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THE INFLUENCE OF SUPERSTRUCTURE DESIGN CRITERIA ON SEISMIC FORCE REDUCTION FACTORS FOR BASE ISOLATED STRUCTURES

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ABSTRACT

In this paper, a contribution to the definition of the force reduction factors for base isolated steel structures is provided. In particular, it is analysed how the design criterion of the superstructure affects the nonlinear behaviour of the entire base isolated system. For this purpose, a comparison between the nonlinear dynamic response of different BIS in which the superstructure is respectively designed with and without allowance for capacity design and for drift limitations is carried out. Investigations on the behaviour of the superstructures through nonlinear static push-over analyses also allow to evidence some mechanical parameters affecting the response of BIS.

KEYWORDS

Base isolated structures, steel frames, seismic design criteria, force reduction factors.

INTRODUCTION

The values for the force reduction factors to be adopted when designing base isolated structures (BIS) have represented a subject to which extensive investigations have been devoted from the research community in the last years. An agreement on the most appropriate value for the force reduction factor is not yet reached due to the numerous physical parameters which influence the seismic behaviour of the BIS. This fact is demonstrated also by the changes of the numerical values advised in the seismic codes in the different years (SEAOC 86, SEAOC 90, UBC 94). Furthermore, the debate on the values which have to be used for the reduction factors is open also due to the philosophy to be pursued in designing BIS, which can be different from the one implemented into the seismic codes for ordinary structures. In fact, it can be agreed that the base isolated structures should provide higher performance with negligible damage in the superstructure under severe earthquakes (Kobori 1994).

In order to give a contribution to the definition of the reduction factors for BIS, in the paper it is analysed how the design criterion of the superstructure affects the nonlinear behaviour of the entire base isolated system. Within this framework some preliminary results have been presented in (De Luca *et Al.* 1994, De Luca *et Al.* 1995), where the effect of design level on performance of BIS and some indications on the values of the force reduction coefficients have been proposed. In this paper, with reference to steel structures, a comparison between the nonlinear seismic response of different BIS in which the superstructure is respectively designed with and without allowance for capacity design, which means comparing strong-columns and weak-columns steel framed superstructures, is carried out. Furthermore, investigations on the behaviour of the superstructures through nonlinear static push-over analyses have allowed to evidence some mechanical parameters affecting the response of BIS and, amongst these, it seems that the overstrength of the superstructure plays a major role.

THE DESIGN OF THE SUPERSTRUCTURE IN BIS

As anticipated in the previous section, the analyses carried out in this paper are aimed to investigate on the influence of the design of the superstructure on the seismic performance of BIS. For this purpose 4 and 8-story steel framed structures, characterized by a plan of 3 bay by 7 bay, have been considered (Fig. 1). In particular, it has been studied the nonlinear dynamic response of plane frames which can be considered representative of the building behaviour since a regular plan-wise distribution of masses, stiffnesses and strengths has been assumed.

The analysed frames are subjected to beam loads equal to 22.5 *N/mm* (dead) and to 10 *N/mm* (live); furthermore, joint vertical perimeter loads equal to 30 *kN* have been applied to consider the presence of exterior claddings. Therefore, based on tributary area, the frames have been designed for a seismically effective weight of 377.5 *kN* at each floor. In the design it has been assumed that the frames are characterized by moment-resisting connections, thus being SMRF.

The isolation devices have been designed to shift the period T_I of the isolated system (complete building structure including devices and superstructure) to 3.0 *sec*. In the analyses, an equivalent viscous damping ν of 10% has been assumed to represent the dissipative properties of the isolators.

Design of frames

For each frame, two different designs of the superstructure have been carried out. Design 1 (D1) corresponds to fixed base structures which fully satisfy the strength and deformability (drift limitations) requirements of UBC (1994); UBC rules for implementing strong column - weak beam mechanism have also been accounted for. Design 2 (D2) is instead obtained from design D1 by relaxing some requirements concerning lateral stiffness and beam-to-column strength ratio, thus leading to smaller member cross sections and, hence, to more flexible structures with possible strong beam - weak column mechanism. The cross section sizes of beams and columns for the designed frames are provided in Fig. 1. It is possible to see that the column cross-sections have been modified every two stories while the beam cross-sections have been reduced every four stories. The second column of Table 1 contains the fundamental periods T_F of the superstructures considered fixed at the base.

Table 1, together with the periods T_F and T_I, provides the values of the reduction factors R_{WI} computed by means of a multimodal response spectrum analysis of the base isolated frames. In particular, reference to the response spectrum proposed by the UBC for the soil profile S1, amplified by a coefficient equal to 1.15 for considering a 3% damping in the superstructure, has been made; in order to incorporate the higher damping of the isolation system (ν=10%), the portion of the UBC standard response spectrum (5% damping) corresponding to periods larger than T_I has been reduced by a coefficient equal to 1.2 (see Fig. 2).

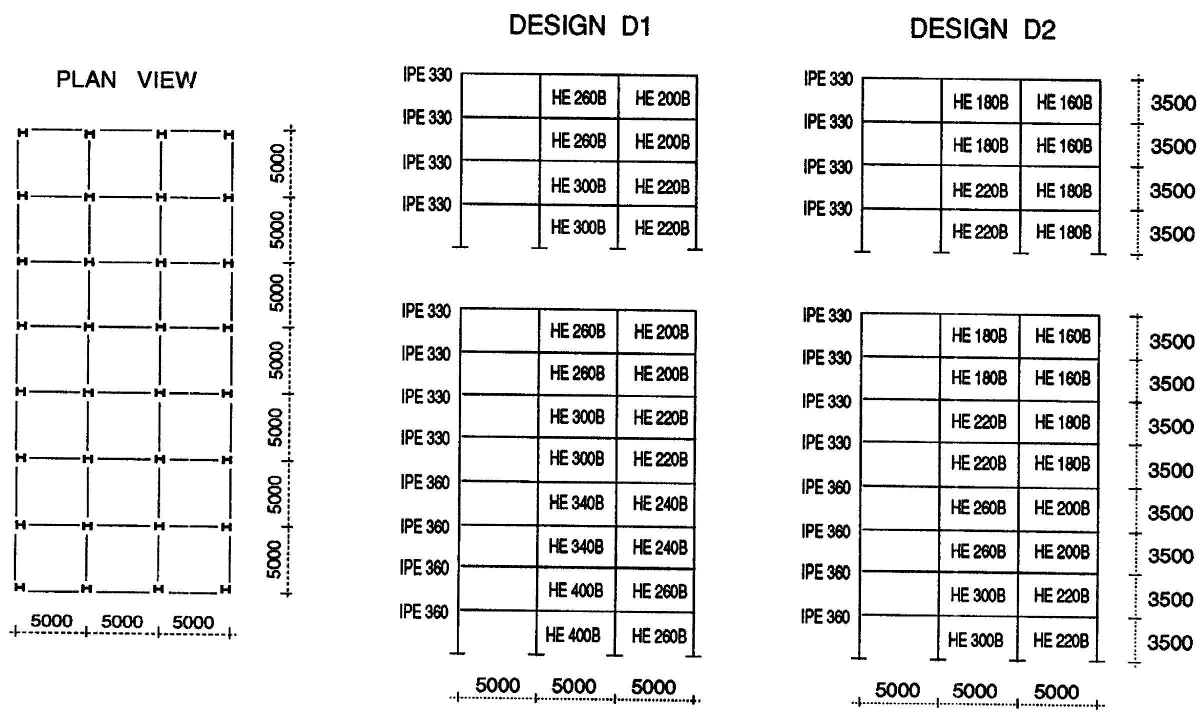


Fig. 1. Plan view and member cross section sizes of the analysed steel framed structures.

Table 1. Reduction factors and design spectral values.

Frame	T_F [sec]	T_I [sec]	R_{WI}	S_{des} [g]	$V_{b,y}/W_t$
4/D1	0.98	3.00	0.78	0.1453	0.1464
4/D2	1.31	3.00	1.08	0.1049	0.1099
8/D1	1.67	3.00	1.22	0.0929	0.0956
8/D2	2.01	3.00	1.45	0.0781	0.0859

The values of R_{WI} are representative of the minimum reduction in the response spectrum ordinates $S_{UBC}(T)$ which is needed to reach the yield level in the highly stressed cross-section. Therefore, the ratio S_{UBC}/R_{WI} at T_I provides the design spectral value S_{des} (commonly considered the most significant spectral ordinate for designing of BIS), which leads to yielding in the superstructure. By considering that $S_{UBC}(T_I)$ at T_I=3.0 *sec* and for 10% damping is equal to 0.1133 *g* and by adopting the R_{WI} values provided in Table 1, the values of S_{des} result equal to the ones reported in the fifth column of the table.

From Table 1, it can be seen that the values of R_{WI} range between 0.78 and 1.45, that is in the proximity of the unity. UBC recommends a force reduction factor equal to 3 for the base isolated steel SMRF, but in order to compare the computed values with the UBC recommended ones, an amplification through a coefficient of at least 1.5 has to be considered. In fact, reduction factors R_{WI} have been evaluated with respect to yielding stresses in the superstructure while UBC considers Allowable Stress Design. Furthermore, the adoption of soil coefficients greater than 1 is very common and, therefore, the examined frames can be considered designed quite in accordance with the UBC provisions.

Figure 2 compares the lateral floor deflections d_s computed for the base isolated 4-story frame D2 by means of the response spectrum analysis to the ones obtained from a push-over analysis of the corresponding fixed base superstructure. In the latter case, the superstructure has been subjected to increasing horizontal forces, uniformly distributed over the height of the frame, and the story displacements at the achievement of the first yielding have been plotted in the figure. The capacity curves provided by the push-over analyses for all frames are instead plotted in Figs. 3*a* and 4*a* and are explained more in detail in the subsequent section.

The last column of Table 1 contains the base shear $V_{b,y}$, nondimensionalised with respect to the frame seismic weight W_t, computed from the push-over analyses at the attainment of yielding in the superstructure. These values are very close to the design spectral value S_{des} reported in the previous column of the same table, thus explaining the excellent agreement between the lateral floor deflections d_s of Fig. 2 and confirming the adequacy of common design which assumes (at least up to yielding) the superstructure acting as a rigid body. Finally, the comparison between S_{des} and $V_{b,y}$ obtained for all the frames evidences that the largest difference affects the values computed for the 8-story frame D2 in which some coupling with higher modes of the superstructure occurs due to the smaller isolation ratio T_I/T_F.

Capacity curves of the superstructures

The capacity curves in terms of base shear - top displacement computed by means of the nonlinear static push-over analyses allow to better examine the effects of the different philosophies adopted in designing the superstructures. In particular, such curves allow to have more data for understanding how the superstructure response, through its properties (overstrength, collapse mechanism, P-Δ sensitivity), affects the response of the complete base isolated system in the nonlinear range of behaviour.

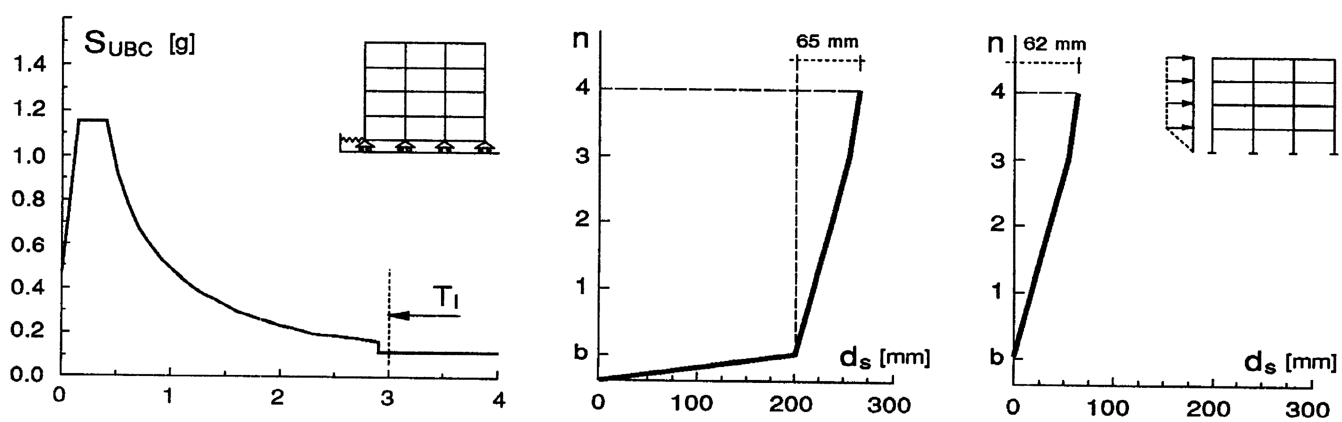


Fig. 2. Lateral floor deflections computed for frame 4/D2 by means of response spectrum analysis and of push-over analysis.

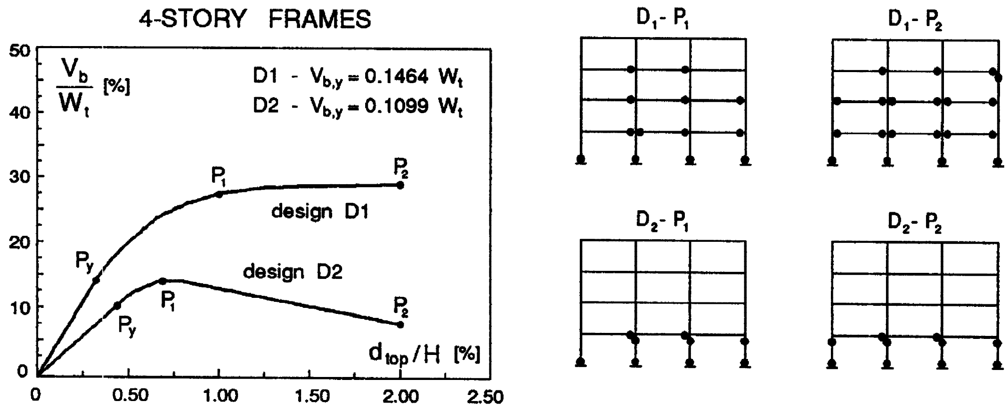


Fig. 3. Base shear - top displacement curves (*a*) and distribution of plastic hinges (*b*) for the superstructures of the 4-story frames.

As anticipated, the base shear - top displacement curves are plotted in Figs. 3*a* and 4*a*; in particular, in each figure the base shear V_b has been nondimensionalised with respect to the frame seismic weight W_t while the top displacement d_{top} is reported as a percentage of the frame total height H. All curves are conventionally plotted up to a displacement equal to the 2% of the total height H. From such figures, it is evident that the frames D1 present a strength capacity larger than the one of the frames D2; furthermore, the capacity curves of frames D2 are characterized by a more pronounced softening branch due to a larger P-Δ sensitivity and to different type of collapse mechanisms.

On each curve of Figs. 3*a* and 4*a*, together with the yielding points P_y, are provided two significant points, P_1 and P_2, for which the distribution of plastic hinges in the frames is plotted in Figs. 3*b* and 4*b*. Such distributions clearly show the different performance of the designed superstructures and, in particular, confirm that frames D1 are characterized by a strong column - weak beam behaviour while frames D2 show a weak column behaviour.

THE INFLUENCE OF THE SUPERSTRUCTURE DESIGN CRITERION

In order to investigate on the influence of the superstructure design criterion on the seismic force reduction factors to be used in BIS, several nonlinear step-by-step dynamic analyses of the previously designed base isolated steel frames have been carried out. Historical and generated earthquakes, also scaled to reach a wide distribution of yielding in the superstructures, have been used for this purpose. The response of the frames has been evaluated in terms of maximum interstory drifts and of maximum plastic rotations, which can give a relevant measure of damage in the superstructure.

Modeling assumptions and input ground motions

The analyses have been carried out by means of the DRAIN 2DX computer program (Prakash *et Al.* 1993) by adopting concentrated plastic hinge member models. Bending - axial interaction, P-Δ effects and a 1% strain hardening have been included in the analyses. The isolation devices have been modelled through linear viscous damping elements which allow to obtain reliable results for what concerns the superstructure response (De Luca *et Al.* 1996).

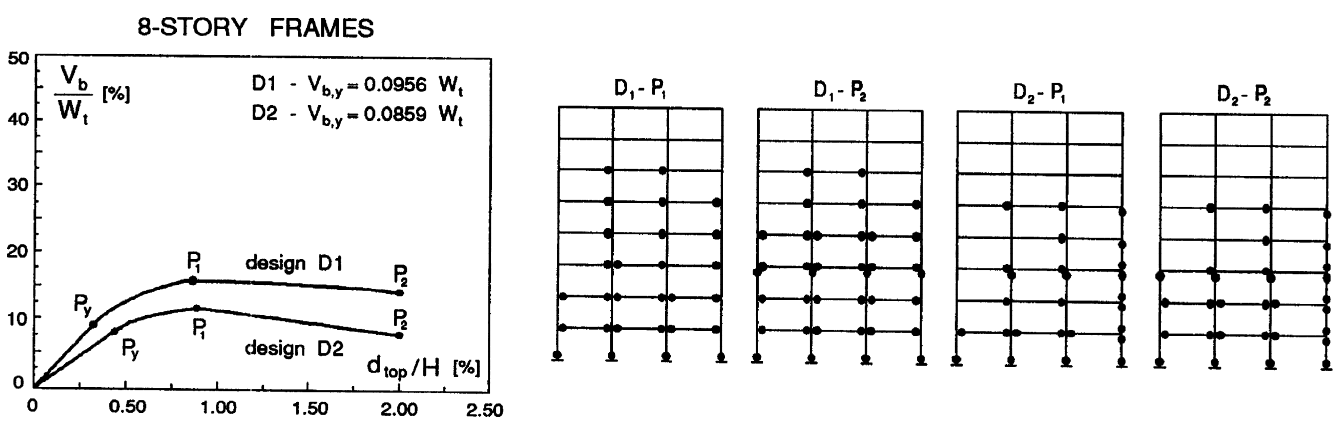


Fig. 4. Base shear - top displacement curves (*a*) and distribution of plastic hinges (*b*) for the superstructures of the 8-story frames.

Table 2. Earthquake records used in the nonlinear dynamic analyses.

Earthquake	Date	Station	Component	Duration [*sec*]	PGA [*g*]	S_a (3 *sec*, 10%) [*g*]
Imperial Valley	18.5.40	El Centro	S00E	53.80	0.348	0.0911
Friuli	6.5.76	Tolmezzo	EW	36.44	0.313	0.0270
Northridge	17.1.94	Newhall	340N/118W	59.98	0.583	0.1280
Hyogoken-nanbu	17.1.95	Kobe	EW3	56.40	0.834	0.1487
Generated	-	-	-	25.00	0.400	0.0984

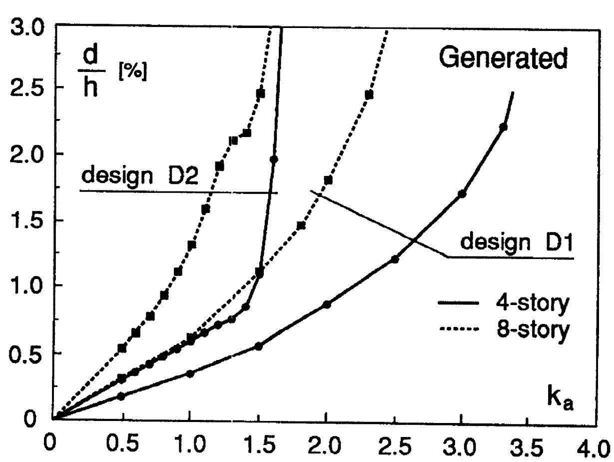
Four records of historical earthquakes, which are representative of different frequency contents, and one generated accelerogram have been used in the analyses. The characteristics of the selected records are reported in Table 2. The simulated accelerogram has been generated compatible with the elastic response spectrum defined by UBC for type S1 soil. The SIMQKE computer program (Gasparini 1976) has been used to generate the signals, by adopting an exponential intensity envelope function, a peak ground acceleration equal to 0.4 *g* and a duration of 25 seconds.

Response of the base isolated frames under scaled records

Figure 5 shows the maximum interstory drifts *d*, nondimensionalised with respect to the story height *h*, obtained for the base isolated frames D1 and D2 subjected to the generated and the Newhall records. On the *x*-axis of the figure it is reported a coefficient k_a which is representative of the amplification in the input ground motion (k_a=1 corresponds to the unscaled records). The values of the interstory drifts represented in the figure are the maximum ones computed for each frame and, in most cases, have been attained at the same story as the coefficient k_a increases. However, similar curves, which are not provided in this paper for brevity reasons, have been obtained in terms of d_{top}/H which is a more global response parameter. The cases in which the curves in terms of d/h are not very regular can be ascribed to a variation of the level at which the maximum interstory drift is attained or to a change in the distribution of plastic hinges.

The behavioural curves in terms of d/h confirm the results anticipated by examining the capacity curves of the superstructures, that is a substantial different response of frames D1 and D2. In particular, for the frames D2 amplifications k_a of about 1.2÷1.5 lead to unacceptable interstory drifts while frames D1 can undergo larger amplifications in the input ground motion.

The results presented in Fig. 5 allow to obtain straightforward indications on the difference in the seismic behaviour of base isolated structures arising from the criterion adopted in designing the superstructure, but do not allow to derive information on the most suitable numerical values to be adopted for the force reduction factor. In fact, the acceptable amplification of the selected records depends on the relation between the record itself and the design earthquake. Relevant indications on the values of the reduction factor can be instead derived if an effective value of such factor, which explicitly accounts for the above difference, is introduced, as it is proposed in the following section.



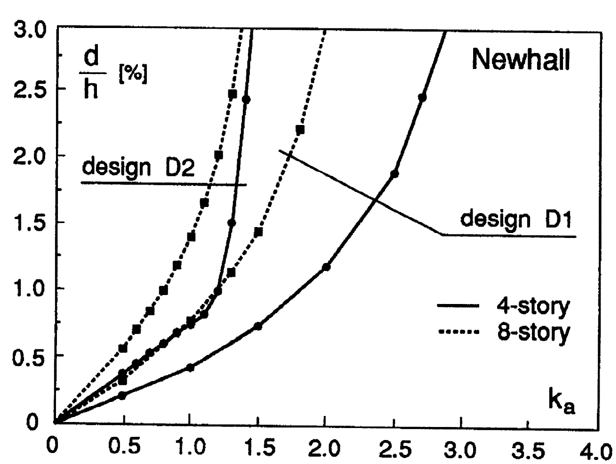


Fig. 5. Maximum interstory drift vs earthquake amplification for frames D1 and D2.

Effective reduction factors and influence of the superstructure design criterion

In order to adequately compare the response of frames subjected to different historical earthquakes there is a need for "nondimensionalising" the selected records so that the same spectral ordinate $S_a(T)$ at the period T_I is achieved. Therefore, in this paper, the reduction factor R_{WI} has been modified by introducing the ratio $S_a(T_I)/S_{UBC}(T_I)$ as follows:

$$R_{WI,eff} = R_{WI} \cdot \frac{k_a \cdot S_a(T_I)}{S_{UBC}(T_I)} \tag{1}$$

The introduction of the coefficient k_a in the expression of $R_{WI,eff}$ allows, though in an approximate way, to investigate on the effect of using different force reduction factors by scaling the earthquake without changing the frame geometry.

Figure 6 reports the maximum interstory drifts d/h as the effective reduction factor $R_{WI,eff}$, computed by Equation (1), varies; in particular, the results computed for the four frames subjected to the selected historical records have been plotted. It is worth noting that such behavioural curves provide similar trends of d/h and similar indications for the most appropriate values of the force reduction factors which should be adopted for the BIS. It has to be evidenced that, being the four selected historical records considerably different in terms of PGA, frequency content and spectral ordinate at T_I, the above results demonstrate the adequacy of the proposed procedure for computing $R_{WI,eff}$ through Equation (1).

The curves of Fig. 6 confirm what already shown in Fig. 5, that is the different behaviour of frames D1 and D2. Furthermore, these curves suggest that the effective reduction factor should be comprised between 1 and 2 since larger values lead to unacceptable interstory drifts. The indications on the values to be adopted for the force reduction factor derived from the above results confirm the ones which have been advised by other authors (Palazzo and Petti 1993, Occhiuzzi *et Al.* 1994, Dolce and Quinto 1994), even though they have been obtained through different procedures. Furthermore, also the dependence of the advisable values for the force reduction factor on the superstructure design criteria, clearly evidenced herein, confirms the conclusion reported in (Calderoni *et Al.* 1995), where it is underlined that reduction factors larger than 1 can be assumed only if the rules for capacity design are adopted for the superstructure.

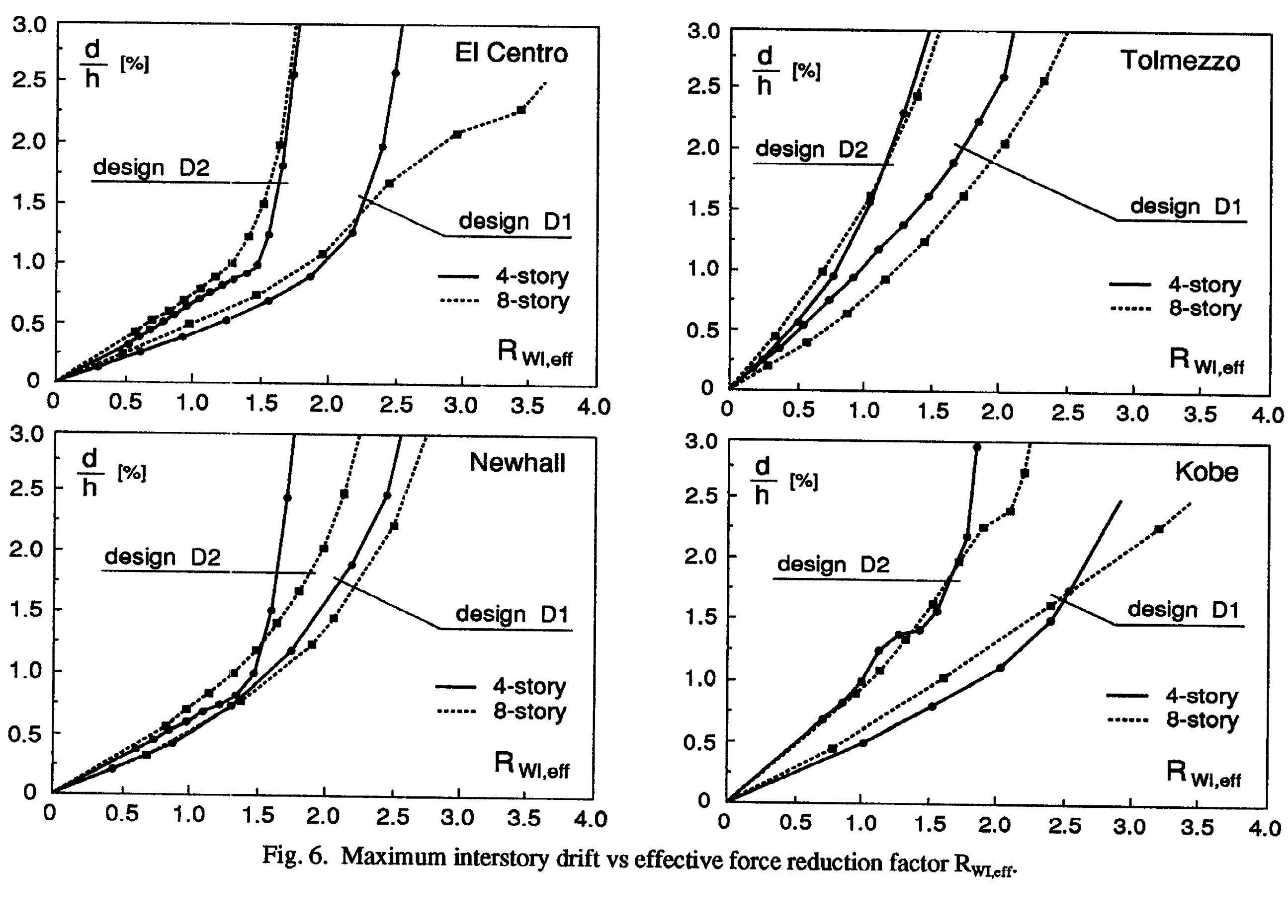


Fig. 6. Maximum interstory drift vs effective force reduction factor $R_{WI,eff}$.

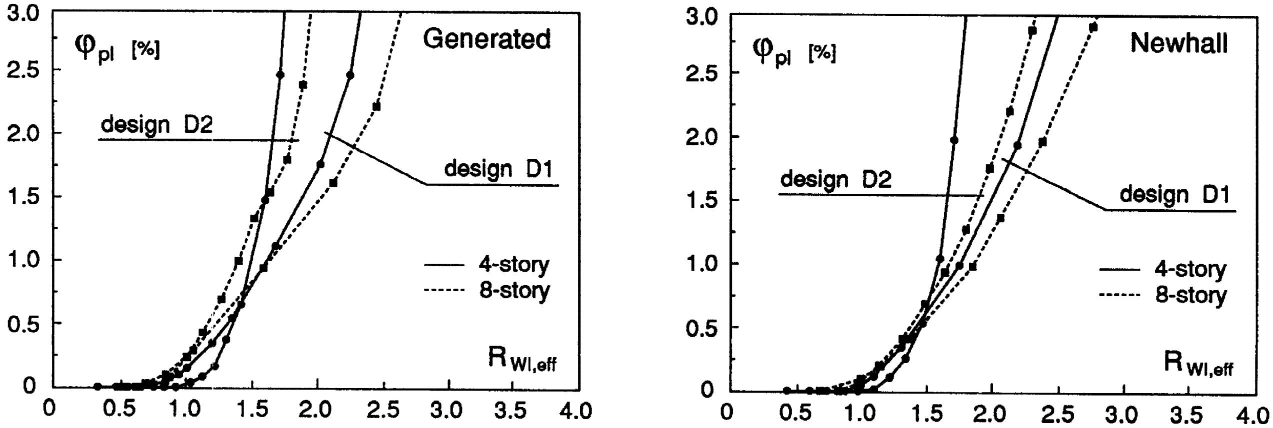


Fig. 7. Maximum plastic rotation vs effective force reduction factor $R_{WI,eff}$.

The results in terms of interstory drifts presented in Fig. 6 are not representative of the effective inelastic actions in the frame members since the ratios d/h contain also an elastic part. Results in terms of maximum plastic rotation φ_{pl} are instead more significant since clearly show the potential damage in the structural members. Such results are presented in Fig. 7, where the maximum value of φ_{pl} computed among all beams and columns are reported on the y-axis, as the $R_{WI,eff}$ varies. Figure 7 shows that the first plastic hinges develop in proximity of $R_{WI,eff}$ factors equal to the unity, thus confirming the adequacy of the procedure for defining the effective reduction factor since $R_{WI,eff}$ equal to 1 represents the threshold beyond which the frames start experiencing inelasticity. The sudden increase in φ_{pl} again occurs at reduction factors ranging between 1 and 2 and, in particular, the sharp increment in φ_{pl} takes place at $R_{WI,eff}$ of about 1.5 for frames D2 and at $R_{WI,eff}$ larger than 2 for frames D1. These results evidence that if the base isolation is a design philosophy which must provide higher performance of the structures (no distribution of plasticity), effective reduction factors ranging between 1 and 1.5 must be adopted in designing the superstructures of BIS.

In order to capture a parameter which can concisely characterize the influence of the superstructure design on the seismic behaviour of BIS and in order to account for the results obtained by means of the push-over analyses, the results of Figs. 6 and 7 have been corrected by considering the overstrength of the frames. Figure 8 presents the same results in terms of maximum plastic rotations of Fig. 7, but on the x-axis it is reported the effective reduction factor $R_{WI,eff}$ divided by a coefficient k_s. This coefficient has been computed through the push-over analyses as the ratio between the maximum base shear $V_{b,max}$ which the frame can bear and the yielding base shear $V_{b,y}$. The obtained curves evidence that such a representation in terms of $R_{WI,eff} / k_s$ leads to a remarkable reduction in the scatter among the values of φ_{pl} relative to the different superstructure designs. This reduction in scatter of results is more evident when the d/h values are plotted, as it is done in Fig. 9.

The above results allow, even though obtained by an approximate procedure (by scaling the input ground motions), to conclude that the values of the force reduction factors to be adopted in designing BIS can be also assumed to be dependent on the overstrength k_s characterizing the superstructure. Therefore, larger reduction factors can be adopted only if the design of the superstructure is affected by a larger overstrength.

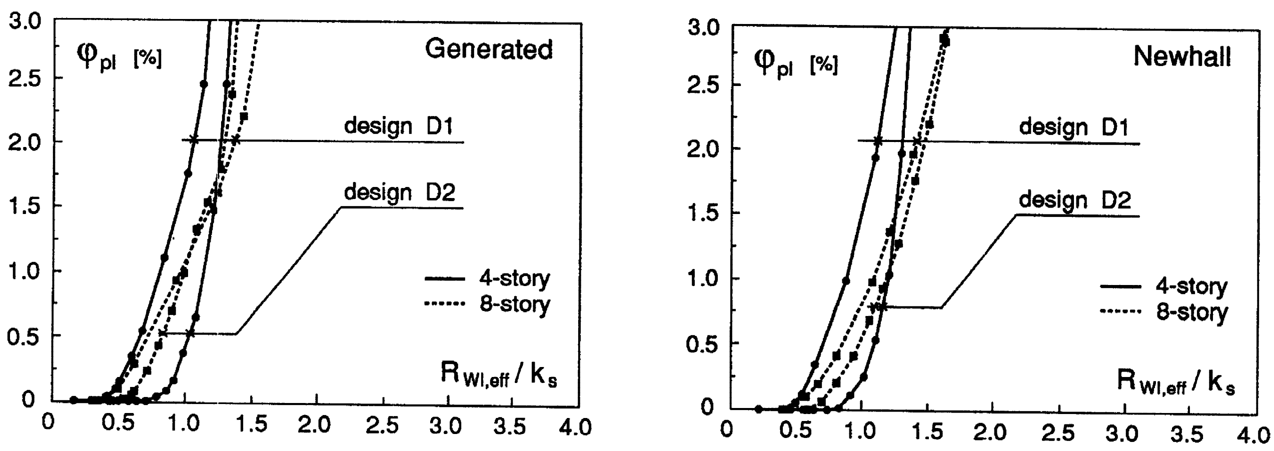


Fig. 8. Maximum plastic rotation vs reduced effective force reduction factor $R_{WI,eff} / k_S$.

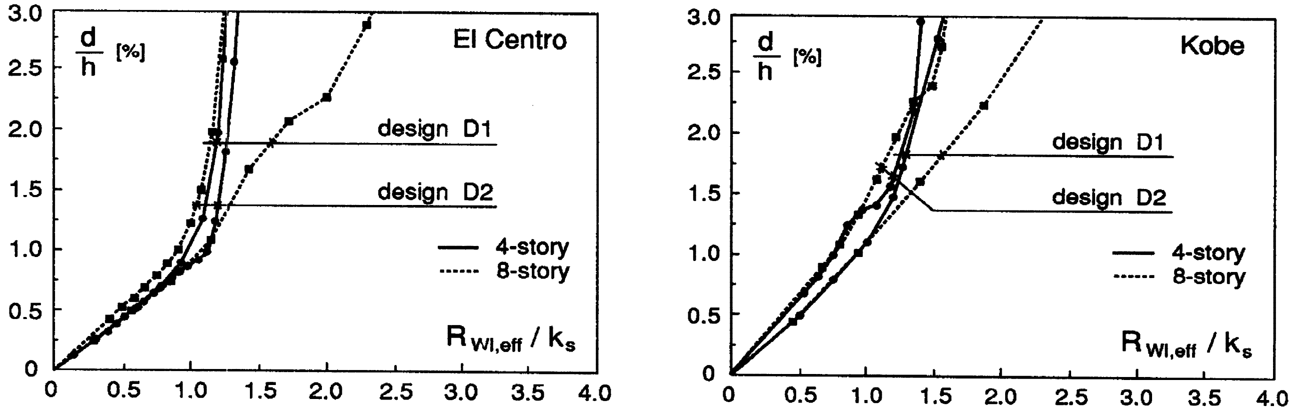


Fig. 9. Maximum interstory drift vs reduced effective force reduction factor $R_{WI,eff} / k_S$.

CONCLUSIVE REMARKS

In this paper, a contribution to the definition of the force reduction factors to be adopted when designing BIS has been provided by analysing how the design criterion of the superstructure affects the nonlinear behaviour of the entire base isolated system. In particular, a comparison between the nonlinear seismic response of base isolated steel SMRF in which the superstructure is respectively designed with and without allowance for capacity design and for drift limitations has been carried out. Several historical input ground motions have been used in the analyses and, in order to adequately compare the response, an effective value of the force reduction factor has been proposed. Results in terms of maximum interstory drifts and of maximum plastic rotations have allowed to conclude that such factor should be comprised between 1 and 2, but the largest values can be adopted only if the design leads to an adequate overstrength in the superstructure.

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