | 0.0084  | 0.0083  | 0.0109  | 0.0127  | 0.0133  | 0.0084   | 0.0083   | 0.0132   |

Fig. 8 Maximum damage index vs maximum drift for the 4-story building; IDA analysis with 11 earthquakes

Fig. 9 Regression analysis for the 4-story building

Table 8 Values of the regression parameters, Eq. 7

| No. of stories, n | a     | b     |
|-------------------|-------|-------|
| 4                 | 0.071 | 0.012 |
| 8                 | 0.053 | 0.011 |
| 12                | 0.047 | 0.007 |

Step 2. Determining the lateral displacement of story $i$, $\Delta_i$, from Eq. (9):
\[ \Delta_i = \omega_\theta \Delta_{i,LS} - \theta_{N,i} \cdot x_{CP_i - CM_i} \] (9)

in which \( \omega_\theta \) is the displacement reduction factor due to higher modes effect, \( \Delta_{i,LS} \) is lateral displacement of the center of mass of the \( i \)-th story at the selected limit state, \( \theta_{N,i} \) is the torsion angle of the \( i \)-th story when its displacement is maximum, and \( x_{CP_i - CM_i} \) is distance between the critical point and the mass center of the \( i \)-th story. \( \omega_\theta \) equals unity for moment frame buildings up to 6 stories and equals 0.85 when number of stories is 16 and larger. It is calculated by linear interpolation in between. \( \Delta_{i,LS} \) is calculated using Eq. (10):

\[ \Delta_{i,LS} = \left( \frac{4H_n - h_i}{4H_n - h_1} \right) \cdot \theta_c h_i \] (10)

where \( h_i \) is height of the \( i \)-th floor from the base level, \( H_n \) is the total height of the building from the same level, \( \theta_c \) is the maximum drift ratio calculated in step 1, and \( h_1 \) is height of the first story.

**Step 3** Calculating the equivalent SDF system characteristics, including the target displacement \( \Delta_d \), the effective mass \( m_e \), equivalent height \( h_e \), ductility factor \( \mu \), and damping ratio \( \zeta \), as follows:

\[ \Delta_d = \frac{\sum_{i=1}^{n} m_i \Delta_i^2}{\sum_{i=1}^{n} m_i \Delta_i} \] (11)

\[ m_e = \frac{\sum_{i=1}^{n} m_i \Delta_i}{\Delta_d} \] (12)

\[ h_e = \frac{\sum_{i=1}^{n} m_i \Delta_i h_i}{\sum_{i=1}^{n} m_i \Delta_i} \] (13)

\[ \mu = \left( \Delta_d \over \Delta_y \right) \geq 1 \] (14)

\[ \zeta = 0.05 + 0.577 \left( \frac{\mu - 1}{\mu} \right) \] (15)

in which \( m_i \) is mass of the \( i \)-th story, \( n \) is number of stories, and \( \Delta_y \) is the effective yield displacement calculated as:

| Park-Ang DI | Damage intensity | Performance level |
|-------------|------------------|-------------------|
| 0.0–0.2     | Minor            | Operational       |
| 0.2–0.5     | Moderate         | Life safety       |
| 0.5–1       | Severe           | Near collapse     |
| > 1         | Collapse         | Total collapse    |

Table 9: The damage intensity for each range of the damage index (Komeili et al. 2012)
\[ \Delta_y = H_e \cdot \theta_y \]  \hfill (16)

where \( \theta_y \) is the effective yield drift as:

\[ \theta_y = 0.65 \varepsilon_y L_b / h_b \]  \hfill (17)

in which \( \varepsilon_y \) is the yield strain of steel, \( L_b \) is the average span length, and \( h_b \) is the average depth of the beams of the story.

Step 4. Determining the elastic and inelastic design displacement spectra. The elastic design displacement spectrum, \( S_d \), is calculated from the following well known equation:

\[ S_d(T) = \frac{T^2}{4\pi^2} S_a(T) \cdot g \]  \hfill (18)

where \( T \) is the natural period, \( S_a \) is the design acceleration spectral value, and \( g \) is acceleration of gravity. \( S_d \) is calculated using Fig. 1. It is shown in Fig. 10.

The inelastic design displacement spectrum is determined by multiplying \( S_d \) by the factor \( R_\xi \). This factor is a function of the equivalent damping ratio \( \xi \) and fault distance as follows:

\[ R_\xi = \begin{cases} 0.07 \left( 0.02 + \frac{1}{\xi} \right)^{0.5} & : \text{at far distances from fault} \\ 0.07 \left( 0.02 + \xi \right)^{0.25} & : \text{at near distances to fault} \end{cases} \]  \hfill (19)

Step 5. Calculating the effective period and stiffness. The effective period \( T_e \) is extracted from the inelastic spectrum at the target displacement \( \Delta_d \). Then the effective stiffness \( k_e \) is determined as follows:

\[ k_e = \frac{4\pi^2 m_e}{T_e^2} \]  \hfill (20)

Step 6. Determining the base shear, \( V_{\text{Base}} \), as:

\[ V_{\text{Base}} = k_e \Delta_d + V_{P-\Delta} < 2.5R_\xi \cdot PGA \cdot m_e + V_{P-\Delta} \]  \hfill (21)

where \( V_{P-\Delta} \) is the base shear due to the P - \( \Delta \) effect. It is calculated as follows:

**Fig. 10** The design displacement spectrum

\[ \begin{array}{c|c|c|c|c|c|c} \text{Period (sec)} & 0 & 2 & 4 & 6 & 8 & 10 \\ \hline \text{Sd (m)} & 0.2 & 0.4 & 0.6 & 0.8 & 1.0 & 1.2 & 1.4 \end{array} \]
Step 7. Calculating the design lateral forces, $F_i$, as:

$$V_{P-\Delta} = C \cdot \frac{P \Delta_d}{H_e} = C \cdot \frac{m_e g \Delta_d}{H_e} \quad (\Delta_y > \Delta_d)$$  

(22a)

$$V_{P-\Delta} = C \cdot \frac{P \Delta_y}{H_e} = C \cdot \frac{m_e g \Delta_y}{H_e} \quad (\Delta_y < \Delta_d)$$  

(22b)

Then, the structural system is designed for the member forces due to application of the above lateral forces. Use of the above procedure results in the following table that lists values of the mentioned parameters for the buildings of this study (Table 10).

Design of the buildings based on the above procedure that began with setting DI at 0.4, results in the fundamental periods of the new buildings to be 1.32, 2.35, and 2.76 s for the 4, 8, and 12-story buildings, respectively. They can be compared with the forced-based design period that, as mentioned in Sect. 2.2, are 1.02, 1.82, and 2.24 s. It shows that taking DI to be equal to 0.4 results in larger periods for the buildings designed according to the proposed damage-based design procedure, compared their forced-based designed counterparts. Clearly it is a clear clue to the fact that the force-based procedure is inherently based on a smaller extent of damage at the occurrence of the design earthquake. Sections of the beams appear to be IPE240 to IPE330, IPE240 to IPE360, and IPE240 to IPE450, for the same order of buildings. The column box sections are

| Table 10 | Values of the design parameters of the buildings |
|----------|-----------------------------------------------|
| Parameter | No. of stories |  |  |  |
|  | 4  | 8  | 12 |
| $\omega_0$ | 1.000 | 0.970 | 0.910 |
| $\Delta_d (m)$ | 0.299 | 0.432 | 0.524 |
| $m_e (ton)$ | 580.374 | 1118.917 | 1477.553 |
| $H_e (m)$ | 9.251 | 17.532 | 25.839 |
| $h_b (m)$ | 0.285 | 0.299 | 0.348 |
| $\theta_y (rad)$ | 0.014 | 0.013 | 0.011 |
| $\Delta_y (m)$ | 0.127 | 0.229 | 0.289 |
| $\mu$ | 2.361 | 1.888 | 1.811 |
| $\zeta$ | 0.156 | 0.136 | 0.132 |
| $R_c$ | 0.631 | 0.669 | 0.678 |
| $T_e (sec)$ | 3.060 | 4.950 | 5.620 |
| $k_e (kN/m)$ | 2446.947 | 1802.799 | 1846.844 |
| $k_e \cdot \Delta_d (kN)$ | 731.209 | 777.988 | 967.939 |
| $V_{P-\Delta} (kN)$ | 77.911 | 143.093 | 162.383 |
| $V_{Base}(kN)$ | 809.119 | 921.081 | 1130.322 |
180 × 20 to 300 × 20, 180 × 20 to 340 × 20, and 220 × 20 to 440 × 30, respectively. The structural member sections are given in detail in Tables 11, 12 and 13.

4 Nonlinear dynamic analysis of the damage-based designed buildings

Nonlinear dynamic analysis of the buildings designed with the proposed damage-based procedure is performed using the ground motions introduced in Sect. 2.4. The maximum damage index in each story under each earthquake is reported in Table 14. For the 4-story building, for example, values of the maximum drift ratios of stories are also given.

As observed, in the most critical cases, the damage index is just close to the DI taken for design of the mentioned building. Of course, after designing for a selected design damage index, according to Fig. 9 it should be expected that the emerged maximum damage index to be somewhat larger than the design value under some earthquakes and to be smaller than that for some others. This is exactly what is seen in Table 14 in the row showing the maximum value between the earthquakes. This fact becomes more obvious when one notes that the nonlinear dynamic analysis has been done here under a suit of ground motions with their scale factors being closest to unity, as mentioned in Sect. 2.4. Then it can be said that the procedure of matching the design DI with a maximum drift and the displacement-based design afterwards, has been successful in limiting the maximum DI under the spectrum-compatible ground motions to the desired value. As seen in Table 14, for medium and repairable damage, DI should be contained within 0.2–0.4. To investigate further, the above procedure is repeated this time for DI = 0.2. In this case, the maximum drift ratio becomes 0.028, 0.018 and 0.014 for 4, 8, and 12-story buildings, respectively. Then heavier

| Story | Interior beams in the X direction | Interior beams in the Y direction | Perimeter beams in the X and Y directions | A-1–2 | A-2–3 | A-3–4 | 1-A-B | 1-B-C | 1-C-D |
|-------|----------------------------------|----------------------------------|------------------------------------------|-------|-------|-------|-------|-------|-------|
| 1     | IPE330                           | IPE300                           | IPE330 IPE180 IPE330 IPE180 IPE330 IPE180 IPE330 IPE180 |
| 2     | IPE330                           | IPE300                           | IPE330 IPE180 IPE330 IPE180 IPE360 IPE180 |
| 3     | IPE300                           | IPE300                           | IPE330 IPE180 IPE330 IPE180 IPE330 IPE180 |
| 4     | IPE240                           | IPE240                           | IPE240 IPE180 IPE240 IPE180 IPE240 IPE180 |

(b)

| Story | Column locations |
|-------|------------------|
|       | Interior | Edge | Corner |
| 1     | Box300 × 20     | Box240 × 20 | Box220 × 20 |
| 2     | Box260 × 20     | Box240 × 20 | Box220 × 20 |
| 3     | Box260 × 20     | Box220 × 20 | Box220 × 20 |
| 4     | Box180 × 20     | Box180 × 20 | Box140 × 20 |
sections are designed for the members. The design sections for the beams are IPE270 to IPE360, IPE300 to IPE400, and IPE300 to IPE500 for the mentioned buildings. The column box sections are 180 × 20 to 300 × 20, 180 × 20 to 340 × 20, and 220 × 20 to 440 × 20 for the same buildings. The fundamental periods are 0.93, 1.64, and 2.02 s, respectively. The maximum damage index in each case is shown in Table 15.

Again, as observed, the proposed damage-based procedure has been successful in limiting the resulting maximum damage index in the most critical case for each building to the presumed value. It is seen also that the maximum story drifts, for example for the 4-story building, are also decreased.

For comparison, in the following three tables, the fundamental periods, base shear, and total member weights of the damage-based designed buildings are compared with their force-based counterparts (Tables 16, 17, and 18).

As observed, by halving DI, the structure’s weight increases only up to 10% compared with the force-based design that may justify such a strictness if architectural and mechanical limitations allow. Moreover, the base shear increases up to 12% and the period reduces up to 10% for DI = 0.2 compared with the force-based quantities. By

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**Table 12** Structural member sections according to the proposed damage-based design procedure for the 8-story building: (a) beams, (b) columns

(a)

| Story | Interior beams in the X direction | Interior beams in the Y direction | Perimeter beams in the X and Y directions |
|-------|----------------------------------|----------------------------------|------------------------------------------|
|       | Interior | Edge | Interior | Edge | A-1-2 | A-2-3 | A-3-4 | 1-A-B | 1-B-C | 1-C-D |
|-------|----------|------|----------|------|-------|-------|-------|-------|-------|-------|
| 1     | IPE300   |      | IPE270   | IPE300| IPE330| IPE160| IPE330| IPE160| IPE360| IPE160|
| 2     | IPE330   |      | IPE300   | IPE360| IPE360| IPE180| IPE360| IPE180| IPE400| IPE180|
| 3     | IPE330   |      | IPE330   | IPE360| IPE330| IPE180| IPE330| IPE180| IPE360| IPE180|
| 4     | IPE330   |      | IPE330   | IPE360| IPE330| IPE180| IPE330| IPE180| IPE360| IPE180|
| 5     | IPE300   |      | IPE270   | IPE300| IPE300| IPE180| IPE300| IPE180| IPE330| IPE180|
| 6     | IPE270   |      | IPE270   | IPE270| IPE270| IPE180| IPE270| IPE180| IPE300| IPE180|
| 7     | IPE240   |      | IPE240   | IPE240| IPE240| IPE180| IPE240| IPE180| IPE240| IPE180|

(b)

| Story | Column locations |
|-------|------------------|
|       | Interior | Edge | Corner |
|-------|----------|------|--------|
| 1     | Box340×20 | Box320×20 | Box240×20 |
| 2     | Box340×20 | Box320×20 | Box240×20 |
| 3     | Box320×20 | Box280×20 | Box220×20 |
| 4     | Box320×20 | Box280×20 | Box200×20 |
| 5     | Box320×20 | Box260×20 | Box200×20 |
| 6     | Box260×20 | Box240×20 | Box200×20 |
| 7     | Box260×20 | Box240×20 | Box180×20 |
| 8     | Box180×20 | Box140×20 | Box140×20 |
observation, it seems that the force-based procedure is consistent approximately with DI = 0.3. Then, the proposed design procedure is flexible enough to make and compare different designs regarding the acceptable damage for the final decision. It should be noted that the results of this section are only to show the trend as establishing certain values needs much more analysis with various cases.

### Table 13 Structural member sections according to the proposed damage-based design procedure for the 12-story building: (a) beams, (b) columns

(a)

| Story | Interior beams in the X direction | Interior beams in the Y direction | Perimeter beams in the X and Y directions |
|-------|----------------------------------|----------------------------------|------------------------------------------|
|       | Interior Edge                    | Interior Edge                    | A-1–2 D-1–2 A-2–3 D-2–3 A-3–4 D-3–4 1-A-B 4-A-B 1-B-C 4-B-C 1-C-D |
| 1     | IPE270 IPE270                    | IPE240 IPE400                    | IPE330 IPE330 IPE400 IPE160 IPE160 |
| 2     | IPE300 IPE400                    | IPE330 IPE360                    | IPE400 IPE160 IPE360 IPE160 IPE330 |
| 3     | IPE330 IPE360                    | IPE330 IPE400                    | IPE400 IPE160 IPE400 IPE160 IPE300 |
| 4     | IPE360 IPE360                    | IPE300 IPE450                    | IPE360 IPE160 IPE360 IPE160 IPE300 |
| 5     | IPE360 IPE360                    | IPE300 IPE450                    | IPE330 IPE160 IPE360 IPE160 IPE300 |
| 6     | IPE330 IPE360                    | IPE300 IPE450                    | IPE300 IPE160 IPE300 IPE180 IPE400 |
| 7     | IPE360 IPE360                    | IPE300 IPE400                    | IPE300 IPE160 IPE300 IPE180 IPE300 |
| 8     | IPE330 IPE360                    | IPE300 IPE450                    | IPE330 IPE160 IPE300 IPE180 IPE300 |
| 9     | IPE330 IPE360                    | IPE300 IPE450                    | IPE330 IPE160 IPE300 IPE180 IPE300 |
| 10    | IPE360 IPE360                    | IPE300 IPE330                    | IPE360 IPE160 IPE300 IPE180 IPE300 |
| 11    | IPE270 IPE270                    | IPE270 IPE270                    | IPE360 IPE160 IPE300 IPE180 IPE300 |
| 12    | IPE220 IPE220                    | IPE220 IPE220                    | IPE240 IPE140 IPE240 IPE140 IPE270 |

(b)

| Story | Column locations |
|-------|------------------|
|       | Interior Edge Corner |
| 1     | Box380 x 25 Box440 x 30 Box340 x 20 |
| 2     | Box380 x 25 Box440 x 30 Box340 x 20 |
| 3     | Box380 x 25 Box380 x 25 Box320 x 20 |
| 4     | Box380 x 25 Box380 x 25 Box320 x 20 |
| 5     | Box380 x 25 Box380 x 25 Box320 x 20 |
| 6     | Box380 x 25 Box380 x 25 Box260 x 20 |
| 7     | Box380 x 25 Box360 x 25 Box240 x 20 |
| 8     | Box340 x 20 Box320 x 20 Box240 x 20 |
| 9     | Box340 x 20 Box320 x 20 Box240 x 20 |
| 10    | Box320 x 20 Box240 x 20 Box240 x 20 |
| 11    | Box320 x 20 Box240 x 20 Box220 x 20 |
| 12    | Box180 x 20 Box220 x 20 Box160 x 20 |
Table 14 Maximum damage index in each story for the damage-based designed buildings with DI = 0.4. (a) 4-story, (b) 8-story, and (c) 12-story building

(a)  

| NGA no. | Story Damage Index | Story Drift Ratio |
|---------|--------------------|------------------|
|         | Story 1 | Story 2 | Story 3 | Story 4 | Story 1 | Story 2 | Story 3 | Story 4 |
| 20      | 0.300   | 0.124   | 0.075   | 0.028   | 0.0225  | 0.0218  | 0.0185  | 0.0158  |
| 68      | 0.073   | 0.060   | 0.028   | 0.012   | 0.0105  | 0.0133  | 0.0121  | 0.0112  |
| 169     | 0.277   | 0.198   | 0.129   | 0.046   | 0.0197  | 0.0224  | 0.0217  | 0.0185  |
| 778     | 0.095   | 0.097   | 0.061   | 0.032   | 0.0120  | 0.0185  | 0.0165  | 0.0152  |
| 826     | 0.109   | 0.111   | 0.078   | 0.018   | 0.0136  | 0.0177  | 0.0160  | 0.0139  |
| 900     | 0.443   | 0.271   | 0.192   | 0.070   | 0.0315  | 0.0338  | 0.0285  | 0.0236  |
| 987     | 0.036   | 0.039   | 0.007   | 0.005   | 0.0090  | 0.0140  | 0.0113  | 0.0106  |
| 1107    | 0.111   | 0.052   | 0.032   | 0.042   | 0.0121  | 0.0143  | 0.0131  | 0.0154  |
| 4853    | 0.209   | 0.161   | 0.087   | 0.027   | 0.0161  | 0.0202  | 0.0186  | 0.0156  |
| 5814    | 0.254   | 0.222   | 0.231   | 0.139   | 0.0198  | 0.0251  | 0.0287  | 0.0301  |
| 6923    | 0.043   | 0.064   | 0.075   | 0.062   | 0.0091  | 0.0140  | 0.0181  | 0.0183  |
| Max     | 0.443   | 0.271   | 0.231   | 0.139   | 0.0315  | 0.0338  | 0.0287  | 0.0301  |
| Ave of Max | 0.177   | 0.127   | 0.090   | 0.044   | 0.0160  | 0.0196  | 0.0185  | 0.0171  |

(b)  

| NGA no. | Story Damage Index | Story Drift Ratio |
|---------|--------------------|------------------|
|         | Story 1 | Story 2 | Story 3 | Story 4 | Story 5 | Story 6 | Story 7 | Story 8 |
| 20      | 0.014   | 0.054   | 0.155   | 0.198   | 0.169   | 0.083   | 0.027   | 0.024   |
| 68      | 0.032   | 0.028   | 0.030   | 0.014   | 0.010   | 0.019   | 0.014   | 0.024   |
| 169     | 0.009   | 0.125   | 0.268   | 0.389   | 0.394   | 0.258   | 0.131   | 0.051   |
| 778     | 0.330   | 0.314   | 0.387   | 0.358   | 0.307   | 0.234   | 0.184   | 0.100   |
| 900     | 0.014   | 0.037   | 0.064   | 0.072   | 0.150   | 0.163   | 0.096   | 0.029   |
| 987     | 0.007   | 0.004   | 0.025   | 0.033   | 0.037   | 0.071   | 0.043   | 0.007   |
| 1110    | 0.030   | 0.113   | 0.160   | 0.115   | 0.071   | 0.159   | 0.135   | 0.046   |
| 4849    | 0.014   | 0.063   | 0.149   | 0.172   | 0.155   | 0.144   | 0.077   | 0.019   |
| 5814    | 0.045   | 0.122   | 0.159   | 0.133   | 0.092   | 0.114   | 0.166   | 0.102   |
| 5988    | 0.007   | 0.054   | 0.078   | 0.055   | 0.025   | 0.037   | 0.024   | 0.033   |
| 6923    | 0.014   | 0.059   | 0.082   | 0.067   | 0.049   | 0.042   | 0.018   | 0.024   |
| Max     | 0.330   | 0.314   | 0.387   | 0.389   | 0.394   | 0.258   | 0.184   | 0.102   |
| Ave of Max DI | 0.047   | 0.088   | 0.142   | 0.146   | 0.133   | 0.120   | 0.083   | 0.042   |

(c)  

| NGA no. | Story Damage Index |
|---------|--------------------|
|         | Story 1 | Story 2 | Story 3 | Story 4 | Story 5 | Story 6 | Story 7 | Story 8 | Story 9 | Story 10 | Story 11 | Story 12 |
| 20      | 0.014   | 0.010   | 0.040   | 0.057   | 0.112   | 0.183   | 0.205   | 0.168   | 0.116   | 0.056   | 0.026   | 0.045   |
| 68      | 0.036   | 0.022   | 0.068   | 0.084   | 0.100   | 0.115   | 0.098   | 0.059   | 0.030   | 0.027   | 0.020   | 0.074   |
| 169     | 0.014   | 0.070   | 0.122   | 0.170   | 0.292   | 0.238   | 0.243   | 0.254   | 0.251   | 0.163   | 0.065   | 0.057   |
| 777     | 0.027   | 0.160   | 0.252   | 0.250   | 0.213   | 0.220   | 0.241   | 0.223   | 0.234   | 0.201   | 0.120   | 0.063   |
| 900     | 0.019   | 0.056   | 0.104   | 0.102   | 0.072   | 0.039   | 0.028   | 0.070   | 0.096   | 0.094   | 0.073   | 0.073   |
5 Conclusions

A procedure for practical damage-based design of structures under seismic loads was presented in this paper. The method was specified for low rise special steel moment frames with vertical and in-plan regularity. In this method, a maximum drift ratio was devised based on the design damage index. For this purpose, the Park-Ang damage index was calculated for 4, 8, and 12-story special steel moment frame buildings designed based on the conventional force-based procedure. Eleven pairs of ground motions were selected and scaled to a design spectrum for nonlinear dynamic analysis. The maximum values of drift ratio and its associated damage index were extracted from the analysis under each earthquake for each building. Use of the pairs of maximum (drift ratio, damage index) resulted in a linear regression equation converting one of the mentioned parameters to the other for each building. By selecting two different values as the design damage indices, the maximum drifts were calculated for both cases and the buildings were designed in the context of displacement-based design. After performing nonlinear dynamic analysis on the designed buildings, maximum damage indices were calculated and reported for each story under each earthquake. Finally, it was concluded that:

1. The proposed procedure begins from selecting a design damage index. Then it simply proceeds to calculate the maximum drift from the developed regression equation and then to other design parameters. It makes a practical step by step trend for seismic design.
2. Results of the damage-based design are in good correlation with the forced-based design. It can be assumed that the force-based design is for a damage index of about 0.3 inherently.
3. For the 11 pairs of spectrum compatible ground motions, the damage-based design resulted in maximum story damage indices that in the worst case just meet the design.
Table 15 Maximum damage index for DI = 0.2 for the damage-based designed buildings

(a)  

| NGA no. | Story Damage Index | Story Drift Ratio |
|---------|---------------------|-------------------|
|         | Story 1 | Story 2 | Story 3 | Story 4 | Story 1 | Story 2 | Story 3 | Story 4 |
| 20      | 0.154   | 0.123   | 0.083   | 0.011   | 0.015   | 0.018   | 0.018   | 0.016   |
| 68      | 0.004   | 0.018   | 0.023   | 0.005   | 0.007   | 0.011   | 0.011   | 0.012   |
| 169     | 0.143   | 0.081   | 0.062   | 0.010   | 0.013   | 0.015   | 0.013   | 0.014   |
| 778     | 0.103   | 0.076   | 0.072   | 0.014   | 0.013   | 0.017   | 0.016   | 0.016   |
| 826     | 0.020   | 0.037   | 0.037   | 0.003   | 0.008   | 0.013   | 0.013   | 0.012   |
| 900     | 0.087   | 0.087   | 0.061   | 0.007   | 0.012   | 0.016   | 0.016   | 0.013   |
| 987     | 0.043   | 0.080   | 0.041   | 0.000   | 0.010   | 0.016   | 0.015   | 0.012   |
| 1107    | 0.067   | 0.053   | 0.052   | 0.016   | 0.010   | 0.014   | 0.015   | 0.016   |
| 4853    | 0.156   | 0.110   | 0.074   | 0.007   | 0.014   | 0.016   | 0.015   | 0.013   |
| 5814    | 0.220   | 0.178   | 0.151   | 0.118   | 0.023   | 0.028   | 0.028   | 0.027   |
| 6923    | 0.032   | 0.033   | 0.061   | 0.045   | 0.008   | 0.012   | 0.015   | 0.019   |
| Max     | 0.220   | 0.178   | 0.151   | 0.118   | 0.023   | 0.028   | 0.028   | 0.027   |
| Ave of Max | 0.093 | 0.080 | 0.065 | 0.022 | 0.012 | 0.016 | 0.016 | 0.015 |

(b)  

| NGA no. | Story Damage Index |
|---------|--------------------|
|         | Story 1 | Story 2 | Story 3 | Story 4 | Story 5 | Story 6 | Story 7 | Story 8 |
| 20      | 0.1585  | 0.1909  | 0.2127  | 0.2053  | 0.1536  | 0.0420  | 0.0173  | 0.0002  |
| 68      | 0.0001  | 0.0002  | 0.0002  | 0.0003  | 0.0003  | 0.0003  | 0.0002  | 0.0001  |
| 169     | 0.0748  | 0.0929  | 0.1065  | 0.0994  | 0.0857  | 0.0873  | 0.0679  | 0.0108  |
| 778     | 0.1424  | 0.0985  | 0.1006  | 0.1317  | 0.1330  | 0.0821  | 0.0394  | 0.0004  |
| 900     | 0.0010  | 0.0347  | 0.0504  | 0.1361  | 0.1981  | 0.1584  | 0.0830  | 0.0056  |
| 987     | 0.0006  | 0.0257  | 0.0480  | 0.0344  | 0.0149  | 0.0310  | 0.0144  | 0.0001  |
| 1110    | 0.0067  | 0.0570  | 0.0788  | 0.0589  | 0.0456  | 0.0477  | 0.0332  | 0.0004  |
| 4849    | 0.1270  | 0.2046  | 0.2272  | 0.2143  | 0.2023  | 0.1346  | 0.0783  | 0.0011  |
| 5814    | 0.0345  | 0.0876  | 0.1056  | 0.0928  | 0.1251  | 0.1595  | 0.1494  | 0.0826  |
| 5988    | 0.0058  | 0.0557  | 0.0973  | 0.1531  | 0.1674  | 0.1169  | 0.0484  | 0.0003  |
| 6923    | 0.0823  | 0.1127  | 0.1196  | 0.0971  | 0.0805  | 0.0492  | 0.0277  | 0.0008  |
| Max DI  | 0.1585  | 0.2046  | 0.2272  | 0.2143  | 0.2023  | 0.1595  | 0.1494  | 0.0826  |
| Ave of Max DI | 0.0576 | 0.0873 | 0.1043 | 0.1112 | 0.1097 | 0.0826 | 0.0508 | 0.0093 |

(c)  

| NGA no. | Story Damage Index |
|---------|--------------------|
|         | Story 1 | Story 2 | Story 3 | Story 4 | Story 5 | Story 6 | Story 7 | Story 8 | Story 9 | Story 10 | Story 11 | Story 12 |
| 20      | 0.001   | 0.038   | 0.102   | 0.174   | 0.212   | 0.224   | 0.187   | 0.124   | 0.063   | 0.043   | 0.021   | 0.027   |
| 68      | 0.000   | 0.004   | 0.009   | 0.006   | 0.003   | 0.002   | 0.001   | 0.024   | 0.041   | 0.036   | 0.034   | 0.026   |
| 169     | 0.006   | 0.082   | 0.122   | 0.136   | 0.114   | 0.117   | 0.114   | 0.166   | 0.123   | 0.065   | 0.037   | 0.029   |
| 777     | 0.152   | 0.168   | 0.197   | 0.205   | 0.209   | 0.226   | 0.211   | 0.190   | 0.178   | 0.146   | 0.079   | 0.033   |
| 900     | 0.001   | 0.027   | 0.052   | 0.067   | 0.078   | 0.114   | 0.158   | 0.186   | 0.160   | 0.087   | 0.056   | 0.033   |
| 987     | 0.001   | 0.024   | 0.039   | 0.031   | 0.013   | 0.004   | 0.007   | 0.050   | 0.078   | 0.067   | 0.031   | 0.015   |
Table 15 (continued)

| NGA no. | Story Damage Index |
|---------|--------------------|
|         | Story 1 | Story 2 | Story 3 | Story 4 | Story 5 | Story 6 | Story 7 | Story 8 | Story 9 | Story 10 | Story 11 | Story 12 |
| 1110    | 0.004   | 0.060   | 0.091   | 0.092   | 0.083   | 0.087   | 0.086   | 0.098   | 0.156   | 0.170   | 0.101   | 0.045   |
| 4849    | 0.003   | 0.065   | 0.121   | 0.151   | 0.161   | 0.115   | 0.174   | 0.167   | 0.133   | 0.078   | 0.035   | 0.022   |
| 5814    | 0.013   | 0.140   | 0.186   | 0.190   | 0.158   | 0.117   | 0.095   | 0.075   | 0.115   | 0.169   | 0.138   | 0.091   |
| 5988    | 0.002   | 0.030   | 0.046   | 0.045   | 0.042   | 0.047   | 0.059   | 0.062   | 0.033   | 0.013   | 0.008   | 0.037   |
| 6923    | 0.003   | 0.044   | 0.052   | 0.036   | 0.016   | 0.015   | 0.045   | 0.055   | 0.065   | 0.039   | 0.015   | 0.021   |
| Max DI  | 0.152   | 0.168   | 0.197   | 0.205   | 0.212   | 0.226   | 0.211   | 0.190   | 0.178   | 0.170   | 0.138   | 0.091   |
| Ave of Max DI | 0.017 | 0.062   | 0.093   | 0.103   | 0.099   | 0.097   | 0.103   | 0.109   | 0.104   | 0.083   | 0.050   | 0.035   |

Table 16 Comparison of the fundamental periods

| No. of stories | Fundamental period (sec) | Ratio of column 2 to 4 | Ratio of column 3 to 4 |
|----------------|--------------------------|------------------------|------------------------|
|                | DI = 0.2 | DI = 0.4 | Force-based |                |                      |                      |
| 4              | 0.93     | 1.32     | 1.02       | 0.91            | 1.29                |
| 8              | 1.64     | 2.35     | 1.82       | 0.90            | 1.29                |
| 12             | 2.02     | 2.76     | 2.24       | 0.90            | 1.23                |

Table 17 Comparison of base shears

| No. of stories | Base shear (kN) | Ratio of column 2 to 4 | Ratio of column 3 to 4 |
|----------------|----------------|------------------------|------------------------|
|                | DI = 0.2 | DI = 0.4 | Force-based |                |                      |                      |
| 4              | 968.38   | 809.12   | 862.67      | 1.12            | 0.94                |
| 8              | 1048.53  | 921.08   | 965.63      | 1.09            | 0.95                |
| 12             | 1293.07  | 1130.32  | 1180.62     | 1.10            | 0.96                |

Table 18 Comparison of total structure weight

| No. of stories | Total structure weight (kN) | Ratio of column 2 to 4 | Ratio of column 3 to 4 |
|----------------|-----------------------------|------------------------|------------------------|
|                | DI = 0.2 | DI = 0.4 | Force-based |                |                      |                      |
| 4              | 426.0    | 383.0    | 407.9       | 1.04            | 0.94                |
| 8              | 982.5    | 866.9    | 926.1       | 1.06            | 0.94                |
| 12             | 2018.8   | 1815.4   | 1900.3      | 1.06            | 0.96                |
4. The developed equation for converting the design damage index to the maximum drift ratio appears to be simply a linear relation with good correlation. Use of this simple equation resulted in acceptable story damage indices in the most critical cases studied. However, regarding simplicity of the building cases considered, more research is needed to extend the equation to a wider range of buildings.

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Declarations

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