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Comparative analysis of seismic elastoplastic behaviour of reinforced concrete and steel frames

B. Calderoni, R. Landolfo, P. Lenza & F. M. Mazzolani
University of Naples, Italy

ABSTRACT: In the present work some common typologies of reinforced concrete and steel framed structures with rigid connections, designed according to the Eurocode 8 provisions, are examined. Their dynamic response to some selected accelerograms is analysed by a step by step procedure which takes into account the plastic evolution of sections up to the ultimate deformation limit. The comparison between elastic and elastoplastic dynamic response allow to evaluate the behaviour factor q corresponding to the frame global ductility and hence to judge the correctness of the values assumed in the design. The performed analysis points out some ambiguities connected to the definition itself of the behaviour factor. Formulations more effective are therefore proposed.

1 INTRODUCTION

The capability of the usual framed structures to withstand a strong earthquake is greatly conditioned by their behaviour beyond the elastic limit, i.e. by the possibility of plastic dissipation of seismic energy. The structural design codes aim therefore to improve the post-elastic dynamic response, by means of criteria for designing and checking cross-sections and prescriptions concerning constructional details. As a sufficient global ductility level of the structure is implicitly or explicitly assumed, present codes impose a design procedure based on a static elastic analysis; the prescribed horizontal forces, evaluated by means of modal analysis or by direct formulations, are hence significantly smaller than the ones needed in the design of structures which are required to remain within the elastic limit. The value of such a reduction coefficient (named "behaviour factor" q by Eurocode 8) is defined a-priori, on the basis of the regularity of a given structure and of the ductility of its sections. The a-posteriori control of the correctness of this assumption is at the moment too complex to be carried out by the structural designer and is therefore the object of the scientific research. Aiming at contributing to it, this paper analyses the dynamic non-linear response of reinforced concrete and steel frames, designed according to Eurocode 8, when submitted to some selected accelerograms (corresponding to the most important Italian earthquakes). From these results some ambiguities connected to the definition of the behaviour factor have been pointed out.

2 NUMERICAL ANALYSIS

2.1 *Geometrical and structural layout*

The behaviour factors have been evaluated referring to a common typology of civil buildings, having a rectangular plan and five storeys. Reinforced concrete and steel frames with rigid connections have been assumed in four different structural layouts, which differ from the direction of floor structure (figures 1 and 2). The comparison among those cases has been finalized to find out the influence of the structural typology and of the ratio between vertical and horizontal loads on the response to seismic events acting along transversal direction.

In the case 1a) the main frames are longitudinal and the rectangular cross-section of columns is alternatively orientated in longitudinal and transversal direction. For this reason and for the different-cross section of beams, the transversal frames at the end of the building and those next to the stairs are remarkably stiffer than the other transversal frames. On the contrary, in the case 1b) all transversal frames present the same stiffness, because of the identical orientation of columns and dimension of T beams.

In the steel structure, columns are always orientated so as to supply the greatest stiffness in transversal direction; hence all transversal frames are identical in spite of the different orientation of floor slabs (fig.2a, 2b).

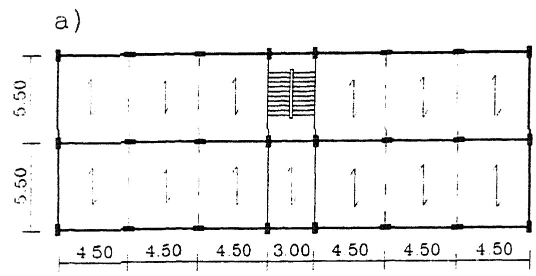
2.2 *Design according to Eurocode 8*

For each of the four schemes under examination, the cross-sections of beams and columns have been previously designed on the basis of good practice (avoiding too many changes of section at different levels) and then checked according to the provisions of Eurocode 8 (EC8), together with those of EC2 and EC3 for r.c. and steel frames respectively. The normalized elastic response spectrum corresponding to soil profile B and a ground acceleration $\alpha = 0.25$ g have been assumed. Thanks to the high regularity of these structures, it seemed to be sufficiently appropriate to perform a simplified dynamic analysis, by verifying that the ultimate limit state of cross-sections is exceeded neither by the effect of conventional horizontal forces, permanent loads at their characteristic value and representative values of imposed loads obtained by means of a factor $\psi = 0.3$, nor by the effect of design values of vertical loads obtained by means

REINFORCED CONCRETE STRUCTURES

STEEL STRUCTURES

a)



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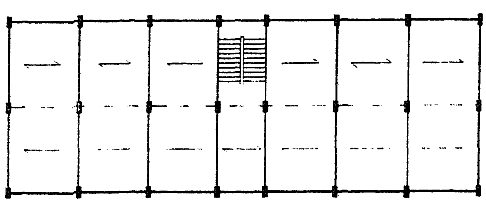
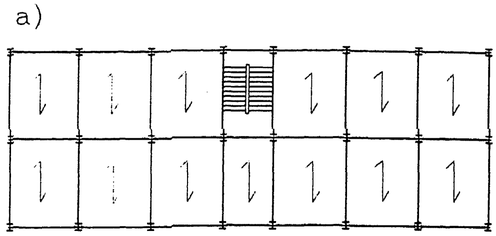


Figure 1

a)



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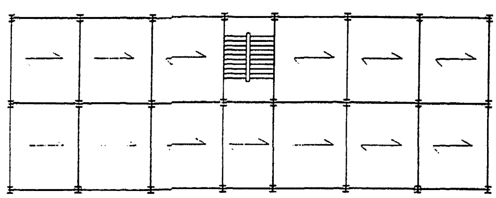
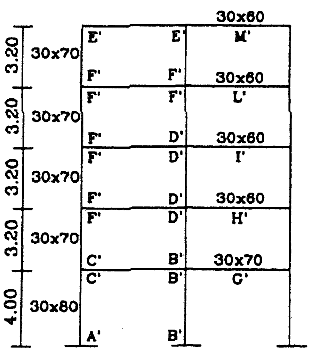


Figure 2

a)

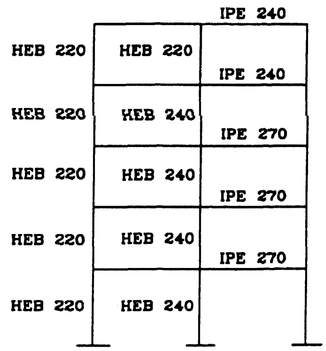
REINFORCED CONCRETE



Beam reinforcement (cm^2)		
	$A_{s,sup}$	$A_{s,inf}$
G	4ϕ20+1ϕ14	4ϕ14
H	4ϕ20	4ϕ14
I	3ϕ20+1ϕ14	4ϕ14
L	3ϕ20	4ϕ14
G'	5ϕ20+1ϕ14	4ϕ20+1ϕ14
H'	5ϕ20+1ϕ14	4ϕ20
I'	5ϕ20	3ϕ20+1ϕ14
L'	3ϕ20+1ϕ14	2ϕ20+1ϕ14
M'	4ϕ14	4ϕ14

STEEL

a)



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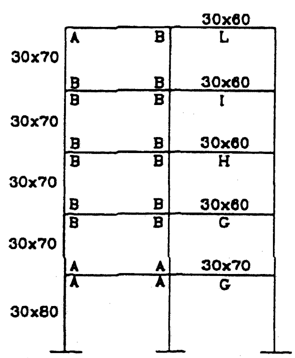

Column reinforcement $A_s=A'_s$ (cm^2)			
A	3ϕ20	C'	4ϕ20
B	2ϕ20+1ϕ14	D'	2ϕ20+3ϕ14
A'	6ϕ20	E'	3ϕ20
B'	5ϕ20	F'	2ϕ20+1ϕ14

b)

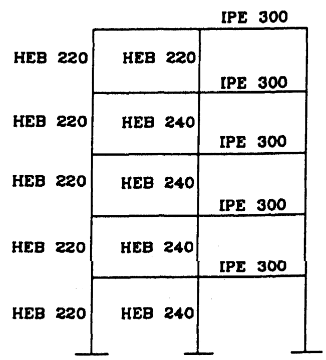

Figure 3

of factors $\gamma_G = 1.35$ and $\gamma_Q = 1.5$.

In the case of r.c. frames, a concrete strength class C20/25 and a steel grade f_{yk} = 380 MPa have been assumed. According to EC8, the respect of the capacity design criterion (which grants an ultimate strength of columns greater than the one of beams and a shear strength greater than the bending one) and of the local ductility criterion (with the prescriptions about longitudinal and transversal reinforcement and constructive details corresponding to H level) allows the use of a behaviour factor $q = 5$.

In the case of steel frames, a steel grade Fe E 235 has been assumed. Because of the structural type (frame with rigid connections), the ratio between the horizontal load multiplier corresponding to the maximum load bearing capacity of the frame α_u and the one corresponding to the point where the most strained cross-section reaches its limit state α_1 has been assumed to be 1.2. The respect of the code limits for b/t ratio of compressed flanges and of the capacity design criterion, slightly different but substantially analogous to the one provided for r.c. structures (Landolfo, Mazzolani, 1990), allows the use of a behaviour factor $q = 6$.

The values of floor masses and of the fundamental period of vibration for the four schemes, shown in table 1, have been used to evaluate horizontal forces; in the case of steel frames, a value of response spectrum corresponding to T=1.2 seconds has been cautiously assumed. The cross-sections of beams and columns and the longitudinal reinforcement are shown in figure 3.

2.3 *Type of analysis*

The numerical analysis has been carried out referring to the most important recent Italian seismic events (table 2) acting along transversal direction. Their elastic response spectra are shown in figure 4.

The analysis of the three-dimensional structure has been simplified by taking into account its symmetry and regularity. The static analysis of the scheme of fig.1a shows that each one of the four stiffer transversal frames bears approximately 1/5 of the horizontal forces; the dynamic analysis has been therefore performed examining a plane frame with 1/5 of the global masses. The behaviour of schemes of fig.1b, 2a, 2b is instead exactly equivalent to that of plane frames with 1/8 of the global masses.

The dynamic analysis has been performed by means of the well-known code DRAIN-2D (Kanaan, Powell, 1973), which integrates step-by-step the differential equations of motion corresponding to a given seismic input, taking into account the possibility of the yielding of the end of beams.

table 1

type	scheme	total weights kN	T seconds
reinforced	a	18210	0.7
concrete	b	18210	0.6
steel	a	11100	1.7
	b	11100	1.5

An elastic-perfectly plastic behaviour of sections of r.c. frames has been assumed, neglecting both the hardening and the degrading stiffness. The ultimate rotation of plastic hinges has been evaluated by assuming a width of the plastic zone nearly equal to the height of the cross-section and an ultimate strain of concrete $\varepsilon_{cu} = 0.35$ % (Giuffrè, Giannini, 1983).

P-δ effect has been taken into account in the case of steel frames, because of their greater deformability. Hardening has been considered too. The ultimate rotation of plastic hinges has been evaluated by means of the formulations proposed by Mitani and Makino (1980).

The response of each frame under the selected accelerograms scaled with increasing peak ground acceleration (PGA) has been evaluated, so as to identify the PGA corresponding to the reaching of the ultimate plastic rotation in the most strained cross-section. The elastic response of the frame to this PGA has been also evaluated.

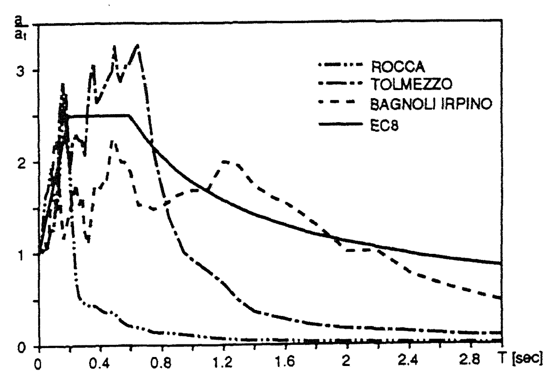

Figure 4.

table 2

Seismic event	Magnitudo	Rec. site	Comp.	Correction	PGA [g]	Lenght [s]
Ancona June 14, 1972 18:55:50	4.8	Rocca	NS	UC Berkeley Geophysics Dept.	0.555	18.98
Friuli May 6, 1976 20:00:13	6.2	Tolmezzo Ambiesta	EW	UC Berkeley Geophysics Dept.	0.266	36.40
Campano-Lucano Nov. 23, 1980 18:34:53	6.5	Bagnoli Irpino	EW	UC Berkeley Geophysics Dept.	0.191	79.15

table 3

structure	seismic event	design		first yielding			ultimate behaviour					q_1
								plastic		elastic		
		$a_{g,d}$	Q_d kN	$a_{g,y}$	Q_y kN	d_y cm	$a_{g,u}$	$Q_{u,p}$ kN	$d_{u,p}$ cm	$Q_{u,e}$ kN	$d_{u,e}$ cm	$\frac{a_{g,u}}{a_{g,y}}$
r.c.	Bagn.Irp.	0.25	411	0.088	461	2.3	0.31	665	18.0	1516	8.1	3.52
a	Tolmezzo	0.25	411	0.048	407	2.7	0.50	760	14.1	4604	28.1	10.42
r.c.	Bagn.Irp.	0.25	257	0.09	300	1.8	0.35	472	14.7	1223	7.3	3.89
b	Tolmezzo	0.25	257	0.043	299	1.7	0.40	435	10.1	2713	16.1	9.30
steel	Bagn.Irp.	0.25	91.5	0.08	127	11.5	0.51	251	61.1	757	68.5	5.93
a	Tolmezzo	0.25	91.5	0.18	129	6.7	1.025	288	41.7	737	38.8	5.69
steel	Bagn.Irp.	0.25	91.5	0.07	139	8.4	0.37	280	42.7	764	43.7	5.29
b	Tolmezzo	0.25	91.5	0.20	146	5.2	0.90	290	27.6	670	23.1	4.50

2.4 *Numerical results*

The results obtained for each of the four schemes are synthesized in table 3. The values related to Ancona's earthquake have not been reported because the spectral characteristics of its accelerogram are such as to not substantially influence the behaviour of selected frames; ultimate plastic rotation was in fact not even reached with PGA = 1.5 g.

For each frame the following values are reported:

a_g = peak ground acceleration, as a ratio to the gravity acceleration g

Q = maximum total shear at the basis of the plane frame

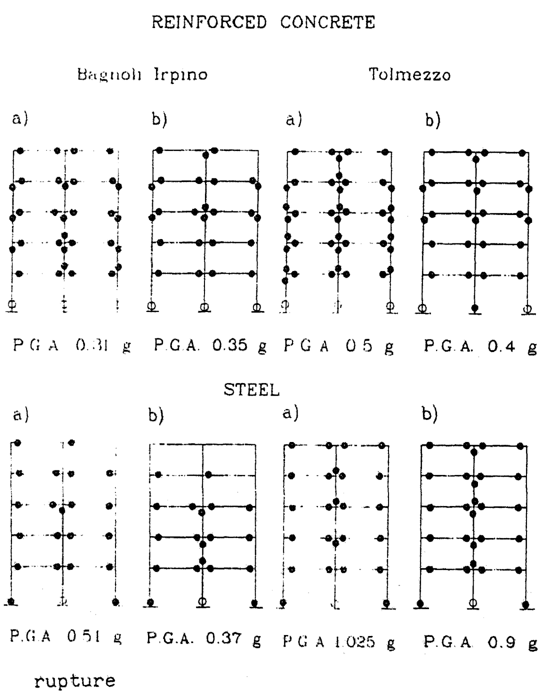

Figure 5

d = maximum horizontal displacement of the upper floor

Such values are referred to the following situations:

a) design (from the dynamic simplified analysis);
b) first yielding (appearing of the first plastic hinge in the frame);
c) ultimate plastic behaviour (reaching of the ultimate plastic rotation in the most strained cross-section);
d) ultimate elastic behaviour (dynamic elastic response to a PGA equal to the one of the above case c).

In table 3 they are identified by means of the subscript d (design), y (first yielding), u (ultimate behaviour, in general), u,p (ultimate plastic), u,e (ultimate elastic), respectively.

The location of plastic hinges at collapse are shown in figure 5. Peak ground accelerations and total shears at the basis of the frames are plotted in figures 6 and 7.

3 ANALYSIS OF RESULTS

Although based on a limited number of cases, the analysis of numerical results points out some general considerations:

- In the case of reinforced concrete structures, the global shear corresponding to first yielding Q_y is only slightly greater than the design forces Q_d, because the reinforcement has been strictly calibrated to stress requirements. On the other side, in the case of steel structures the difference between yielding and design values is more remarkable, because of the use of commercial sections which often are considerably stronger than the design requirements.
- The PGA corresponding to first yielding ($a_{g,y}$) is strongly dependent on the seismic event and on the fundamental period of vibration of the structure. Comparing the elastic response spectra of the examined earthquakes to the one provided by EC8 (fig.3), it can be clearly noted that for r.c. structures (T = 0.6 - 0.7 s) the Tolmezzo event gives a magnification greater than the one supplied by the code, and even greater than the above mentioned design over-strength: $a_{g,y}$ is therefore smaller than the design value (α/q = 0.05); on the contrary, the Bagnoli Irpino event gives a smaller magnification and a greater value of $a_{g,y}$. In case of steel structures (T = 1.5 -

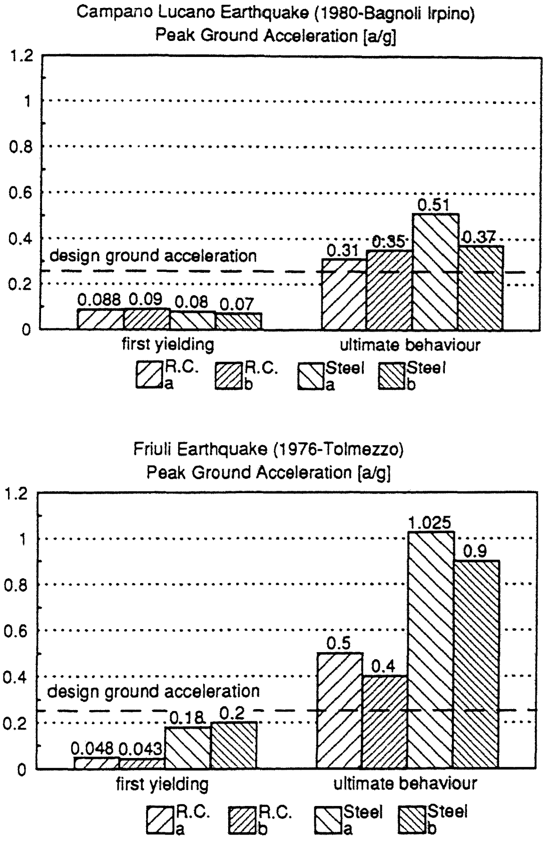

Figure 6

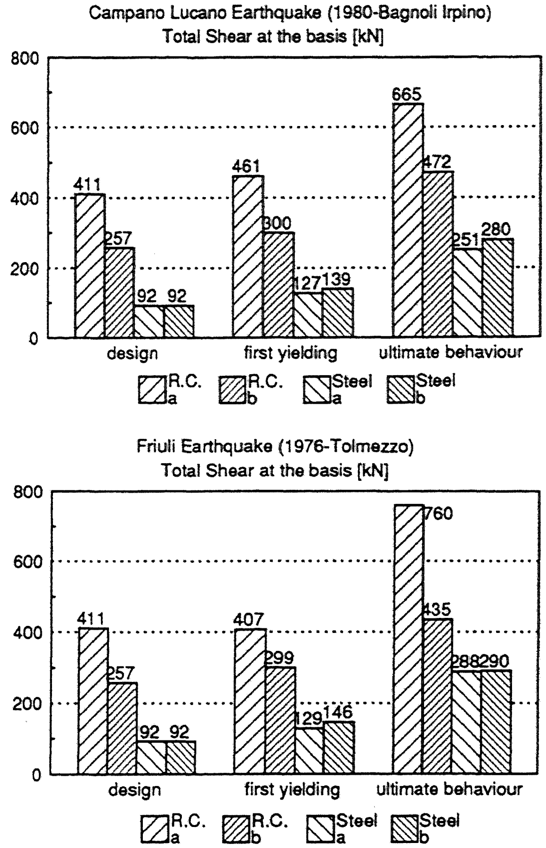

Figure 7

1.7 s) the Tolmezzo event gives really small magnification and hence yielding PGA up to nearly five times the design one (α/q = 0.042); the Bagnoli Irpino event gives a magnification slightly greater than the design one, but the over-strength is such as to allow also in this case PGA greater than the design one.

- The global shear corresponding to the ultimate plastic behaviour Q_{up} (achieved with the yielding of the end of columns) is always considerably greater (1.5 - 2 times) than the first yielding shear Q_y (always corresponding to the yielding of a beam end), pointing out the effectiveness of the capacity design criterion; the increase is greater in case of steel structures because of the above mentioned necessity to use commercial sections.

- For each structure, the PGA corresponding to the ultimate plastic behaviour a_{gu} strongly depends on the seismic event considered. The Tolmezzo elastic response spectrum shows a sharp reduction of effects on structures having a period greater than 0.7 seconds; for the examined r.c. schemes, the increase of period due to the appearing of plastic hinges during the seismic event allows the structure to bear really great increments of ground acceleration (up to ten times the yielding ground acceleration); the steel schemes are, as already shown, not significantly affected by this event and can thus reach very great values of ultimate PGA, but the increment with respect to the yielding value is not equally relevant (nearly five times). The Bagnoli Irpino event mantains its maximum effect up to period T = 1.3 s; the increase of period of yielded r.c. schemes doesn't correspond to a reduction of seismic effects, allowing them to bear smaller ground acceleration increments (less than four times); the reduction is present, but it is slight, in the case of steel schemes which can bear an increment of nearly six times.

- In all examined cases the collapse ground acceleration a_{gu} is greater than the design value (α = 0.25); from this point of view, the design criteria appear then to be correct and the assumed values of the behaviour factor q appear adequate both for r.c. and steel structures.

- The behaviour of the examined schemes is not very significantly affected by the orientation of floor slabs; the relative intensity of vertical and horizontal loads as well as the small differences in stiffness and in the first period of vibration appear therefore to be of minor importance in the examined cases.

4 CRITERIA FOR ASSESSING THE Q-FACTORS

The results of the dynamic analyses may be used to evaluate the actual values of the behaviour factor q for each structure under a given seismic event, in order to control the correctness of the values aprioristically assumed in the design. The behaviour factor is usually defined as the ratio between the ground acceleration which produces the structural collapse a_{gu} and the one which produces the first yielding a_{gy}. As expected, such values q_1 (shown both in the last column of table 3 and in table 4) are strongly dependent upon the seismic event. The correlation among elastic response spectra of the earthquakes, first period of vibration of the schemes and behaviour factors appears to be justified, as outlined in the previous section. Nevertheless, the values are in the most cases smaller than those assumed in the design, giving the wrong impression that the assumed behaviour factors q are not adequate (opposite to the conclusion of the previous section, based on the analysis of the collapse ground acceleration). Moreover, the high values of q in the case of r.c. structures under the Tolmezzo event could allow to think that the r.c. schemes may be safer than the steel ones, while the collapse ground acceleration is always greater in the case of steel structures.

These apparent contradictions are mainly connected to the value of ground acceleration which produces the first yielding. First of all, this value is strongly dependent upon the magnification of the ground acceleration, expressed by the elastic response spectrum which in the examined seismic events is clearly different from the one defined by EC8. Secondly, the value is affected by the unavoidable design over-strength (more relevant in the case of steel structures). The judgement of the correctness of the assumed values of behaviour factors must therefore rely upon different criteria. The design value of q may be compared with the values q_2 or q_3, shown in table 4 and defined as:

q_2 = ratio between the collapse ground acceleration ($a_{g,u}$) and the acceleration used to evaluate the design forces ($a_{g,d} / q$); in this way the influence of the different magnification of ground acceleration produced by a given seismic event is overpassed;

q_3 = the value q_2 reduced by the over-strength ratio, i.e. by the ratio between first yielding total shear Q_y and design forces Q_d; in this way also the influence of the design over-strength is eliminated.

The values obtained are always greater than those assumed, consistently with the capability of the examined schemes to withstand greater PGA than those required. Moreover, for each seismic event the values provided by the steel structures are 1.5 to 2 times those supplied by the r.c. structures, in accordance with the greater ductility of steel structures and the greater PGA which they can withstand.

table 4

structure	seismic event	q_1	q_2	q_3
r.c.	Bagn.Irp.	3.52	6.2	5.5
a	Tolmezzo	10.42	10.0	10.0
r.c.	Bagn.Irp.	3.89	7.0	6.0
b	Tolmezzo	9.30	8.0	6.9
steel	Bagn.Irp.	5.93	12.2	8.8
a	Tolmezzo	5.69	24.6	17.4
steel	Bagn.Irp.	5.29	8.9	5.9
b	Tolmezzo	4.50	21.6	13.5

5 CONCLUSIONS

The analysis of the dynamic elasto-plastic response of some significant schemes subjected to different seismic events, although not exhaustive, gives useful indications for the assessment of the behaviour factor.

For a given structure, both the ultimate displacement and the q-factor strictly depend upon the elastic response spectrum of the earthquake. The wideness of the range of the evaluated values points out the difficulties and contradictions which may be met while extending the results of the single-degree-of-freedom model to the actual multi-degree-of-freedom schemes. In order to reduce such incertainties, a more accurate evaluation of the behaviour factor should then require to test the scheme under a number of earthquakes so high as to perform a statistical elaboration of the results. Furthermore, the definition itself of q-factor should be revised in order to judge the correctness of the q-values aprioristically assumed in the design. The formulations proposed in the previous section appear to be adequate to this purpose.

Nevertheless, in every examined case the peak ground acceleration which produces the collapse is acceptable in comparison to the design requirements. However, in spite of the complexity of their numerical application, the capacity design and the local ductility criteria, imposed by Eurocode 8, are useful to recall the attention of the designer to the post-elastic behaviour of the structure, allowing to involve all structural elements in the plastic dissipation of seismic energy.

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