9. Faggiano B, Mazzolani F.M., (1999): Proposals for improving the steel frame ductility by weakening. XII Congresso C.T.A, Construire in Acciaio: Struttura e Architettura, Napoli, Italy, 269-280.
10. Iwankiw N.R, Carter C.J. (1996): The dogbone: A new idea to che on. Modern Steel Constructions, Vol. 36, No. 4, 18-23.
11. Mazzolani F.M., Piluso V., (1996): Theory and Design of Seismic Resistant Steel Frames. E & FN Spon London, U.K.
12. Paulay T., Bachmann H., Moser K., (1990): Erdbenbemessung von Stahlbetonhochbauten. Birkhauser Verlag.
13. Plumire A. (1996): reduced beam section: A safety concept for structures in seismic zones. Buletinul Stiintific al Universitatii "Politehnica" din Timisoara, Vol. 41 (55), Fasc. 2, 46-60.
14. Plumire A. (1999): General report on local ductility. Proceedings of the Stability and Ductility of Steel Structures, SDSS Colloquium, Timisoara, Eds. D Dubina and M. Ivanyi, Special Issue in J. Construct. Steel Research.
15. SAC, (1996): Connection test summaries, FEMA 267/SAC-96-02, SAC Joint Venture, Sacramento, California, USA.
16. Sabol T.A, Engelhardt M.D. (1996): Overview of the AISC Northridge moment connection test program. J. Construct. Steel Research, Vol. 46, No. 1-3, CD paper No 857.
17. Tremblay R, Tcobotarev N., Filiatraut A., (1997): Seismic performance of RBS connections for steel moment resisting frames: Influence of loading rate and floor slab. Behaviour of Steel Structures in Seismic Areas, STESSA' 97. Eds. F.M. Mazzolani and H. Akiyama, E & FN Spon London, 664-671.
18. Srivanich W., Shen J., Kitjasateanphun J., (1999): Seismic performance of cover-plate strengthened frames. The Struct. Design of Tall Buildings, No. 8, 215-246.

of the aforementioned factors, it is better to minimize the composite floor-slab action with adequate detailing measures. So, in order to allow the complete formation of plastic hinge, it is recommended to eliminate the shear studs, to create a free zone, or in a simpler way to decoupling the concrete slab from the column flange using an elastically compressible material. (Fig. 10a,b,c).

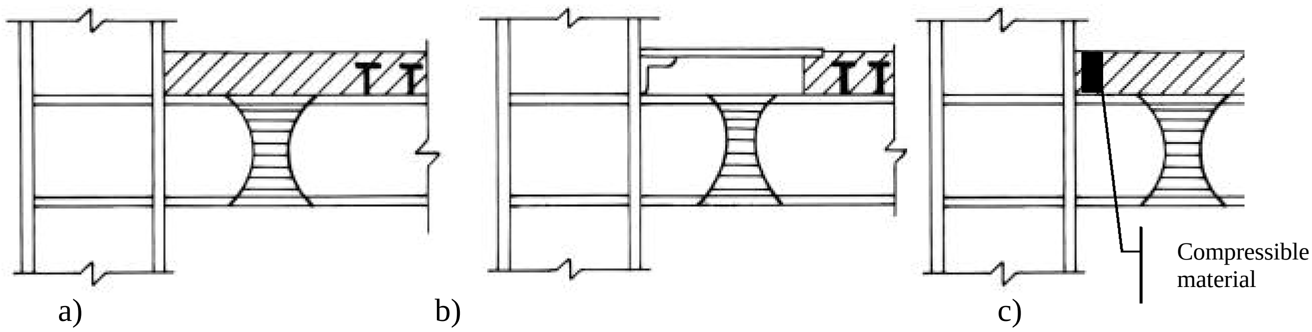

Fig. 10 Floor-slab constructional details

5. CONCLUSIONS

New seismic design provisions based on constructional details must be developed and introduced in modern seismic codes. The paper outlines some alternative solutions in order to promote a reliable and predictable plastic mechanism based on constructional details through using of capacity criteria. The importance of plastic hinge position must be recognized in design assumptions. Strengthening solutions may led in oversized columns in case of high vertical loads or high material variation, while reduced beam sections represents a more attractive solutions, from this point of view. In case of hybrid frames specific criteria must be respected assuring that the design yield strength is equal with the real yield strength, otherwise the intended plastic mechanism may not be achieved.

Using such constructional details, which improve the inelastic performance, an extension in classification of moment frames may obtain, allowing the implementation of performance based design in earthquake design of steel moment resisting frames.

6. REFERENCES

1. Anastasiadis A, Gioncu V. (1998): Influence of joint details on the local ductility of steel moment resisting frames, 3rd National Greek Conference on Steel Structures, Thessaloniki, 30-31 October, Greece, 311-319.
2. Anastasiadis A (1999): Ductility problems of steel moment resisting frames, Ph. D dissertation, University “Politehnica” Timisoara, Romania.
3. Anastasiadis A., Mateescu G., Gioncu V., Mazzolani F.M. (1999): Reliability of joints for improving the ductility of MR frames, Proceedings of the Stability and Ductility of Steel Structures, SDSS Colloquium, Timisoara, Eds. D Dubina and M. Ivanyi, Elsevier, U.K, 259-268.
4. Chen S.J., Yeh C. H., Chu J.M, (1996): Ductile steel beam-to-column connections for seismic resistance., J.Structural Eng., Vol. 122, No. 11, 1292-1299.
5. Chen S.J., Chu J.M., Chou Z.L., (1997): Dynamic behavior of steel frames with beam flanges shaved around connection. J. Construct. Steel Research, Vol. 42, No. 1. 49-70.
6. Chen S.J., (1998): Effects of floor slab on the seismic behavior of steel beam-to-column connections with reduced beam section. J. Construct. Steel Research, Vol. 46, No. 1-3, CD paper No 218.
7. Engelhardt M, Sabol T. A., (1995): Lessons learned from the Northridge earthquake: Steel moment frame performance., A new Direction in Seismic Design, Tokyo 9-10 October, 1-14.
8. Engelhardt M Winneberger T., Zekany A.J., Potyraj T.J., (1996): The dogbone connection: Part II. Modern Steel Construction. Vol. 36, No. 8, 46-55.

3.3 Hybrid MR-frames

Exploiting the capacity design philosophy, one can obtain a SC-WB system, simply, using columns with higher steel grade than beams (e.g. columns from Fe 510 and beams from Fe 360). In the same time, it is easier to satisfy the capacity criteria, which in some cases led to thick doubler plates or thick welds or bolts at the beam-to-column connection. Additionally, this concept allows yielding of beam at column face due to inherent difference of strength between beam and column. It is obvious that such a procedure requires the correct determination of mechanical properties, especially yield stress, otherwise it is questionable. Recent experience from Northridge earthquake has confirmed this fact. Hybrid frames, with beams designed as ASTM A36 steel and columns as ASTM A572 Grade50, have been widely used in U.S practice. One of the main factors for brittle damage , for these frames, was the high level of actual yield strength in ASTM A36 beams (≈ 340 MPa) as compared with the minimum specified value (250 MPa). However, the influence of yield stress variation was recognized in EC-8 Part 1.3, clause " Control of the design and construction". So, in order to design reliable hybrid frames the aforementioned requirements must be respected.

The effectiveness of hybrid frames, in terms of plastic rotation and storey drift, was demonstrated from dynamic inelastic analysis in limited number of low-rise steel moment frames (2bays-3 stories), using near and far source accelerograms [2] (Fig. 9a,b). Among the other advantages with the hybrid concept an ordinary moment frame may be translated in a special moment resisting frame. (Fig. 9a).

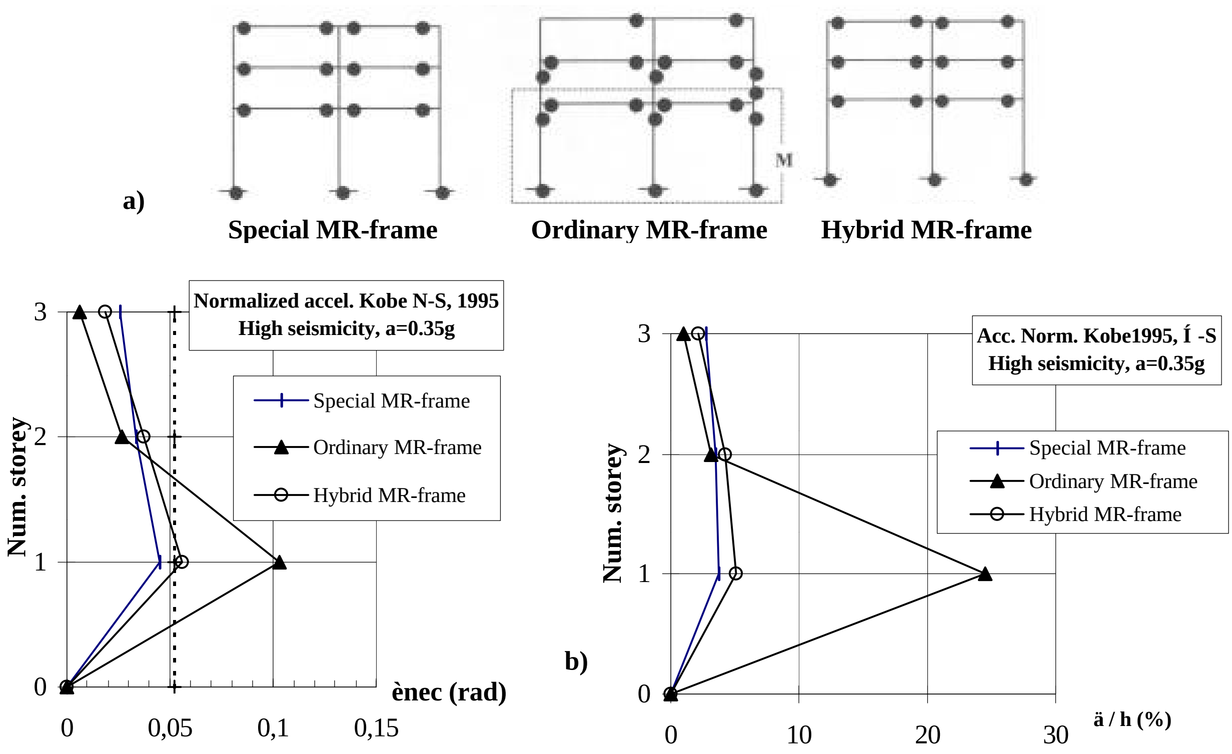

Fig. 9 Performance of hybrid frames

4. FLOOR SLAB CONSTRUCTIONAL DETAILS

Generally, the presence of concrete floor-slab has a positive action, providing lateral bracing to the top beam flange, and a negative action, which prevents the free local buckling of the top beam flange also adding stains to the bottom beam flange. Analyzing selected preliminary results of research on floor-slab effects indicate that the beam-buckling mode depends on beam size, detailing of connection zone, stress condition (e.g positive or negative moment), while the increase of beam flexural strength, of about 10-20%, depends on connection detailing, stress condition and appearance of lateral-torsional buckling [6,17]. Taking into account the difficulty in quantification where the notations used in relation (4) were explained above. Relation (4) accounts the influence of strain-hardening effects, yield stress variation, loading conditions. Drift requirements, which govern the seismic design of steel moment frames, provide an overstrength, so, some concerns raised about the inherent strength reduction of the RBS on frames strength does not represent a real problem, counterbalanced with the oversized members. In contrast, weakening as compared with strengthening strategy reduces the shear force in panel zone, dimensions at the weldements, and it is easier to satisfy a SC-WB mechanism. Using the relations (1), (2), (4), and assuming the formation of plastic hinge at the same distance, L_p, for the weakening and strengthening solution, one can see in figure 8 that in case of weakening the plastic moment at the column face, $M_{c.f}$, was reduced due to the greater reduction of $M_{p.red}$, while in case of strengthening the movement of plastic hinge promote the increasing of moment at the column face.

Table 1. Example of direct sizing for constant reduced beam section

Commercial section	$M_{p.b}$ / qL^2	L_p / L	$M_{p.red}$ / $M_{p.b}$	b_1 / b	
				0.95 $M_{p.red}$ / $M_{p.b}$	0.90 $M_{p.red}$ / $M_{p.b}$
IPE 300	0.25	0.040	0.843	0.692	0.627
		0.070	0.729	0.527	0.470
		0.10	0.620	0.367	0.319
	0.50	0.040	0.881	0.748	0.679
		0.070	0.794	0.622	0.559
		0.10	0.710	0.497	0.444

a) Detail of RBS **b) Design concept**

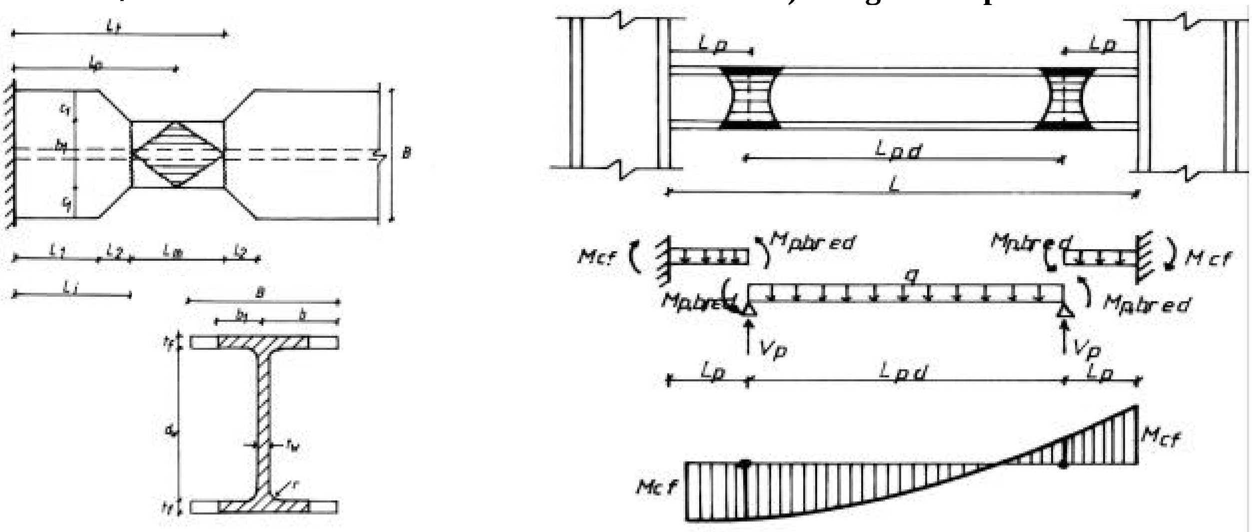

Fig. 7 Representation of RBS and design concept for weakened solutions

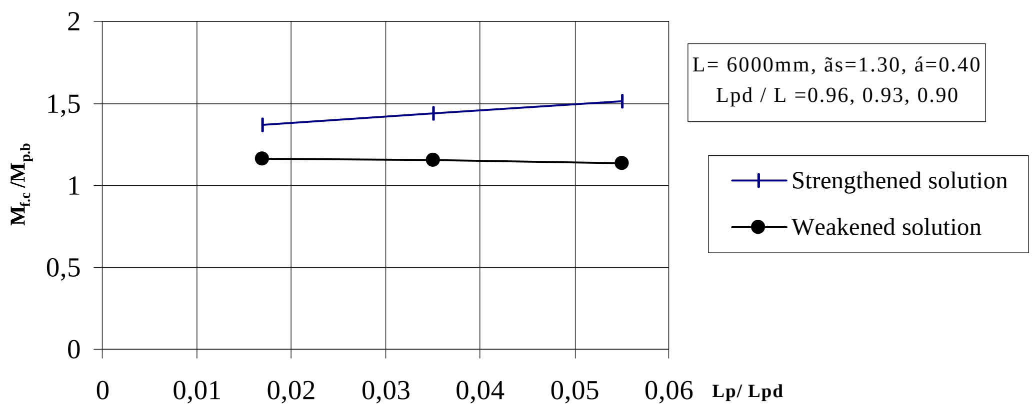

Fig. 8 Moment demands at the column face for strengthened and weakened solutions

must be sized avoiding stress concentration, having a length of about 1.20…1.50c, where c is the beam reduction. Generally, a reduction of about 35-45% is sufficient to provide an improved ductile behaviour. Finally, the critical zone, L_m, is approximately selected, of about $0.5h < L_m < 0.7h$, or analytically calculated creating a segment, within the full length of RBS, for a complete formation of plastic hinge.

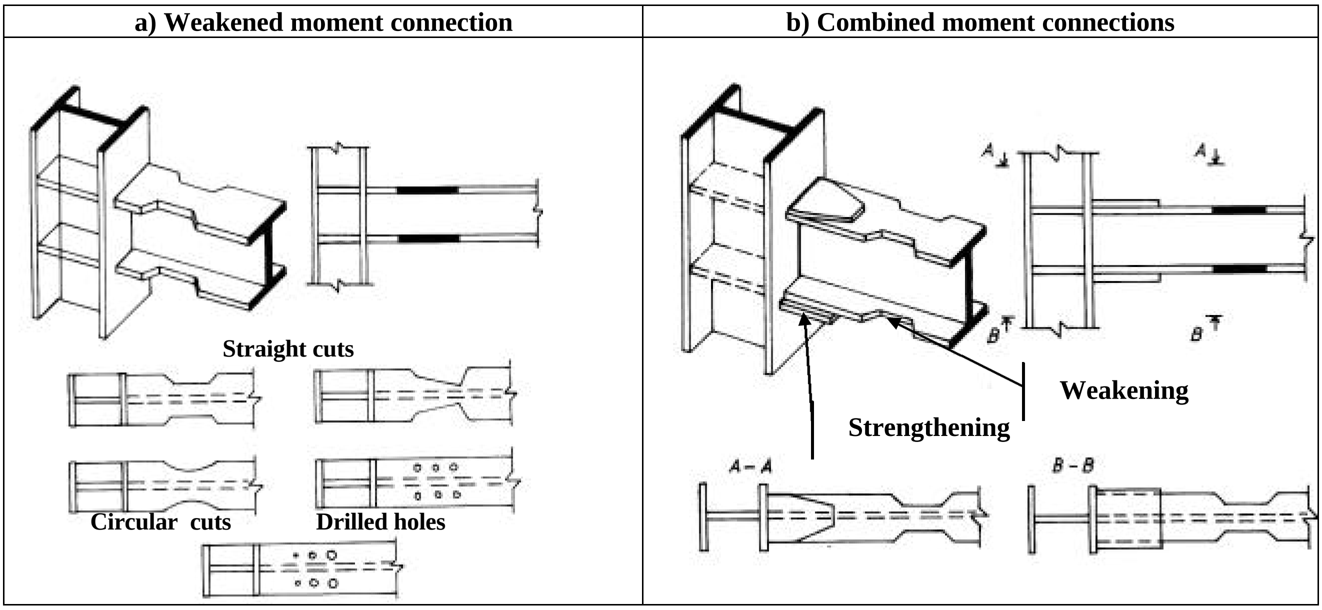

Fig. 6 Illustrative representation of different weakening and combined solutions

Considering that beam with reduced section belongs to a frame, fig. 7a,b, and a weakening in bending at the critical zone with 5-10% as compared with the full beam plastic moment, $M_{p.b}$, the reduced plastic moment, $M_{p.red}$, results:

$$\frac{M_{p.red}}{M_p} = (0.90...0.95)\left[\left(\frac{2L_p}{L} - 1\right) + \frac{1}{2\alpha}\frac{L_p}{L}\left(1 - \frac{L_p}{L}\right)\right] \tag{2}$$

where á a coefficient introducing the influence of vertical loads, L_p is a distance from the column face to the assumed center of plastic hinge, and L is the beam span. Analyzing the above equation can be obtained a relation for direct sizing of the RBS:

$$\frac{b_1}{b} = \left(1 + \frac{d^2 t_w}{4bt_f(d + t_f)}\right)\frac{M_{p.red}}{M_{p.b}} - \frac{d^2 t_w}{4bt_f(d + t_f)} \tag{3}$$

where b_1,b, d, t_f, t_w, geometrical dimensions as illustrated from figure 7. Using the aforementioned relations, (2), (3), it is easy to make tables for direct determining of the reduced plastic moment or the reduction of a beam section, Table 1a,b. Tabulating the main influencing parameters for commercial profiles a standardization can be obtained in order to reduce the fabrication cost.

Using the calculation concept from figure 9b results the moment at the column face, $M_{f.c}$, in order to satisfy the SC-WB mechanism ($\sum M_c \geq \sum M_{f.c}$):

$$M_{c.f} = \left[\gamma_s M_{p.red}\left(\frac{L_p}{L_{pd}} + 1\right) + \frac{M_{p.b}}{\alpha}\frac{L_p}{L^2}\left(\frac{L_{pd}^2}{L_p}\frac{L_p}{L_{pd}} - \frac{L_p^2}{L_{pd}}\frac{L_{pd}}{L_p}\right)\right] \tag{4}$$

and strength variation must be well defined. Also, weld fracture mitigation measures must be taken, avoiding premature brittle failures.

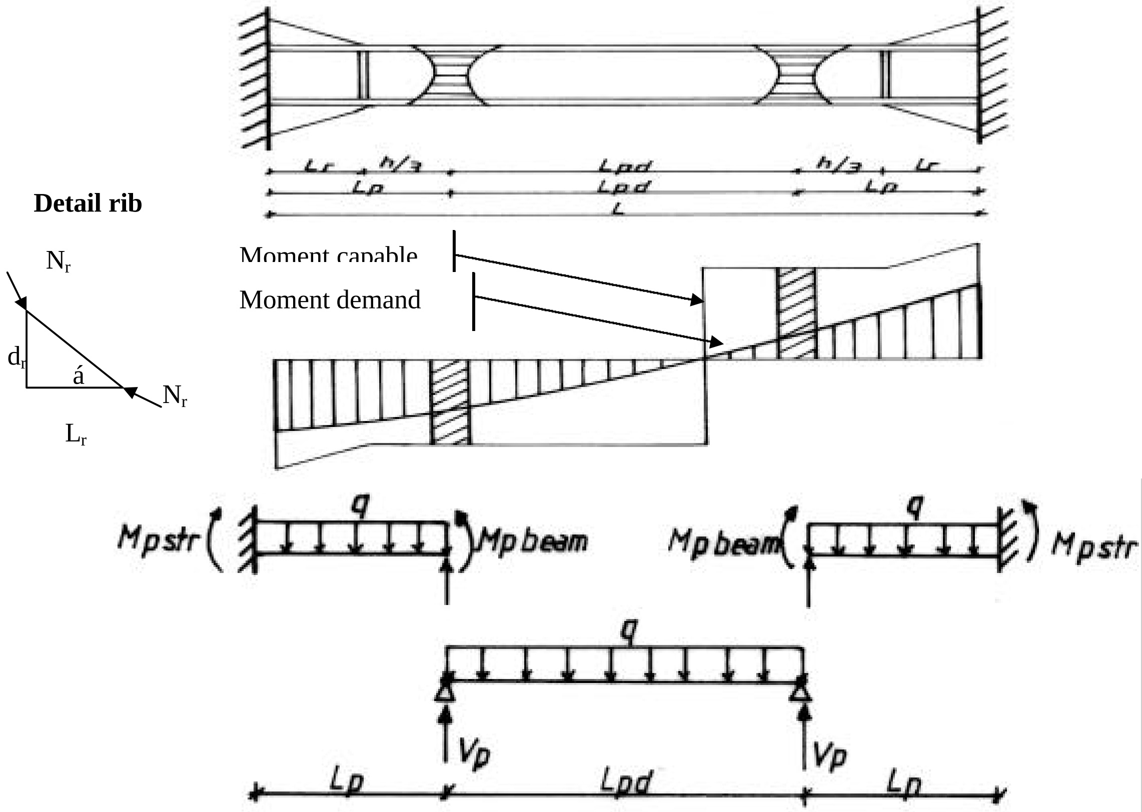

Fig. 4 Design concept for the strengthened moment connections

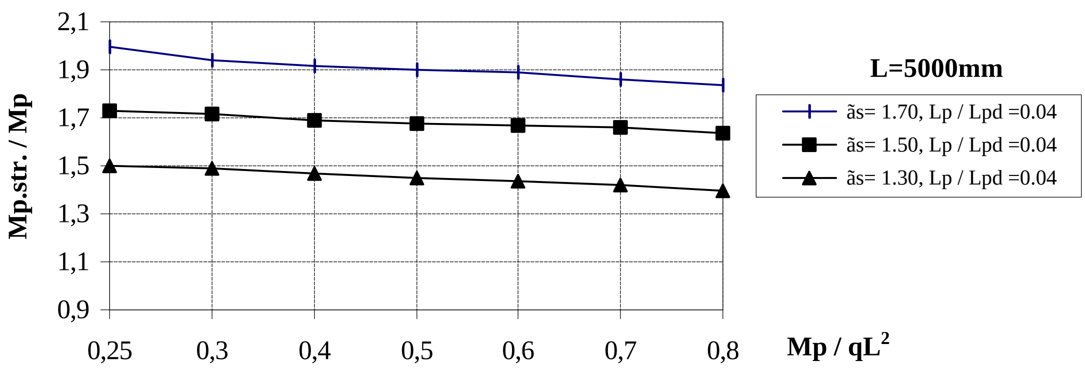

Fig. 5 Influence of reinforcing on moment demands

3.2 MR-frames with weakened connections

A rational alternative is the weakening of beam section at a selected position, near the beam-to-column interface, creating conditions for the concentration of inelastic action in the weakest beam zone. A variety of reduced beam sections, RBS, with straight, tapered and circular cutting for both beam flange were proposed. (Fig. 6a). An alternative could be a proper combination of both strengthening and weakening strategies. (Fig. 6b). Testing of RBS has confirmed the effectiveness of such solutions, achieving the target plastic rotation of 0.03 rad [4,5,10, 13,17].

Additionally, analytical investigations demonstrated the improved ductility of moment frames with RBS [1,3,9]. The weakened beam section with straight cuts is schematically illustrated in figure 9a. Tapered and circular beam sections are detailed elsewhere [1,8]. The beam distance, L_1, is selected, of about $h/6 < L_1 \leq h/3$ from the column face, in order to protect the heat affected zone as well as to create a sufficient distance for the formation of plastic hinge. The transition zone, L_2,

iii) the length of plates must not be very long, because the overstress remain at the column face due to the fact that the beam plastic hinge will be developed in a very low stress zone.

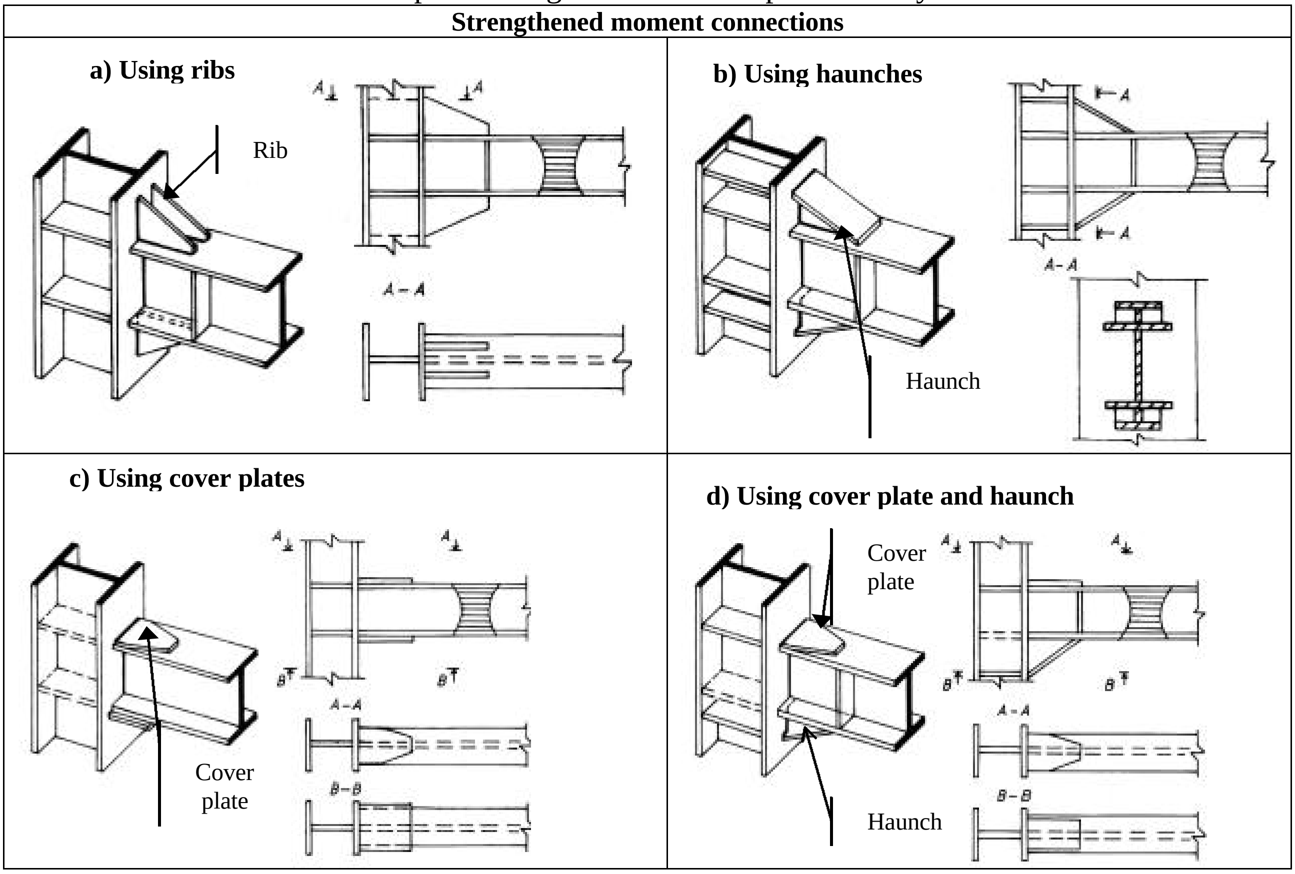

Fig. 3 Joints with strengthened connections

According to the aforementioned criteria, the strengthened zone sized with 5-10% bending moment higher than beams' plastic moment, $M_{p.str}$ = 1.05…1.10$M_{p.b}$, while the local detailing depends on solution chosen (ribs, haunches etc). For instance, in case of ribs the plate length, L_r, it is recommended to taken about 0.5h…h, h is the beam depth (Fig. 4). The ribs height must taken equal to d_r=L_r taná, where á is 25°-35°, as a function of strength and stability requirements. In order to satisfy the strength and stability requirements, it is recommended, a simplified check with N_r = V_p / siná ≤ f_y A_r for strength and a limit of d_r /t_r ≤ 36å (treated as a half of web depth) for stability. Using the calculation concept from figure 4c, the strengthened plastic moment, $M_{p.str}$, results from the following relation, which takes into account the influence of geometry, material randomness, loading conditions:

$$\frac{M_{p.str}}{M_{p.b}} = \gamma_s \left[\left(\frac{2L_p}{L_{pd}} + 1 \right) + \frac{1}{2\alpha} \frac{L_{pd}^2}{L^2} \left(\frac{L_p}{L_{pd}} - \frac{1}{2} \frac{L_p^2}{L_{pd}^2} \right) \right] \quad (1)$$

where:

$ã_s$- A coefficient that accounts the overstrength of the connection, strain-hardening effects, and yield strength variation. So, $ã_s$ = (1.05…1.1) 1.20 $f_{y.max}$ / $f_{y.min}$ =1.26…1.32 $f_{y.max}$ / $f_{y.min}$.

á- A coefficient introducing the influence of vertical loads, á= M_p /qL^2.

L_p- Distance between column face and plastic hinge, L_p= L_r +h/3.

L_{pd}- Distance between the assumed plastic hinges.

Representing the relation (1) in figure 5, one can see that the increasing of vertical loads requires the increasing of strengthening. As a consequence, in case of high vertical loads it is difficult to satisfy a SC-WB mechanism. So, in order to obtain an economical design strain-hardening effects

v) Influence of vertical loads: It is very important to notice that the magnitude of vertical loads acting on beams decide for the location of the plastic hinges in the beams as well as for the shape of moment diagram.

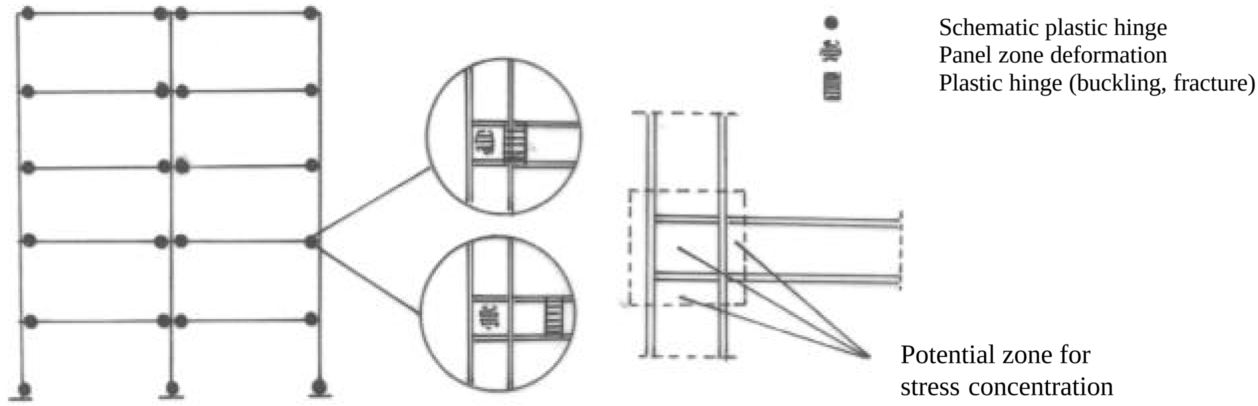

Fig 1. Schematic representation of a global plastic mechanism

Geometry and configuration of joint	• Type of joint (rigid, semi-rigid, pinned) • Conceptual conformation (weakening, strengthening) • Geometrical detailing (stiffeners, doubler plates) • Panel zone yielding
Composite floor slab action	SC-WB ($M_{p+slab} < M_c$) $\longrightarrow$ WC-SB ($M_{p+slab} > M_c$)
Yield strength	$f_{y.real} > f_{y.design}$ $\longrightarrow$ $M_{pb.real} > M_{pb.design}$ $\longrightarrow$ $M_{pb} > M_c$ $\longrightarrow$ WC-SB
Overstrength	• Commercial sections • Drift requirements • Strain hardening effects
Vertical loads	Seismic loading Seismic + Vertical loading condition

Fig 2. Factors affecting the location of plastic hinge

3. DUCTILE DESIGN ALTERNATIVES BASED ON CONSTRUCTIONAL DETAILS

3.1 MR-frames with strengthened connections

In order to force the development of a plastic hinge away from the column face additional elements such ribs, haunches, cover plates, or a combination, were proposed. (Fig. 3). Experimental research has confirmed the relocation of a plastic hinge just after or at some distance from the toes of cover plates or ribs [15]. Furthermore, both experimental and analytical studies demonstrated the improvement pf local as well as of global ductility of steel moment frames [1,3,15,18]. The main disantvantage of this solution as compared with the weakening one is the increasing of column size in order to maintain the SC-WB mechanism. As a consequence may result thicker doubler plates or continuity plates and supplemental welding.

For an efficient design of strengthened connections the following considerations must be taken into account:

i) the plastic hinge must be formed at sufficient distance from the column face, protecting the susceptible connection zone. So, plates used for strengthening must satisfy strength and stability requirements;

ii) the length of plates must be adequately chosen in order to create bending and not shear failure mechanism. Thus, the span L_{pd} must be limited ($L_{pd} / h > 4.0$);

discussed in the paper. However, considering the damage observations from Northridge earthquake, it was noticed little difference in the performance of frames designed with panel zone yielding as compared with those without panel zone yielding [7].

The recent experience from Northridge and Kobe demonstrated the importance to consider in capacity design the location of plastic hinges in the beams, even in the case of formation of a global plastic mechanism. The vast majority of brittle damage occurred at the end of beams, at the beam-to- column connection zone, initiated at very low levels of inelastic demand without any sign of plastic deformation in the beams, and also in many cases, while the frames behave elastically. Some recent experimental tests in U.S show that the conventional connection configuration is not sufficient to fulfil plastic rotation requirements (i.e. larger than 0.03rad) for steel moment-resisting frames in high seismicity areas [15,16]. From the other side, European experimental tests demonstrated that fully welded joints achieve the target ductility when the base material and welding conditions were properly defined [14]. However, the difference in design, detailing, and practice between Europe and US. must be considered. In any case, some measures should be introduced in seismic design in order to avoid the unexpected behaviour of a structure due to randomness of earthquake characteristics, of material properties, of uncertainties in execution quality.

Taking into account the response of moment resisting frames and especially frame beams, which exhibit maximum bending moments at the member ends, the solution is to move the formation of plastic hinge within the beam span but away from the beam-to-column interface in order to protect this sensible zone reducing the stress levels in the connection. In this way, some problems related to the potential fragility of welds, poor workmanship, or poor base and weld material toughness could be minimized. Different authors propose such constructional details, which improve the inelastic performance of steel moment resisting frames [1,4,8,13]. So, in order to relocate the plastic hinge, connections must adequately strengthened or weakened. An alternative solution must be a hybrid frame, with well-controlled mechanical properties, having columns from higher steel grade than beams. Using the aforementioned solutions it is possible to obtain a capacity design with failure mode and ductility control based on the fulfilment of the member hierarchy criterion in conjunction with adequate detailing of the connection zone. For a proper ductile design it is necessary to underline the importance of some factors affecting the development of a strong column-weak beam, SC-WB, mechanism. Geometry and configuration of the joint, presence of composite floor slab action, yield strength, overstrength, and magnitude of vertical loads directly influence the location of plastic hinge. (Fig. 2):

i) Geometry and configuration of the joint: Generally, the type of joint (rigid, semi-rigid, etc) and the detailing of the joint region (weakening, strengthening, etc) dictates the location of plastic hinges. Experimental evidence had confirmed that in case of rigid nodes the use of doubler plates increase the load carrying capacity, in the same time, decreasing the energy absorption capacity. The use of stiffeners can contribute to the increase of energy dissipation when do not modify the deformation mechanism [11].

ii) Presence of composite floor slab action: In many cases the strong interaction between floor slab and steel beam increase the strength of the beam changing the ductile mechanism of SC-WB in a WC-SB mechanism. In the same time, the concrete floor slab does not permit the local buckling of the upper beam flange, which can work as a filter contributing to dissipation of the input energy.

iii) Yield strength: Due to random variability of yield strength the intend SC-WB mechanism may be changed in a WC-SB affecting the plastic hinge formation, as well as, the ultimate behaviour of the frame.

iv) Overstrength: In case of steel structures it is possible to oversize some members, due to the necessity of using commercial sections. Furthermore, in earthquake design of steel moment frames member sizes are governed by drift requirements, which can be larger than those required by strength. Therefore, another search for overstrength is provided. Strain-hardening effects may cause an overstrength, when it is not considered, modifing the ductile mechanism.

IMROVED DUCTILE DESIGN OF STEEL MR-FRAMES BASED ON CONSTRUCTIONAL DETAILS

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Abstract: Steel moment resisting frames performed poorly during the Northridge and Kobe earthquakes. A large number of brittle damage at the beam-to-column connection was observed, caused by a numerous factors including earthquake characteristics, inefficient design and execution practices. In the paper on introducing the ductile approach, the importance of hinge location and some general factors affecting the performance of joint region are discussed. Then the new ductile design of steel moment frames based on strengthening or weakening of joint is presented. An alternative solution of hybrid frames is also briefly discussed.

1. INTRODUCTION

It is widely recognized the randomness of seismic phenomena, which very rarely reoccurs with the same characteristics (magnitude, epicentral distance, contents of frequencies e.t.c). So, one of the most important problems in carrying out proper design is to select a design earthquake that adequately represent the ground motion, and in particular the motion that would lead the structure to its critical response, as well as, to predict the inelastic response of the structure. The quantification of the aformentioned problems is not an easy task, introducing uncertainties in analysis and design of structures. Taking into account that the structure behaves different in case of near-source and far-source earthquakes, the problem become more difficult. Therefore, in order to minimize these uncertainties steel moment resisting frames must be proportioned in such a way providing a member hierarchy in the energy dissipation mechanism, independently from the type of ground motion. Using the capacity design concept, which requires the formation of plastic hinges in beams rather than in columns, it is possible to avoid undesirable collapse mechanisms (such as soft storey mechanisms), in the same time obtaining a failure mode control.

Unfortunately, during the Northridge (1994) and Kobe (1995) earthquakes unexpected brittle damage was mainly observed at the beam-to-column connections were dissipative zones are devoted to absorb the seismic energy. Among the factors influencing the unacceptable performance of steel moment frames were the design concept and constructional details. After the experience of these earthquakes some changes were made concerning the design concept and the detailing of the dissipative zones.

The aim of this paper is first to present the concept of ductile design and some factors affecting the achievement of a ductile behaviour during severe earthquakes. Second, the new constructional details, as well as, a simplified concept using different quality of steels, which improve the ductile performance of steel moment frames, are critically presented and discussed.

2. DUCTILE DESIGN APPROACH

It is well-recognized that ductile steel moment frames designed to resist severe earthquakes should be capable of withstanding substantial inelastic deforamtion, developing a global plastic mechanism [11, 12]. (Fig.1). Generally with the formation of plastic hinges only at the beams and at the column bases the structural system could achieve a high inelastic performance. According to the local configuration and design concept of the connection zone we can obtain a plastic hinge at the end of beam or away from the column face with or without inelastic deformation of panel zone, Fig. 1. Now, modern aseismic codes as EC-8 specifying only the formation of inelastic deformations at the end of beams. Concerning the panel zone yielding, it is a problem under debate, which is not