The use of "Dog-Bone" for the Seismic Improvement of Steel MRFs

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Abstract: - This work is devoted to the strengthening of steel moment resisting frame designed in order to bear vertical loads only. In particular, the idea is based on the attainment of improvement of seismic performance by simply trimming the flanges of the beam ends. This strategy can be applied by considering both the results of the theory of plastic mechanism control and the rules assuring the yielding of reduced beam sections (RBS) when seismic loads are applied to the structure. It is important to underline that the results of such strategy is not always effective. In fact, there are several condition that are to be satisfied in order to obtain an actual seismic improvement. Notwithstanding, when these conditions are satisfied, the cost of intervention can be considered as negligible. For this reason this strategy can be very interesting and the rules applied in this work can clarify which is the effect of RBS taking into account all the parameters playing a role in the final design:, i.e. existing column sections, resistance and ductility of existing connections, vertical loads acting in seismic load combination, amount of the reduction of beam section and its distance from the connection. By means of a worked example the effectiveness of the proposed procedure is shown.

Key-Words: - collapse mechanism, existing structure, reduced beam section, soft storey.

1 Introduction

In 1980s, during a research project financed by the Luxembourg steel producer ARBED and the European Union with the aim of increasing the ductility of the structure by promoting the development of plastic hinges in the beams rather than in the columns, the first idea of RBS was introduced by A. Plumier [1]. At that time the idea was patented by ARBED, and, due to the reduction of the beam flange width by means of a "dog-bone" shape at a proper distance from the column flange, RBS connections have been also called "dog-bone" connections (Fig. 1).

In 1994 Northridge earthquake and in 1995 Kobe earthquake a lot of unexpected damages to steel moment-resisting frames were observed. These damages were mainly due to the failure of welded beam-to-column connections. For these reasons ARBED waived any licensing fees and claims and RBS connections started to be investigated by a lot of researchers [2-18]

Since that time one of the main objective of the research concerning the "dog-bone" connections has been the development of design rules able to promote the beam yielding for safeguarding the beam-to-column connections. [2-22].

So, it can be concluded that structures in high seismicity zones are normally designed to resist severe earthquakes by dissipating the input energy by means of inelastic deformations and, in order to maximize this effect, plastic hinges need to be developed at beam ends rather than in the columns in case of moment resisting frames (MRFs) [23-42].

Fig. 1. Typical shape of a "Dog-Bone" connection

However also in the case of other structural typologies the need to avoid the yielding of columns is always the desired goal and the development of a global mechanism is one of the main design objective [43-64]. When we have an existing structure designed according to old seismic codes or even with no particular rules for seismic protection the same design objectives above recalled became relevant. In fact, in those cases the structure has been designed with no particular rules for the development of a dissipative collapse mechanism. In addition, the beam to column connections have a very poor dissipative behavior and have no over strength with respect to the beam plastic moment. The aim of this work is to set up a procedure able to assure a better collapse mechanism with respect to the original one and, at the same time, also the protection of the connections.

2 Reduced Beam Sections for Seismic Improvement

As it is well known when we need to retrofit a steel structure in order to improve its seismic resistance we can add material to different zones of the structure. In particular we can add steel plate to columns in order to increase their resistance. In this way we can move the plasticization from the column to the beam ends. But at this point another problem appears: the connections do not have the over-strength which can guarantee the yielding of the beam ends rather than the connections and , in addition, the connections themselves cannot provide the ductility required to assure the development of a dissipative mechanism. In fact, as already mentioned, also in the case of connections designed to resist to seismic action (Kobe and Northridge) the performance exhibited were inadequate due to the brittle failure of weldings.

For this reasons, generally also the retrofitting of beam to column connections become mandatory.

In this context the strategy of reduced beam section can be a very economical solution, because the cut of beam flanges can be considered as a negligible cost. In fact, the realization of "dog-bone" at the ends of each beam could solve both the problem of avoiding a very poor dissipative mechanism and the problem of avoiding the yielding of beam to column connections.

In addition, it is important to underline that the weakened beam section is characterized by the decrease, with respect to the original section, of the width-to-thickness ratio of the flanges, i.e. a reduced local slenderness, which leads to the improvement of the plastic rotation capacity.

The first problem to be solved is the one concerning the location of RBS is the beam and the amount of the reduction. Regarding this point, we have to apply the results found in [65].

3 Location of "Dog Bone"

If we consider that seismic action can be represented by means of an appropriate distribution of increasing horizontal forces, it is preliminarily necessary to observe which is the shape of the bending moment diagram of a generic beam subjected to both horizontal forces and vertical loads (Fig. 2).

We can apply the superposition principle by considering separately the effect of vertical loads and the effect of horizontal forces (Fig. 2).

Therefore, the resulting bending moment diagram is given in Fig. 3, where the sections corresponding to the beam ends (sections 1 and 5), those corresponding to the "dog-bone" locations (section 2 and 4) and that corresponding to the maximum bending moment (section 3) have been pointed out.

It is evident that the design parameters are the location of the "dog-bones" (which is denoted with the distance a in Fig. 3 and the magnitude of the weakening characterising the "dog-bones". This second parameter can be expressed in non-dimensional form as:

$$m_{db} = \frac{M_{p.db}}{M_P} \tag{1}$$

where $M_{p.db}$ is the plastic moment of the weakened beam section and M_p is the plastic moment of the complete beam section.

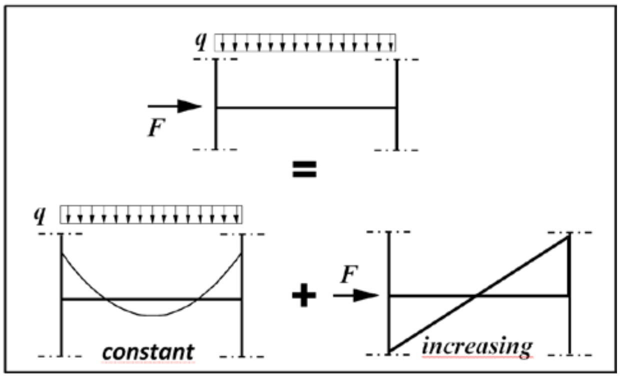

Fig. 1. Bending moment due to vertical loads and seismic forces.

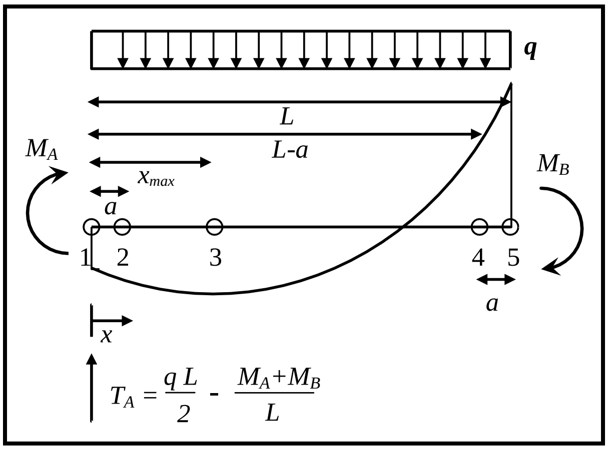

Fig. 2. Total beam bending moment diagram.

In this phase of the design procedure the m_{db} value can assumed as fixed, while the location a of the "dog-bones" is to be properly selected.
It is important to note that at the left side of the beam (beam sections 1 and 2) the bending moments due to vertical loads and horizontal forces have an opposite sign (one is anticlock-wise and another is clock-wise), while at the right side (beam sections 3 and 4) they have the same sign (clock-wise).

Due to this consideration it is obvious that for increasing values of horizontal forces the first plastic hinge develops in beam section 4 or 5 rather than in beam section 1 or 2.

So the first problem to be solved is to find the conditions assuring that sections 1, 2, 3 and 5 remain in elastic range, while section 4 yields when seismic horizontal forces increase.

To this aim it is useful to consider the expression of bending moment at the generic section x:

$$M(x) = M_A + T_A x - q\frac{x^2}{2} = = M_A + \left(q\frac{L}{2} - \frac{M_A + M_B}{L}\right)x - q\frac{x^2}{2} \quad (2)$$

And the value of x_{max} representing the abscissa where the bending moment has its maximum value:

$$x_{max} = \frac{L}{2} - \frac{M_A + M_B}{qL} \quad (3)$$

Using Eq. (2) and (3) the bending moment in sections 1,2,3,4 and 5 can be expressed as:

Section 1 $$M(0) = M_A \quad (4)$$

Section 2
$$M(a) = q\frac{a(L-a)}{2} - M_B\frac{a}{L} + \left(1 - \frac{a}{L}\right)M_A \quad (5)$$

Section 3
$$M(x_{max}) = q\frac{L^2}{8} + \frac{M_A - M_B}{2} + \frac{(M_A + M_B)^2}{2qL^2} \quad (6)$$

Section 4
$$M(L-a) = q\frac{a(L-a)}{2} + M_A\frac{a}{L} - \left(1 - \frac{a}{L}\right)M_B \quad (7)$$

Section 5 $$M(L) = -M_B \quad (8)$$

The conditions to be fulfilled in order to assure that sections 1, 2, 3 and 5 remain in elastic range, while section 4 yields when seismic horizontal forces increase are given by the following relationships:

Section 1 $$M_A < M_P \quad (9)$$

Section 2 $$M(a) < m_{db} M_P \quad (10)$$

Section 3 $$M(x_{max}) < M_P \quad (11)$$

Section 4 $$M(L-a) < -m_{db} M_P \quad (12)$$

Section 5
$$M(L) > -M_P \Rightarrow -M_B > -M_P \Rightarrow M_B < M_P \quad (13)$$

It is easy to recognize that by combining the yielding condition of “dog-bone” of the right side (Eq. (12), section 4) with the value of bending moment at the abscissa $x=L-a$ given by Eq.(7), an expression of M_B as a function of M_A can be obtained:

$$M_B = M_A\frac{a}{(L-a)} + q\frac{La}{2} + \frac{L}{L-a}m_{db}M_P \quad (14)$$

This expression represents the relation occurring between the end moments when the first plastic hinge develops at section 4 corresponding to the right “dog-bone”.

By means of Eqs. (14) and (2), it is possible to express the design requirements (9), (10), (11) e (13) as follows:

$M_A < M_{A1}$ with $M_{A1} = M_p$ (15)

$M_A < M_{A2}$ with (16)

$$M_{A2} = \frac{m_{db} L}{L-2a} M_P - q\frac{a(L-a)}{2}$$

$M_A < M_{A3}$ with

$$M_{A3} = \sqrt{2q(l-a)^2(1+m_{db})M_P} + -\left(\frac{q(L-a)^2}{2} + m_{db} M_P\right) \quad (17)$$

$M_A < M_{A5}$ with

$$M_{A5} = \frac{(1-m_{db})L - a}{a} M_P - q\frac{L(L-a)}{2} \quad (18)$$

Obviously, the first plastic hinge develops in the right "dog-bone" provided that Eq. (12) is satisfied. Under this condition, it is required that the second plastic hinge develops either in the left "dog-bone" or an intermediate beam section. On the contrary, the yielding of the beam ends close to the beam-to-column connections has to be prevented, because, as already stated, the use of "dog-bones" is also aimed at the protection of beam-to-column connections.

It is easy to recognise that increasing the seismic horizontal forces, i.e. increasing M_A, relationships (15), (16), (17) and (18) allow to identify the section where the second plastic hinge develops. To this scope, it is sufficient to control what is the minimum limit value among M_{A1}, M_{A2}, M_{A3}, M_{A5}. In other words, it is sufficient to identify the first relationship to be unsatisfied as far as M_A increases.

Therefore, all the yielding conditions can be expressed by means of the limit values M_{Ai} (with i=1,2,3,5) of the bending moment M_A occurring at the first beam end. In particular, the condition:

$M_{A3} < M_{A2}$ **condition A** (19)

identifies the *a* values assuring that the yielding of the beam in the section where the maximum sagging moment occurs (section 3) precedes the yielding of the left "dog-bone" (section 2); the condition:

$M_{A3} < M_{A5}$ **condition B** (20)

identifies the *a* values assuring that the beam yielding (section 3) precedes the yielding of the connection B (section 5); the condition:

$M_{A2} < M_{A5}$ **condition C** (21)

identifies the *a* values assuring that the left "dog-bone" yielding (section 2) precedes the yielding of the connection B (section 5); the condition:

$M_{A3} < M_{A1}$ **condition D** (22)

identifies the *a* values assuring that the beam yielding (section 3) precedes the yielding of the left connection A (section 1); finally, the condition:

$M_{A2} < M_{A1}$ **condition E** (23)

identifies the *a* values assuring that the yielding of the left "dog-bone" (section 2) precedes the yielding of the left connection A (section 1). It is evident that conditions (20), (21), (22), (23) have to be absolutely satisfied, because they assure the development of the second plastic hinge either in the left "dog-bone" or in the intermediate beam section where the maximum sagging moment occurs, while the yielding of the connections at the beam ends is prevented. In other words, relationships (20), (21), (22) and (23) are the design requirements.

Conversely, condition (19), depending on its fulfilment or not, can be used to discern if the second plastic hinge develops in the left "dog-bone" or in the intermediate beam section.

Such conditions give rise to the following non-dimensional relationships:

- **condition A**:

$$4\left(\frac{a}{L}\right)^3 + \left(4\sqrt{2(1+m_{db})\frac{M_p}{qL^2}} - 8\right)\left(\frac{a}{L}\right)^2 + \left(5 + 4m_{db}\frac{M_p}{qL^2} - 6\sqrt{2(1+m_{db})\frac{M_p}{qL^2}}\right)\left(\frac{a}{L}\right) - 4m_{db}\frac{M_p}{qL^2} - 1 + 2\sqrt{2(1+m_{db})\frac{M_p}{qL^2}} < 0 \quad (24)$$

whose solutions are:

$$\frac{a}{L} < \frac{a_3}{L} \quad \text{and} \quad \frac{a_2}{L} < \frac{a}{L} < \frac{a_1}{L} \quad (25)$$

where:

$$\frac{a_1}{L} = 1$$

$$\frac{a_2}{L} = \frac{1}{2} - \sqrt{\frac{(1+m_{db})}{2}\frac{M_p}{qL^2}} + \sqrt{\frac{(1-m_{db})}{2}\frac{M_p}{qL^2}} \quad (26)$$

$$\frac{a_3}{L} = \frac{1}{2} - \sqrt{\frac{(1+m_{db})}{2}\frac{M_p}{qL^2}} - \sqrt{\frac{(1-m_{db})}{2}\frac{M_p}{qL^2}}$$

- **condition B:**

$$-\left(\frac{a}{L}\right)^3+\left(1-2\sqrt{2(1+m_{db})\frac{M_p}{qL^2}}\right)\left(\frac{a}{L}\right)^2+\left(2(1-m_{db})\frac{M_p}{qL^2}+2\sqrt{2(1+m_{db})\frac{M_p}{qL^2}}\right)\left(\frac{a}{L}\right)-2(1-m_{db})\frac{M_p}{qL^2}<0 \quad (27)$$

whose solution are:

$$\frac{a_6}{L}<\frac{a}{L}<\frac{a_5}{L} \quad \text{and} \quad \frac{a}{L}>\frac{a_4}{L} \quad (28)$$

where:

$$\frac{a_4}{L}=1 \quad ; \quad \frac{a_5}{L}=-\sqrt{2(1+m_{db})\frac{M_p}{qL^2}}+2\sqrt{\frac{M_p}{qL^2}} \quad \frac{a_6}{L}=-\sqrt{2(1+m_{db})\frac{M_p}{qL^2}}-2\sqrt{\frac{M_p}{qL^2}} \quad (29)$$

- **condition C:**

$$-2\left(\frac{a}{L}\right)^4+5\left(\frac{a}{L}\right)^3-4\left(1+\frac{M_p}{qL^2}\right)\left(\frac{a}{L}\right)^2+\left(1+2(3-m_{db})\frac{M_p}{qL^2}\right)\frac{a}{l}-2(1-m_{db})\frac{M_p}{qL^2}<0 \quad (30)$$

whose solutions are:

$$\frac{a}{L}<\frac{a_8}{L} \quad \text{and} \quad \frac{a}{L}>\frac{a_7}{L} \quad (31)$$

where:

$$\frac{a_7}{l}=1 \quad ; \quad \frac{a_8}{l}=\frac{1}{2}+\frac{1}{6}\sqrt[3]{T}-6\frac{\frac{2}{3}\frac{M_p}{ql^2}-\frac{1}{12}}{\sqrt[3]{T}} \quad (32)$$

with:

$$T=-108m_{db}\frac{M_p}{qL^2}+3\left[-3+72\frac{M_p}{qL^2}-576\left(\frac{M_p}{qL^2}\right)^2+1296m_{db}^{\,2}\left(\frac{M_p}{qL^2}\right)^2+1536\left(\frac{M_p}{qL^2}\right)^3\right]^{1/2} \quad (33)$$

- **condition D:**

condition D can be written as follows (by expressing relationship (22) by means of M_{A1} and M_{A3} values given by Eqs. (15) and (17), respectively):

$$-\left[\frac{a}{L}+\left(1-\sqrt{2(1+m)\frac{M_p}{qL^2}}\right)\right]^2<0 \quad (34)$$

therefore, condition *D* is always satisfied.

- **condition E:**

By means of Eq.(15) and (16) this condition provides:

$$2\left(\frac{a}{L}\right)^3-3\left(\frac{a}{L}\right)^2-\left(4\frac{M_p}{qL^2}-1\right)\left(\frac{a}{L}\right)+2(1-m)\frac{M_p}{qL^2}>0 \quad (35)$$

In order to show that this condition is always verified when the condition C is verified, it is useful to write the condition C (Eq.(31)) in the following way:

$$\left(\frac{a}{L}-1\right)\left[-2\left(\frac{a}{L}\right)^3+3\left(\frac{a}{L}\right)^2-\left(1+4\frac{M_p}{qL^2}\right)\left(\frac{a}{L}\right)++2(1-m)\frac{M_p}{qL^2}\right]<0 \quad (36)$$

Being *a/L<1* this relation is equivalent to require:

$$\left[-2\left(\frac{a}{L}\right)^3+3\left(\frac{a}{L}\right)^2-\left(1+4\frac{M_p}{qL^2}\right)\left(\frac{a}{L}\right)++2(1-m)\frac{M_p}{qL^2}\right]>0 \quad (37)$$

Now it is easy to verify that the first member of Eq. (35) is greater than the first member of Eq.(37) when the following relation is satisfied:

$$-4\left(\frac{a}{L}\right)^3+6\left(\frac{a}{L}\right)^2-2\left(\frac{a}{L}\right)<0 \quad (38)$$

The solutions of Eq. (38) are:

$$\frac{a}{L} > 1 \qquad \text{and} \qquad 0 < \frac{a}{L} < \frac{1}{2} \tag{39}$$

Now it can be observed that, being $a/L < 1/2$ (which means that a “dog-bone” cannot be located beyond the midspan), Eq. (38) is always true, and so condition *E* is always satisfied if condition *C* is satisfied. Therefore, in the range $0<a/L<1/2$, which is the significant one from the design point of view, only the three conditions *A*, *B* and *C* remain to be analysed. These three remaining condition provide the following significant solutions (32):

condition A

$$\frac{a}{L} < \frac{a_3}{L} \qquad \text{and} \qquad \frac{a}{L} > \frac{a_2}{L} \tag{40}$$

which is obtained from Eqs. (25) and (26);

condition B $\quad \frac{a}{L} < \frac{a_5}{L} \quad (41)$

which is obtained from Eqs. (28) and (29) by observing that a_6 provides negative values which are not significant;

condition C $\quad \frac{a}{L} < \frac{a_8}{L} \quad (42)$

which is obtained from Eqs. (31) and (32). Therefore, taking into account that condition *A* has to be used only to recognise the location of the second plastic hinge which can develop either at the left “dog-bone” or at an intermediate beam section, it means that conditions *B* and *C* show the existence of an upper bound concerning the parameter *a* expressing the "dog-bone" location (this upper bound value is given by the minimum value between a_5 and a_8).

Therefore, the design solution concerning the "dog-bone" location can be expressed as follows: the smallest value between a_5 and a_8 is the upper bound of *a*, while the location of the second plastic hinge depends on a_2 and a_3 value; in particular, according to Eq. (40), if $a < a_3$ or $a > a_2$ the second plastic hinge develops in the intermediate beam section, where the maximum sagging moment occurs, otherwise the second plastic hinge occurs at the left “dog-bone”.

In addition, when relation (40) is satisfied, the location x_{max} of the second plastic hinge where the maximum sagging moment occurs can be determined by solving the following equation:

$$T(x) = T_A - qx = \frac{qL}{2} - \frac{M_A + M_B}{L} - qx = 0 \tag{43}$$

which provides:

$$x_{\max} = \frac{L}{2} - \frac{M_A + M_B}{qL} = \frac{L-a}{2} - \frac{M_A}{q(L-a)} - \frac{m_{db} M_p}{q(L-a)} \tag{44}$$

The value of M_A to be used in relationship (43) is equal to M_{A3} consistently with condition A expressed by Eq. (19).
By substituting the M_{A3} value in Eq. (44) x_{max} becomes:

$$x_{\max} = L - a - \left(\frac{2M_p (1 + m_{db})}{q} \right)^{1/2} \tag{45}$$

As the expression for computing a_8/L is particularly complex, in order to identify the governing limit value of *a/L*, a numerical analysis has been carried out. As an example for a given value of m_{db} , by varying the non-dimensional parameter M_p/qL^2 in the range between 1/16 and zero, which covers all the possible design situations, the values of a_2, a_3, a_5 and a_8 have been computed.

The results of this numerical analysis is presented in table 1 for m_{db} equal to 0.5. In addition, the curves representing the values of a_2, a_3, a_5 and a_8 are plotted in Fig. 4. From the analysis of the above recalled figures it is evident the existence of two limit values of qL^2/M_p which are $q_{lim1}L^2/M_p$ and $q_{lim2}L^2/M_p$. The first one represents the value for which a_2, a_5 and a_8 are coincident while the second one represents the value for which a_3, a_5 and a_8 are coincident ([66])

These values have been highlighted in bold type in Tables 1-6. Such limit values can be easily determined by means of relationships (26) and (29) providing a_2, a_3, and a_5. In fact, q_{lim1} can be obtained by equating a_2 and a_5, while q_{lim2} can be obtained by equating a_3 and a_5.

The following relationships are thus obtained:

$$q_{\lim 1} = \frac{4M_p}{L^2} \left(5 - \sqrt{8(1 - m_{db})} - \sqrt{8(1 + m_{db})} - \sqrt{1 - m_{db}^2} \right) \tag{46}$$

$$q_{\lim 2} = \frac{4M_p}{L^2} \left(5 + \sqrt{8(1 - m_{db})} - \sqrt{8(1 + m_{db})} - \sqrt{1 - m_{db}^2} \right) \tag{47}$$

qL^2/M_p	a_2/L	a_3/L	a_5/L	a_8/L
0.01	-3.1603	-13.1603	2.6795	0.2498
0.50	-0.0176	-1.4319	0.3789	0.2381
1.00	0.1340	-0.8660	0.2679	0.2260
1.61	***0.2113***	***-0.5774***	***0.2113***	***0.2113***
2.00	0.2412	-0.4659	0.1895	0.2020
3.00	0.2887	-0.2887	0.1547	0.1792
4.00	0.3170	-0.1830	0.1340	0.1588
5.00	0.3363	-0.1109	0.1198	0.1412
6.00	0.3506	-0.0577	0.1094	0.1263
7.00	0.3617	-0.0163	0.1013	0.1137
8.00	0.3706	0.0171	0.0947	0.1031
9.00	0.3780	0.0447	0.0893	0.0942
10.00	0.3843	0.0680	0.0847	0.0865
10.68	***0.3880***	***0.0820***	***0.0820***	***0.0820***
11.00	0.3896	0.0881	0.0808	0.0800
12.00	0.3943	0.1057	0.0773	0.0743
13.00	0.3985	0.1211	0.0743	0.0693
14.00	0.4022	0.1349	0.0716	0.0650
15.00	0.4055	0.1473	0.0692	0.0611
16.00	0.4085	0.1585	0.0670	0.0577

Table 1. Values of a_2, a_3, a_5 and a_8 for $m_{db} = 0.5$.

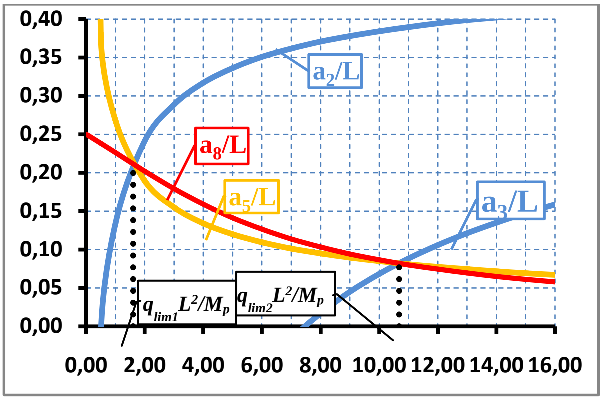

Fig. 3. Limit values a_2/L, a_3/L, a_5/L and a_8/L *for* $m_{db} = 0.5$

As a conclusion, the design solution concerning the "dog-bone" location and its influence on the location of the second plastic hinge could be expressed as follows:

case $q<q_{lim1}$**:** the design requirement is $a < a_8$, while, regarding the development of the second plastic hinge, if $a < a_2$ the yielding of the second "dog-bone" occurs, otherwise the yielding of the beam develops;

case $q_{lim1} < q < q_{lim2}$**:** the design requirement is $a < a_5$, while, regarding the development of the second plastic hinge, if $a > a_3$ the yielding of the second "dog-bone" occurs, otherwise the yielding of the beam develops;

case $q > q_{lim2}$**:** the design requirement is $a < a_8$, while, regarding the development of the second plastic hinge, it always develops at the intermediate beam section where the maximum sagging moment occurs.

Actually, the result obtained for the case $q<q_{lim1}$ is to be better specified. In fact, in this case the second plastic hinge can develops only in the "dog-bone". In order to clarify this aspect, the relation (45) providing the location of maximum bending moment is to be considered. As it is obvious, the second plastic hinge can develop in the beam only if the value expressed by relation (45) is positive. In fact, when the value of x_{max} is negative, the maximum bending moment in the beam is obtained in the point A, and, as a consequence, condition A loses its meaning.

In order to understand which are the conditions required for obtaining a positive value of x_{max} the following relation is to be analysed.

$$x_{\max} \geq 0 \Rightarrow L - a - \left(\frac{2M_p(1+m_{db})}{q}\right)^{1/2} \geq 0 \qquad (48)$$

From relation (48) the following condition can be easily obtained:

$$\frac{a}{L} < 1 - \sqrt{\frac{2M_p}{qL^2}(1+m_{db})} = \frac{a_9}{L} \qquad (49)$$

In addition, the comparison between a_2/L and a_9/L provides:

$$\frac{a_2}{L} < \frac{a_9}{L} \Rightarrow$$

$$\frac{1}{2} - \sqrt{\frac{(1+m_{db})}{2}\frac{M_p}{qL^2}} + \sqrt{\frac{(1-m_{db})}{2}\frac{M_p}{qL^2}} < 1 - \sqrt{\frac{2M_p}{qL^2}(1+m_{db})} \Rightarrow \qquad (50)$$

$$q > \frac{4M_p}{L^2}\left[1 + \sqrt{1 - m_{db}^{\ 2}}\right] = q_9$$

The obtained value of q_9 is always greater than q_{lim1} because:

$$q_9 > q_{lim\,1} \Rightarrow \frac{4M_p}{L^2}\left[1 + \sqrt{1-m_{db}^{\ 2}}\right] > \frac{4M_p}{L^2}\left(5 - \sqrt{8(1-m_{db})} - 2\sqrt{2(1+m_{db})} - \sqrt{1-m_{db}^{\ 2}}\right) \Rightarrow \qquad (51)$$

$$2\sqrt{1-m_{db}^{\ 2}} > 0$$

Which is always verified. So, it can be concluded that the value of a_2/L can be completely neglected. In fact, from Fig. 5 it is obvious that if

$q>q_{lim1}$ then a_2 does not play any role, because the solution is found for $a < a_5$ and, as a consequence, $a < a_2$ so that the second plastic hinge will develop in the dog-bone. When $q<q_{lim1}$ there are two possibilities: if $a < a_9$ we are in the same condition already recalled, if $a > a_9$ then then $x_{max} < 0$ and the second plastic hinge develops again in the dog bone. For this reason, when $q<q_{lim1}$ the design requirement is $a < a_8$, and the yielding of the second "dog-bone" occurs. From a graphical point of view the situation is represented in Fig. 6.

The same results is obtained also for different value of m_{db} as reported in Fig. 7 and Fig. 8. In order to clarify the solution of the problem, it is of fundamental importance to highlight that the design goal consists in the protection of the beam-to-column connections, i.e. yielding of both "dog-bones" or yielding of one "dog-bone" and of a beam cross section (the one where the maximum bending moment is achieved). In fact, the limit value of a/L can be obtained varying m_{db} for a fixed vertical load q and beam plastic moment M_p.

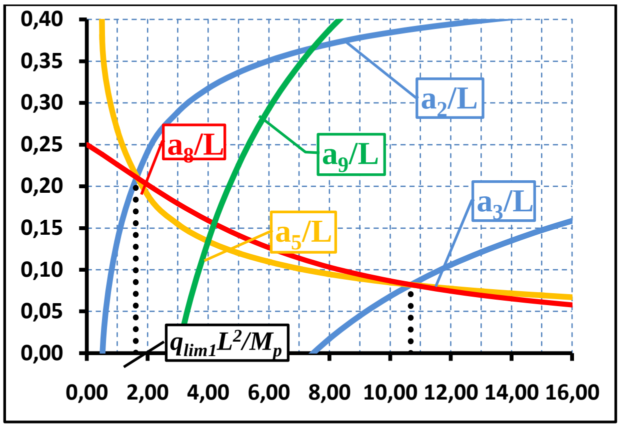

Fig. 4. Limit values a_2/L, a_3/L, a_5/L, a_8/L *and* a_9/L *for* m_{db} = 0.5

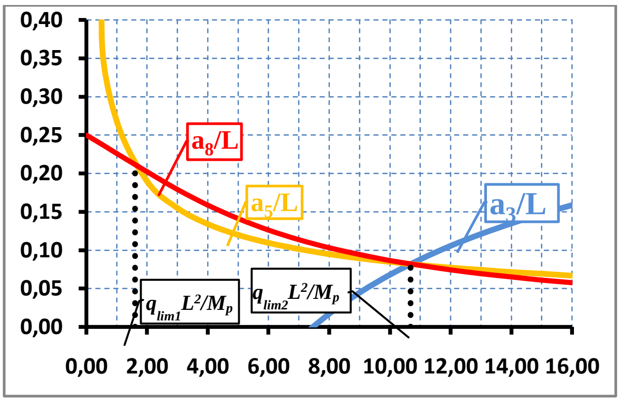

Fig. 5. Limit values a_3/L, a_5/L and a_8/L *for* m_{db}=0.5

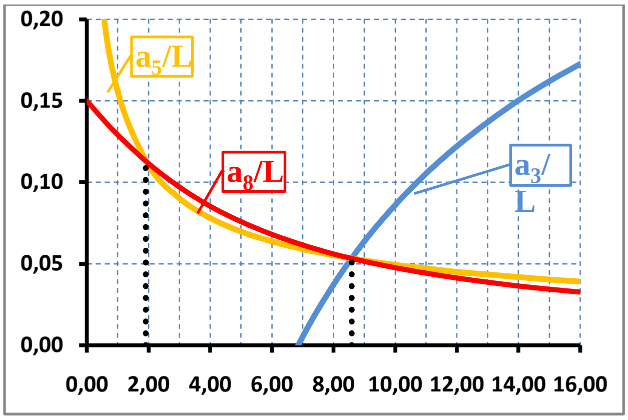

Fig. 6. Limit values a_3/L, a_5/L and a_8/L *for* m_{db}=0.7

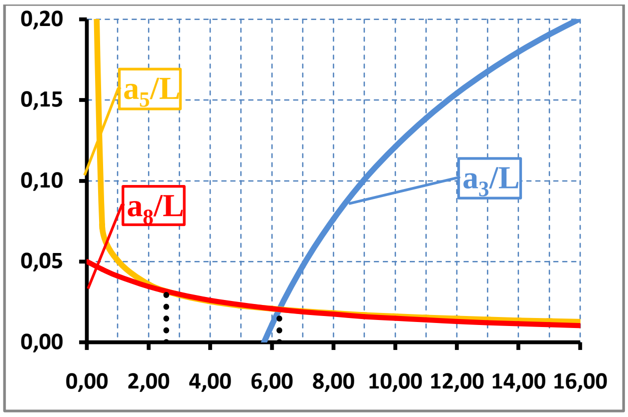

Fig. 7. Limit values a_3/L, a_5/L and a_8/L *for* m_{db}=0.9

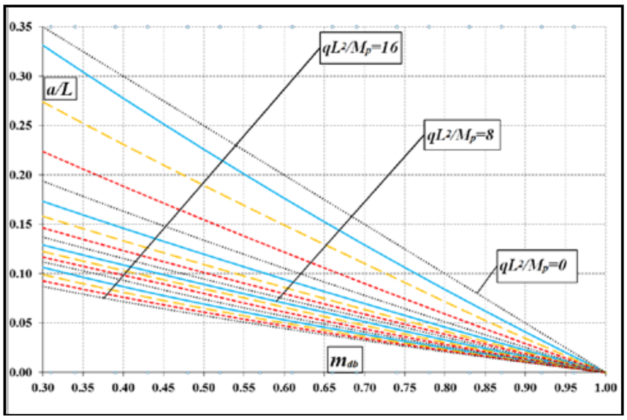

Fig. 8. Design abacus for "dog-bone" location

In other words, for a given qL^2/M_p the curve representing the upper limit of *a/L* as a function of m_{db} can be obtained as depicted in Fig. 9. This figure is, substantially, a design abacus for "dog-bone" location. In fact, it includes all the design variables expressed in non-dimensional form.

The abacus can be useful also to understand the role played by several parameters. The numerical values used to build Fig. 9 can be found in [65]. One of the main result is constituted by the fact that increasing the vertical load, the admissible *a/L* value decreases.

In order correctly apply the abacus of Fig. 9 it is important to clarify the meaning of a and L parameters. In fact, a represents the distance between the beam to column connections and the middle point of the dog-bone, while L represents the distance between the two connections as depicted in Fig. 10. So that L is different from the length L_i depicted in Fig. 10 which is the bay span. Obviously the relation between L and L_i is given by:

$$L = L_i - H_{c1}/2 - H_{c2}/2 \tag{52}$$

where H_{c1} and H_{c2} are the heights of the column sections.

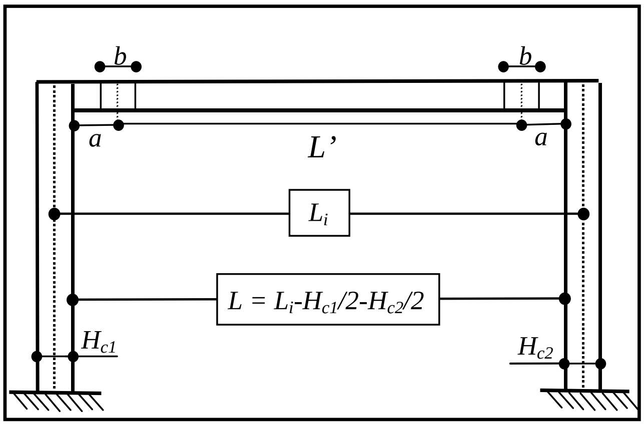

Fig. 9. Difference between L and L_i

4 Application of the Theory of Plastic Mechanism Control

The theory of plastic mechanism control (TPMC) has been developed, applied and verified for a lot of structural typology [33-48]. In this case it can be applied for the evaluation of the effectiveness of the RBS. In fact, by using the results of such theory we can determine the conditions assuring the development of a collapse mechanism better than the original one, in particular we can understand if the soft storey mechanism (when this is the collapse mechanism of the original structure) can be avoided or not by simply trimming the beam flanges.

TPMC procedure is based on the kinematic theorem of plastic collapse and on the concept of mechanism equilibrium curve. In particular, it is observed that the collapse mechanism of a frame subjected to horizontal forces can, basically, belong to three collapse mechanism typologies, so that the failure mode control can be obtained by analysing $3n_s$ collapse mechanisms, being n_s the number of storeys. Moreover, the design procedure accounts for the influence of second order effects by extending the kinematic theorem of plastic collapse to the concept of mechanism equilibrium curve.

In fact, the plastic moments of the columns are derived by imposing that, within a given displacement range depending on the plastic rotation supply of members and connections, the mechanism equilibrium curve corresponding to the global mechanism has to lie below the mechanism equilibrium curves corresponding to all the remaining 3ns-1 kinematically admissible mechanisms.

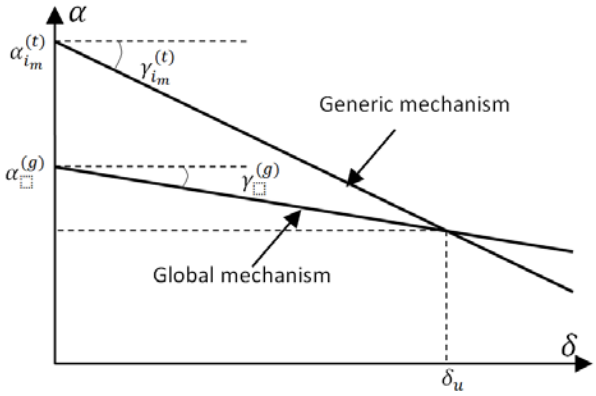

Fig. 10. Design conditions

In order to understand the results of the analyses herein presented, it is useful to remember that the main result of the design procedure for failure mode control is the sum, at each storey, of the plastic moments of the columns required to assure a collapse mechanism of global type:

$$M_k = \sum_{i=1}^{n_c} M_{i,k} \tag{53}$$

where $M_{i,k}$ is the plastic moment (reduced due to the influence of the axial force) of the i*th* column of the k*th* storey and n_c is the number of columns. Assuming that both dog-bones are yielded, in the case of global type mechanism, the kinematically admissible multiplier of horizontal forces is given by [31]:

$$\alpha^{(g)} = \frac{\sum_{i=1}^{n_c} M_{c.i.1}}{\sum_{k=1}^{n_s} F_k h_k} + \frac{2\sum_{k=1}^{n_s}\sum_{j=1}^{n_b} m_{db.jk}\left(\frac{L_{jk}}{L_{jk}-2d}\right) M_{b.jk}}{\sum_{k=1}^{n_s} F_k h_k} \tag{54}$$

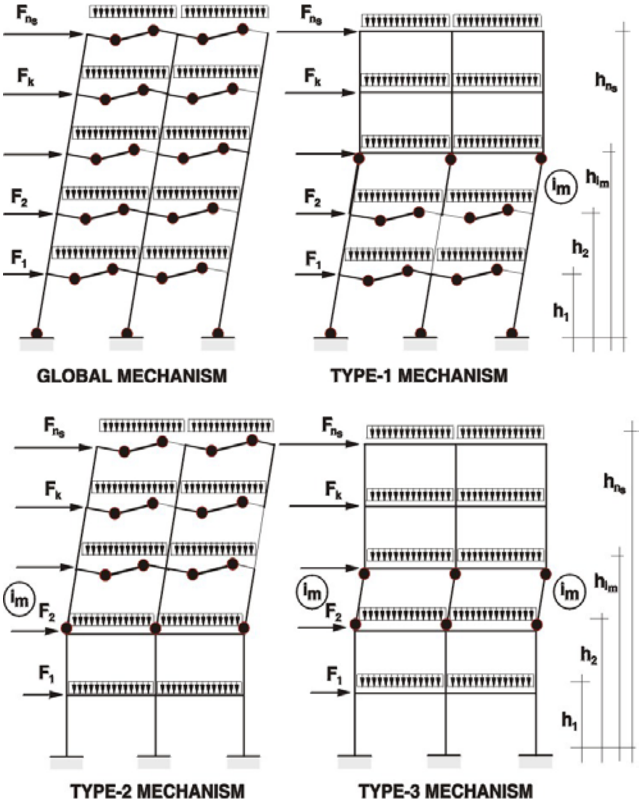

Fig. 11. Collapse mechanism typologies

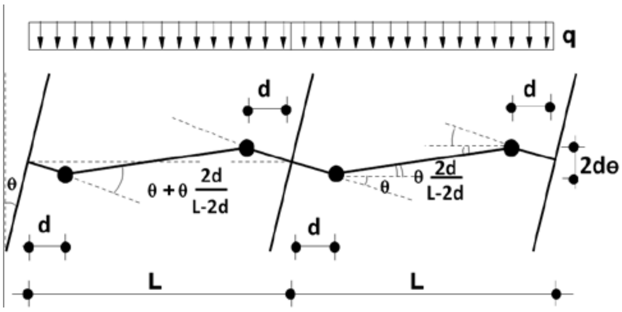

Fig. 12. Internal work in RBSs

where F_k and h_k are, respectively, the seismic force applied at k-th storey and the k-th storey height with respect to the foundation level; $M_{c.i.k}$ is the plastic moment of i-th column of k-th storey reduced due to the contemporary action of the axial force; n_c, n_b and n_s are the number of columns, bays and storeys, respectively. With respect to the relation reported in [31] there is the term $m_{db}(L_{jk}/(L_{jk}-2d))$ which accounts for the internal work obtained from Fig. 13. It is important to underline that the vale d represented in Fig. 13 is given by the sum of the distance *a* (from the midpoint of dog-bone to the connection) and the half height Hc/2 of the column section as showed in Fig. 10.

Regarding the slope $\gamma^{(g)}$ of the mechanism equilibrium curve, it is given by:

$$\gamma^{(g)} = \frac{1}{h_{ns}} \frac{\sum_{k=1}^{n_s} V_k h_k}{\sum_{k=1}^{n_s} F_k h_k} \tag{55}$$

where V_k is the total vertical load acting at k-th storey. With reference to i_mth mechanism of type-1, the kinematically admissible multiplier of seismic horizontal forces is given by:

$$\alpha_{i_m}^{(1)} = \frac{\sum_{i=1}^{n_c} M_{c.i.1} + 2\sum_{k=1}^{i_m-1}\sum_{j=1}^{n_b} m_{b.jk}\left(\frac{L_{jk}}{L_{jk}-2d}\right) M_{b.jk}}{\sum_{k=1}^{i_m} F_k h_k + h_{i_m}\sum_{k=i_m+1}^{n_s} F_k} + \frac{\sum_{i=1}^{n_c} M_{c.i.i_m}}{\sum_{k=1}^{i_m} F_k h_k + h_{i_m}\sum_{k=i_m+1}^{n_s} F_k} \tag{56}$$

while the slope of the mechanism equilibrium curve is:

$$\gamma_{i_m}^{(1)} = \frac{1}{h_{i_m}} \frac{\sum_{k=1}^{i_m} V_k h_k + h_{i_m}\sum_{k=i_m+1}^{n_s} V_k}{\sum_{k=1}^{i_m} F_k h_k + h_{i_m}\sum_{k=i_m+1}^{n_s} F_k} \tag{57}$$

With reference to i_mth mechanism of type-2, the kinematically admissible multiplier of seismic horizontal forces is given by:

$$\alpha_{i_m}^{(2)} = \frac{\sum_{i=1}^{n_c} M_{c.i.i_m}}{\sum_{k=i_m}^{n_s} F_k (h_k - h_{i_m-1})} + \frac{2\sum_{k=i_m}^{n_s}\sum_{j=1}^{n_b} m_{b.jk}\left(\frac{L_{jk}}{L_{jk}-2d}\right) M_{b.jk}}{\sum_{k=i_m}^{n_s} F_k (h_k - h_{i_m-1})} \tag{58}$$

while the slope of the mechanism equilibrium curve is:

$$\gamma_{i_m}^{(2)} = \frac{1}{h_{n_s} - h_{i_m-1}} \frac{\sum_{k=i_m}^{n_s} V_k (h_k - h_{i_m-1})}{\sum_{k=i_m}^{n_s} F_k (h_k - h_{i_m-1})} \tag{59}$$

Finally, with reference to i_mth mechanism of type-3, the kinematically admissible multiplier of horizontal forces, for $i_m = 1$, is given by:

$$\alpha_1^{(3)} = \frac{2\sum_{i=1}^{n_c} M_{c.i.1}}{h_1 \sum_{k=1}^{n_s} F_k} \tag{60}$$

and, for $i_m > 1$, is given by:

$$\alpha_{i_m}^{(3)} = \frac{2\sum_{i=1}^{n_c} M_{c.i.i_m}}{\left(h_{i_m} - h_{i_m-1}\right)\sum_{k=i_m}^{n_s} F_k} \tag{61}$$

In addition, the corresponding slope of the mechanism equilibrium curve is given by

$$\gamma_{i_m}^{(3)} = \frac{1}{h_{i_m} - h_{i_m-1}} \frac{\sum_{k=i_m}^{n_s} V_k}{\sum_{k=i_m}^{n_s} F_k} \qquad (62)$$

It is important to underline that, for any given geometry of the structural system, the slope of mechanism equilibrium curve attains its minimum value when the global type mechanism is developed.

This issue assumes a paramount importance in TPMC exploiting the extension of the kinematic theorem of plastic collapse to the concept of mechanism equilibrium curve.

In fact, according to the kinematic theorem of plastic collapse, extended to the concept of mechanism equilibrium curve, the design conditions to be fulfilled in order to avoid all the undesired collapse mechanisms require that the mechanism equilibrium curve corresponding to the global mechanism has to be located below those corresponding to all the undesired mechanisms within a top sway displacement range, δ_u, compatible with the ductility supply of structural members

$$\alpha_0^{(g)} - \gamma^{(g)}\delta_u \leq \alpha_{i_m}^{(t)} - \gamma_{i_m}^{(t)}\delta_u$$
$$for \quad i_m = 1,2,3,\dots,n_s \quad t = 1,2,3 \qquad (63)$$

The condition to avoid type 1, type 2 and type 3 mechanisms can be expressed as [31]:

$$\sum_{i=1}^{n_c} M_{c.i.i_m}^{(1)} \geq \left(\alpha^{(g)} + \gamma_{i_m}^{(1)}\delta_u\right) * \left(\sum_{k=1}^{i_m} F_k h_k + h_{i_m} \sum_{k=i_m+1}^{n_s} F_k\right) + - \sum_{i=1}^{n_c} M_{c.i.1} - 2 \sum_{k=1}^{i_m-1} \sum_{j=1}^{n_b} m_{b.jk} \left(\frac{L_{jk}}{L_{jk} - 2d}\right) M_{b.jk} \qquad (64)$$

$$\sum_{i=1}^{n_c} M_{c.i.i_m}^{(2)} \geq \left(\alpha^{(g)} + \gamma_{i_m}^{(2)}\delta_u\right) * \sum_{k=i_m}^{n_s} F_k (h_k - h_{i_m-1}) + -2 \sum_{k=i_m}^{n_s} \sum_{j=1}^{n_b} m_{b.jk} \left(\frac{L_{jk}}{L_{jk} - 2d}\right) M_{b.jk} \qquad (65)$$

$$\sum_{i=1}^{n_c} M_{c.i.i_m}^{(3)} \geq \left(\alpha^{(g)} + \gamma_{i_m}^{(3)}\delta_u\right) * \frac{\left(h_{i_m} - h_{i_m-1}\right)}{2} \sum_{k=i_m}^{n_s} F_k \qquad (66)$$

If all the conditions expressed by relations (64), (65) and (66) are satisfied than a collapse mechanism of global type is obtained. In our case we can observe that by decreasing the value of $m_{db,jk}$ then the value of $\alpha^{(g)}$ of equation (54) decreases and, as a consequence, the relation (66) could become satisfied even if in its original condition ($m_{db,jk} = 1$) it was not satisfied.

If by introducing the smallest admissible values of $m_{db,jk}$ in the above equations the relation (66) cannot be satisfied then we can conclude that the strategy of dog-bones for the existing structure is not effective, because it is prone to develop a soft storey mechanism both with and without dog-bones.

On the contrary, when the existing structure does not satisfy the relation (66) while by introducing dog-bones this relation is satisfied, then a considerable improvement in seismic behaviour can be obtained.

5 A Case Study

In order to show the effectiveness of the proposed procedure we can consider the structure depicted in

Fig. 14. It was designe according to the old italian seismic code (DM96). By means of a simply push over analysis, made with Sap2000 computer program, it shows a great vulnerability to seismic action. In fact, as represented in Fig. 15 a soft storey mechanism develops. By applying the proposed procedure, it is easy to recognize that a weakening of all the beams can be realized. The maximum amount of the section reduction which allows to verify all the serviceability requirement is equal to 0.7 M_p.

If we assume an available ductility of columns equal to 0.04 rad, then the value of the top sway displacement δ_u can be determined. If such reduction is realized in each beam of the structure, the condition expressed by the relation (66) can be evaluated. In this case this condition can be satisfied for each i_m value, i.e. for each storey.

With the aim to verify this conclusion another push over analysis has been made on the structure with dog bones. The result of the push over is reported in Fig. 16. From this figure we can observe that a soft storey has been avoided and, in addition, a consistent number of beams have been significantly involved in the collapse mechanism. In fact, the green colour shows that both the two of the first storey columns and two dog bones have achieved a plastic rotation of 0.04 rad which is the value assumed as the maximum admissible value both for beams and columns.

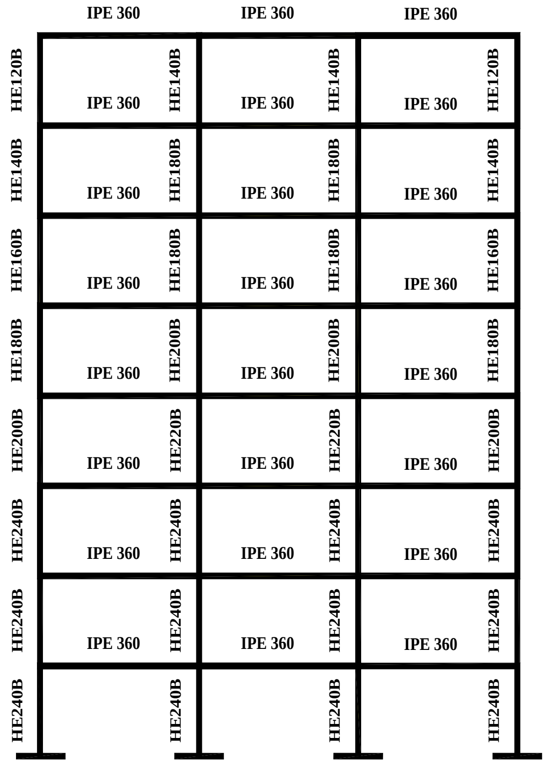

Fig. 13. Existing structures

color of yielded beams indicates that the rotation is less than 1%.

In Fig. 17 the push over curve of both structures have been reported. The analyses have been stopped when the available ductility (0.04 rad) has been achieved at least in one element. It is evident a greater ductility available for the structure with dog bones.

Finally, in order to have a further confirmation of the effectiveness of the proposed procedure, also non-linear dynamic analyses have been carried out.

In particular the three real earthquakes reported in table 2 have been considered.

From these analyses the differences in seismic behavior are actually significant. Analyses have been repeated by progressively increasing the multiplier of the earthquake. For each structure and for each considered earthquake the value of the multiplier corresponding to the attainment of a rotation greater than 0.04 rad has been determined. Finally these value has been compared and an increase in seismic performance for the structure with dog bones of 14%, 9.5% and 21% for Northridge, Imperial Valley and Santa Barbara, respectively, has been found (Table 3).

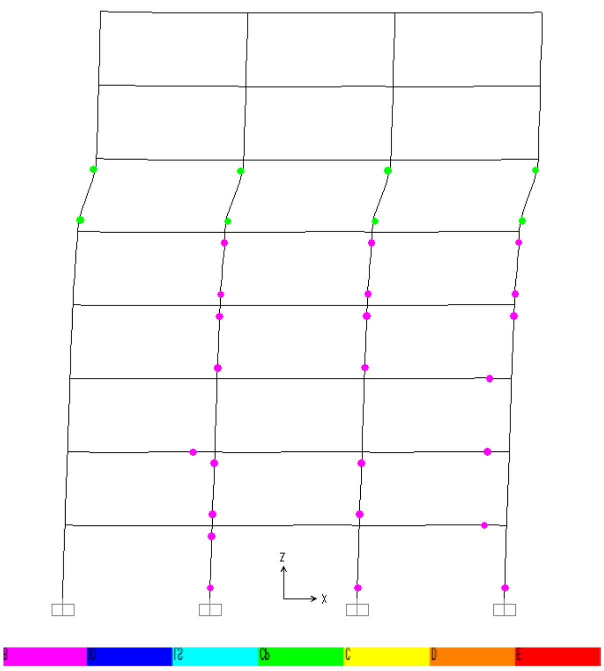

Fig. 14. Soft storey mechanism developed by the existing structure

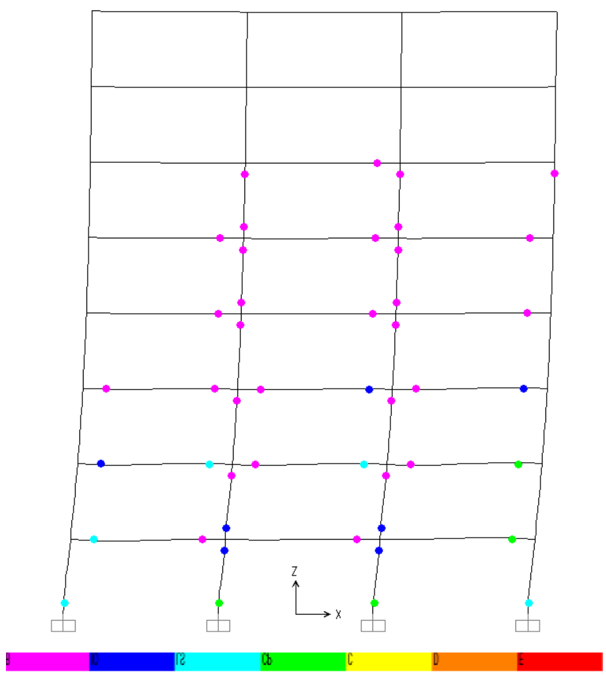

Fig. 15. Collapse mechanism of the structure with dog bones.

On the contrary in the existing structure only few beams are involved in the collapse mechanism, but their contribution to the dissipation is very low. In fact, in in Fig. 15 the

In addition, if we consider then in the existing structure the hinges develop in the connections, a bigger increment in seismic behavior is obtained.

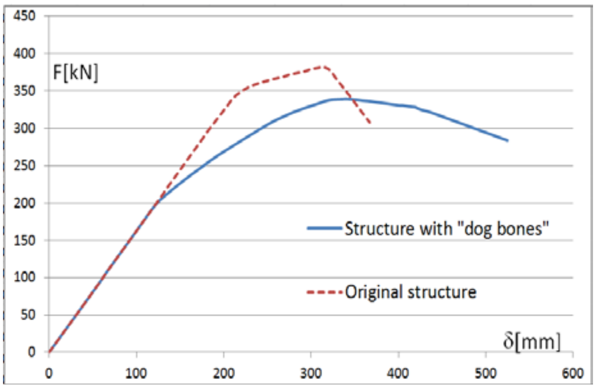

Fig. 16. Comparison between push over curve of existing structure and structure with dog bones.

Earhquake	Date	PGA/g	Length [sec]
Northridg	1994/01/17	0.252	39.99
Imperial Valley	1979/10/15	0.370	28.35
Santa Barbara	1978/08/13	0.102	12.57

Table 2. Considered earthquakes

In fact, assuming that the limit ductility of connections is equal to 0.02 rad the increase can be of 50% for the Imperial Valley earthquake as reported in table 3. As an example of the results found by non-linear dynamic analyses, in Fig. 18 all the plastic hinges developed in the structures for the Imperial Valley seismic input are represented.

The result are very similar to the ones obtained with push-over analyses. So it can be concluded that in this case less is more, because a significant seismic improvement has been achieved by simply trimming the flanges of the beams. Obviously further improvement can be obtained by increasing same column sections according to the relation (64), (65) and (66).

Earhquake	connections ductility 0.04 rad	connections ductility 0.02 rad
Northri	14%	14%
Imperial Valley	9.5%	50%
Santa Barbara	21%	28%

Table 3. Percentage increase of PGA carried by the structure with dog bones with respect to the original structure

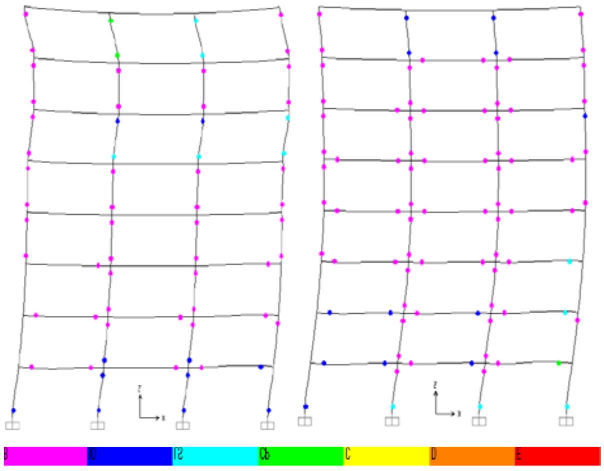
Fig. 17. Comparison between existing structures (left one) and structure with dog bones (right one) when subjected to Imperial Valley earthquake.

6 Conclusion

In the present paper the problem of strengthening a steel moment resisting frame in seismic zone has been considered. The idea and the developed example of this work is based on the attainment of improvement of seismic performance by simply trimming the flanges of the beam ends. An example of significant seismic improvement has been showed. The strategy can be very interesting because it requires no additional materials and the cost to cut the beams is really negligible if compared with the obtainable results.

It is important to underline that this methodology is not always effective. In fact in some cases the introduction of dog bones can determine a negligible seismic improvement due to the development of a soft storey mechanism which could be avoided only by means of an increase of some column sections.

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