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Evaluation of Damage Distribution in Elements of Dual Frames

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ABSTRACT

During severe earthquakes, many structures yield and experience large inelastic deformations. Design procedure in current seismic design codes, is based on elastic behavior of structures and considers inelastic deformations implicitly. This fact results in inadequacy of current design practice as researches show. Taking into account inelastic behavior of structural elements may better predict seismic responses.
In the present study, three steel structures with dual system consisting of intermediate moment-resisting and concentric-braced frames that are widely used in medium and high-rise buildings, have been designed based on allowable stress design method. Then, inelastic seismic responses have been determined under three earthquake records by the PERFORM 3D software and hysteretic energy and damage state of structural elements have been evaluated in detail. Finally, by column strengthening, it has been tried to reduce structural damages. The results indicate that this approach is an efficient technique to make the damage distribution uniform among structural members..

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1. INTRODUCTION

Damages of moment-resisting frames during past earthquakes show their vulnerability because of excessive displacement under seismic loads specially in high-rise structures. On the other hand, concentric-braced frames are stiff but brittle, and this leads to collapse of these frames under lateral loads. Dual steel frames with moment-resisting and concentric-braced frames combine two aforementioned structural systems to improve seismic performances. These dual systems can be used in design and retrofit of mid- and high-rise structures.

Current design practice of dual steel frames is the strength criterion. The criterion is based on elastic behavior of structures and considers the inelastic deformations indirectly. Meanwhile, during severe earthquakes, structures yield and experience inelastic deformations. Inadequacy of this design method is proven nowadays by researches and observed structural damages in past earthquakes. To be more realistic in assessment of seismic performance of structures, the inelastic characteristic of elements should be taken into account in the analysis procedure.

Various static and dynamic analysis procedures have different levels of accuracy in characterizing structural model properties and applied loads. Being still in use, equivalent static method is the simplest practical one adapted by seismic design codes. In spite of simplicity, this procedure is limited to regular and linear structures. Response spectrum analysis and static pushover procedure are applicable to a significantly broader range of structures. Meanwhile, nonlinear time-history analysis and dynamic pushover procedure are expected to predict seismic demands more realistically. Although such analyses are hindered by complexity and high computational effort [1], they are capable of considering almost any type of nonlinearity in material and geometry.

The present study focuses on evaluation of dual steel frames consisting of intermediate moment-resisting and concentric-braced frames designed according to the strength-based provisions. In this way, three 3-, 5- and 8-story 3D models have been designed based on allowable stress design (ASD) method of UBC-97 [2]. The models have been analyzed under three pairs of strong ground motion records by inelastic time-history

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analysis in Perform 3D software, and Park-Ang damage index has been used as damage measure in evaluating performance of structural elements. Damage distribution among all members of buildings has been determined. Results show that damage distribution is not uniform in height of the structure and among members of each story. To overcome this non-uniformity, column strengthening has been selected as the retrofit method. Nonlinear time-history analysis of the strengthened structures shows that this approach is an efficient technique for improving seismic performance of dual steel systems.

2. SEISMIC DAMAGE INDICES

Finding clear quantitative measures for representation of structural damages is an important issue in damage evaluation. Generally, the structural damage is defined in terms of either economics or safety/strength considerations. Economic damage indices are usually defined as the ratio of repair to replacement cost for the structure or structural element while safety/strength damage indices, used in the present research, are normally related to deterioration of structural resistance.

A large number of seismic damage models have been proposed in the literature, ranging from simple approaches using ductility ratio or inter-story drift to complicated definitions taking into account the effects of both deformation and energy [3-6]. The damage indices use many different local and global parameters to reflect the best estimate of the damage status of the structure. For instance, "Park and Ang" [7] and "Bozorgnia and Bertero" [8] consider deformation and energy dissipation of the structure and "Krawinkler and Zohrei" [9] uses the low-cycle fatigue theory.

TABLE 1. The relation between damage index and damage state [12]

Degree of damage	Physical Appearance	Damage Index	State of Building
Slight	Sporadic occurrence of cracking	< 0.1	No Damage
Minor	Minor cracks; partial crushing of concrete in columns	0.1-0.25	Minor Damage
Moderate	Extensive large cracks; spalling of concrete in weaker elements	0.25-0.4	Repairable
Severe	Extensive crashing of concrete; disclosure of buckled reinforcement	0.4-1.0	Beyond Repair
Collapse	Partial or total collapse of building	>1.0	Loss of Building

In fact, the damage of structures is a combined effect of the response magnitude and the number of load cycles. For steel structures, these two effects can be easily modeled by accumulation rules such as that of Miner [10]. The case of reinforced concrete structures is complicated by the concrete-reinforcement interaction. For these structures, the level of the load, the number of cycles to failure and their relationship need to be simulated by appropriate acceleration factors [11].

Among the many damage measures available, the Park - Ang damage index appears to be the most promising due to its simplicity and extensive calibration against experimentally-observed damages in reinforced concrete structures. Although it is less reliable in the case of steel structures, its confidence can be improved by selecting suitable values of the model parameters [12, 13]. Park- Ang damage index, $D_{PA,I}$, is defined as a linear combination of the ductility and energy dissipation indices:

$$D_{PA,I} = \frac{U_{\max}}{U_u} + \frac{\beta}{Q_r\ U_u}\sum dE \quad (1)$$

where:

$U_{\max}$ = The maximum deformation response under ground motion

U_u = The ultimate deformation under monotonic loading

$\sum dE$ = Dissipated hysteretic energy

Q_r = Yield strength

β = A nonnegative constant

values of β about 0.15, derived by fitting test results, are used in the literature for reinforced concrete structures [14, 15], while in the case of steel structures a value of β =0.025, used in present study, can be adopted [16].

3. STRUCTURAL MODELING PROCEDURE

The PERFORM 3D (VER 4.0.1) software [17] has been used for numerical modeling and analysis of the structures. In a detailed finite element model, each beam or column is divided into a number of elements along its length as shown in Figure 1 schematically.

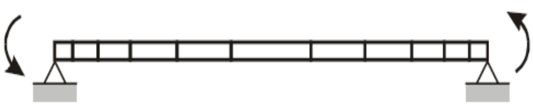


Figure 1. Detailed Finite Element Model [17]

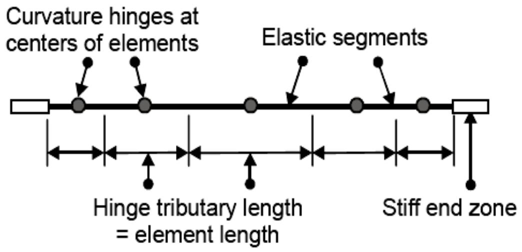


Figure 2. Finite Element Model with Hinges [17]

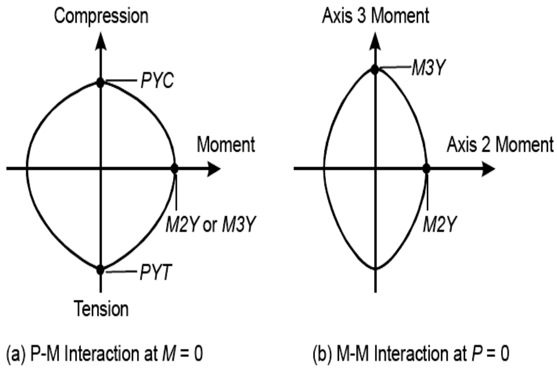


Figure 3. P-M-M Yield Surface for Steel Elements [17]

Many different finite element models with low or high order might be used in modeling process. Generally, these models are based on either moment-curvature or fiber stress-strain relationships. PERFORM 3D software provides two options, namely curvature hinges and fiber segments. Figure 2 shows a finite element model using curvature hinges.

For an elastic structure, the goal of finite element analysis is to get a close approximation of the exact responses. In the case of a beam element, this means determining accurate values of bending moments, shear forces and displacements. Generally, as the element mesh gets finer, the result gets more accurate. For an inelastic structure, an additional goal is to determine sufficiently-accurate inelastic deformations for calculating demand-capacity ratios. The demand-capacity measure might be the curvature or the fiber strain which is closely related to the curvature. The problem is that as the mesh is made finer, the maximum calculated curvature or strain usually gets progressively larger. This arises from the fact that beam theory for inelastic behavior predicts very large localized curvatures at the points of maximum bending moment, usually at the beam ends. Indeed, for an elastic-perfectly-plastic moment-curvature relationship, the maximum curvature after yield is theoretically infinite. Trilinear behavior has been considered for all elements modeled in PERFORM 3D. Interaction of axial forces and bending moments has been considered in the trilinear model for column elements, making the model substantially more complex than elastic-perfectly-plastic (EPP) model. In the EPP behavior, there is only one yield surface and the surface does not move, but in the trilinear behavior, there are two surfaces and the inner one move around. Figure 3 shows the yield surface that PERFORM uses for axial force and moment interaction (P-M-M) for steel elements [17].

Nonlinear components can have complex properties, and the forms for input of component properties may appear to be complex. In this paper trilinear behavior, brittle strength loss in force-deformation relationship for inelastic components has been considered. Each material and each basic structural component has one or more actions or forces and corresponding deformations. The relationship between the two is the F-D relationship. In a structural component, brittle strength loss can be caused by a number of effects, including tensile fracture, concrete crushing, concrete shear failure and buckling. When a component loses strength, the lost strength is redistributed to adjacent components [17].

4. NONLINEAR DYNAMIC ANALYSIS OF STRUCTURES

The studied steel structures with dual system of intermediate moment-resisting and concentric-braced frames have been designed according to the requirements of the ASD method of UBC-97. All of the structures are symmetrical. The beams and columns have general I-shape sections, and the box sections are used for the bracings.

Plans and geometry of the structural models are shown in Figure 4. All models have the same plan. A bay width of 4 m and a story height of 3.2 m, common values in residual buildings, have been used in the models. Some characteristics of the structural models are shown in Table 2. All buildings have been designed considering a response reduction factor, R, of 7, corresponding to the dual system of intermediate moment-resisting and concentric-braced frames. Building importance class III, site subsoil of type C and site seismicity category 1 have been considered for models based on UBC-97 classification. In addition to the dead loads and seismic loads, snow loads and live loads have been taken into account, as well as lateral loads due to column sway. After designing and detailing the structures, nonlinear time-history analyses are carried out for evaluating the structural seismic responses. In this way, the hysteretic behavior of the beams, columns and braces, incorporating stiffness degradation, strength deterioration and non-symmetric responses, is specified. Degrading parameters have been chosen from experimental results of cyclic force–deformation characteristics of typical components of the studied structures [17].

Three pairs of strong ground motion records for site class C have been used as input for nonlinear time-history analysis. Characteristics of these records are given in Table 3. For each pair of records, the square root of the sum of the squares (SRSS) of the 5%-damped response spectrum of the normalized horizontal components has been constructed. The records have been scaled such that the average SRSS spectrum does not fall below 1.4 times the design spectrum in period range 0.2T to 1.5T, where T is the fundamental period of the structure [2]. The response spectra of the scaled acceleration records of Loma Prieta, Northridge and San Fernando earthquakes and the design spectrum of UBC-97 are shown in Figure 5.

TABLE 2. Basic design properties of the studied structures

Structure	Natural Period (s)	Base Shear (ton)	Mass of Structure (ton)
3-Story	0.526	95.21	103.56
5-Story	0.783	160.47	174.54
8-Story	1.227	205.56	282.88

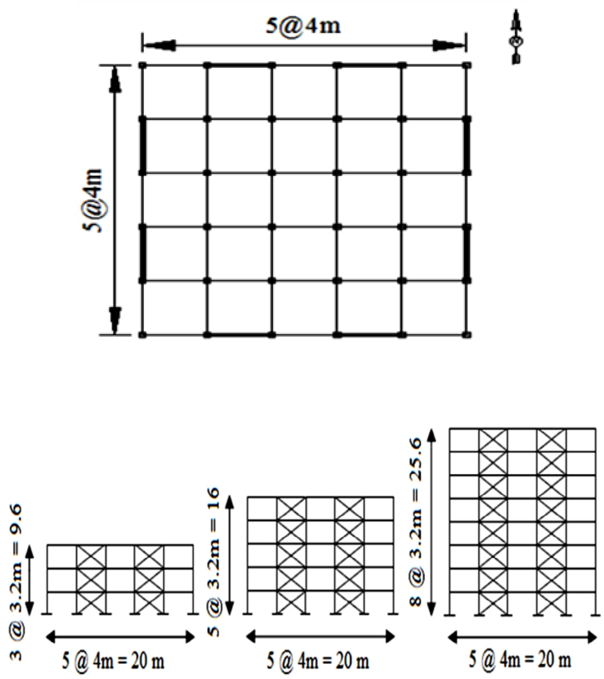


Figure 4. Configuration of studied models

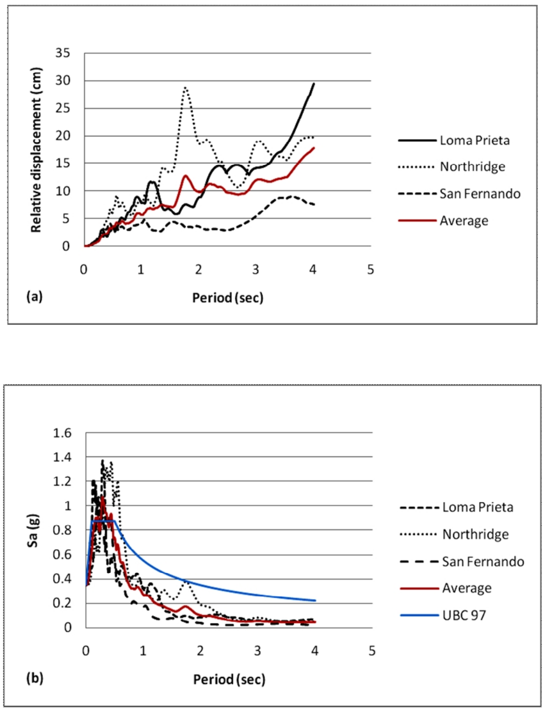


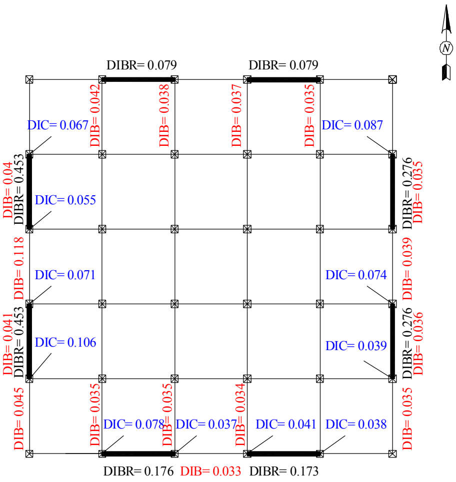
Figure 5. Response Spectra of the Scaled Acceleration Records and the UBC Design Spectrum, (a) Relative Displacement, (b) Acceleration.

TABLE 3. Strong ground motion parameters

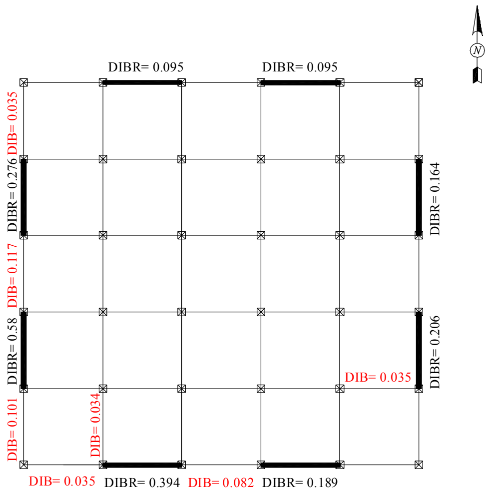
Earthquake:	Northridge	Loma Prieta
Year:	1994	1989
Station:	Covia-S Grand Ave.	Apeel 3E Hayward
Component:	GRA074 GRA344	A3E000 A3E090
PGA (g):	0.066 0.062	0.078 0.084
Magnitude (Ms):	6.7	6.9
Duration (sec):	35	40

5. EVALUATION OF NONLINEAR DYNAMIC ANALYSIS RESULTS IN PLAN

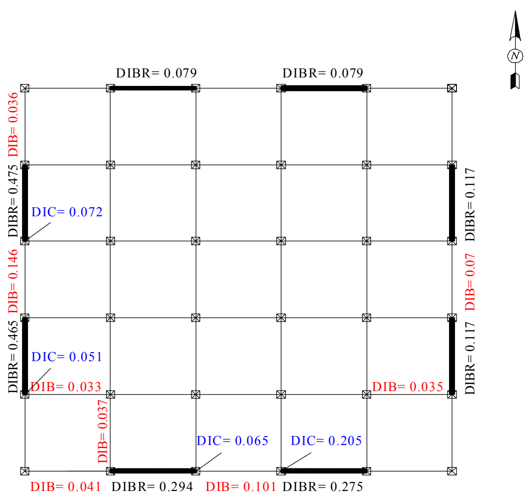
Usually perpetual structural damage is seen in the end of earthquake. Therefore, distribution of damage in this time is expressive of the permanent damage of the building. The structures have been subjected to two scaled horizontal components of acceleration records, simultaneously. Damage values of structural elements are averaged under three earthquakes. Average values are shown in Figure 6 for 8-story building.



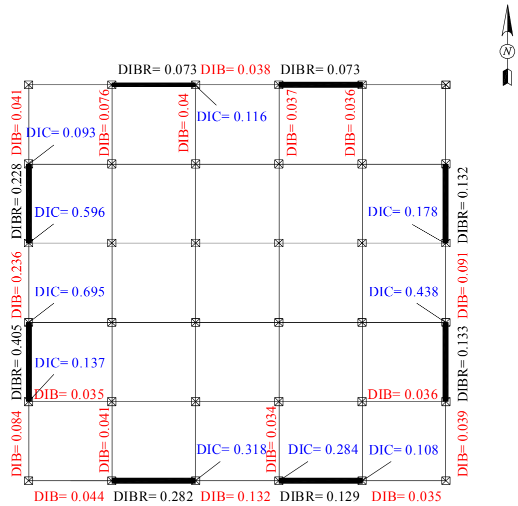
Damage index of story 1



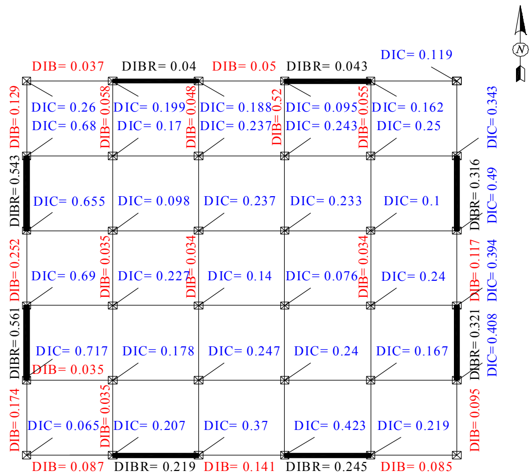
Damage index of story 2



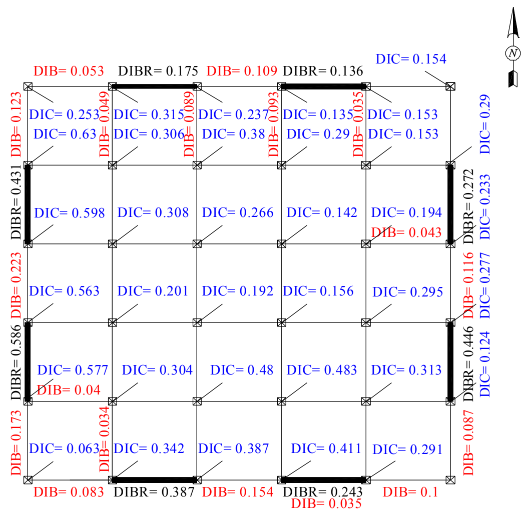
Damage index of story 3



Damage index of story 4



Damage index of story 5



Damage index of story 6

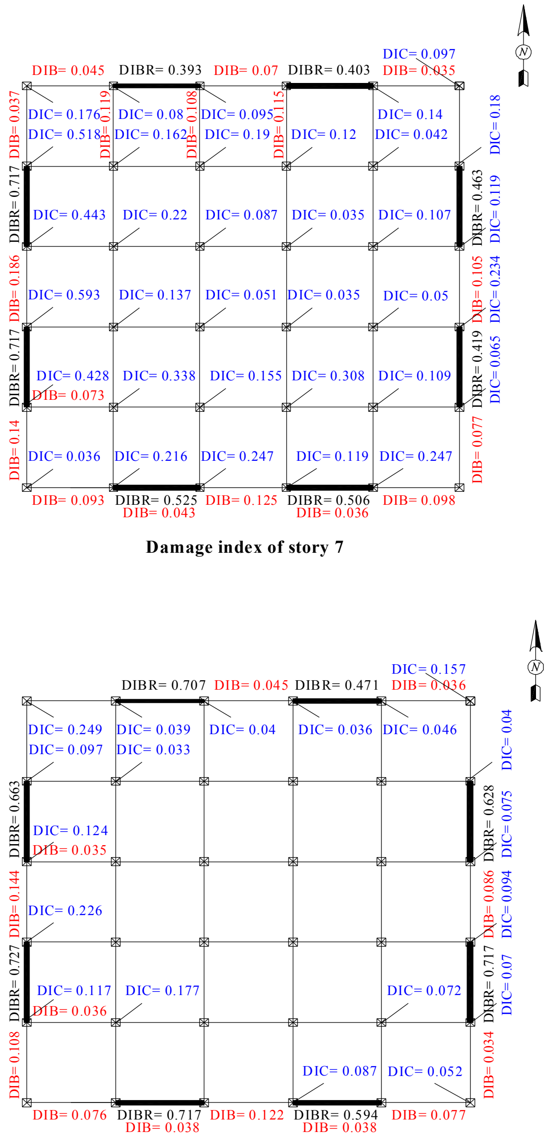


Figure 6. Damage indices for members of 8-story building

In these figures, damage indices are shown only for the damaged members, i.e. the members that have experienced inelastic deformation. Also, the braced spans are highlighted in the figures. Because of the large number of analyses with large stiffness matrixes, e.g., 388 nodes and 1024 elements for the 8-story building, evaluating the damage distribution in all parts of the buildings is very time-consuming and difficult. In the following figures:

DIBR: Damage index of brace
DIC: Damage index of column
DIB: Damage index of beam

As shown in Figures 6, maximum damage is seen in the external braced frames. In these frames, columns connected to braces have maximum damage due to high absorption of hysteretic energy and high displacement. Therefore, to improve seismic performance of the structures, the sections of these columns have been strengthened about 10 - 13%.

Absorption of hysteretic energy in members of higher stories is more than that in members of lower stories, and consequently, higher stories have more damage. So, coulmn-strenghtening has been done only in higher stories. Speaking precisely, in 8-story building, columns connected to braces have been strengthened from story 3 to the upper stories. This procedure has been done for 3rd, 4th and 5th stories in 5-story building and for all stories in 3-story building. The results of analysis after the column-strengthening can be seen in Figure 7 for 8-story building.

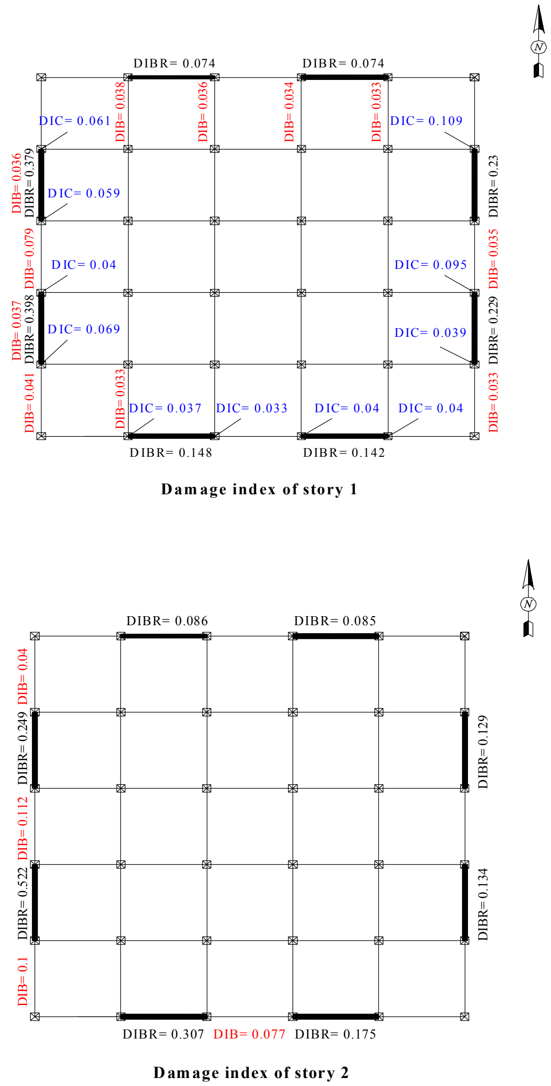


Figure 7. Damage indices for members of 8-story building after strengthening

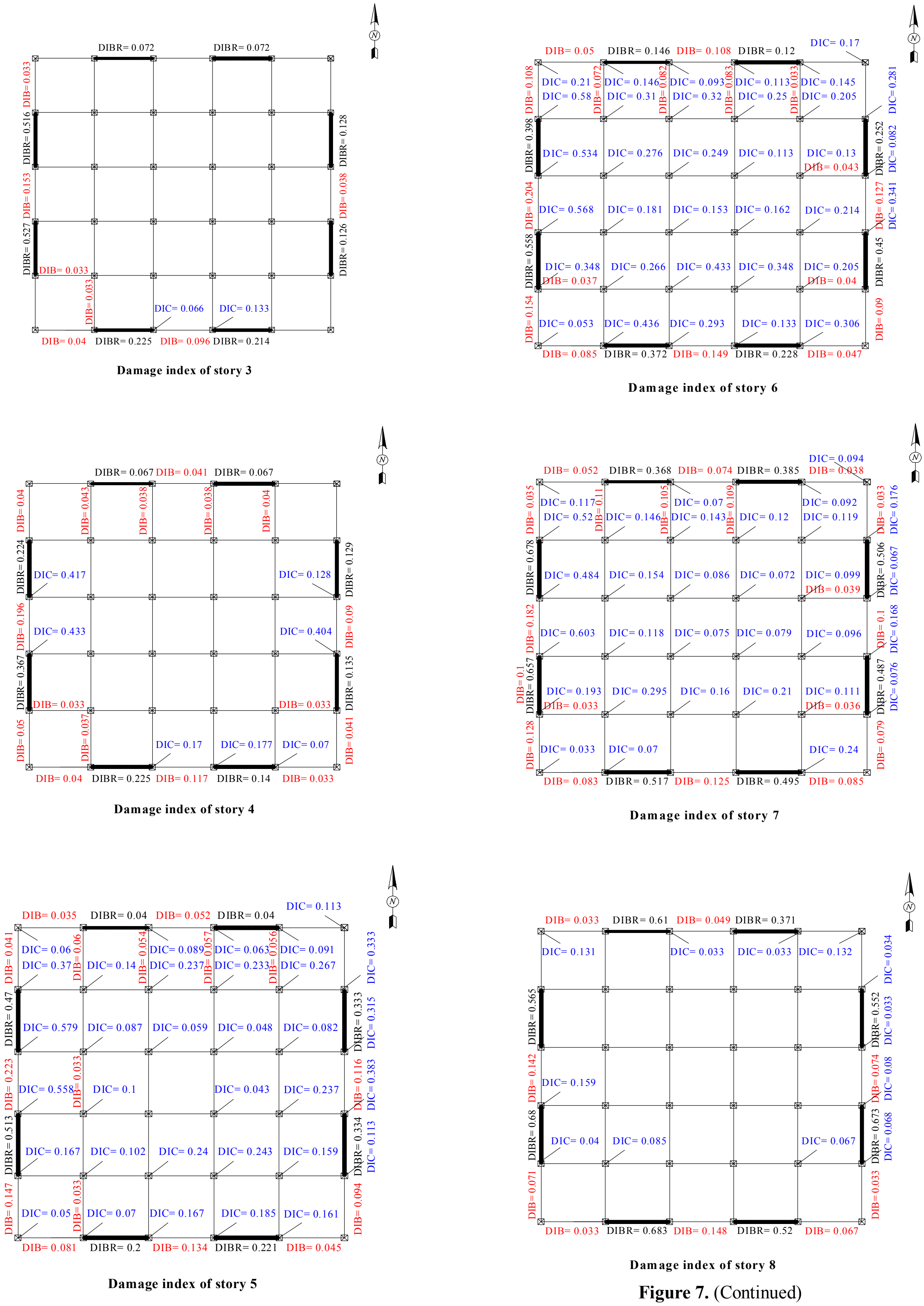
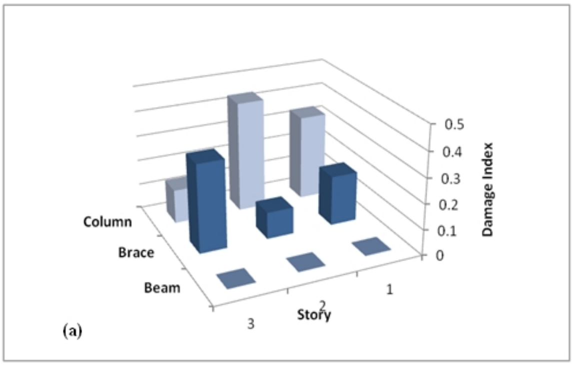
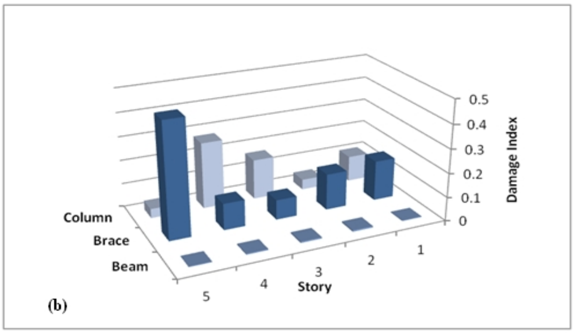


Figure 7. (Continued)

6. DISTRIBUTION OF DAMAGE IN PARTS OF EACH STORY

For evaluation of seismic behavior of structures, the contribution of the beams, columns and braces of 3-, 5- and 8-story buildings in the level of damage have been determined and are shown in Figures 8 and 9. The Level of damage in the elements of the buildings has been computed in the end of earthquake record because in this time, the damages will be stable and maximal. These damage values are averaged under Loma Prieta, Northridge and San Fernando earthquakes. The figures show that in all stories of the studied buildings, damage values of beams are less than those of other elements and that in the lower and higher stories, the braces have maximum damages. In the strengthened buildings, damage values get closer together in the columns and braces, but the values in the columns are a little more than those in the braces. The damage distributions show that structural design according to the requirements of the current seismic design codes has not good performance in structural members and does not result in uniform distribution of damage among members of buildings. Therefore, several authors have discussed the need for an improvement of the current earthquake-resistant-design methods in order to consider the structural damage with inelastic behavior concept. In this way, the column strengthening has been proposed in this study to have a uniform damage distribution among the members of the structure.





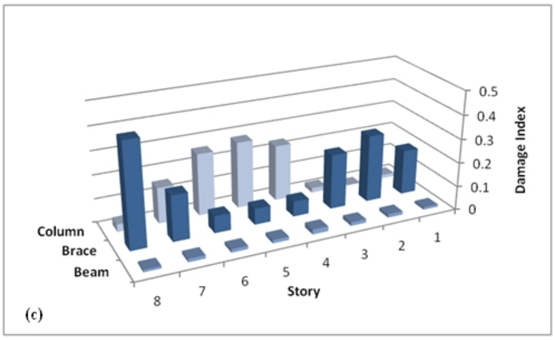
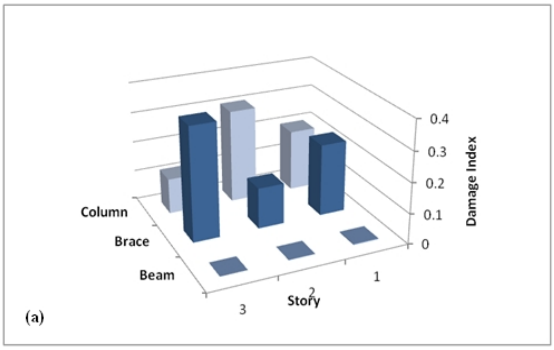
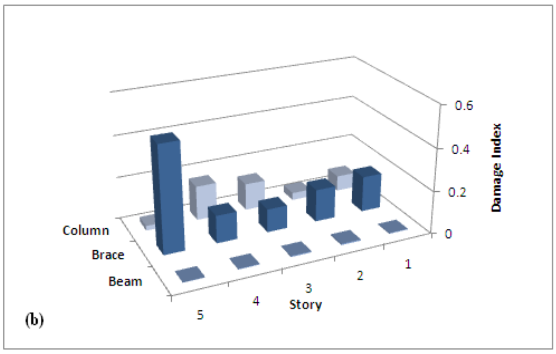


Figure 8. Comparison of Damage Distribution among the Members of (a): 3-Story Building, (b): 5-Story Building and (c): 8-Story Building before Strengthening.





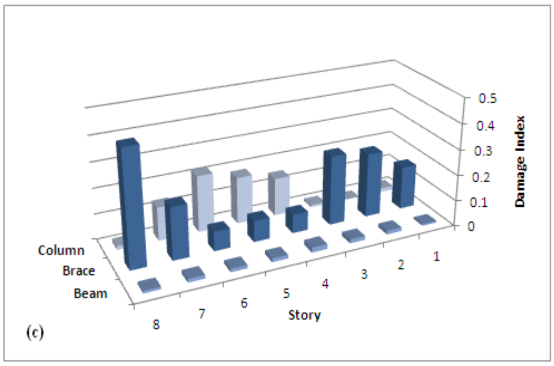


Figure 9. Comparison of Damage Distribution among the Members of (a): 3-Story Building, (b): 5-Story Building and (c): 8-Story Building after Strengthening.

7. CONCLUSIONS

In this study, the distribution of damage among structural members of dual steel buildings was evaluated. The damage indices in all members of dual steel buildings with various numbers of stories were evaluated by nonlinear time-history analysis using PERFORM 3D software. The results of the study can be summarized as follows:

1. In spite of the uniform distribution of strength in the height of the structures, the damage values don't have that uniformity and don't have specific height-wise regularity.
2. Most of the damaged columns and beams are those connected to the external braced frames. For example, 95% of damaged beams in the 3-story building are seen in the external frames. Therefore, these frames in the dual systems consisting of intermediate moment-resisting and concentric-braced frames have low strength and need to be strengthened.
3. Most of the energy imported to the external braced frames is absorbed by the braces and in next rank, by the columns.
4. In plans of the buildings, quite a regular distribution of damage can be seen, so that the damage values around the center of the floors of the stories is less than those in external frames.
5. In all parts of the buildings, beams have less damage values compared to other members, and in up and down stories, braces have more damage values than other members.

In this paper, only the column section strengthening have been used for strengthening of buildings, but other methods such as the brace strengthening and the bay width reduction can be used and compared with the method applied in this paper.

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چکیده

طی زلزله های شدید سازه های زیادی تسلیم شده و تغییر شکل‌های غیر خطی بزرگی را تجربه کرده اند. فرآیند طراحی در آیین‌نامه‌های طراحی لرزه‌ای فعلی بر پایه رفتار ارتجاعی سازه‌ها با منظور داشتن اثر تغییر شکل‌های غیرارتجاعی به صورت غیرمستقیم است. محققین ثابت کرده‌اند که این موضوع سبب عدم کفایت آیین نامه‌های موجود در طرح لرزه‌ای سازه‌هاست و در نظر گرفتن رفتار غیر الاستیک اعضا سازه‌ای ممکن است پاسخ لرزه‌ای سازه‌ها را بهتر آشکار سازد. در این تحقیق سه سازه فولادی دوگانه با شکل پذیری متوسط با مهاربند هم‌مرکز، که به طور وسیعی در ساختمان‌های متوسط و بلند مرتبه استفاده می شوند، انتخاب شده و بر اساس روش طراحی تنش مجاز طرح شده‌اند. سپس، این سازه‌ها تحت اثر سه زلزله قرار گرفته و با نرم افزار PERFORM 3D (VER 4.0.1) تحلیل دینامیکی غیرخطی شدند و انرژی هیسترزیس و خرابی همه بخش‌های مختلف سازه محاسبه شدند. سر انجام، با تقویت ستون، تلاش شده است تا تخریب‌های سازه‌ای کاهش یابد. نتایج نشان می‌دهد که این روش تکنیک مناسبی برای توزیع یک‌نواخت خسارت بین اعضا سازهای می‌باشد.

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