Seismic behaviour of limited ductility buildings

A.H. BARBAT[a] J.C. VIELMA[b] S. OLLER[a]

[a]*Technical University of Catalonia, UPC, Barcelona, Spain*

[b] *Lisandro Alvarado University, UCLA, Barquisimeto, Venezuela*

Rezumat: În această lucrare se studiază cele mai importante aspecte ale comportării şi proiectării seismice a clădirilor de beton armat cu ductilitate limitată, aşa cum ar fi cele cu planşee ciupercă sau cu grinzi plane. Aceste tipuri de structuri sunt cele mai utilizate in Spania la proiectarea noilor clădiri şi mai multe norme de proiectare din lume nu recomandă utilizarea lor în zone seismice. Comportarea seismică acestor clădiri se examinează aici utilizând o analiză neliniară incrementală (pushover analysis) care permite obținerea curbelor lor de capacitate şi se compară cu comportarea clădirilor pe cadre proiectate cu ajutorul normelor spaniole şi din Statele Unite ale Americii. Cele mai importante rezultate ale acestui studiu arată că doar clădirile pe cadre au suficientă ductilitate şi suprarezistență pentru a garanta un bun comportament seismic. Comportamentul clădirilor cu ductilitate limitată este puternic influențat de tipologia structurală; chiar atunci când se utilizaează armatură ductilă sau o bună armare a secțiunii transversale, creştera ductilității nu este semnificativă.

Abstract: The most important aspects of the seismic design and behaviour of reinforced concrete buildings with limited ductility, like the buildings with waffled slabs or flat beams, are examined in this work. The structures with these typologies are the most used in Spain for new buildings and many seismic codes do not recommend their use in seismic areas. The expected seismic performance of these structures is studied herein by means of incremental non linear structural analysis (pushover analysis) which provides capacity curves. Their behaviour is compared with that of buildings with moment resisting frames designed according to the Spanish EHE and NCSE-02 codes and also to the ACI-318 (2005) and IBC-2003. The most important results of the study show that only the moment-resisting framed buildings exhibit sufficient ductility and overstrength to guarantee a stable seismic behaviour. The behaviour of limited ductility buildings is strongly influenced by the structural type; even if they are reinforced with ductile steel or if their confinement if improved, they exhibit slightly higher ductility.

Keywords: seismic design, structural analysis, higher ductility

1. Introduction

Among the building typologies used nowadays in the seismic areas of Spain, the most frequent have flat beams and waffled slabs (Barbat et al. 2006 and 2008). Earthquake-resistant codes, in general, and Spanish code NCSE-02, in particular, assign ductility values of two to these buildings and classifies them as restricted ductility buildings. These values are fixed by the code on the premise that buildings expressly designed to have low ductility have a low capacity of energy dissipation and a non-adequate seismic behaviour. The adequacy of the response of a structure to a given seismic threat can be evaluated by using an incremental nonlinear structural analysis providing capacity curves (Erberik and Elnashai 2006), examining especially the structural ductility and the overstrength. It has to be noted that restricted ductility buildings have been not extensively studied yet using this procedure. In the past, capacity and performance-based procedures have been used mostly in evaluating the seismic behaviour of moment-

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resisting frames (Mwafi and Elnashai 2002; Fragiacomo *et al.* 2006). It has to be also mentioned that, apart from the UBC-97 and the IBC-2003, no other earthquake-resistant code directly refers to overstrength values, which are very important in the determination of response reduction factors (Vielma *et al.* 2006).

The objective of this article is to calculate the ductility and overstrength values of buildings with restricted ductility by means of pushover analysis. The drift values corresponding to the yielding point are obtained by using the idealized bilinear form of the capacity curve (Park 1988). Once the non-linear response is determined, the benefits of improving the ductility of the steel reinforcements and the longitudinal and transversal confinement are evaluated. Finally, the non-linear response of the buildings with restricted ductility is compared with that of two moment-resisting framed buildings: one with intermediate ductility, designed according to the Spanish EHE guidelines specifications; and the other one with high ductility, designed according to ACI-318 code specifications.

2. Description of the studied buildings

To elucidate how structural typology and design influence the global response of building structures, four buildings with different characteristics have been designed and studied. The first two buildings, one of which has waffled slabs, and the other flat beams, have restricted ductility and are designed using low reduction factors. The third building, with moment-resisting frames, is designed according to the Spanish EHE guidelines and has medium to high ductility values. Finally, the fourth building with moment-resisting frames is designed according to the ACI-318 code specifications in order to fulfil requirements for high ductility.

2.1 Building with waffled slabs

The reinforced concrete building with waffled slabs (design ductility μ=2) has ribs which run along the lines that join the ends of its columns. It has three stories: the first is 4,5 m high, whereas the other two are 3,0 m high —a typical configuration for a building whose ground floor is intended for commercial use—. It has four spans parallel to the x-axis and three spans parallel to the y-axis (see figure 1a). Figure 1a also shows an equivalent frame of this building. The slab has bidirectional ribs, whereby the ribs are orthogonal to each other. The total depth of the slabs is 30 cm.

2.2 Framed building with flat beams

In the case of the structure with flat beams (design ductility μ=2), a unidirectional slab is supported on these beams (see Figure 1b). The flat beams are used both in the direction that receives the slab ribs and in that of the bracing. The story layout of the building is similar to that of the building with waffled slabs, except that the columns have been aligned with what could be defined as the resistant lines of the orthogonal frames, as observed in Figure 1b.

Just as in the case of the building with waffled slabs, the ground floor of the flat beam building is the tallest and the effect of weak ground floor is expected. However, the remaining stories have the same height and number of spans. Figure 1b shows the typical plan and elevation views of this building.

2.3 Moment-resisting framed buildings

Two buildings were designed to study the response of moment-resisting framed buildings: one according to the Spanish EHE and NCSE-02 codes (design ductility μ=4); the second one using ACI-318 (2005) and IBC-2003 (design ductility μ=6). Both frames are geometrically similar to the building with flat beams. The slabs of the building are unidirectional. Seismic design criteria are added in order to increase the column dimensions, thereby yielding a structure with strong columns and weak beams.

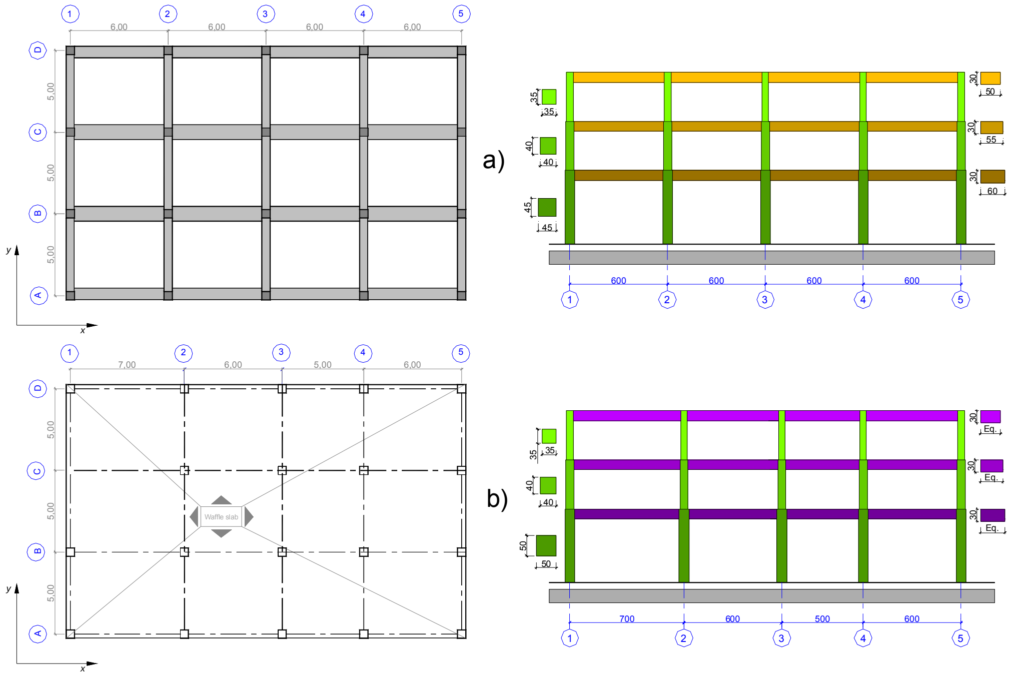

Figure 1. a) Typical plan and elevation view of the building with waffled slabs and b) the framed building with flat beams

3. Pushover analysis

By applying the modal analysis foreseen in the Spanish seismic code NCSE-02, the equivalent seismic forces corresponding to all the levels of the building have been calculated (Barbat *et al.* 2005 and 2007). The same inelastic spectrum was also used to calculate the seismic forces applied to the building with waffled slabs.

The aim of the non-linear analysis was to obtain a more realistic response of the buildings designed according to the linear elastic method outlined in the code NCSE-02. This allows a clear demonstration of how adequate earthquake-resistant design measures improve structural ductility, while also revealing how non-linear response challenges certain simplifications made during the elastic structural analysis.

3.1 Equivalent mechanical models of the buildings

The results were calculated using 2D models of the buildings defining for each of them representative frames. The non-linear analysis was performed with a finite element program (PLCd 1991) which enables modelling reinforced concrete as a composite material to which the Mixing Theory was applied. Discretization of the frames was performed with elements whose lengths vary in function of the column and beam zones with special confinement requirements. These confinement zones were designed according to the general dimensions of the structural elements, the diameters of the longitudinal steel, the clear of spans and the story heights.

3.2 Calculation method: pushover analysis

To evaluate the inelastic response of the four structures, a pushover analysis was performed by applying a set of lateral forces representing the seismic actions corresponding to the first vibration mode. The lateral forces were gradually increased starting from a zero value, passing through the value which induces the transition from elastic to plastic behaviour, and ultimately reaching the value which corresponds to the ultimate drift (*i.e.* the point at which the structure can no longer support any additional load and collapses). Before subjecting the structure to lateral loads simulating seismic action, it was first loaded with the gravity loads, in agreement with the combinations applied in the elastic analysis.

The non-linear static response obtained via finite element techniques was used to generate the idealized bilinear expression shown in Figure 2, which has a secant segment from the origin to a point on the capacity curve that corresponds to a 75% of the maximum base shear (Park 1988). The second segment, which represents the branch of plastic behaviour, was obtained by finding the intersection of the aforementioned segment with another, horizontal, segment which corresponds to the maximum base shear. The use of this compensation procedure guarantees that the energies dissipated by the ideal, bilinear, system and by the more realistic finite element calculated model, are equal (see Figure 2).

In this case of a simplified non-linear analysis, there are two variables that characterize the quality of the seismic response of buildings. The first one is the structural ductility μ, defined as $\mu = \Delta_u / \Delta_y$,where Δy is the yield drift and Δu is the ultimate drift; these values can be obtained from the idealized capacity curve shown in Figure 2.

The second variable influencing on the quality of the seismic response of a building is the overstrength $R_R = V_y / V_d$, where V_d is the design base shear and V_y is the yielding base shear. The overstrength R_R is like a safety factor applied in the seismic design.

3.3 Non-linear response of the building with waffled slabs

The capacity curve of the buildings with waffled slabs, shown in Figure 2, is calculated with a mechanical model similar to the *equivalent frame* defined in the code ACI-318 (ACI Committee 318 2005). The analysis is performed by means of the finite element method, using damage and plasticity constitutive models, and the Mixing Theory (PLCd 1991; Barbat et al. 1997; Mata et al. 2007 and 2008; Faleiro et al. 2008). To control the energy dissipation and to ensure the correct behaviour of the structure, approximate mean values for strength and fracture energy were used for each constituent material (i.e. steel and concrete) (Car *et al.* 2000 and 2001).

The structural ductility of the calculated frame is $\mu = 2,91/1,85 = 1,57$, where Δy and Δu are obtained from the idealized capacity curve of Figure 2. The obtained value is lower than the design value $\mu = 2$ foreseen by the Spanish code NCSE-02 for this structural type. The overstrength is $R_R = V_y/V_d = 1,92$, that is, the structure exhibits high overstrength level. The ductility value calculated for this structural class suggests that the ductility factor values considered in the NCSE-02 earthquake-resistant code should be revised.

The low ductility response of the buildings with waffled slabs can be attributed to the formation of plastic hinges in the transition points between the abacus and the ribs of the slab at the first floor. The elements of the slabs are subjected to bending induced by gravitational loads, as well as to the demands of the seismic forces; hence, the zones which require special reinforcement are those closest to the slab-column node and to the middle of the span, where the greatest bending moments frequently appear. However, efficient confinement in the central slab zone is technically complicated. The described effect suggests a possible mechanism for structural failure during earthquakes and, consequently, the low level of ductility of the structure.

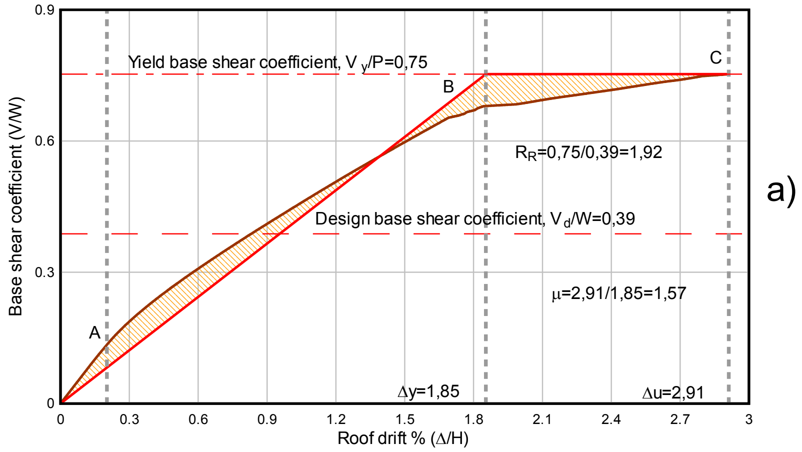

Figure 2. Capacity curve and its idealized form for waffled slabs buildings

3.4 Non-linear response of the framed building with flat beams

It is technically difficult to reinforce adequately flat beams in order to assure a ductile behaviour of the structure, what justifies the low ductility value suggested by the Spanish seismic code NCSE-02. Figure 3 shows the global response of the framed building with flat beams reaching the ultimate drift (*i.e.* the drift before total structural collapse) which, together with the yield drift, enables calculating the structural ductility. The ductility obtained for the building with flat beams is 1,54, a value which raises some concern, given that the NCSE-02 earthquake-resistant code recommends a response reduction factor of 2.

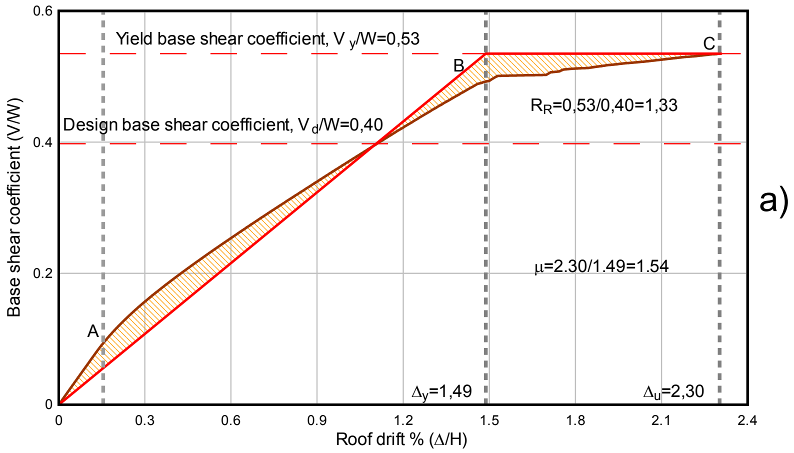

Figure 3. Idealized capacity curve for the exterior frame of the building with flat beams

The response of the building with flat beams shows that the stability of the structure depends on the behaviour of the beams. This is an important aspect to consider when deciding between using a moment-resisting frame or a frame with flat beams, given that the latter shows lower ductility values than those prescribed by the code and, consequently, can have lower response reduction factors R.

3.5 Non-linear response of the moment-resisting framed buildings

The response of the moment-resisting framed buildings was calculated and compared with the results obtained for the limited ductility structures. Figure 4 shows the capacity curve obtained for the building designed according to the Spanish codes. The curve clearly illustrates how this structural type is capable to sustain a stable ductile response, which is reflected by the high value for the final drift. Based on the idealized bilinear curve of Figure 4, a ductility factor of 5,17 is obtained —a value higher than that considered in the design, which was 4—. This means that buildings with deep beams have a ductile response to seismic forces, as well as adequate overstrength.

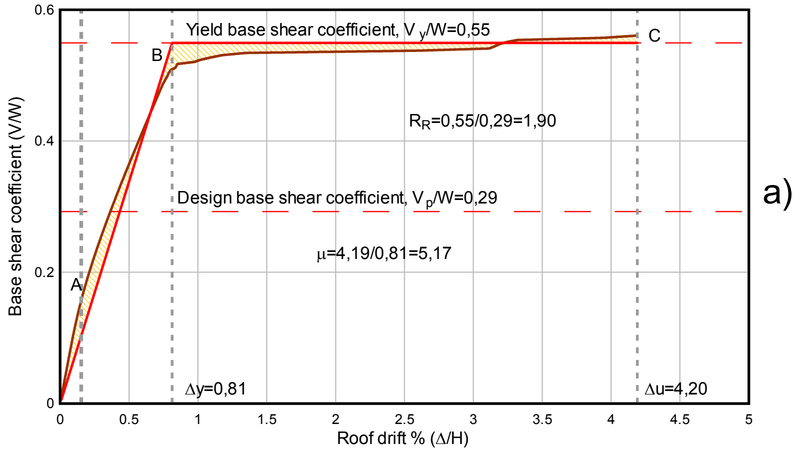

Figure 4. Idealized capacity curve for the moment-resisting framed building designed according to the Spanish EHE guidelines

Figure 5 show the capacity curve for the building designed according to ACI-318. The main difference between this building and the former (see Figure 4) is, by one hand, that the Spanish earthquake-resistant code limits the ductility factor for this class of buildings to four and, by the other hand, that the Spanish code requires less transversal and longitudinal reinforcement than the ACI-318 (2005) code, which enables greater dissipation capacity. The non-linear response of the studied moment-resisting framed buildings is typical for reinforced concrete low-rise structures which generally undergo plastic hinges at the base of their ground floor columns. This general tendency stems from the fact that designing buildings with strong columns and weak beams implies the predominance of gravitational loads on the beams which, ultimately, require larger cross sections than those of the columns.

4. Possibilities of improving the seisimc repsonse of buildings with restricted ductility

The results of the non-linear analysis of the buildings with restricted ductility raise the question: Can their seismic behaviour be improved at the design stage, to reach the maximum ductility values prescribed in the code NCSE-02 while maintaining the same structural type? This section discusses this possibility for buildings with either waffled slabs or flat beams, based on the pushover analysis performed using finite element models. The improved responses are finally compared with those obtained for buildings with moment-resisting frames.

With the aim of studying the influence of the steel type on the non-linear response of buildings with waffled slabs, steel with different mechanical characteristics are considered. The buildings were calculated by considering for the reinforcement either welded ductile steel (WD), whose characteristics makes it recommendable for the design of structures according to the EHE and EC-8 specifications or welded steel (W) (see Table 1).

For both cases, the yield stresses B400 and B500 were considered.

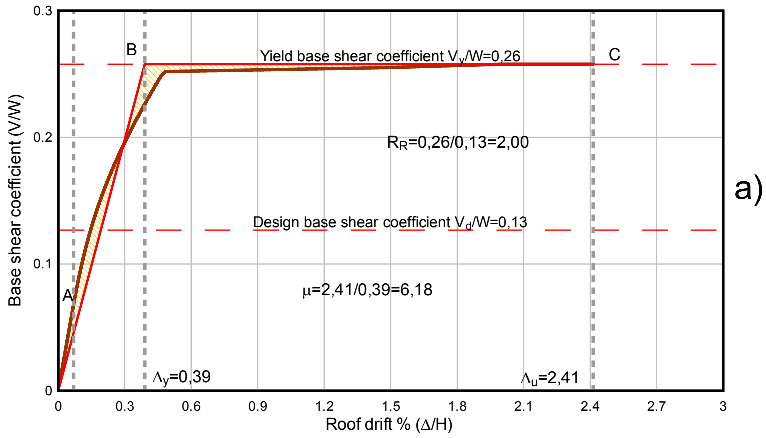

Figure 5. Idealized capacity curve for the moment-resisting framed building designed according to ACI-318 (2005)

Table 1 Characteristics of the steel recommended for the design of ductile reinforced concrete buildings

	Code			
	Eurocode 8		EHE	
Steel type	B	C	B 400 WD	B 500 WD
Yield stress fy (N/mm²)	400 to 600	400 to 600	400	500
Ultimate stress fs (N/mm²)	-	-	480	575
Ratio fs/fy	≥ 1,08	≥ 1,15 and ≤ 1,35	≥ 1,20 and ≤ 1,35	≥ 1,15 and ≤ 1,35
Maximum strain ε_{max} (%)	≥ 5,0	≥ 7,5	≥ 9,0	≥ 8,0
Ultimate strain, ε_u (%)	-	-	≥ 20,0	≥ 16,0

The results of the pushover analyses are shown in Figure 7, which reveals that frames reinforced with ductile steel have only a slightly more ductile response than do those reinforced with non-ductile steel. Hence, the global response of the building is influenced to a much greater extent by the general configuration and the structural typology chosen than by the characteristics of the reinforcement steel.

The behaviour of buildings with flat beams that are reinforced with ductile (WD) or non-ductile (W) steels, and with yield stress values of 400 or 500, has been also studied. Just as in the case of the buildings with waffled slabs, the ductile capacity of this type of building was found to be far more influenced by the structural type than by the type of steel (see Figure 7).

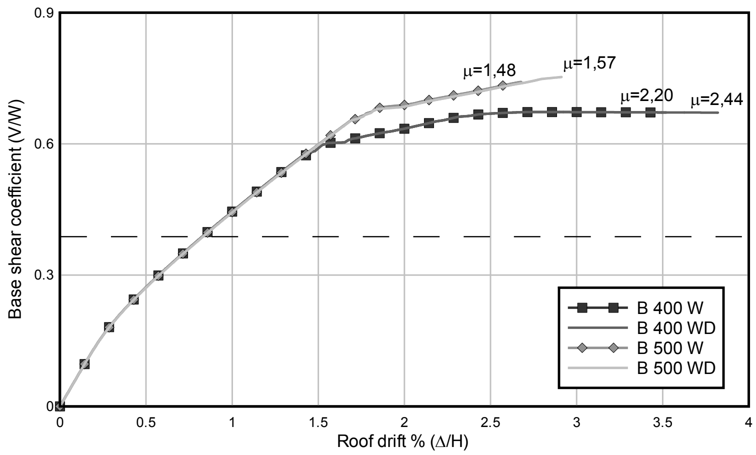

Figure 6. Capacity curves for the building with waffled slabs reinforced with either ductile steel (WD) or non-ductile steel (W)

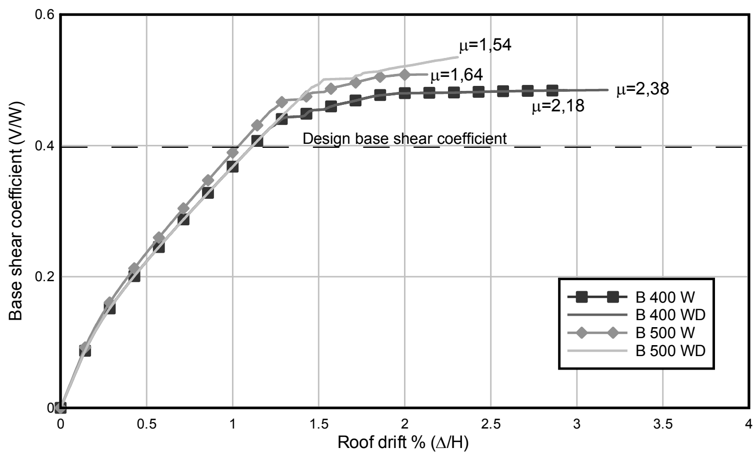

Figure 7. Capacity curve for the building with flat beams reinforced with steel of different mechanical characteristics

Finally, Figure 8 shows the same results obtained for the moment-resisting frame building reinforced with different types of steel. Observe that, in this case, increasing the ductility of the steel leads to a major increase in structural ductility.

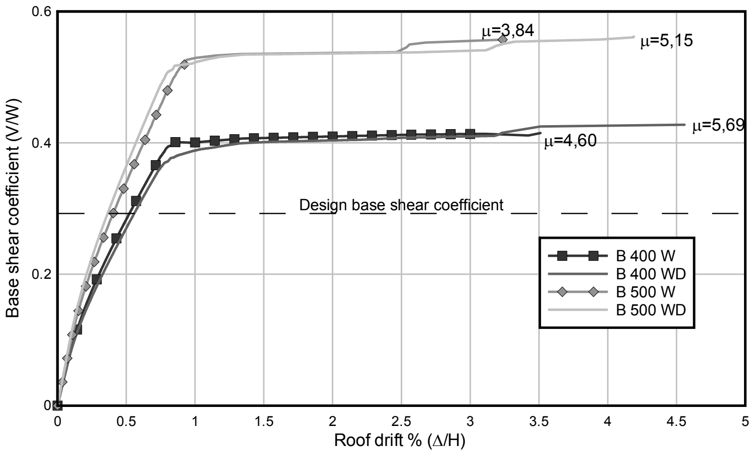

Figure 8. Capacity curve for the moment-resisting frame building reinforced with steel of different mechanical characteristics

5.Conclusions

A procedure of non-linear static analysis with force control has been used the yield drifts of the analyzed structures have been established using the idealized bilinear capacity curves.

Among the studied cases, only the moment-resisting framed buildings exhibit sufficient ductility and overstrength to guarantee a stable behaviour, including for ductility values higher than the design ones. The obtained results also confirmed the premise that greater resistance leads to less ductility: structures modelled with B500 WD steel have higher overstrength and lower ductility than do those built with B400 WD steel.

The global behaviour of buildings with flat beams and with waffled slabs is influenced in great part by the structural type. If these buildings are reinforced with WD steel, they exhibit slightly higher ductility than if reinforced with W steel. However, for the case of moment-resisting framed buildings, the use of WD steel instead of W steel provides a substantial increase in the ductility. Moreover, the ductile response of the buildings with flat beams cannot be greatly improved via confinement of its elements; good confinement is only advantageous for buildings with moment-resisting frames.

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