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On the seismic behavior of steel buildings, designed according to Eurocode 8 provisions, subjected to near-fault or to long duration seismic motions

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Abstract

An assessment study of the seismic behavior of steel buildings, including their foundation system, designed according to Eurocode 8 provisions is presented. This assessment study is performed for two types of seismic motions that are not explicitly addressed in the context of Eurocode 8, i.e., the near-fault and the long duration seismic motions. In particular, by means of non-linear time-history analyses and taking into account soil-structure interaction (SSI) effects, seismic response results of the steel buildings are obtained. These seismic response results involve the maximum values for the residual interstorey drift ratio of the steel buildings as well as for the permanent settlement and tilting of their foundation system. It is concluded that when subjected to the aforementioned kinds of seismic motions, steel buildings, designed according to Eurocode 8 provisions, exhibit in the majority of cases unacceptable seismic behaviour no matter if the SSI effects are included or not. On the other hand, the seismic behaviour of their foundation system is always acceptable.

Key words: steel buildings, Eurocode 8, near-fault seismic motions, long duration seismic motions, soil-structure interaction

1. Introduction

Current version of Eurocode 8 [1] does not include a specific design procedure for steel buildings subjected either to near-fault or to long duration seismic motions. Near-fault seismic motions have been repeatedly reported in literature to impose large inelastic seismic demands to steel buildings, whereas long duration seismic motions are mainly associated with fatigue and deterioration/degradation phenomena to the structural members and connections of steel buildings. It is also known that for both these types of seismic motions, soil-structure interaction (SSI) effects may also be significant because they can alter the force and displacement distribution induced to a steel building and its foundation. SSI effects to steel buildings are usually taken account only for the case of very soft soil [1].

This paper discusses the overall seismic behavior of 2-, 5- and 8-storey steel buildings, designed according to Eurocode 8 [1, 2] provisions, when they are subjected to near-fault or to long duration seismic motions. This overall behaviour is assessed on the basis of seismic response results with respect to both the steel building and its foundation. These response results are obtained by nonlinear time-history analyses and include the maximum values for: i) the residual interstorey drift ratio (RIDR) of the steel buildings and ii) the residual settlement and tilting of their foundation. In order to evaluate the potential effects of SSI to seismic response, the steel buildings are assumed to be founded on a soil of class B, C or D, following the soil classification of Eurocode 8 [1]. The overall seismic behavior of the 2-, 5- and 8-storey steel buildings, when subjected to near-fault or to long duration seismic motions, is considered to be acceptable or unacceptable on the basis of satisfaction or not of specific seismic performance criteria.

It is concluded that the seismic behavior of the steel buildings under study is most likely unacceptable for the case of near-fault seismic motions, whereas it exhibits a mixed pattern for the case of long duration seismic motions. On the other hand, the seismic behavior of the foundation of the steel buildings under study is always acceptable for both types of seismic motions.

2. Steel buildings, SSI effects and seismic motions

The 2-, 5- and 8-storey steel buildings under study, comprised by moment-resisting and concentrically braced frames (dual MRF-CBF), are shown in Fig. 1. They are regular in plan (in both orthogonal directions there are three bays of 6.0 m span each) and elevation (each storey has 3.0 m height). The orientation of columns for the steel buildings of Fig. 1 is shown in Fig. 2. At each floor level a rigid composite slab is considered and the values assumed for dead and live loads are 8 kN/m^2 and 3 kN/m^2, respectively. The yield strength of columns is 355 MPa, whereas that of beams and braces is 235 MPa and 275 MPa, respectively. All connections between main beams and columns are designed as moment-resisting ones, assigning to them an overstrength in order to ensure the formation of the plastic hinge to the beam. Simple shear connections are assigned to the ends of the secondary floor beams and pinned connections to the ends of the braces. The braces intersect at their mid length and are simulated as fixed in plane direction and pinned in out of plane direction. The design spectrum of Eurocode 8 [1] for an assumed peak ground acceleration (PGA = 0.36g), behavior (reduction) factor equal to 3 and soil class B, is considered for the calculation of the design seismic load. The sections of steel members obtained following the design requirements of Eurocodes 3 [3] and 8 [1] are shown in Table 1.

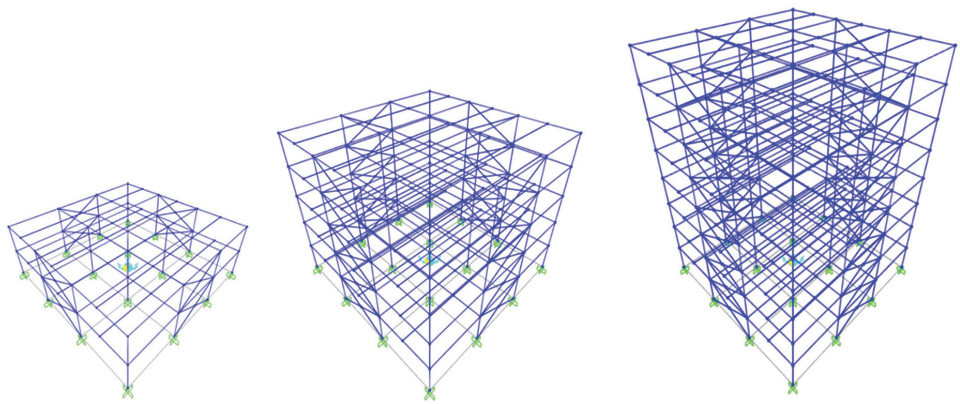

Figure 1. 2-, 5- and 8-storey steel buildings

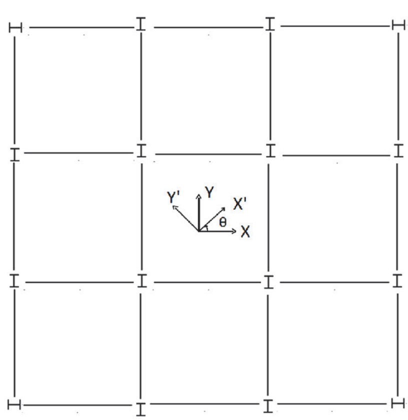

Figure 2. Orientation of columns for the 2-, 5- and 8-storey steel buildings

Table 1. Sections of steel beams, braces and columns

Steel structure	Beams	Braces	Columns
2-storey	IPE 450	CHS 219.1 x 5.0	HEM 320
5-storey	IPE 500	CHS 273.0 x 5.6	HEM 600
8-storey	IPE 500	CHS 355.6 x 6.3	HEM 700

For the 2-, 5- and 8-storey steel buildings of Fig. 1, the foundation type selected is that of a rigid mat having an area 20 x 20 m^2 and thickness of 0.3 m, 0.6 m and 0.8 m, respectively. The rigid mat foundations have been designed according to Eurocode 8 [2]. SSI is now introduced to the numerical model of the steel buildings of Fig.1 upon the assumption that the rigid mat foundation is constructed on a soil of class B, C or D according to the soil classification adopted in [1]. Since SSI effects are expected to be negligible for soil class B and of importance for soil classes C and D, it is considered that fixed base conditions correspond to soil class B, whereas compliant base conditions correspond to soil classes C and D.
To simulate the effects of SSI for the cases of soil classes C and D, the three-dimensional discrete mass-stiffness-damping model of [4] is utilized. More specifically, a set of masses-dashpots-springs is assumed to act at the centre of the bottom area of the rigid mat and it is assigned to all its six modes of vibration. To calculate the values of mass-damping-stiffness parameters associated with each mode of vibration (i.e. two horizontal, one vertical, two rocking and one torsional) of the rigid mat foundation, Table 2, taken from [4], is provided. In this table, ν, G, V_s are the Poisson's ratio, shear modulus and shear wave velocity, respectively, of the soil medium, m and m_v are the mass of the foundation and a virtual soil mass, respectively. The values of G, V_s and ρ assumed for soil classes C and D are shown in Table 3. It should be recalled that in order to take into account the non-linear soil deformations for soil classes C and D due to relative large values of ground acceleration, the effective values of G, i.e., those corresponding to a reduction of 84 % (most unfavorable case) from its initial value [2], are finally employed. Parameter in Table 2 is the half-length of a square foundation.

Table 2. Formulae for mass, springs and dashpots

	Mass (inertia) ratio, β	Equivalent radius, r_0	Virtual soil mass (inertia), m_v	Static stiffness K	Damping C
Vertical	$\frac{(1-\nu)}{4}\frac{m}{\rho r_0^3}$	$\frac{2a}{\sqrt{\pi}}$	$\frac{0.27m}{\beta}$	$\frac{4.7Ga}{1-\nu}$	$\frac{0.8a}{V_s}K$
Horizontal	$\frac{(7-8\nu)}{32(1-\nu)}\frac{m}{\rho r_0^3}$	$\frac{2a}{\sqrt{\pi}}$	$\frac{0.095m}{\beta}$	$\frac{9.2Ga}{1-\nu}$	$\frac{0.163a}{V_s}K$
Rocking	$\frac{3(1-\nu)}{8}\frac{m}{\rho r_0^5}$	$\frac{2a}{\sqrt[4]{3\pi}}$	$\frac{0.24m}{\beta}$	$\frac{4.0G\alpha^3}{1-\nu}$	$\frac{0.6a}{V_s}K$
Torsion	$\frac{m}{\rho r_0^5}$	$\frac{2a}{\sqrt[4]{3\pi}}$	$\frac{0.045m}{\beta}$	$8.31G\alpha^3$	$\frac{0.127a}{V_s}K$

Table 3. Shear modulus, shear velocity and density assumed for soil classes C and D

Soil Class	G_{eff}	V_s	ρ
C	$0.16G$	270 m/sec	1.8 Mg/m^3
D	$0.16G$	180 m/sec	1.9 Mg/m^3

The design of steel buildings of Fig. 1 with SSI included (via the set of masses-dashpots-springs presented above) is performed using the design spectrum of Eurocode 8 [1] (for PGA equal to 0.36g, and behaviour factor equal to 3) that corresponds first to soil class C and then to soil class D. The sections of steel members finally obtained for these soil class cases are the same with those given in Table 1 for soil class B (fixed base conditions) even though the stress ratios calculated from the interaction (member design) equations of [3] are different for each one of the soil classes considered. The only exception is the assignment of HEM700 columns to the 5-storey steel building for soil classes C and D.

The set of near-fault and long duration accelerograms used for the non-linear time-history analyses of the steel buildings of Fig. 1 are shown in Table 4. In this table, some details associated with the recorded accelerograms, i.e., the earthquake name, location, year and moment magnitude, are also provided. The two, as recorded, horizontal components of these accelerograms are applied to the structural axes of Fig. 2 with varying angle of seismic incidence θ, i.e., 0°, 45° and 90°. Thus, the number of nonlinear time-history analyses performed for each one of the steel buildings of Fig. 1 is 189 (21 accelerograms x 3 values of θ x 3 base-soil conditions).

Table 4. Near-fault accelerograms considered

No.	Earthquake, Location, Year	Recording station	M_w
1.	San Fernando, California, 1971	Pacoima Dam	6.6
2.	Superstition Hills, California, 1987	Parachute Test Site	7.3
3.	Loma Prieta, California, 1989	Los Gatos	6.5
4.	Cape Mendocino, Alaska, 1992	Petrolia	7.0
5.	Landers, California, 1992	Lucerne Valley	7.3
6.	Northridge, California, 1994	Rinaldi Receiving St.	6.7
7.	Northridge, California, 1994	Newhall	6.7
8.	Northridge, California, 1994	Sylmar Converter St.	6.7
9.	Kobe, Japan, 1995	Takatori	6.9
10.	Christchurch, New Zealand, 2011	Resthaven	6.3

Table 5. Long duration accelerograms considered

No.	Earthquake, Location, Year	Recording station	M_w
1.	Valparaiso, Chile, 1985	Llolleo	7.9
2.	Michoachan, Mexico, 1985	SCT	8.0
3.	El Salvador, El Salvador, 2001	Observatorio	7.6
4.	El Salvador, El Salvador, 2001	Santa Tecla	7.6
5.	Denali, Alaska, 2002	Taps Pump Station 10	7.9
6.	Ica Pisca, Peru, 2007	ICA2	8.0
7.	Maule, Chile, 2010	Angol	8.8
8.	Maule, Chile, 2010	Constitution	8.8
9.	Tohoku-Oki, Japan, 2011	Hirono	9.0
10.	Tohoku-Oki, Japan, 2011	Sendai	9.0
11.	Tohoku-Oki, Japan, 2011	Tsukidate	9.0

The non-linear time-history analyses are conducted in SAP 2000 [5], considering both material and geometrical nonlinearities. Beams and columns are modelled as standard frame elements assuming a bilinear hysteresis model and concentrated plastic hinges at their ends. For the case of columns, plastic hinges are formed as a result of the interaction between axial load and biaxial bending while for the case of beams, plastic hinges are formed as a result of uniaxial bending only. For the case of long duration accelerograms, in order to account for the stiffness degradation of columns and beams, monotonic backbone curves, e.g., those proposed by Lignos et al. [6] are employed. The steel braces are modelled as truss elements to which concentrated plastic hinges with isotropic strain hardening are assigned to their ends and at their intersection. The limits of permissible plastic rotations of the plastic hinges formed in beams, columns and braces are those defined in ASCE 41-17 [7] for specific seismic performance levels.
The innate viscous damping is considered 3 % for the first and the last mode significant mode of the response. The non-linear time-history analyses for the steel buildings of Fig.1 are initially performed assuming fixed base conditions and then assuming a compliant ground (soil classes C or D) where SSI has to be taken into account. The inclusion of SSI in non-linear time-history analyses is obtained with the aid of the 'link element' of SAP 2000 [5] which can effectively reproduce the set of masses-dashpots-springs presented above.

3 Assessment of seismic behavior

To assess the seismic behavior of the steel buildings of Fig. 1 for the cases of the seismic motions of Tables 4-5, the satisfaction or not of the following criteria is checked: i) the plastic hinge rotations of the bottom storey columns and of all storey beams are lower than the life-safety level [7] and there is no formation of a soft-storey mechanism; ii) the maximum value computed for the RIDR does not surpass the threshold value of 0.5

% [8]; iii) yielding of the braces takes place first and in line with the design principles. Brace fracture is an anticipated failure and it is traced by computing the plastic hinge (brace end) rotation and comparing it with the one provided by the empirical formula of [9] in which an upper bound of 0.25 rad is set; iv) the permissible level of deformation associated with the rigid mat foundation, i.e., its residual settlement δ and tilting ω, is defined by the moderate damage limits of [10]. The number of cases in which one or more of the above-mentioned criteria is violated, are considered as failures of the steel buildings studied.

3.1 2-storey steel buildings

Table 6 reveals the number of failures for the 2-storey steel buildings when subjected to the near-fault seismic motions of Table 4. It is observed that the fixed base steel buildings fail in total to 13 out of 30 near-fault seismic motions and θ combinations, whereas the steel buildings founded on compliant ground, i.e., on soil types C and D, do not fail. The mat foundation of the 2-storey steel buildings on compliant ground exhibits no damage. The maximum results found for settlement δ and tilting ω of the mat foundation are $\delta = 6.6 \cdot 10^{-3}$ (soil type C), $\delta = 1.56 \cdot 10^{-2}$ (soil type D), $\omega = 4.61 \cdot 10^{-4}$ (soil type C) and $\omega = 8.77 \cdot 10^{-4}$ (soil type D).

Table 6. Number of failures for the 2-storey steel buildings under near-fault seismic motions.

Steel structure-foundation, θ	Number of failures Steel building	Number of failures Foundation
2-storey, fixed, 0°	4/10	-
2-storey, fixed, 45°	4/10	-
2-storey, fixed, 90°	5/10	-
2-storey, soil type C, 0°	0/10	0/10
2-storey, soil type C, 45°	0/10	0/10
2-storey, soil type C, 90°	0/10	0/10
2-storey, soil type D, 0°	0/10	0/10
2-storey, soil type D, 45°	0/10	0/10
2-storey, soil type D, 90°	0/10	0/10

Table 7 reveals the number of failures for the 2-storey steel buildings when subjected to the long duration seismic motions of Table 5. It is observed that the fixed base steel buildings fail in total to 3 out of 33 long duration seismic motions and θ combinations, whereas the steel buildings founded on compliant ground, i.e., on soil types C and D, do not fail. It should be also noted that in 9 out of 33 cases, the fixed base steel buildings respond elastically, whereas the steel buildings founded on soil classes C and D exhibit elastic response in only 2 out of 66 cases. For the no failure cases, the maximum value computed for the brace end rotation is 0.051 rad indicating no fracture in view of the maximum permissible value of 0.133 rad obtained using the formula of [9]. This value of 0.133 rad is surpassed only in the 3 aforementioned failure cases. The mat foundation of the 2-storey steel building on compliant ground exhibits no damage. The maximum results found for settlement δ and tilting ω of the mat foundation are $\delta = 6.6 \cdot 10^{-3}$ (soil type C), $\delta = 1.57 \cdot 10^{-2}$ (soil type D), $\omega = 5.4 \cdot 10^{-4}$ (soil type C) and $\omega = 1.08 \cdot 10^{-3}$ (soil type D).

Table 7. Number of failures for the 2-storey steel buildings under long duration seismic motions

Steel structure-foundation, θ	Number of failures Steel building	Number of failures Foundation
2-storey, fixed, 0°	1/11	-
2-storey, fixed, 45°	1/11	-
2-storey, fixed, 90°	1/11	-
2-storey, soil type C, 0°	0/11	0/11
2-storey, soil type C, 45°	0/11	0/11
2-storey, soil type C, 90°	0/11	0/11
2-storey, soil type D, 0°	0/11	0/11
2-storey, soil type D, 45°	0/11	0/11
2-storey, soil type D, 90°	0/11	0/11

3.2 5-storey steel buildings

Table 8 reveals the number of failures for the 5-storey steel buildings when subjected to the near-fault seismic motions of Table 4. It is observed that out of 30 near-fault seismic motions and θ combinations, the fixed base steel structures fail to 27, whereas the steel buildings founded on compliant ground fail to 17 and 21, for soil categories C and D, respectively. The mat foundation of the 5-storey steel buildings on compliant ground exhibits no damage. The maximum results found for settlement δ and tilting ω of the mat foundation are $\delta = 1.5 \cdot 10^{-2}$ (soil type C), $\delta = 3.7 \cdot 10^{-2}$ (soil type D), $\omega = 1.05 \cdot 10^{-3}$ (soil type C) and $\omega = 2.25 \cdot 10^{-3}$ (soil type D).

Table 8. Number of failures for the 5-storey steel buildings under near-fault seismic motions

Steel structure-foundation, θ	Number of failures Steel building	Number of failures Foundation
5-storey, fixed, 0°	9/10	-
5-storey, fixed, 45°	9/10	-
5-storey, fixed, 90°	9/10	-
5-storey, soil type C, 0°	5/10	0/10
5-storey, soil type C, 45°	6/10	0/10
5-storey, soil type C, 90°	6/10	0/10
5-storey, soil type D, 0°	7/10	0/10
5-storey, soil type D, 45°	7/10	0/10
5-storey, soil type D, 90°	7/10	0/10

Table 9. Number of failures for the 5-storey steel buildings under long duration seismic motions

Steel structure-foundation, θ	Number of failures Steel building	Number of failures Foundation
5-storey, fixed, 0°	0/11	-
5-storey, fixed, 45°	1/11	-
5-storey, fixed, 90°	0/11	-
5-storey, soil type C, 0°	7/11	0/11
5-storey, soil type C, 45°	7/11	0/11
5-storey, soil type C, 90°	7/11	0/11
5-storey, soil type D, 0°	7/11	0/11
5-storey, soil type D, 45°	7/11	0/11
5-storey, soil type D, 90°	7/11	0/11

The total number of failure cases for the 5-storey steel buildings when subjected to the long duration earthquakes of Table 5, including the 3 values considered for the angle of seismic incidence θ, are presented in Table 9. According to this table, the fixed base steel buildings fail in total to 1 out of 33 cases studied, whereas the steel buildings founded on soil classes C and D, fail to 42 out of 66 cases studied. Moreover, in 3 out of 33 cases, the fixed base steel buildings respond elastically, whereas the steel buildings founded on soil classes C or D never exhibit an elastic response. For the no failure cases, the maximum value computed for the brace end rotation is 0.11rad indicating no fracture in view of the maximum permissible value of 0.122 rad obtained using the formula of [9]. It is stressed that in all 42 failure cases identified, at least four braces fractured, i.e., the brace end rotation surpassed the maximum permissible value of 0.122 rad. The maximum values computed for residual settlement δ and tilting ω of the mat foundation are $\delta = 1.56 \cdot 10^{-2}$ (soil type C), $\delta = 3.75 \cdot 10^{-2}$ (soil type D), $\omega = 9.6 \cdot 10^{-4}$ (soil type C) and $\omega = 2.02 \cdot 10^{-3}$ (soil type D). These values indicate that the mat foundation exhibits no damage.

3.3 8-storey steel buildings

Table 10 reveals the number of failures for the 8-storey steel buildings when subjected to the near-fault seismic motions of Table 4. It is observed that out of the 30 near-fault seismic motions - θ combinations considered for each base condition, the fixed base steel buildings fail to 28, whereas the steel buildings on compliant base fail to 18 and 19, for soil categories C and D, respectively. Nevertheless, the mat foundation of the 8-storey steel buildings on compliant ground, exhibits no damage. The maximum results found for settlement δ and tilting ω of the mat foundation are $\delta = 2.8 \cdot 10^{-2}$ (soil type C), $\delta = 6.1 \cdot 10^{-2}$ (soil type D), $\omega = 1.43 \cdot 10^{-3}$ (soil type C) and $\omega = 3.16 \cdot 10^{-3}$ (soil type D).

Table 10. Number of failures for the 8-storey steel buildings under near-fault seismic motions

Steel structure-foundation, θ	Number of failures Steel building	Number of failures Foundation
8-storey, fixed, 0°	9/10	-
8-storey, fixed, 45°	9/10	-
8-storey, fixed, 90°	10/10	-
8-storey, soil type C, 0°	6/10	0/10
8-storey, soil type C, 45°	6/10	0/10
8-storey, soil type C, 90°	6/10	0/10
8-storey, soil type D, 0°	5/10	0/10
8-storey, soil type D, 45°	7/10	0/10
8-storey, soil type D, 90°	7/10	0/10

The total number of failure cases for the 8-storey steel building-foundation systems under the action of the 11 long duration seismic motions of Table 5, including the 3 values considered for the angle of seismic incidence θ, are presented in Table 11. According to the results shown in this table, the fixed base steel buildings fail in total to 24 out of 33 cases studied, whereas the steel buildings founded on soil classes C and D, fail to 54 out of 66 cases studied. For the no failure cases, the maximum value computed for the brace end rotation is 0.067rad indicating no fracture in view of the maximum permissible value of 0.109rad obtained using the formula of [9]. It should be stressed that in all 54 failure cases identified, at least eight braces fractured, i.e., the brace end rotation surpassed the maximum permissible value of 0.109rad. The maximum values computed for residual settlement δ and tilting ω of the mat foundation are $\delta = 2.8 \cdot 10^{-2}$ (soil type C), $\delta = 6.6 \cdot 10^{-2}$ (soil type D), $\omega = 1.92 \cdot 10^{-3}$ (soil type C) and $\omega = 3.91 \cdot 10^{-3}$ (soil type D). In view of these values for δ and ω, the mat foundation exhibits no damage.

Table 11. Number of failures for the 8-storey steel buildings under long duration seismic motions

Steel structure-foundation, θ	Number of failures Steel building	Number of failures Foundation
8-storey, fixed, 0°	8/11	-
8-storey, fixed, 45°	8/11	-
8-storey, fixed, 90°	8/11	-
8-storey, soil type C, 0°	9/11	0/11
8-storey, soil type C, 45°	9/11	0/11
8-storey, soil type C, 90°	9/11	0/11
8-storey, soil type D, 0°	9/11	0/11
8-storey, soil type D, 45°	9/11	0/11
8-storey, soil type D, 90°	9/11	0/11

4. Discussion and conclusions

Regarding the seismic behaviour of steel buildings subjected to near-fault seismic motions the main type of failure observed, independent of the base conditions, is that of the creation of a soft-storey mechanism at higher stories, something that, as it seems, cannot be avoided by the dimensionless slenderness and overstrength requirements for the braces given in [1]. A second type of failure observed, mainly to the fixed base steel buildings, is that of the premature yielding of a column, i.e., before the yielding of the braces. This type of failure is interpreted as a violation of the axial resistance of the column because of increased seismic loads. Thus, the amplification factor of [1], used to obtain the design axial force of the column, has to be revised for the case of near-fault seismic action.

The main type of failure for steel buildings subjected to long duration seismic motions is that of the formation of a soft-storey mechanism as a result of several simultaneous brace fractures in conjunction with major damage induced to the beams. The prolonged duration of the seismic motion further leads to drift concentration at specific storeys, rendering the distribution of deformation demands to other storeys impossible. Thus, to rely on the dimensionless slenderness and overstrength requirements for the braces as given in [1], does not seem to prevent the formation of soft-storey mechanism.

The foundation design rules of [1, 2] are considered to be adequate for steel buildings subjected either to near-fault or to long duration seismic motions.

Taking into account the pronounced unfavourable seismic behavior of steel buildings studied, it is concluded that in the next version of Eurocode 8 [1]: a) a specific design methodology for steel buildings under near-fault and long duration seismic motions should be included and b) SSI effects should be taken into account in the seismic analysis of steel buildings subjected to these two kinds of seismic motions.

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