DESIGN ASPECTS AFFECTING THE SEISMIC BEHAVIOUR OF STEEL MRF BUILDINGS: ANALYSIS OF THREE CASE STUDIES

E MELE[1], A DE LUCA[2] And L DI SARNO[3]

SUMMARY

The seismic performance of three partial perimeter and spatial moment resisting frames is analysed in order to establish if the perimeter configuration has a detrimental effect on the seismic behaviour of buildings. For these frames, designed for three different seismic areas, a comparison is presented in terms of ultimate capacity and inelastic demands, expressed through different parameters, i.e. interstory drifts, beam and column maximum plastic rotations, hysteretic energy. The results of both inelastic static and dynamic analyses allow to conclude that the behaviour of the perimeter and spatial MRFs is very similar, provided that design criteria are consistent.

INTRODUCTION

Following the recent earthquakes in California (Northridge, 1994) and Japan (Hyogoken-Nanbu, 1995), widespread and unexpected brittle fractures were detected in welded steel beam-to-column connections of several steel frame structures. Different opinions have been advanced on the causes of the poor performance observed during the earthquakes, also relating the erratic connection behaviour to global design choices, namely the lack of structural redundancy resulting from the U.S. current trend in the MRF design philosophy, consisting in the adoption of perimeter, or even partial perimeter configuration, for resisting lateral actions, while the interior elements support only gravity loads.

A great deal of research activity is presently devoted to understand the causes of the observed behaviour. In U.S.A. the SAC Steel Project, was *"formed specifically to address both immediate and long-term needs related to solving problems of the WSMF connections"* (SAC, 1995). In this context some frame office buildings were designed according both to "pre-Northridge" design practice (UBC, 1994) and to the Interim Guidelines (SAC, 1995) developed by the SAC.

The present paper investigates on the seismic performance of partial perimeter and spatial MRFs in order to establish if the first structural configuration has detrimental effects on the seismic behaviour of buildings, giving rise to concentrations of rotation demands in the moment resisting beam-to-column connections. In particular the seismic behaviour of three frame buildings, designed within the SAC Steel Project (according to the pre-Northridge design practice) for three different seismic regions is assessed by performing nonlinear static and dynamic analyses. For the three buildings an alternative structural configuration, more close to the European design practice, i.e. spatial MRF, is also designed and analysed.

For all frames is presented a comparison in terms of ultimate capacity and inelastic demands, expressed through different parameters, i.e. interstory drifts, beam and column plastic rotations, hysteretic energy.

ANALYSED BUILDINGS

Structures and Ground Motions

The structures analysed in this paper are 3-story office buildings, designed according to UBC 1994 provisions for three different seismic zones, respectively Los Angeles (Seismic Zone 4), Seattle (Seismic Zone 2A) and Boston (Seismic Zone 2B). The three building structures have been designed by three U.S. professional firms,

[1] *Università degli Studi di Napoli 'Federico II', P.le Tecchio n°80, 80125, Napoli, Italy. E.mail: elenmele@unina.it*
[2] *Università degli Studi di Napoli 'Federico II', P.le Tecchio n°80, 80125, Napoli, Italy. E.mail: adeluca@unina.it*
[3] *Università degli Studi di Salerno, via Ponte don Melillo n), Italy. E.mail: luigi.disarno@mbox.diima.unisa.it*

i.e. Brandow & Johnston Associates (Los Angeles building), KPFF Consulting Engineers (Seattle building) and LeMessurier Consultants (Boston building), and have been extensively analysed by several researchers (Gupta and Krawinkler, 1998) in the context of SAC studies on system performance.
As shown in figure 1, all frames have the same rectangular floor plan (56.08x37.80 m): the structural geometry consists of six 9.15 m bays along major side and four 9.15 m bays in the orthogonal direction. The interstory height is 3.96 m.

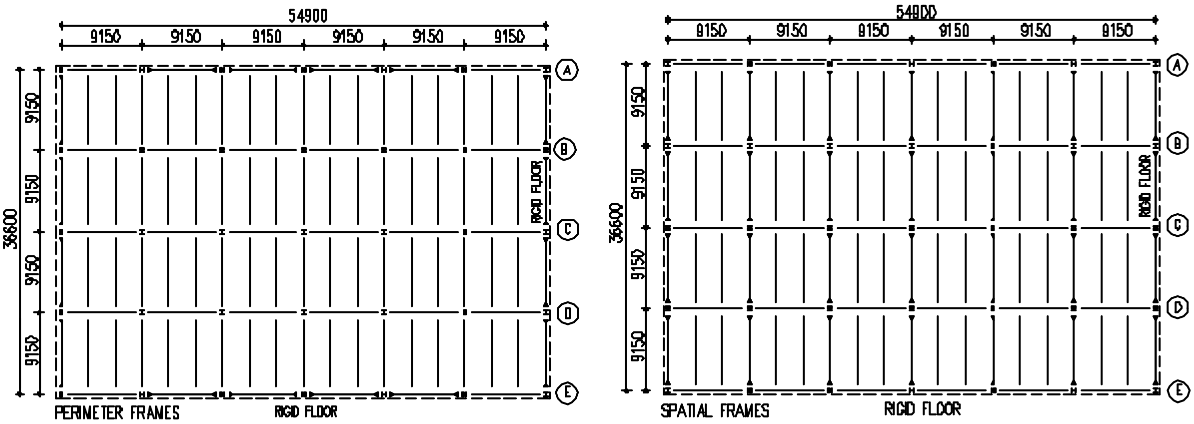

Figure 1 - Typical plan layout

At each story level there is a floor system of 76.2 mm metal decking with 63.5 mm of NWC fill.
These three buildings have MRF perimeter configuration along both directions and interior gravity frames with simple shear connections. In the Los Angeles perimeter frame there are only three moment resisting bays; this last frame has also a weak axis column.
The governing design criterion of the alternative spatial MRF configurations has been: stiffness and strength close to the corresponding perimeter structure values. This condition (same fundamental periods and close yield strength capacity) is the fundamental requirement for consistent comparison between performances, as shown in next paragraphs. The frame elevations and the element cross sections are provided in figure 2.

The non linear dynamic analyses have been carried out with three well-known historical records, characterised by different frequency contents and peak ground acceleration values (table 1).

Table 1. Seismic inputs

Earthquake	Date	Station	Component	Duration [sec]	PGA [g]
Imperial Valley	18.5.40	El Centro	S00E	53.80	0.348
Northridge	17.1.94	Newhall	340N/118W	59.98	0.583
Hyogoken-Nanbu	17.1.95	Kobe	EW3	56.40	0.834

Structural Models

The non-linear dynamic analysis computer program DRAIN-2DX (Prakash et Al., 1993) was used to evaluate both the static and the dynamic responses of perimeter and spatial MRFs. The frames are modelled as two-dimensional assemblages of plastic hinge (lumped plasticity) beam-column element, including bending-axial interaction and P-Δ effects.

In the SAC building design ASTM A36 steel, with nominal yield strength equal to 248 MPa (36 ksi) and Grade 50 steel (345 MPa=50 ksi) have been respectively adopted for beams and columns. The plastic hinges at the end of beam elements have been modelled with a linearized biaxial plastic domain (Bending Moment-Axial Load), with both horizontal and vertical branches corresponding to 10% of ultimate bending and axial force values. The analyses have been performed assuming material strain hardening of 1%, while the equivalent structural damping has been fixed as 3% of the critical value.

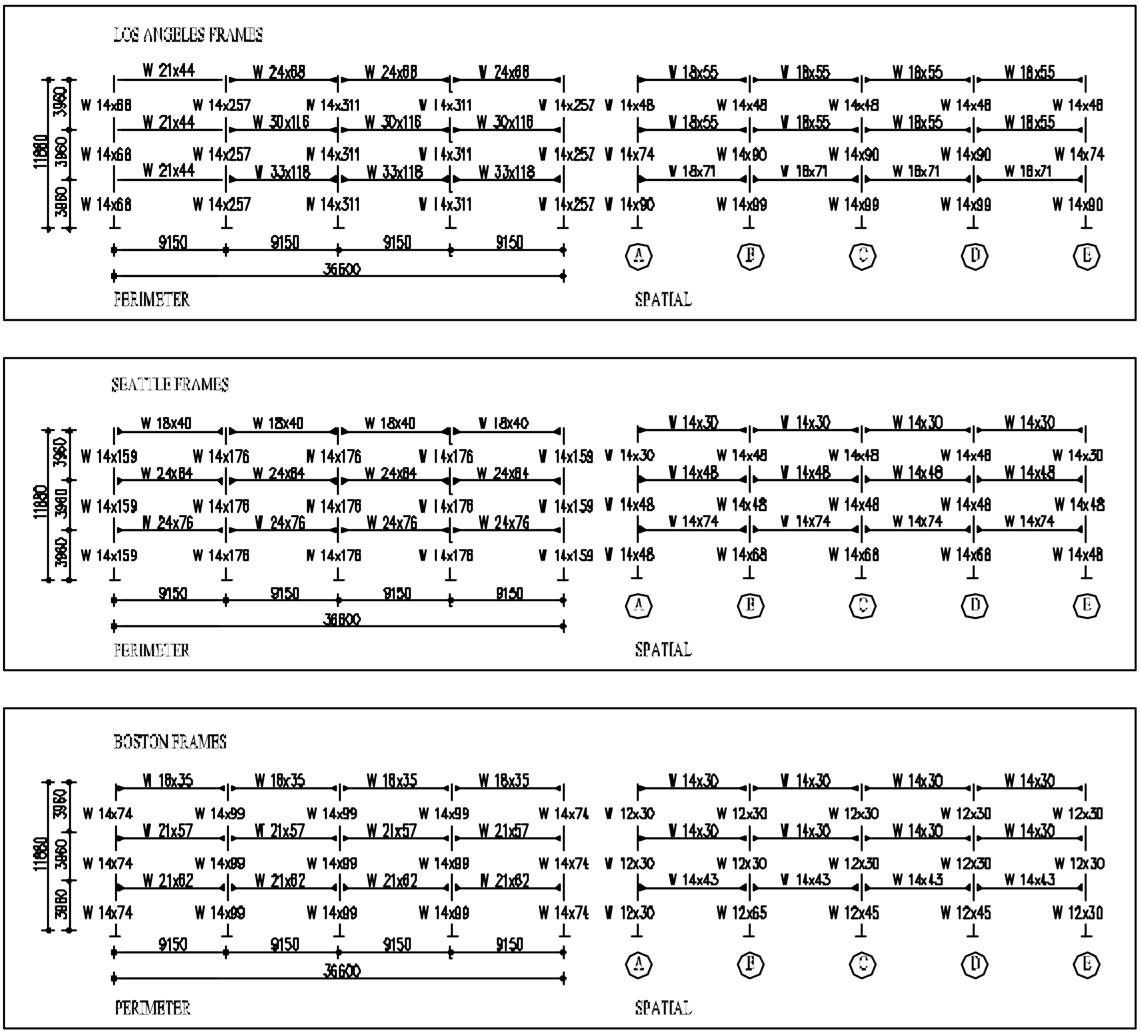

Figure 2 - Frame elevations

Beam loads (self weight and exterior walls) and joint vertical loads, provided in figure 3, are the same for the two frame configurations. The floor mass values (table 2) have been computed by considering ½ floor tributary area in the case of the perimeter structures, while 1/7 of the total mass is attributed to each frame in the spatial frame configurations. The total masses are the same for the three buildings, namely 2950 $kN{\cdot}s^2/m$. The total seismic load (dead plus live loads without reduction) is W_{tot} = 28926 kN.

The fundamental vibration periods of the analysed frames, listed in the table 3 for both partial and perimeter frames, are higher (usually twice larger) than the values calculated with the empirical formula for steel MRFs suggested by UBC Method A: the code formula value, equal for all considered building structures, is 0.55 sec. The floor equivalent static forces and the design base shear provided by the UBC at T=0.55 are provided in table 4. In the design of the spatial MRFs, the code drift limit of 1/400 under this force distribution, has been the most stringent requirement.

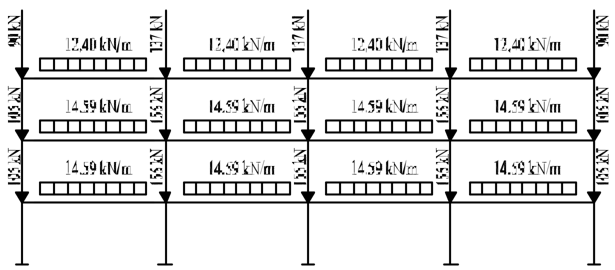

Figure 3 - Dead plus live vertical loads.

Table 2. Floor masses

Typology	M_1 [$kN{\cdot}s^2/m$]	M_2 [$kN{\cdot}s^2/m$]	M_3 [$kN{\cdot}s^2/m$]
Perimeter	478.65	478.65	518.06
Spatial	136.50	136.50	148.00

Table 3. Fundamental vibration periods.

PERIMETER FRAMES			
Seismic Zone	T_0 [s]	T_1 [s]	T_2 [s]
Los Angeles	1.010	0.327	0.171
Seattle	1.325	0.425	0.214
Boston	1.771	0.563	0.296

SPATIAL FRAMES			
Seismic Zone	T_0 [s]	T_1 [s]	T_2 [s]
Los Angeles	1.012	0.331	0.171
Seattle	1.321	0.422	0.227
Boston	1.777	0.529	0.308

Table 4. Equivalent horizontal loads.

	PERIMETER FRAMES			SPATIAL FRAMES			BUILDINGS	
Seismic Zone	F_1 [kN]	F_2 [kN]	F_3 [kN]	F_1 [kN]	F_2 [kN]	F_2 [kN]	$V_{UBC,BASE}$ [kN]	$V_{UBC,BASE}/W_{TOT}$ [%]
Los Angeles	189	378	596	54	108	170	2323	8.03
Seattle	95	189	297	27	54	85	1161	4.01
Boston	71	142	223	20	40	64	871	3.01

RESULTS

Building Resistance

The nonlinear static behaviour of the perimeter and spatial MRFs has been assessed through pushover analyses. The results of the analyses, carried out both on the single frames and on the complete buildings, are provided in figure 4, according to three seismic regions (L.A., Seattle, Boston). The charts provide the base shear V_b versus the top displacement d_{top}, normalised to the building height H. In the curves referring to the entire building structure, the base shear has been normalised to the seismic weight W_{tot}.

The curves point out that the yield strength capacity of the frames is quite higher than the relative design base shears. The nonlinear branch starts approximately at base shear values equal to 10% and 25% W_{tot}, respectively for the Boston and L.A. building. The design (Allowable Stress Design) values of base shear are equal to 3.01% (Boston) and 8.03% (Los Angeles), thus considering the 1.5 ratio of yield to allowable stress, an overstrength factor of approximately 2 can be derived. The same ovestrength value is also reported by Osteraas and Krawinkler (1990) with reference to low-rise steel frame building structures. This significant overstrength is expected to reduce the inelastic deformation demands in the structural elements.

The pushover analyses results also evidence that the design of the buildings is mainly governed by stiffness requirements, as confirmed by the intersection of the vertical line at 1/400 and of the horizontal line at V_{UBC} on approximately the same point of the curve. Finally, the comparison between spatial and perimeter MRFs curves in figure 4 underlines that the ultimate strength capacity of the spatial MRFs is consistently higher than the perimeter MRFs, though the yield displacements are similar. The close values of yield displacements is related to the approach in the design of spatial MRFs herein adopted, which was aimed to match the structural stiffness of the corresponding perimeter MRFs.

Drifts and Plastic Rotations

In the figures 5 and 6 the maximum values of top displacement (normalised to the building height H_{tot}) and of interstory drift d_{int} (normalised to the interstory height h_{int}) obtained from the inelastic dynamic analyses are reported. The values of global drift are generally less than 3%, both for perimeter and spatial frames. An exception is the case of the Seattle building perimeter frame under the Kobe seismic input, which gives 3.50%. The maximum interstory drifts provided in figure 6 are generally lower than 4%, which is the interstory drift capacity (IDC) required in (SAC, 1999) for SMRFs. The spatial frames experience slightly larger values than the corresponding perimeter frames, and under the Kobe signal, the interstory drift exceeds 4%.

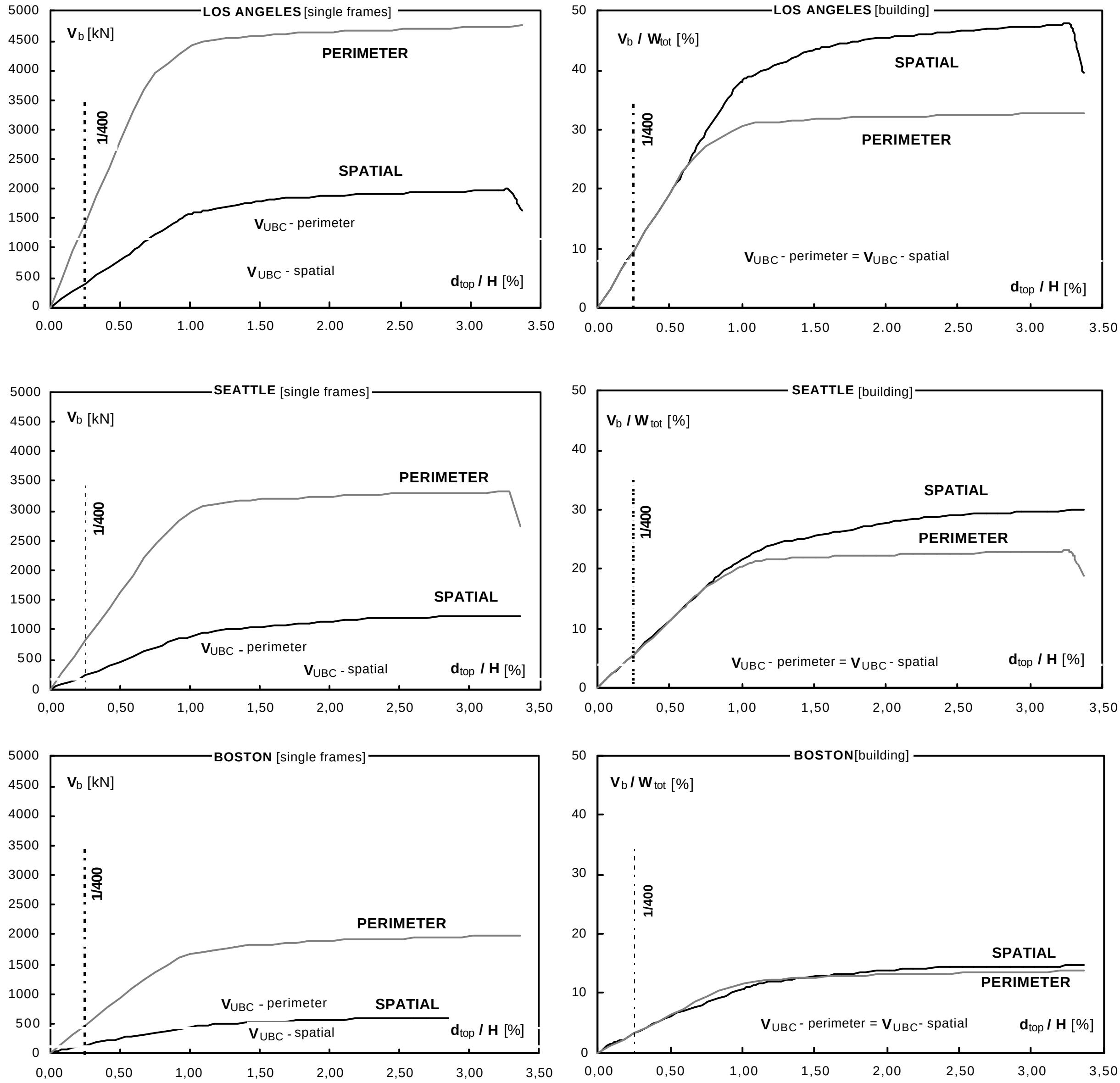

Figure 4 - Push-over curves for single frames and the buildings

The maximum plastic rotations in the beam and column elements deriving from the time history analyses are provided in figure 7. The values of plastic rotation demands at the beam ends of the perimeter and spatial MRFs are quite close and do not generally exceed the value of 0.03 rad, defined within the SAC Steel Project as the “target plastic rotation” of beam-to-column connections. The response under the Kobe earthquake induces the largest plastic rotations in the beam elements, up to values of 0.05 rad. The rotation demands in the columns are lower than the beam ones, since Strong Column Weak Beam design philosophy has been adopted for both the MRF configurations. Larger rotations are required to the spatial configuration columns than to the perimeter frame columns. The location of the plastic hinges is usually at the base of the first story columns or, in few cases, at the top of the third story columns.

As already noticed in (Mele et Al., 1995), the comparison between interstory drifts and plastic rotations shows close values, particularly in the case of perimeter MRFs, thus confirming the potential of plastic rotation prediction starting from the interstory drift values. Since inelastic interstory drift can be related to the elastic interstory drift by means of displacement amplification factor C_d, thus a reasonable prediction of plastic rotation demands can be obtained by means of a simple elastic analysis.

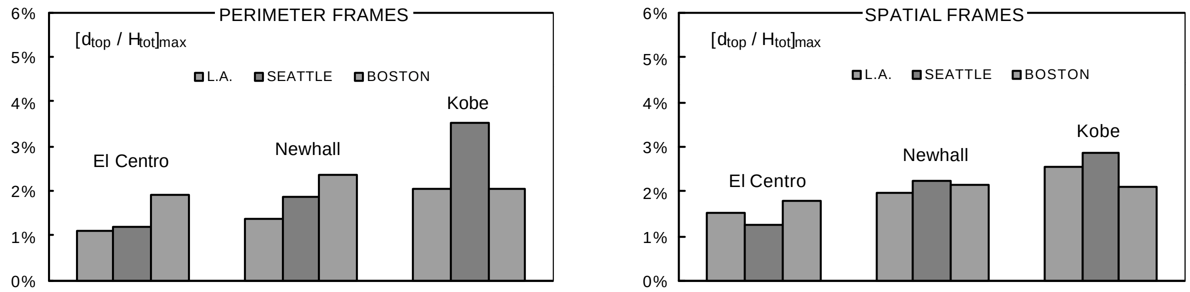

Figure 5 - Maximum top displacements

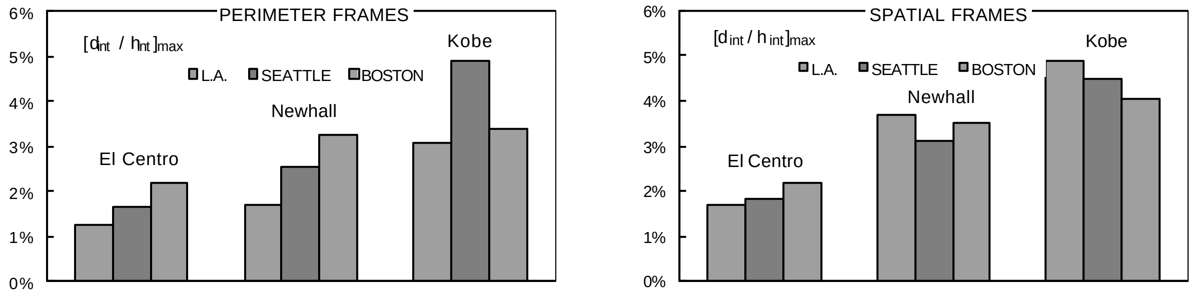

Figure 6 - Maximum interstory drifts

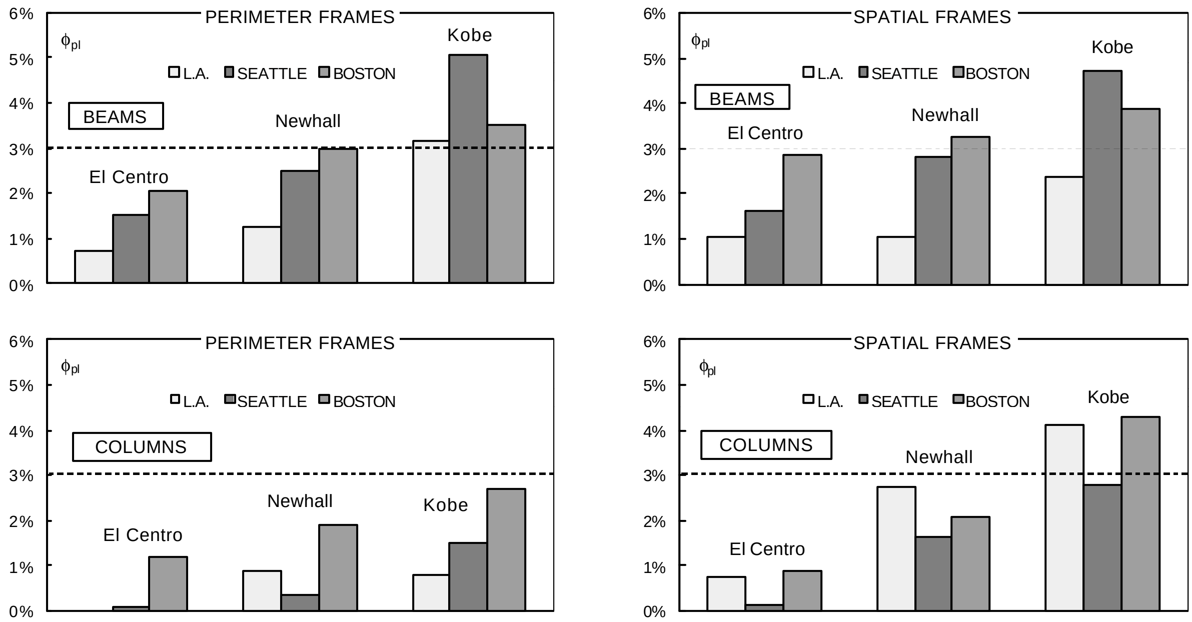

Figure 7 - Maximum plastic rotations

Dissipated Hysteretic Energy

The inelastic time history analyses also provide the hysteretic energy dissipated during the ground motion. In figure 8 the hysteretic energy histories in the three perimeter and in the three spatial MRF buildings, under the Newhall and Kobe seismic inputs, are respectively provided. It can be observed that the energy dissipated by the three pairs (L.A., Seattle, Boston) of buildings configurations (perimeter, spatial) is quite close. Therefore, also in terms of hysteretic energy, the dynamic response of the perimeter and spatial MRFs is very similar, pointing out their equivalent behaviour.

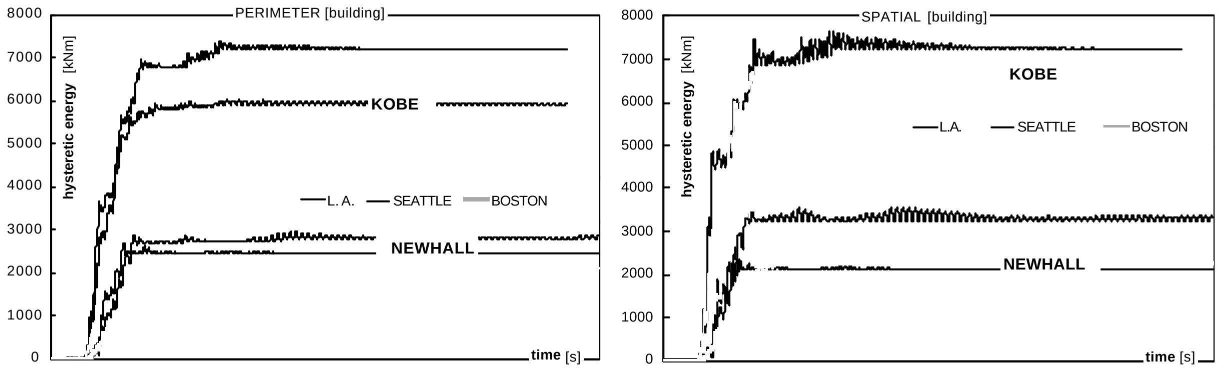

Figure 8 - Hysteretic energy dissipated by the buildings

CONCLUSIONS

The results of the static analyses show similar level of global strength capacity of the perimeter and spatial MRF buildings. The inelastic dynamic analyses evidence that the behaviour of the perimeter and spatial buildings is practically equivalent also from the point of view of the inelastic demand under different seismic inputs. Both the perimeter and the spatial frames show a good seismic behaviour, since they provide a global distribution of the plastic deformation within the entire frame, proved by the fact that the interstory drift ratios are comparable to the maximum plastic rotations and that the maximum plastic rotation values are generally smaller than 0.03 radians. The value of 0.03 rad, already found in (Mele et Al., 1995; De Luca & Mele, 1997) as an upper limit of plastic rotation demand in SCWB frames and identified in (SAC, 1995) as a reference capacity value for moment resisting welded connections, is confirmed.

The results provided in terms of building ultimate capacity and global and local inelastic demands allow to conclude that the behaviour of the perimeter and the spatial MRF buildings is very similar, provided that the designs are consistent.

On the basis of these results, it seems therefore possible to state that:

- even though the partial perimeter buildings have a smaller number of plastic regions, there is no concentration of plastic rotation requirements. This is mainly due to the different relative capacity of the member cross-sections, which leads to the same hysteretic energy globally dissipated by the buildings;
- the reason of the poor performance of the welded steel MRFs, observed in the Northridge earthquake, is not attributable to the design choice of the perimeter frames. Among others, a possible cause of the observed damage can be the large strain demand in the very deep beams used in the perimeter frames.

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