Journal of
Mechanics of
Materials and Structures

DIRECT DAMAGE-CONTROLLED DESIGN OF PLANE STEEL MOMENT-RESISTING FRAMES USING STATIC INELASTIC ANALYSIS

George S. Kamaris, George D. Hatzigeorgiou and Dimitri E. Beskos

Volume 4, Nº 7-8 September 2009

mathematical sciences publishers

JOURNAL OF MECHANICS OF MATERIALS AND STRUCTURES
Vol. 4, No. 7-8, 2009

DIRECT DAMAGE-CONTROLLED DESIGN OF PLANE STEEL MOMENT-RESISTING FRAMES USING STATIC INELASTIC ANALYSIS

GEORGE S. KAMARIS, GEORGE D. HATZIGEORGIOU AND DIMITRI E. BESKOS

A new direct damage-controlled design method for plane steel frames under static loading is presented. Seismic loading can be handled statically in the framework of a push-over analysis. This method, in contrast to existing steel design methods, is capable of directly controlling damage, both local and global, by incorporating continuum damage mechanics for ductile materials in the analysis. The design process is accomplished with the aid of a two-dimensional finite element program, which takes into account material and geometric nonlinearities by using a nonlinear stress-strain relation through the beam-column fiber modeling and including P-δ and P-Δ effects, respectively. Simple expressions relating damage to the plastic hinge rotation of member sections and the interstorey drift ratio for three performance limit states are derived by conducting extensive parametric studies involving plane steel moment-resisting frames under static loading. Thus, a quantitative damage scale for design purposes is established. Using the proposed design method one can either determine damage for a given structure and loading, or dimension a structure for a target damage and given loading, or determine the maximum loading for a given structure and a target damage level. Several numerical examples serve to illustrate the proposed design method and demonstrate its advantages in practical applications.

1. Introduction

Current steel design codes, such as AISC [1998] and EC3 [2005], are based on ultimate strength and the associated failure load. In both codes, member design loads are usually determined by global elastic analysis and inelasticity is taken into account indirectly through the interaction equations involving design loads and resistances defined for every kind of member deformation. Instability effects are also taken in an indirect and approximate manner through the use of the effective length buckling factor, while displacements are checked for serviceability at the end of the design process. Seismic design loads are obtained with the aid of seismic codes, such as AISC [2005] and EC8 [2004]. In this case the global analysis can be elastostastic as before, spectral dynamic, static inelastic (push-over) or nonlinear dynamic.

Damage of materials, members, and structures is defined as their mechanical degradation under loading. Control of damage is always desirable by design engineers. Even though current methods of design [AISC 1998; EC3 2005; AISC 2005; EC8 2004] are associated with ultimate strength and consider inelastic material behavior indirectly or directly, they are force-based and cannot achieve an effective control of damage, which is much better related to displacements than forces. For example, the percentage of the interstorey drift ratio (IDR) of seismically excited buildings is considered a solid basic

Keywords: continuum damage mechanics, damage control, steel structures, design methods, beam-column, finite element method, second order effects, elastoplastic behavior.

indicator of the level of damage, as suggested by the HAZUS99-SR2 User's Manual [FEMA 2001]. Even the displacement-based seismic design method [Priestley et al. 2007], in which displacements play the fundamental role in design and are held at a permissible level (target displacements), does not lead into a direct and transparent control of damage.

To be sure, there are many works in the literature dealing with the determination of damage in members and structures, especially in connection with the seismic design of reinforced concrete structures. More specifically, damage determination of framed buildings at the local and global level can be done with the aid of damage indices computed on the basis of deformation and/or energy dissipation, as shown by Park and Ang [1985] and Powell and Allahabadi [1988], for example. On the other hand, the finite element method has been employed in the analysis of steel and reinforced concrete structures in conjunction with a concentrated inelasticity (plasticity and damage) beam element in [Florez-Lopez 1998]. Damage determination in reinforced concrete and masonry structures has also been done by employing continuum theories of distributed damage in the framework of the finite element method [Cervera et al. 1995; Hatzigeorgiou et al. 2001; Hanganu et al. 2002]. Note that in all these references, the approach is to determine damage as additional structural design information, and cannot lead to a structural design with controlled damage.

Here we extend the direct damage-controlled design (DDCD) method, first proposed in Hatzigeorgiou and Beskos [2007] for concrete structures, to structural steel design. The basic advantage of DDCD is the dimensioning of structures with damage directly controlled at both local and global levels. In other words, the designer can select a priori the desired level of damage in a structural member or a whole structure and direct his design in order to achieve this preselected level of damage. Thus, while the DDCD deals directly with damage, inelastic design approaches, such as [AISC 1998; EC3 2005; AISC 2005; EC8 2004; Priestley et al. 2007] are concerned indirectly with damage. Furthermore, the a priori knowledge of damage, as it is the case with DDCD, ensures a controlled safety level, not only in strength but also in deflection terms. Thus, the present work, unlike all previous works on damage of steel structures, develops for the first time a direct damage-controlled steel design method, which is not just restricted to damage determination as an additional structural design information.

More specifically, the present work develops a design method for plane steel moment-resisting frames under static monotonic loading capable of directly controlling damage, both at local and global level. Seismic loading can be handled statically in the framework of a push-over analysis. Local damage is defined pointwise and expressed as a function of deformation on the basis of continuum damage mechanics theory for ductile materials [Lemaitre 1992]. On the other hand, global damage definition is based on the demand-and-capacity-factor design format as well as on various member damage combination rules. The method is carried out with the aid of the two-dimensional finite element program DRAIN–2DX [Prakash et al. 1993], which takes into account material and geometric nonlinearities, modified by the authors to employ damage as a design criterion in conjunction with appropriate damage levels. Material nonlinearities are implemented in the program by combining a nonlinear stress-strain relation for steel with the beam-column fibered plastic hinge modeling. Geometric nonlinearities involve P-δ and P-Δ effects. Thus, the proposed method belongs to the category of design methods using advanced methods of analysis [Chen and Kim 1997; Kappos and Manafpour 2001; Vasilopoulos and Beskos 2006; 2009], which presents significant advantages over the code-based methods. Local buckling can be avoided by using only class 1 European steel sections, something which is compatible with the inelastic analysis employed herein. Furthermore, all structural members are assumed enough laterally braced in order to avoid lateral-torsional buckling phenomena. Using the proposed design method one can either determine damage for a given structure and loading, or dimension a structure for a target damage and given loading, or determine the maximum loading for a given structure and a target damage level.

2. Stress-strain relations for steel

Essential features of a steel constitutive model applicable to practical problems should be, on the one hand the accurate simulation of the actual steel behavior and on the other hand the simplicity in formulation and efficiency in implementation in a robust and stable nonlinear algorithmic manner. In this work, a multilinear stress-strain relation for steel characterized by a good compromise between simplicity and accuracy and a compatibility with experimental results, is adopted. The stress-strain (σ, ε) relation in tension for this steel model is of the form

$$\sigma = E\varepsilon \;\text{ for } \varepsilon \leq \varepsilon_y, \qquad \sigma = \sigma_y + E_h(\varepsilon - \varepsilon_y) \;\text{ for } \varepsilon_y < \varepsilon \leq \varepsilon_u, \qquad \sigma = \sigma_u \;\text{ for } \varepsilon_u < \varepsilon. \tag{1}$$

Equation (1) describes a trilinear stress-strain relation representing elastoplastic behavior with hardening, as shown in Figure 1, with E and E_h being the elastic and the inelastic moduli, respectively, ε_y and ε_u the yield and the ultimate strains, respectively and σ_y and σ_u the yield and ultimate stress, respectively. The negative counterpart to (1) can be adopted for the compression stress state, as shown in Figure 1. Similar stress-strain curves have been proposed earlier by, for example, [Gioncu and Mazzolani 2002]; European and American steels exhibit a stress-strain behavior similar to that of Figure 1. Thus, the model (1) can effectively depict the true behavior of structural steel.

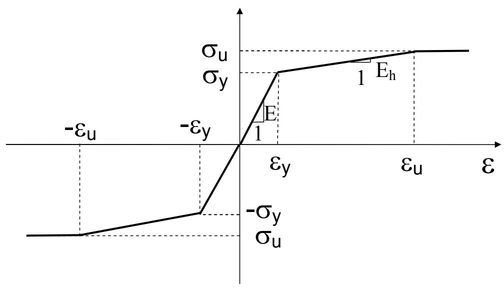

Figure 1. Stress-strain relation for steel.

3. Local damage

Local damage is usually referred to a point or a part of a structure and is one of the most appropriate indicators about their loading capacity. In the framework of continuum damage mechanics, the term "local" is associated with damage indices describing the state of the material at particular points of the structure, and the term "global" with damage indices describing the state of any finite material volume of the structure. Thus, global damage indices can be referred to any individual section, member, substructure, or the whole structure. This categorization of damage in agreement with continuum mechanics principles stipulating that constitutive models are defined at point level and all other quantities are obtained by integrating pointwise information.

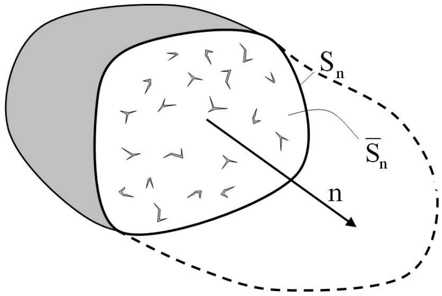

Figure 2. Cross section of a damaged material.

Continuum damage mechanics has been established for materials with brittle or ductile behavior and attempts to model macroscopically the progressive mechanical degradation of materials under different stages of loading. For structural steel, damage results from the nucleation of cavities due to decohesions between inclusions and the matrix followed by their growth and their coalescence through the phenomenon of plastic instability. The theory assumes that the material degradation process is governed by a damage variable d, the local damage index, which is defined pointwise, following Lemaitre [1992], as

$$d = \lim_{S_n \to 0} \frac{S_n - \bar{S}_n}{S_n}, \tag{2}$$

where S_n stands for the overall section in a damage material volume, $\bar{S}_n$ for the effective or undamaged area, while $(S_n - \bar{S}_n)$ denotes the inactive area of defects, cracks, and voids (Figure 2). This index corresponds to the density of material defects and voids and has a zero value when the material is in the undamaged state and a value of unity at material rupture or failure.

The main goal of continuum damage mechanics is the determination of initiation and evolution of the damage index d during the deformation process. Lemaitre [1992], by assuming that damage evolution takes place only during plastic loading (plasticity induced damage) was able to propose a simple damage evolution law, as shown in Figure 3, which can successfully simulate the behavior of steel or other ductile materials. Damage index d is represented by a straight line in damage-strain space, with end points at $d = 0$ for $\varepsilon = \varepsilon_y$, and $d = 1$ for $\varepsilon = \varepsilon_u$, where strain values are assumed to be absolute. This damage evolution law can be expressed as

$$d = 0 \quad \text{for } \varepsilon \le \varepsilon_y, \qquad d = \frac{\varepsilon - \varepsilon_y}{\varepsilon_u - \varepsilon_y} \quad \text{for } \varepsilon_y < \varepsilon \le \varepsilon_u. \tag{3}$$

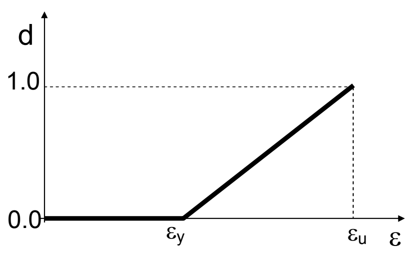

Figure 3. Damage-strain curve for steel.

A similar linear damage evolution law was proposed in [Florez-Lopez 1998]. Both laws are supported by experiments. One can observe that while the damage evolution law for concrete [Hatzigeorgiou and Beskos 2007] was derived by appropriately combining basic concepts of damage mechanics and a nonlinear stress-strain equation for plain concrete, the damage evolution law (3) for steel was taken directly from the literature [Lemaitre 1992].

4. Global damage

Global damage is referred to a section of a member, a member, a substructure, or a whole structure and constitutes one of the most suitable indicators about their loading capacity. Several methods to determine an indicator of damage at the global level have been presented in the literature. In general, these methods can be divided into four categories involving the following structural demand parameters: stiffness degradation, ductility demands, energy dissipation, and strength demands. According to the first approach, one of the most popular ways is to relate damage to stiffness degradation indirectly, that is, to the variation of the fundamental frequency of the structure during deformation [DiPasquale and Cakmak 1990]. However, this approach is inappropriate for the evaluation of the global damage of a substructure or its impact on the overall behavior. Furthermore, in order to evaluate the complete evolution of global damage with loading, a vast computational effort is needed due to the required eigenvalue analysis at every loading step. An alternative way to determine global damage is by computing the variation of the structural stiffness during deformation, as in [Ghobarah et al. 1999]; but again, evaluation of the global damage evolution requires heavy computations at every loading step. Many researchers determine damage in terms of the IDR. Whereas macroscopic quantities such as IDRs are good indicators of global damage in regular structures, this is not generally the case in more complex and/or irregular structures. Damage determination has also been done with the aid of damage indices computed on the basis of ductility (defined in terms of displacements, rotations or curvatures) and/or energy dissipation, as is evident in the method of [Park and Ang 1985] for framed concrete buildings or in the review article [Powell and Allahabadi 1988]. For the computation of damage in steel structures under seismic loading, one can mention [Vasilopoulos and Beskos 2006; Benavent-Climent 2007]. Note that all these indices are appropriate for seismic analyses only. They are not applicable to other types of problems, such as static ones; see [Hanganu et al. 2002].

In this work, for the section damage index D_s of a steel member, the following expression is proposed

$$D_S = \frac{c}{d} = \frac{\sqrt{(M_S - M_A)^2 + (N_S - N_A)^2}}{\sqrt{(M_B - M_A)^2 + (N_B - N_A)^2}}. \tag{4}$$

In the above, the bending moments M_A, M_S, and M_B and the axial forces N_A, N_S, and N_B as well as the distances c and d are those shown in the moment M – axial force N interaction diagram of Figure 4 for a plane beam-column element. The bending moment M_S and axial force N_S are design loads incorporating the appropriate load factors in agreement with EC3 [2005].

Figure 4 includes a lower bound damage curve, the limit between elastic and inelastic material behavior and an upper bound damage curve, the limit between inelastic behavior and complete failure. Thus, damage at the former curve is zero, while at the latter curve is one. Equation (4) is based on the assumption that damage evolution varies linearly between the above two damage bounds. These

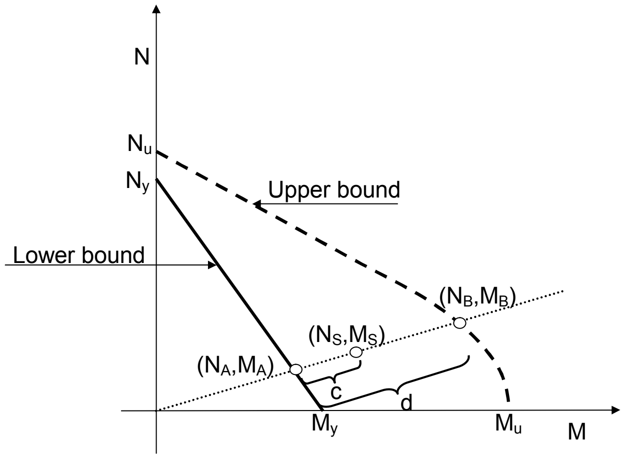

Figure 4. Section damage definition.

lower and upper bound curves can be determined accurately with the aid of the beam-column fibered plastic hinge modeling described in the next section. For their determination, the resistance safety factors are taken into account in agreement with EC3. The bound curves of Figure 4 can also be determined approximately by code type of formulae. Thus, the lower bound curve can be expressed as

$$\frac{M}{M_y} + \frac{N}{N_y} = 1, \tag{5}$$

where N_y and M_y are the minimum axial force and bending moment, respectively, which cause yielding, while the upper bound curve can be expressed as

$$\frac{M}{M_u} + \left(\frac{N}{N_u}\right)^2 = 1, \tag{6}$$

where N_u and M_u are the ultimate axial force and bending moment, respectively, which cause failure of the section. Equations (5) and (6) can be used for the construction of the bounding curves of Figure 4. The provisions in EC3 give a M-N interaction formula similar to (6), with the hardening effect not taken into account, that is, with $\sigma_u = \sigma_y$ or equivalently, $N_u = N_y$. Furthermore, since EC3 allows inelastic analysis only for section class 1, the proposed method is limited to sections of that class.

The section damage index proposed in (4) represents an extension of (3) from strains (or stresses) to forces and moments, i.e., stress resultants. Expressions for damage in terms of stress resultants are also mentioned in [Lemaitre 1992]. By contrast, Florez-Lopez [1998] uses *generalized effective stress*, which corresponds to bending moment, by analogy with the definition of *effective stress*, which corresponds to inelastic stress. His formulation, however, includes only bending moments, without any interaction with axial forces.

It should be noted that the proposed section damage index corresponds to the aforementioned fourth type of damage indicators, which are related to the strength demand approach. More specifically, this index is based on the demand-and-capacity-factor design format. There is an analogy or correspondence between the capacity ratio of interaction equations of EC3 and the proposed damage index; see Figure 4. This format is similar to the one implemented for performance evaluation of new and existing steel moment-resisting structures in the FEMA standards 350 and 351, respectively [FEMA 2000a; 2000b]. The member damage index D_M is taken as the largest section damage index, along the member. This is a traditional and effective assumption in structural design; see [Kappos and Manafpour 2001].

Therefore,

$$D_M = \max(D_S). \tag{7}$$

To provide an overall damage index that is representative of the damage state of a complex structure, the member damage indices must be combined in a rational manner to reflect both the severity of the member damage and the geometric distribution of damage within the overall structure. Various weighted-average procedures have been proposed for combining the member damage indices into an overall damage index. Thus, for a structure composed of m members, the overall damage index, D_O, has the form

$$D_O = \left(\frac{\sum_{i=1}^{m} D_{M,i}^2 W_i}{\sum_{i=1}^{m} W_i} \right)^{1/2}, \tag{8}$$

where $D_{M,i}$ and W_i denote the damage and weighting factor of the i-th member. This expression is in agreement with the fact that the most damaged members affect the overall damage much more than the undamaged (elastic) members. Park and Ang [1985], assuming that the distribution of damage is correlated with the distribution of plastic strain energy dissipation, applied (8) with the weighting factors to correspond to the amount of plastic strain energy dissipation. Similar assumptions have been proposed elsewhere; e.g., in [Powell and Allahabadi 1988]. However, all these approaches are exclusively applied to seismic problems where the external loads have a cyclic form. It is evident that the amount of plastic strain energy dissipation is an inappropriate measure for static monotonic problems. For this reason, the overall damage index D_O is assumed here to be of the form [Cervera et al. 1995]

$$D_O = \left(\frac{\sum_{i=1}^{m} D_{M,i}^2 \Omega_i}{\sum_{i=1}^{m} \Omega_i} \right)^{1/2}, \tag{9}$$

where Ω_i denotes the volume of the i-th member. This relation reflects both the severity of the member damage and the geometric distribution of damage within the structure.

5. Global damage levels

5.1. ***Introduction.*** Damage is used here as a design criterion. Thus, the designer, in addition to a method for determining damage, also needs a scale of damage in order to decide which level of damage is acceptable for his design. Many damage scales can be proposed in order to select desired damage levels associated with the strength degradation and capacity of a structure to resist further loadings. Table 1 provides the three performance levels, immediate occupancy (IO), life safety (LS), and collapse prevention (CP), associated with modern performance-based seismic design with the corresponding limit response values (performance objectives) in terms of interstorey drift ratio (IDR), θ_{pl} (plastic rotation at member end), μ_θ (local ductility), and d (damage) as well as the relevant references. The selection of the appropriate damage level depends on various factors, such as the importance factor or the "weak beams – strong columns" rule in seismic design of structures. Thus, for example, nuclear power plants should be designed with zero damage and plane frames with 60% and 30% maximum damage in beams and columns, respectively. The proposed design method uses the damage level scale that has been derived

Index	Source	Performance level IO	LS	CP
IDR	[Leelataviwat et al. 1999]	1–2%	2–3%	3–4%
	[SEAOC 1999]	1.5%	3.2%	3.8%
	[Vasilopoulos and Beskos 2006]	0.5%	1.5%	3%
(transient)	[FEMA 1997]	0.7%	2.5%	5%
(permanent)	[FEMA 1997]	negligible	1%	5%
θ_{pl}/θ_y	[FEMA 1997]	≤ 1	≤ 6	≤ 8
μ_θ	[FEMA 1997]	2	7	9
damage	[Vasilopoulos and Beskos 2006]	$\leq 5\%$	$\leq 20\%$	$\leq 50\%$
	[ATC 1985]	0.1–10%	10–30%	30–60%

Table 1. Performance levels and corresponding limit response values given by several sources.

with the aid of extensive parametric studies on plane frames and corresponds to the three performance levels of the FEMA 273 code [FEMA 1997]. It should be noted that damage characterizations (such as minor and major) given by modern seismic codes are qualitative and very general, and hence inappropriate for use in practical design. In contrast to them, the proposed values of damage indices can be easily used in practical design.

The following subsections provide details concerning the parametric studies conducted herein for the derivation of simple expressions relating damage to the plastic hinge rotation of the member sections and the IDR of the plane steel frames considered to be used for the construction of a practical quantitative damage scale.

5.2. ***Frame geometry and loading.*** A set of 36 plane steel moment-resisting frames was employed for the parametric studies. These frames are regular and orthogonal with storey heights and bay widths equal to 3 m and 5 m, respectively. Furthermore, they are characterized by a number of storeys n_s with values 3, 6, 9, 12, 15, and 20 and a number of bays n_b with values 3 and 6. The frames were subjected to constant uniform vertical loads $1.35G + 1.5Q = 30$ kN/m and horizontal variable loads $1.35W$, where G, Q, and W correspond to dead, live, and wind loads, respectively. The material properties taken from structural steel grade S235, were divided by a factor of 1.10 for compatibility with EC3 provisions. The frames were designed in accordance with EC3 [2005] and EC8 [2004].

Data for the frames, including values for n_s, n_b, beam and column sections, and first and second natural periods, are presented in the table on the next two pages, taken from [Karavasilis et al. 2007].

5.3. ***Proposed global damage level values.*** The previously described plane steel frames were analyzed by the computer program DRAIN–2DX [Prakash et al. 1993]. Use was made of its beam-column element with two possible plastic hinges at its ends modeled by fibers. During the analyses, the vertical loads of the frames remained constant, while the horizontal ones were progressively increased in order to identify the damage corresponding to each performance level of Table 1. Damage was calculated at section and structural levels by using expressions (4), (7), and (9). In addition, the interstorey drift ratio and the plastic hinge rotation at the end of each member were computed. The latter was computed in the form

#	n_s	n_b	columns and beams (see caption on next page)	T_1/sec	T_2/sec
1	3	3	240-330(1-3)	0.73	0.26
2	3	3	260-330(1-3)	0.69	0.21
3	3	3	280-330(1-3)	0.65	0.19
4	3	6	240-330(1-3)	0.75	0.23
5	3	6	260-330(1-3)	0.70	0.21
6	3	6	280-330(1-3)	0.66	0.20
7	6	3	280-360(1-4) 260-330(5-6)	1.22	0.41
8	6	3	300-360(1-4) 280-330(5-6)	1.17	0.38
9	6	3	320-360(1-4) 300-330(5-6)	1.13	0.37
10	6	6	280-360(1-4) 260-330(5-6)	1.25	0.42
11	6	6	300-360(1-4) 280-330(5-6)	1.19	0.40
12	6	6	320-360(1-4) 300-330(5-6)	1.15	0.38
13	9	3	340-360(1) 340-400(2-5) 320-360(6-7) 300-330(8-9)	1.55	0.54
14	9	3	360-360(1) 360-400(2-5) 340-360(6-7) 320-330(8-9)	1.52	0.53
15	9	3	400-360(1) 400-400(2-5) 360-360(6-7) 340-330(8-9)	1.46	0.51
16	9	6	340-360(1) 340-400(2-5) 320-360(6-7) 300-330(8-9)	1.57	0.55
17	9	6	360-360(1) 360-400(2-5) 340-360(6-7) 320-330(8-9)	1.53	0.53
18	9	6	400-360(1) 400-400(2-5) 360-360(6-7) 340-330(8-9)	1.47	0.51
19	12	3	400-360(1) 400-400(2-3) 400-450(4-5) 360-400(6-7) 340-400(8-9) 340-360(10) 340-330(11-12)	1.90	0.66
20	12	3	450-360(1) 450-400(2-3) 450-450(4-5) 400-450(6-7) 360-400(8-9) 360-360(10) 360-330(11-12)	1.78	0.62
21	12	3	500-360(1) 500-400(2-3) 500-450(4-5) 450-450(6-7) 400-400(8-9) 400-360(10-11) 400-330(12)	1.72	0.60
22	12	6	400-360(1) 400-400(2-3) 400-450(4-5) 360-400(6-7) 340-400(8-9) 340-360(10) 340-330(11-12)	1.90	0.67
23	12	6	450-360(1) 450-400(2-3) 450-450(4-5) 400-450(6-7) 360-400(8-9) 360-360(10) 360-330(11-12)	1.78	0.63
24	12	6	500-360(1) 500-400(2-3) 500-450(4-5) 450-450(6-7) 400-400(8-9) 400-360(10-11) 400-330(12)	1.72	0.61
25	15	3	500-300(1) 500-400(2-3) 500-450(4-5) 450-400(6-7) 400-400(8-12) 400-360(13-14) 400-330(15)	2.29	0.78
26	15	3	550-300(1) 550-400(2-3) 550-450(4-5) 500-400(6-7) 450-400(8-12) 450-360(13-14) 450-330(15)	2.22	0.75
27	15	3	600-300(1) 600-400(2-3) 600-450(4-5) 550-450(6-7) 500-450(8-9) 500-400(10-12) 500-360(13-14) 500-330(15)	2.10	0.72
28	15	6	500-300(1) 500-400(2-3) 500-450(4-5) 450-400(6-7) 400-400(8-12) 400-360(13-14) 400-330(15)	2.30	0.78
29	15	6	550-300(1) 550-400(2-3) 550-450(4-5) 500-400(6-7) 450-400(8-12) 450-360(13-14) 450-330(15)	2.21	0.75
30	15	6	600-300(1) 600-400(2-3) 600-450(4-5) 550-450(6-7) 500-450(8-9) 500-400(10-12) 500-360(13-14) 500-330(15)	2.10	0.72

#	n_s	n_b	columns and beams (see caption)	T_1/s	T_2/s
31	20	3	600-300(1) 600-400(2-3) 600-450(4-5) 550-450(6-10) 500-450(11-13) 500-400(14-16) 450-400(17) 450-360(18-19) 450-330(20)	2.82	0.97
32	20	3	650-300(1) 650-400(2-3) 650-450(4-5) 600-450(6-10) 550-450(11-13) 550-400(14-16) 500-400(17) 500-360(18-19) 500-330(20)	2.76	0.94
33	20	3	700-300(1) 700-360(2) 700-400(3) 700-450(4-5) 650-450(6-10) 600-450(11-13) 600-400(14-16) 550-400(17) 550-360(18-19) 550-330(20)	2.73	0.93
34	20	6	600-300(1) 600-400(2-3) 600-450(4-5) 550-450(6-10) 500-450(11-13) 500-400(14-16) 450-400(17) 450-360(18-19) 450-330(20)	2.75	0.96
35	20	6	650-300(1) 650-400(2-3) 650-450(4-5) 600-450(6-10) 550-450(11-13) 550-400(14-16) 500-400(17) 500-360(18-19) 500-330(20)	2.70	0.93
36	20	6	700-300(1) 700-360(2) 700-400(3) 700-450(4-5) 650-450(6-10) 600-450(11-13) 600-400(14-16) 550-400(17) 550-360(18-19) 550-330(20)	2.67	0.92

Table 2. Steel moment-resisting frames considered in parametric studies. In the central column, the expression 240-330(1-3) means that the first three storeys have columns with HEB240 sections and beams with IPE330 sections. The numbers in parentheses always refer to a range of storeys or single storey.

θ_{pl}/θ_y, where θ_y is the rotation at yielding expressed in FEMA [1997] as

$$\theta_y = \frac{M_{pl} L}{6EI}, \tag{10}$$

where L is the member length, E is the modulus of elasticity of the material and I is the moment of inertia of the section. When members, such as columns, are subjected to an axial compressive force P, the right-hand side of (10) is multiplied by the factor $1-(P/P_y)$, where P_y is the axial yield force of the member.

This subsection presents the results of the parametric studies. Figure 5 shows the variation of the section damage index D_S versus the ratio θ_{pl}/θ_y for low-rise (3 and 6 storeys) and high-rise (9, 12, 15 and 20 storeys) frames, respectively. Figure 6 shows the variation of the overall damage index D_O versus IDR for low- and high-rise frames respectively. Using the method of least squares the mean values of these variations were determined and plotted as straight line segments in Figures 5–6. The analytical expressions of these lines are of the following form

For the low rise frames:

$$D_s = 12.526 \cdot \left(\frac{\theta_{pl}}{\theta_y}\right) \text{ for } \frac{\theta_{pl}}{\theta_y} \le 2.2 \qquad \text{and} \qquad D_s = 3.54 \cdot \left(\frac{\theta_{pl}}{\theta_y}\right) + 20.14 \text{ for } \frac{\theta_{pl}}{\theta_y} > 2.2 \tag{11}$$

$$D_O = 4.67 \cdot IDR. \tag{12}$$

For the high rise frames:

$$D_s = 2.42 \cdot \left(\frac{\theta_{pl}}{\theta_y}\right) \tag{13}$$

$$D_O = 0.94 \cdot IDR. \tag{14}$$

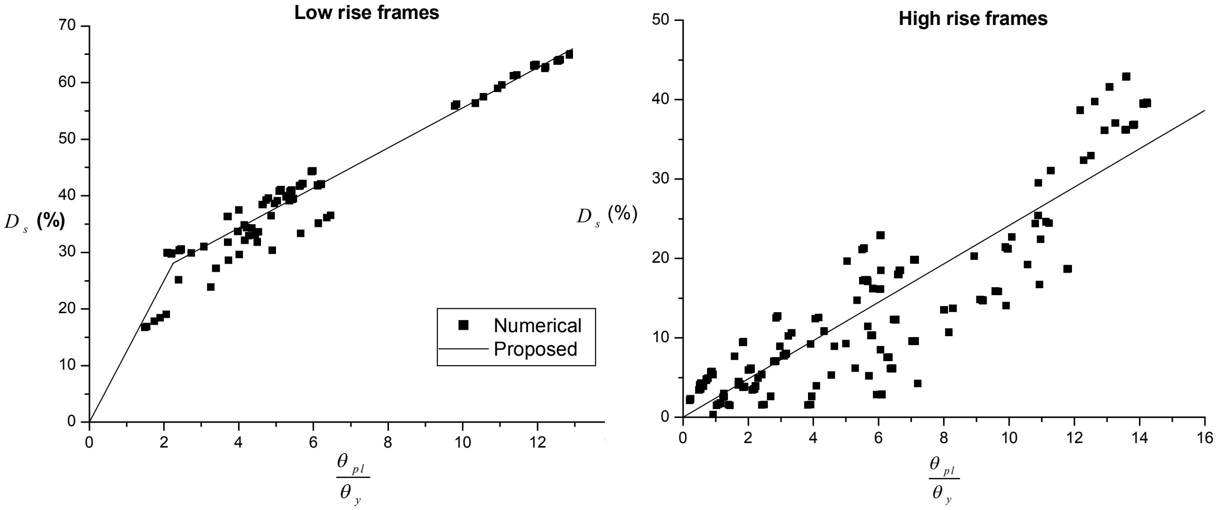

Figure 5. D_s versus θ_{pl}/θ_y curves for low- and high-rise frames.

The coefficient of determination R^2 in (11) and (13) is 0.96 and 0.79 respectively, showing that there is good correlation between the section damage and the plastic hinge rotation. On the contrary, the correlation between structure damage and the IDR is not so good as the coefficient of determination is 0.53 and 0.72 for (12) and (14), respectively.

Using the values of θ_{pl} and IDR given in FEMA [1997] for the three performance levels of Table 1 into (11)–(14), a section and overall damage scale is constructed for low- and high-rise frames and given in Table 4. The low values of damage in the high rise frames in that table can be explained by the instabilities caused in the analyses due to the concentration of damage in one or two sections and the P-δ and P-Δ effects. In the case of structural damage, this concentration combined with the definition of D_O in (9) explains these very small values. It is apparent from (9) that even if one has large values of

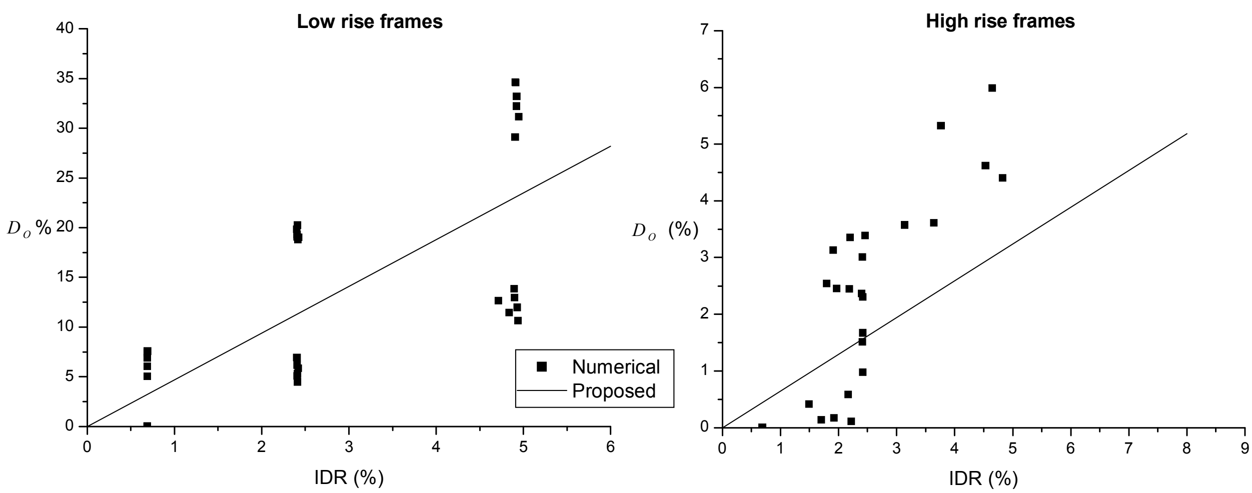

Figure 6. D_O versus IDR curves for low- and high-rise frames.

section damage in a few sections, the overall damage will have a small value because of the small or zero values in other sections. For this reason, the overall damage index is not considered as a representative one, and the section damage index is used in the applications.

6. Direct damage-controlled steel design

The application of the proposed DDCD method to plane steel members and framed steel structures is done with the aid of the DRAIN–2DX [Prakash et al. 1993] computer program, modified properly by the authors to perform both analysis and design. This program can statically analyze with the aid of the finite element method plane beam structures taking into account material and geometric nonlinearities. Material nonlinearities are accounted for through fiber modeling of plastic hinges in a concentrated plasticity theory (element 15 of DRAIN–2DX). Geometric nonlinearities include the P-δ effect (influence of axial force acting through displacements associated with member bending) and the P-Δ effect (influence of vertical load acting through lateral structural displacements), which are accounted for by utilizing the geometric stiffness matrix.

The beam-column section is subdivided in a user-defined number of steel fibers (Figure 7). Sensitivity studies have been undertaken to define the appropriate number of fibers for various types of sections. For example, for an I-section under axial force and uniaxial bending moment one can have satisfactory accuracy by dividing that section into 30 fibers (layers). Thus, for every structural steel member, selected sections are divided into steel fibers and the stress–strain relationship of (1) is used for tension and compression.

In the analysis, every member of the structure needs to be subdivided into several elements (usually three or four) along its length to model the inelastic behavior more accurately. The analysis leads to highly accurate results, but is, in general, computationally intensive for large and complex structures. Figure 8 shows the flow chart of the modified DRAIN–2DX for damage-controlled steel design.

Using this modified DRAIN–2DX, the user has three design options at his disposal in connection with damage-controlled steel design:

(i) determine damage for a given structure under given loading,

(ii) dimension a structure for given loading and given target damage, or

(iii) determine the maximum loading a given structure can sustain for a given target damage.

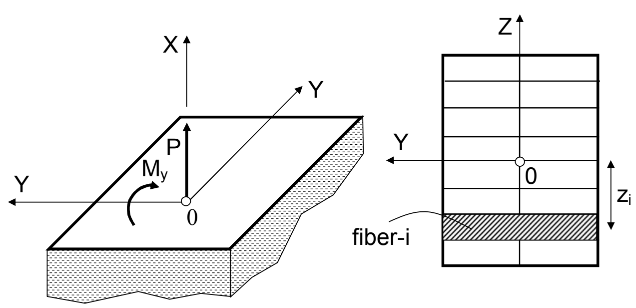

Figure 7. Fiber modeling of a general section.

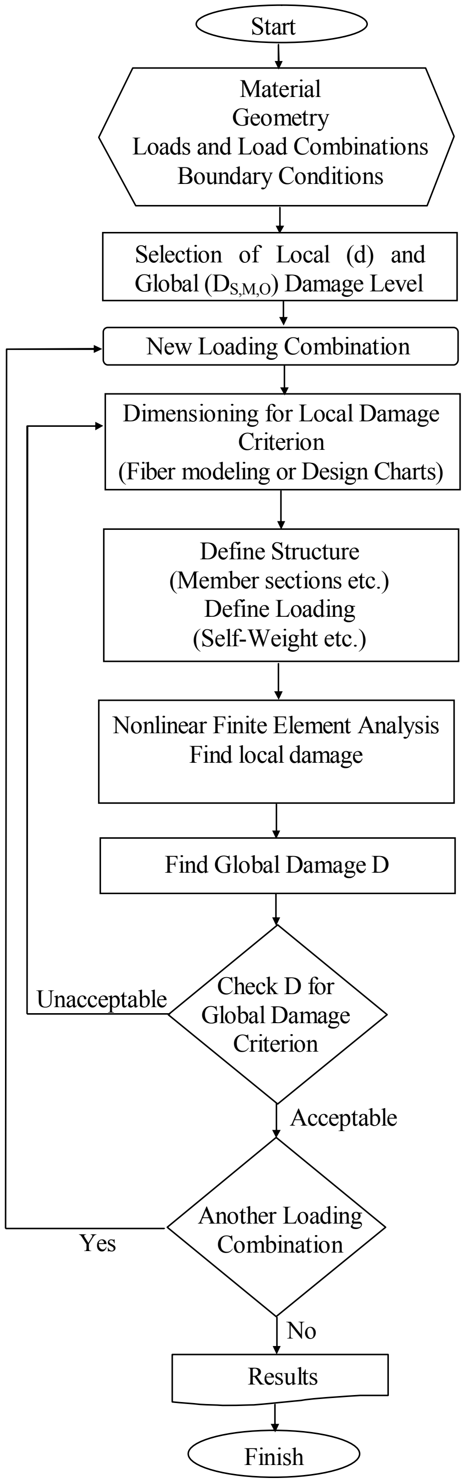

Figure 8. Flowchart of the modified program DRAIN–2DX [Prakash et al. 1993].

The first option is the one usually chosen in current practice. The other two options are the ones which actually make the proposed design method a direct damage-controlled one.

7. Examples of application

This section describes two numerical examples to illustrate the use of the proposed design method and demonstrate its advantages.

7.1. ***Static design of a plane steel frame.*** A plane two bay – two storey steel frame is examined in this example. Figure 9 shows the geometry and loading of the frame. Columns consist of standard HEB sections, while beams of standard IPE sections. The beams are subjected to uniform vertical loads $G = 15.0\,\text{kN/m}$ and $Q = 20.0\,\text{kN/m}$, where G and Q correspond to permanent and live loads, respectively. Additionally, the frame is subjected to horizontal wind loads $W = 12.6\,\text{kN}$ at the first floor level and $W = 22.2\,\text{kN/m}$ at the second. Steel is assumed to follow the material properties of steel grade S235 with trilinear stress-strain curve. Without loss of generality, only one loading combination of EC3 is examined here, that corresponding to $1.35(G + Q + W)$.

In the following, the frame is studied for the three design options of the proposed design method. Initially, the first design option, related to the determination of damage for a given structure and known loading, is examined. In this case, the structure is designed according to the EC3 method. In order to design this frame, four different member sections are determined, as shown in Figure 9: (a) columns of the first floor, (b) columns of the second floor, (c) beams of the first floor, and (d) beams of the second floor.

The most appropriate standard sections have been found to be those in Table 3. These sections have been obtained on the basis of a first order elastic analysis according to EC3. In order to determine the damage level, the structure is analyzed by the modified DRAIN–2DX program [Prakash et al. 1993], taking into account inelasticity and second order phenomena. The damage determined in all the members was found equal to zero (Table 3) indicating linear elastic behavior of the structure.

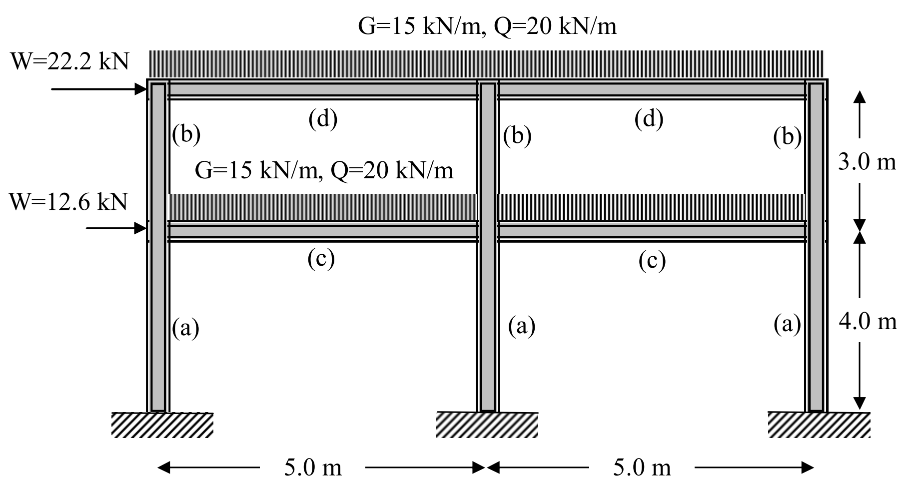

Figure 9. Geometry and loads for the frame of Section 7.1.

Member		EC3 Sections	EC3 Capacity ratio	EC3 Damage	Proposed method – DDCD Sections	Proposed method – DDCD Damage
columns	(a)	HEB-180	0.742	0.0%	HEB-160	0.0%
	(b)	HEB-140	0.821	0.0%	HEB-140	24.3%
beams	(c)	IPE-360	0.686	0.0%	IPE-240	73.7%
	(d)	IPE-330	0.842	0.0%	IPE-270	20.0%

Table 3. Design of two-dimensional frame for the structure of Figure 9.

The second design option has to do with member dimensioning for a preselected target damage level and known loading. Thus, using the modified DRAIN–2DX program, one can determine the most appropriate sections in order to have the selected target (maximum) damage at members, for the same loading combination as above. Two different damage levels are considered by setting the maximum member damage equal to 25% and 75% for columns and beams, respectively. The sections found appear in Table 3. For those sections, the computed values of maximum member damage D_S become 24.2% and 73.7% for columns and beams, very close from below to the preselected (target) values of 25% and 75%. It is evident that the acceptance of greater damage levels decreases the sizes of the sections.

Finally, the third design option associated with the determination of maximum loading for a given structure and preselected target damage is examined. Use is made again of the modified DRAIN–2DX program. The examined structure is assumed to consist of the standard sections obtained in the second design option (see Table 3). In this case, vertical (permanent and live) loads are assumed to remain the same. Thus, allowing maximum values of damage $D_S = 30\%$ and 0% for beams and columns, respectively, one can determine the maximum wind load. The allowable maximum wind load is found to be 11.5 and 20.2 kN for the first and second floor, respectively.

7.2. ***Seismic design of a plane steel frame by push-over.*** Consider an S235 plane steel moment-resisting frame of three bays and three storeys. The bay width is assumed to be 5 m and the storey height 3 m. The load combination $G + 0.3Q$ on beams is equal to 27.5 KN/m. HEB profiles are used for the columns and IPE profiles for the beams. The frame was designed according to EC3 [2005] and EC8 [2004] for a peak ground acceleration equal to 0.4 g, a soil class D and a behaviour factor $q = 4$ with the aid of the SAP2000 program [2005] in conjunction with the capacity design requirements of EC8. Thus, for a design base shear of 355 kN, the following column and beam sections were obtained for the three storeys: (HEB280-IPE360) + (HEB260-IPE330) + (HEB240-IPE300). The maximum elastic top floor displacement was found equal to 0.0465 m. Thus, according to EC8, the corresponding inelastic displacement will be $0.0465q = 0.186$ m, following the well known equal displacement rule.

The frame is subsequently analyzed using static inelastic push-over analysis with an inverted triangle type of profile of horizontal forces. The forces are progressively increased until the maximum inelastic displacement of the frame reaches the previously computed one of 0.186 m.

The damage distribution in the frame is shown in Figure 10. It is observed that plastic hinges are formed both in beams and columns, which implies that in reality the capacity design requirement is not satisfied. Damage values are up to about 47% in the beams and up to 26% in columns (44% at their

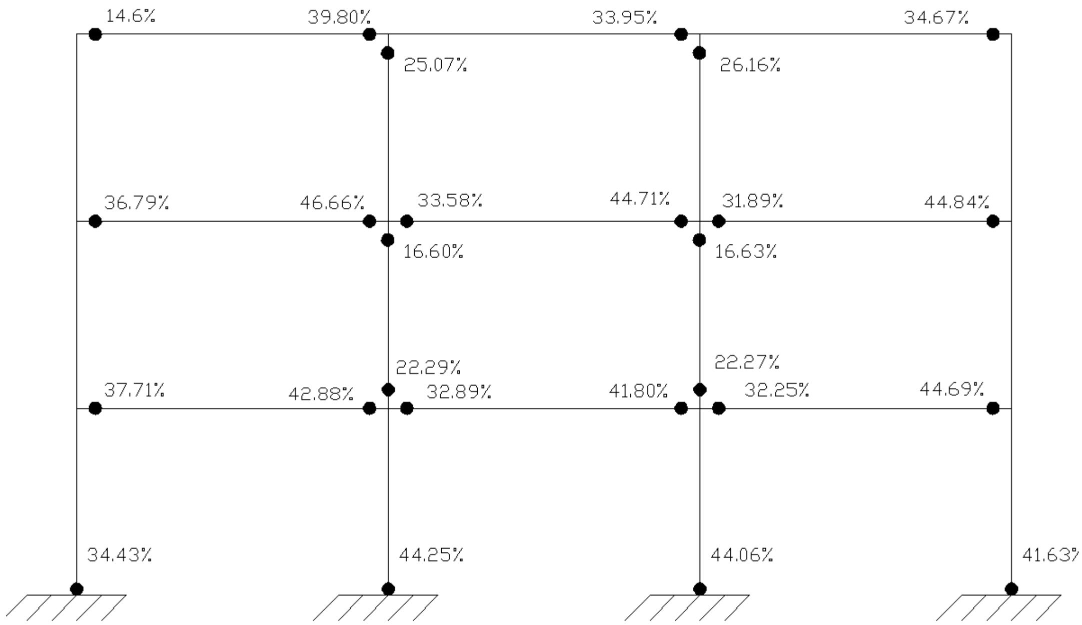

Figure 10. Damage distribution in the frame of Section 7.2 designed according to EC3 and EC8.

bases). The DDCD can overcome this drawback of formation of plastic hinges in the columns, because it can directly control damage and plastic hinge formation in the frame. Indeed, this frame is designed for the CP performance level of Table 4 by assuming target damage of 45% in the beams and 0% in all columns except those of the first floor where the target damage at their bases is 40%. For this target damage distribution and design base shear computed with the aid of the EC8 spectrum, the sections of the frame are obtained. For the resulting frame the push-over curve is used to determine the elastic displacement for the aforementioned base shear. This displacement is multiplied by q in order to find the maximum inelastic one and hence the corresponding base shear from the push-over curve. For this base shear the distribution of damage is obtained. If this distribution is in accordance with the target one, the selected sections are acceptable. Otherwise, the sections are changed and the previous procedure is repeated. Thus, for the damage distribution of Figure 11 with damage values up to about 44% in the beams and up to 37% in column bases, the column and beam sections for the three storeys of the frame were found to be (HEB300-IPE330) + (HEB300-IPE330) + (HEB280-IPE300). This selection results in a global collapse mechanism satisfying completely the capacity design requirement.

Performance level	Low rise frames		High rise frames	
	D_s	D_O	D_s	D_O
IO	$\leq 13\%$	$\leq 3\%$	$\leq 3\%$	$\leq 1\%$
LS	$\leq 40\%$	$\leq 12\%$	$\leq 15\%$	$\leq 2\%$
CP	$\leq 50\%$	$\leq 24\%$	$\leq 20\%$	$\leq 5\%$

Table 4. Performance levels and corresponding section and structural damage.

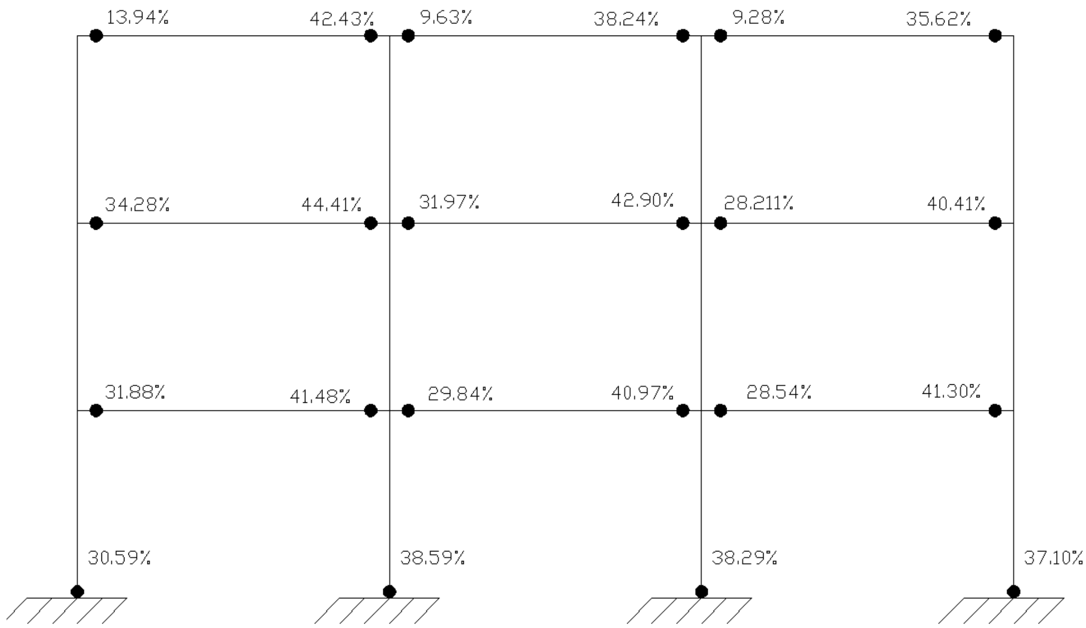

Figure 11. Damage distribution in the frame of Section 7.2 designed according to DDCD.

8. Conclusions

This paper introduced the direct damage-controlled design (DDCD) method for structural steel design. The method

- works with the aid of the finite element method incorporating material and geometric nonlinearities, a continuum mechanics definition of damage and a damage scale derived on the basis of extensive parametric studies;
- allows the designer to either determine the damage level for a given structure and known loading, or dimension a structure for a target damage level and known loading, or determine the maximum loading for a given structure and a target damage level;
- can also be used for the case of seismic loading in the framework of the static inelastic (push-over) analysis providing a reliable way for achieving seismic capacity design.

References

[AISC 1998] "Load and resistance factor design: structural members, specifications and codes", 2nd revision of the 2nd ed., American Institute of Steel Construction, Chicago, 1998.

[AISC 2005] "Seismic provisions for structural steel buildings", standard AISC 341-05, American Institute of Steel Construction, Chicago, 2005.

[ATC 1985] "Earthquake damage evaluation data for California", standard ATC-13, Applied Technology Council, Redwood City, CA, 1985.

[Benavent-Climent 2007] A. Benavent-Climent, "An energy-based damage model for seismic response of steel structures", *Earthquake Eng. Struct. Dyn.* **36**:8 (2007), 1049–1064.

[Cervera et al. 1995] M. Cervera, J. Oliver, and R. Faria, "Seismic evaluation of concrete dams via continuum damage models", *Earthquake Eng. Struct. Dyn.* **24**:9 (1995), 1225–1245.

[Chen and Kim 1997] W. F. Chen and S. E. Kim, *LRFD steel design using advanced analysis*, CRC Press, Boca Raton, FL, 1997.

[DiPasquale and Cakmak 1990] E. DiPasquale and A. S. Cakmak, "Detection of seismic structural damage using parameter-based global damage indices", *Probab. Eng. Mech.* **5**:2 (1990), 60–65.

[EC3 2005] "Eurocode 3: Design of steel structures, part 1-1: general rules for buildings", standard EN 1993-1-1, European Committee on Standardization (CEN), Brussels, 2005.

[EC8 2004] European Committee for Standardization (CEN), "Eurocode 8: design of structures for earthquake resistance, part 1: general rules, seismic actions and rules for buildings", standard EN 1998-1, Brussels, 2004.

[FEMA 1997] "NEHRP guidelines for the seismic rehabilitation of buildings", standard FEMA-273, Building Seismic Safety Council, Federal Emergency Management Agency (FEMA), Washington, D.C., 1997, Available at http://www.wbdg.org/ccb/FEMA/ARCHIVES/fema273.pdf.

[FEMA 2000a] FEMA, "Recommended seismic design criteria for new steel moment-frame buildings", standard FEMA-350, Federal Emergency Management Agency (FEMA), Sacramento, CA, 2000, Available at http://www.fema.gov/plan/prevent/earthquake/pdf/fema-350.pdf.

[FEMA 2000b] FEMA, "Recommended seismic evaluation and upgrade criteria for existing welding steel moment-frame buildings", standard FEMA-351, Federal Emergency Management Agency (FEMA), Washington, D.C., 2000, Available at http://www.fema.gov/plan/prevent/earthquake/pdf/fema-351.pdf.

[FEMA 2001] "HAZUS99 User's manual", Service Release 2, Federal Emergency Management Agency (FEMA), Washington, D.C., 2001.

[Florez-Lopez 1998] J. Florez-Lopez, "Frame analysis and continuum damage mechanics", *Eur. J. Mech. A Solids* **17**:2 (1998), 269–283.

[Ghobarah et al. 1999] A. Ghobarah, H. Abou-Elfath, and A. Biddah, "Response-based damage assessment of structures", *Earthquake Eng. Struct. Dyn.* **28**:1 (1999), 79–104.

[Gioncu and Mazzolani 2002] V. Gioncu and F. M. Mazzolani, *Ductility of seismic resistant steel structures*, Spon Press, London, 2002.

[Hanganu et al. 2002] A. D. Hanganu, E. Onate, and A. H. Barbat, "A finite element methodology for local/global damage evaluation in civil engineering structures", *Comput. Struct.* **80**:20-21 (2002), 1667–1687.

[Hatzigeorgiou and Beskos 2007] G. D. Hatzigeorgiou and D. E. Beskos, "Direct damage-controlled design of concrete structures", *J. Struct. Eng. (ASCE)* **133**:2 (2007), 205–215.

[Hatzigeorgiou et al. 2001] G. D. Hatzigeorgiou, D. E. Beskos, D. D. Theodorakopoulos, and M. Sfakianakis, "A simple concrete damage model for dynamic FEM applications", *Int. J. Comput. Eng. Sci.* **2**:2 (2001), 267–286.

[Kappos and Manafpour 2001] A. J. Kappos and A. Manafpour, "Seismic design of R/C buildings with the aid of advanced analytical techniques", *Eng. Struct.* **23**:4 (2001), 319–332.

[Karavasilis et al. 2007] T. L. Karavasilis, N. Bazeos, and D. E. Beskos, "Behavior factor for performance-based seismic design of plane steel moment resisting frames", *J. Earthq. Eng.* **11**:4 (2007), 531–559.

[Leelataviwat et al. 1999] S. Leelataviwat, S. C. Goel, and B. Stojadinović, "Toward performance-based seismic design of structures", *Earthquake Spectra* **15**:3 (1999), 435–461.

[Lemaitre 1992] J. Lemaitre, *A course on damage mechanics*, Springer-Verlag, Berlin, 1992.

[Park and Ang 1985] Y. J. Park and A. H. S. Ang, "Mechanistic seismic damage model for reinforced concrete", *J. Struct. Eng. (ASCE)* **111**:4 (1985), 722–739.

[Powell and Allahabadi 1988] G. H. Powell and R. Allahabadi, "Seismic damage prediction by deterministic methods: concepts and procedures", *Earthquake Eng. Struct. Dyn.* **16**:5 (1988), 719–734.

[Prakash et al. 1993] V. Prakash, G. H. Powell, and S. Campbell, *DRAIN-2DX base program description and user guide*, version 1.10, University of California, Berkeley, CA, 1993.

[Priestley et al. 2007] M. J. N. Priestley, G. M. Calvi, and M. J. Kowalsky, *Displacement-based seismic design of structures*, IUSS Press, Pavia, Italy, 2007.

[SAP2000 2005] *SAP2000: Static and dynamic finite element analysis of structures*, version 9.1.4, Computers and Structures, Inc., Berkeley, CA, 2005.

[SEAOC 1999] “Recommended lateral force requirements and commentary”, known as the *SEAOC Blue Book*, 7th ed., Seismology Committee, Structural Engineers Association of California, Sacramento, CA, 1999.

[Vasilopoulos and Beskos 2006] A. A. Vasilopoulos and D. E. Beskos, “Seismic design of plane steel frames using advanced methods of analysis”, *Soil Dyn. Earthq. Eng.* **26**:12 (2006), 1077–1100. Corrigendum in **27**:2 (2007), 189.

[Vasilopoulos and Beskos 2009] A. A. Vasilopoulos and D. E. Beskos, “Seismic design of space steel frames using advanced methods of analysis”, *Soil Dyn. Earthq. Eng.* **29**:1 (2009), 194–218.

Received 8 Dec 2008. Accepted 13 Mar 2009.

GEORGE S. KAMARIS: kamaris@upatras.gr
Department of Civil Engineering, University of Patras, 26500 Patras, Greece

GEORGE D. HATZIGEORGIOU: gchatzig@env.duth.gr
Department of Environmental Engineering, Democritus University of Thrace, 67100 Xanthi, Greece

DIMITRI E. BESKOS: d.e.beskos@upatras.gr
Department of Civil Engineering, University of Patras, 26500 Patras, Greece