THE USE OF STEEL RBS FOR THE INCREASE OF WOODEN BEAMS DUCTILITY

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Abstract

Reduced Beam Section (RBS) have been introduced, with reference to steel structure, after the 1994 Northridge, CA Earthquake, due to the brittle failure of beam flange-column flange weldments. In the last decades the connections with RBS have been studied both from an analytical point of view and from an experimental point of view. A lot of experimental tests demonstrated that RBS connections designed according to the most modern seismic codes are able to protect the beam to column connection due to the yielding of the adjacent RBS. In this paper, a new idea regarding the use of steel RBS is presented: the possibility of using a steel RBS in a wooden beam. In this case, RBSs should constitute dissipative zones of the structure, leading to a much better seismic behavior of structure. In fact available ductility of steel is much higher if compared to the available ductility of wood.

At this aim, the yielding of RBSs should precede the yielding not only of the beam to column connections, but also of all the intermediate wooden sections.

In the work the role played by vertical loads and by the amount of section reduction are analyzed and accounted for, and in addition, the possibility that the beam to column connections are realized with a partial strength connection is considered.

Keywords: Reduced Beam Section, Wooden Beam, Dog-Bone connection.

1. Introduction

According to the design philosophy of structures in seismic zone, structures should remain in elastic range during frequent seismic events, i.e. those having a return period similar to the service life of the structure. On the contrary, in the case of severe earthquakes, i.e. those having low probability of occurrence, damage of both structural and nonstructural elements coming from the development of dissipative mechanisms is accepted [1-5]. Therefore, only in the case of destructive earthquakes, the available ductility of the structure is to be exploited in order to dissipate the seismic input energy. Obviously, the dissipation should involve only particular zones of the structure, called dissipative zones, which have to be properly chosen and designed [6-15].

In fact, the column yielding has to be absolutely avoided, because, due to the action of axial forces, they exhibit a poor ductility behaviour. Moreover, the failure modes which can result from column hinging could involve a limited number of dissipative zones. For these reasons, aiming at the complete development of the plastic reserves of the structure, modern seismic codes provide simple design criteria whose goal is the prevention of local failure modes.

In the seismic design of steel moment resisting frames the use of full-strength connections having an over-strength with respect to the connected beam is generally required. In fact, in this way the exploiting of the beam plastic rotation capacity can be obtained. It is important to underline that the design objective can be achieved only if the random material variability and the over-strength of the connection due to the strain-hardening occurring before the flange local

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buckling are considered. These over-strengths can significantly affect the structural detail of connection requiring additional elements like cover plates, haunches, etc.; which considerably increase the connection cost. An alternative solution is constituted by RBSs, because they can protect the beam to column connection by forcing the plastic hinge in a section of beam away from the column face [16-28]. For this reason experimental investigations [29-32] on the cyclic response of partial-strength connections increased in last years. RBS connection constitutes a particular typology of partial-strength connections, because its resistance is lower than the one of the connected beam.

An alternative name of RBS connections is “dog-bones” because of its shape. In fact, due to the reduction of the beam flange width, the shape is similar to the classical "dog-bone" (Figure 1).

Figure 1: Typical shape of a “Dog-Bone” connection

The wooden structures do not possess the ability to deform in the plastic range, not being the wood a ductile material. In addition the connections of such structures have a limited capacity to have deformations in the plastic range. For these reasons, the use of steel RBS is proposed. It can be constituted by an IPE profile connected to each of the two ends of the wooden beam. In addition, in order to obtain the exact amount of reduction, a part of flanges can be removed. In this way the classical steel “dog-bone” connection can be realized (Figure 2). As an example, in Figure 2 a way of realizing steel RBS for wooden beam is represented. Obviously, the advantage above mentioned can be achieved only with a design procedure able to consider the "dog-bone" location with respect to the beam-to-column connection, the definition of the magnitude of the weakening to give to the steel part, the amount of vertical load acting on the beam and the possibility of having a partial strength connection between beam and column.

All these parameters play a role in the development of plastic hinges in the beam or in RBS and they need to be accounted for if the design goal is to assure the development of plastic hinges only in the steel “dog-bones” leaving the entire wooden beam and the beam-to-column connections in elastic range.

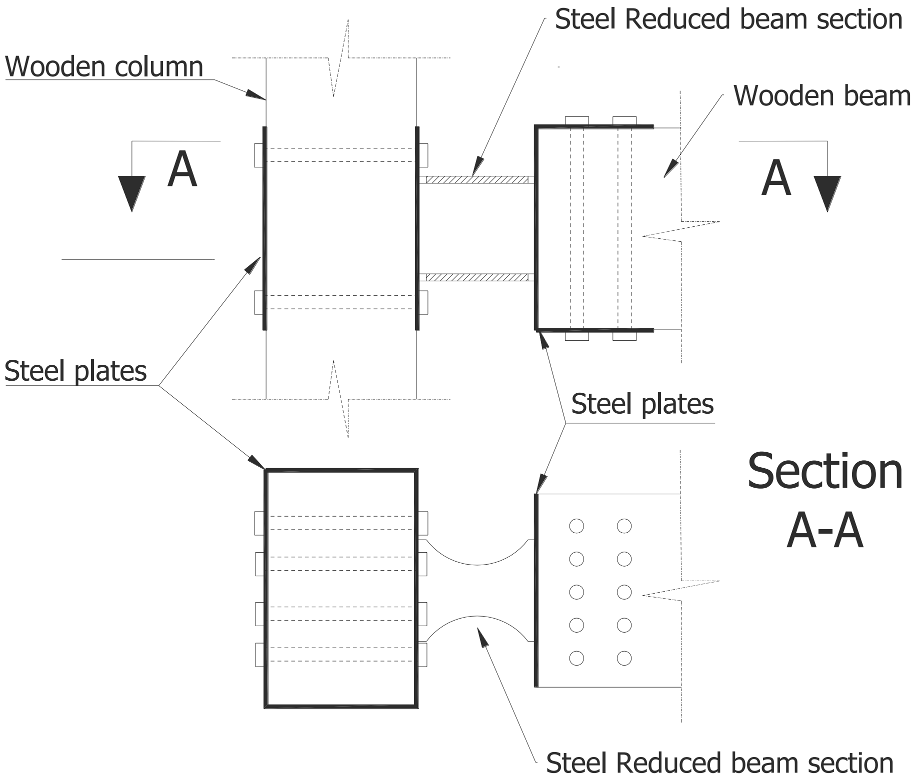

Figure 2: The steel “Dog-Bone” for a wooden beam

2. Development of first plastic hinge

In order to achieve the design goal, a very important problem is to be solved in seismic design of Moment Resisting Frames (MRFs): the identification of zones subjected to yielding in the beams. At this aim, the force acting on the beam in the seismic condition are to be identified. It is well known that in seismic condition, the loads on the structures are constituted by an appropriate distribution of horizontal and vertical forces. By considering the two effects separately, the superposition principle can be applied as reported in Figure 3 and the total bending moment diagram can be easily obtained as reported in Figure 4, where the sections corresponding to the beam ends are identified with the numbers 1 and 5, the sections corresponding to the RBSs are called 2 and 4 and, finally, the section where the maximum bending moment occurs is characterised by number 3.

It is evident that the aim of the procedure herein presented is to assure the development of plastic hinges in sections 2 and 4 when section 1, 3 and 5 are in elastic range.

In order to find the most general solution, the non-dimensional resistance of beam to column connections can be expressed by means of following parameter :

$$m_c = \frac{M_{p,con}}{M_b} \tag{1}$$

where $M_{p,con}$ is the plastic moment of the beam to column connection and M_b is the plastic moment of the wooden beam. Other important design parameters are: the location of the "dog-bones" (which is denoted with the distance a in Figure 4), and the magnitude of the weakening characterising the "dog-bones" expressed as:

$$m_{db} = \frac{M_{p,db}}{M_b} \tag{2}$$

where $M_{p,db}$ is the plastic moment of the weakened section of the IPE steel profile. To obtain the desired result, we assume m_{db} as fixed, while the location a of the "dog-bones" is considered as variable and is to be properly selected. It is easy to check that for sections 1 and 2 the bending moments due to vertical loads and horizontal forces have an opposite sign, in particular one is anticlock-wise and another is clock-wise, while for sections 3 and 4 they have the same sign, i.e. clockwise.

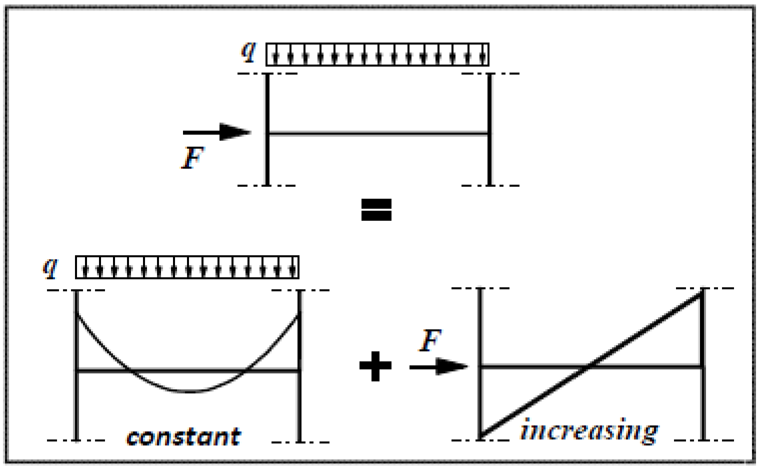

Figure 3: Bending moment due to vertical loads and seismic forces.

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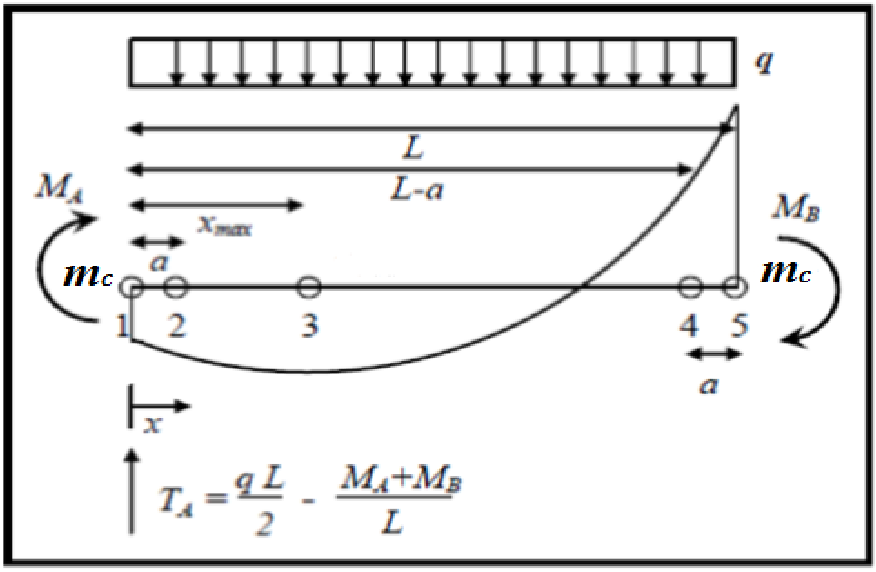

Figure 4: Total beam bending moment diagram.

For these reasons, plastic hinge develops in beam sections 4 or 5 rather than in beam sections 1 or 2 when horizontal forces increase. In particular we are interested in determining the conditions assuring the yielding of section 4, while sections 1, 2, 3 and 5 remain elastic, when seismic horizontal forces increase. At this aim let us consider the expression of bending moment at the generic section x [33]-[34]:

$$M(x) = M_A + q\frac{L}{2}x - \frac{(M_A + M_B)}{L}x - q\frac{x^2}{2} \quad (3)$$

And the value of x_{max} of the abscissa where the bending moment reaches its maximum value [33]-[34]:

$$x_{max} = \frac{L}{2} - \frac{M_A + M_B}{ql} \quad (4)$$

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

Section 1 $$M(x = 0) = M_A \quad (5)$$

Section 2

$$M(x = a) = M_A + q\frac{L}{2}a - \frac{(M_A + M_B)}{L}a - q\frac{a^2}{2} = q\frac{a(L-a)}{2} + M_A\left(1 - \frac{a}{L}\right) - M_B\frac{a}{L} \quad (6)$$

Section 3 $$M(x = x_{max}) = q\frac{L^2}{8} + \frac{(M_A - M_B)}{2} + \frac{(M_A + M_B)^2}{2qL^2} \quad (7)$$

Section 4 $$M(x = L - a) = q\frac{a(L-a)}{2} + M_A\frac{a}{L} - M_B\left(1 - \frac{a}{L}\right) \quad (8)$$

Section 5 $$M(x = L) = -M_B \quad (9)$$

As above recalled, 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, can be expressed in the following way:

Section 1 => $M_A < m_c M_b$ (10)

Section 2 => $M(x = a) < m_{db} M_b$ (11)

Section 3 => $M(x = x_{max}) < M_b$ (12)

Section 4 => $M(x = L\text{-}a) = -m_{db} M_b$ (13)

Section 5 $M(x{=}L) > -m_c M_b$ => $-M_B > -m_c M_b$ => $M_B < m_c M_b$ (14)

Now if we combine the yielding condition of “dog-bone” (Eq.(13)) with the expression of bending moment at the abscissa $x=L-a$ given by Eq. (8), the following relation between M_A and M_B can be found:

$$M_B = q\frac{aL}{2} + M_A\frac{a}{(L-a)} + m_{db}M_b\frac{L}{(L-a)} \tag{15}$$

This equation represents the relation existing between the end moments of the beam when the first plastic hinge occurs at section 4 (the right “dog-bone”) . By using Eq. (15) and (3), the design requirements (10), (11), (12) and (14) can be expressed as:

$$M_A < M_{A1} \quad with \quad M_{A1} = m_c M_b \tag{16}$$

$$M_A < M_{A2} \quad with \quad M_{A2} = \frac{m_{db}L}{L-2a}M_b - q\frac{a(L-a)}{2} \tag{17}$$

$$M_A < M_{A3} \quad with \quad M_{A3} = \sqrt{2q(L-a)^2(1+m_{db})M_b} - \left(\frac{q(L-a)^2}{2} + m_{db}M_b\right) \tag{18}$$

$$M_A < M_{A5} \quad with \quad M_{A5} = \frac{(m_c - m_{db})L - am_c}{a}M_b - q\frac{L(L-a)}{2} \tag{19}$$

When the four inequalities reported above are satisfied, then the first plastic hinge develops in the right “dog-bone” if Eq. (13) is verified. In addition, the design goal requires that the second plastic hinge occurs in the left “dog-bone” when horizontal forces further increase. On the contrary, the yielding of section where the maximum bending moment occurs (section 3) and the section of the beam ends (section 1 and 5 close to the beam-to-column connections) has to be prevented, because, on one hand we want to avoid the yielding of wooden section and, on the other hand, we want to protect the beam-to-column connections.

3. Location of second plastic hinge

It is plain to see that as the seismic horizontal forces increase, M_A value increases, and relationships (16), (17), (18) and (19) allow to identify the section where the second plastic hinge develops. At this aim, it is sufficient to check which is the minimum limit value among M_{A1}, M_{A2}, M_{A3} and M_{A5}, in other words we can determine which condition is the first one to be unsatisfied when horizontal forces increase. In order to solve this problem, it is useful to compare the limit value M_{Ai} (with i=1,2,3,5). In particular the following conditions can be analysed:

$$\mathrm{M_{A2}} < \mathrm{M_{A3}} \quad condition\ A \tag{20}$$

This relation allows to determine the values of a which assure that the yielding of left “dog-bone” (section 2) precedes the yielding of the of the beam in the section where the maximum bending moment is reached (section 3);

$$\mathrm{M_{A2}} < \mathrm{M_{A5}} \quad condition\ B \tag{21}$$

This relation identifies the a values able to assure that the yielding of the left “dog-bone” (section 2) precedes the yielding of the connection B (section 5);

$$\mathrm{M_{A2}} < \mathrm{M_{A1}} \quad condition\ C \tag{22}$$

This relation allows to determine the values of a which assure that the yielding of left "dog-bone" (section 2) precedes the yielding of the connection A (section 1).

Conditions (20), (21) and (22) have to be absolutely satisfied, because they assure the development of the second plastic hinge in the left "dog-bone", while the yielding of the connections at the beam ends and of section where the maximum bending moment is achieved is prevented. In other words, relationships (20), (21) and (22) constitutes the design requirements.

Starting from the above conditions the following non-dimensional relationships can be generated:

◊ *Condition A*:

$$4\left(\frac{a}{L}\right)^3+\left[4\sqrt{\frac{M_b}{qL^2}(m_{db}+1)}-8\right]\left(\frac{a}{L}\right)^2+\left[4m_{db}\frac{M_b}{qL^2}+5-6\sqrt{\frac{2M_b}{qL^2}(m_{db}+1)}\right]\left(\frac{a}{L}\right) \\ +2\sqrt{\frac{2M_b}{qL^2}(m_{db}+1)}-1-4m_{db}\frac{M_b}{qL^2}>0 \tag{23}$$

The solution of the equation (23) is given by:

$$\frac{a_1}{L}<\frac{a}{L}<\frac{a_2}{L} \tag{24}$$

where

$$\frac{a_1}{L}=\frac{1}{2}-\sqrt{\frac{(1+m_{db})}{2}\frac{M_b}{qL^2}}-\sqrt{\frac{(1-m_{db})}{2}\frac{M_b}{qL^2}} \tag{25}$$

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

◊ *Condition B*:

$$-2\left(\frac{a}{L}\right)^4+5\left(\frac{a}{L}\right)^3-4\left(1+\frac{M_b}{qL^2}m_c\right)\left(\frac{a}{L}\right)^2+\left[1+2\frac{M_b}{qL^2}(3m_c-m_{db})\right]\left(\frac{a}{L}\right) \\ -2\frac{M_b}{qL^2}(m_c-m_{db})<0 \tag{27}$$

Whose solution is:

$$\frac{a}{L}<\frac{a_3}{L} \tag{28}$$

where:

$$\frac{a_3}{L}=\sqrt[3]{T_1+\sqrt{T_2}}+\sqrt[3]{T_1-\sqrt{T_2}}+\frac{1}{2} \tag{29}$$

with T_1 and T_2 given by:

$$T_1 = \left[-\frac{M_b m_{db}}{2qL^2}\right] \quad ; \quad T_2 = \left[-\frac{M_b m_{db}}{2qL^2}\right]^2 + \left[2\frac{M_b m_c}{3qL^2} - \frac{1}{12}\right]^3 \tag{30}$$

◊ *Condition C*:

By means of Eq.(16) and (17) this condition provides:

$$2\left(\frac{a}{L}\right)^3 - 3\left(\frac{a}{L}\right)^2 - \left(4m_c\frac{M_b}{qL^2} - 1\right)\left(\frac{a}{L}\right) + 2(m_c - m_{db})\frac{M_b}{qL^2} > 0 \tag{31}$$

With the aim of showing that this condition is always verified if condition *B* is verified, it is useful to write the condition *B* (Eq. (27)) as follows:

$$\left(\frac{a}{L} - 1\right)\left[-2\left(\frac{a}{L}\right)^3 + 3\left(\frac{a}{L}\right)^2 - \left(4m_c\frac{M_b}{qL^2} + 1\right)\left(\frac{a}{L}\right) + 2(m_c - m_{db})\frac{M_b}{qL^2}\right] < 0 \tag{32}$$

So that the product between the two terms in parentheses is lower than zero, if a/L <1 (as always is), the term in square brackets must necessarily be greater than zero:

$$\left[-2\left(\frac{a}{L}\right)^3 + 3\left(\frac{a}{L}\right)^2 - \left(4m_b\frac{M_b}{qL^2} + 1\right)\left(\frac{a}{L}\right) + 2(m_b - m_{db})\frac{M_b}{qL^2}\right] > 0 \tag{33}$$

Now it is easy to verify that the first member of Eq. (31) is greater than the first member of Eq. (33) when the following condition is satisfied:

$$-4\left(\frac{a}{L}\right)^3 + 6\left(\frac{a}{L}\right)^2 - 2\left(\frac{a}{L}\right) < 0 \tag{34}$$

the solution of this inequality is:

$$0 < \frac{a}{L} < \frac{1}{2} \quad and \quad \frac{a}{L} > 1 \tag{35}$$

Assuming that *a/L* is always greater than zero and lower than ½ (because the reduced section cannot be realized beyond the midspan) it can be concluded that condition *C* is verified if condition *B* is verified.

So that the final result can be summarized as:

$$\frac{a_1}{L} < \frac{a}{L} < min\left\{\frac{a_2}{L}; \frac{a_3}{L}\right\} \tag{36}$$

From a graphical point of view the results of these relationships are presented in Figure 5, Figure 6 and Figure 7 for m_{db} equal to 0.60-0.80 and m_c greater than m_{db}. It is easy to check that if a value of m_c equal or smaller than m_{db} is considered, then the values obtained for a_3/L are negative and according to eq. (36) no solution is available. This result appears to be evident because the bending moment at the connection B (section 5) is always greater than the bending moment occurring in the adjacent "dog-bone", as a consequence, if the resistance of the connection is smaller than the resistance of the "dog-bone", the yielding of the latter cannot precede the yielding of the former.

The obtained result can be furtherly simplified by means of the following considerations: if $a_2/L > a_3/L$ then $min\{a_2/L;\ a_3/L\} = a_3/L$ and a so a_2 does not play any rule and can be neglected. In the opposite case when $a_2/L < a_3/L$, from a numerical analysis it is easy to check that the point where the maximum bending moment is obtained is outside of the beam, i.e. the value of x_{max} is negative. In this case, the requirement provided by *condition A* is not effective and, as a consequence, a_2 does not play any rule and can be neglected.

So that it can be concluded that the final result is:

$$\frac{a_1}{L} < \frac{a}{L} < \frac{a_3}{L} \tag{37}$$

For the given vertical load q, the given value of m_{db}, m_c and M_b, if the location of the "dog-bones" respects the condition (37) then plastic hinges will develop in both "dog-bones" leaving the wooden beam and the beam to column connections in elastic range when horizontal forces increase.

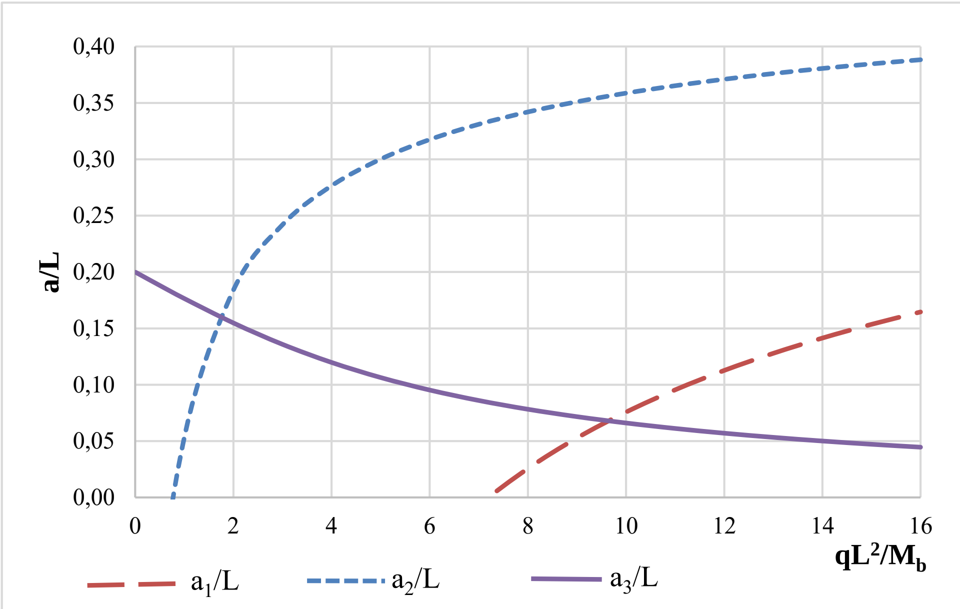

Figure 5: *Limit values of a1/L, a2/L and a3/L for m_{db} = 0.60 and m_c=1.00.*

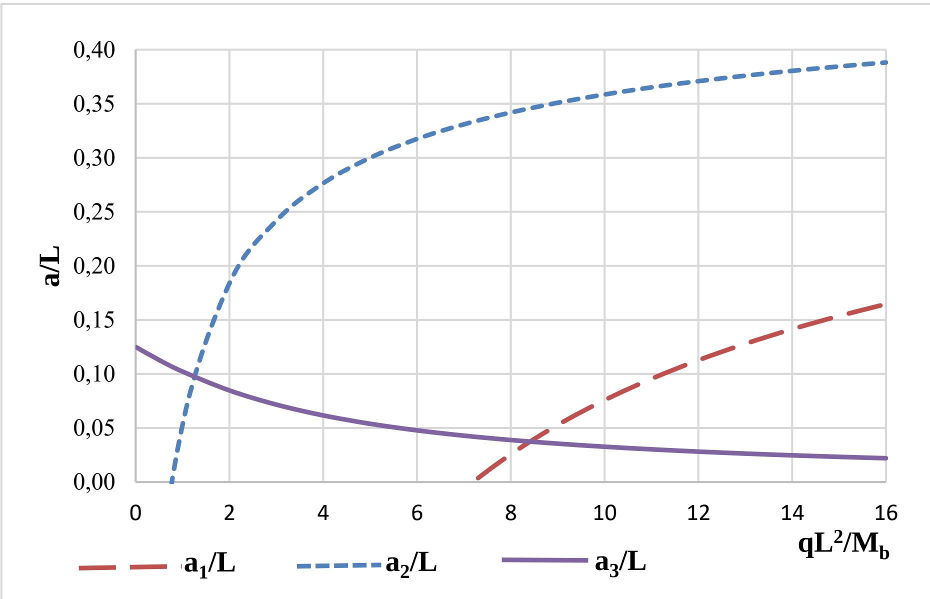

Figure 6: Limit values of a_1/L, a_2/L and a_3/L for m_{db} = 0.70 and m_c=0.80.

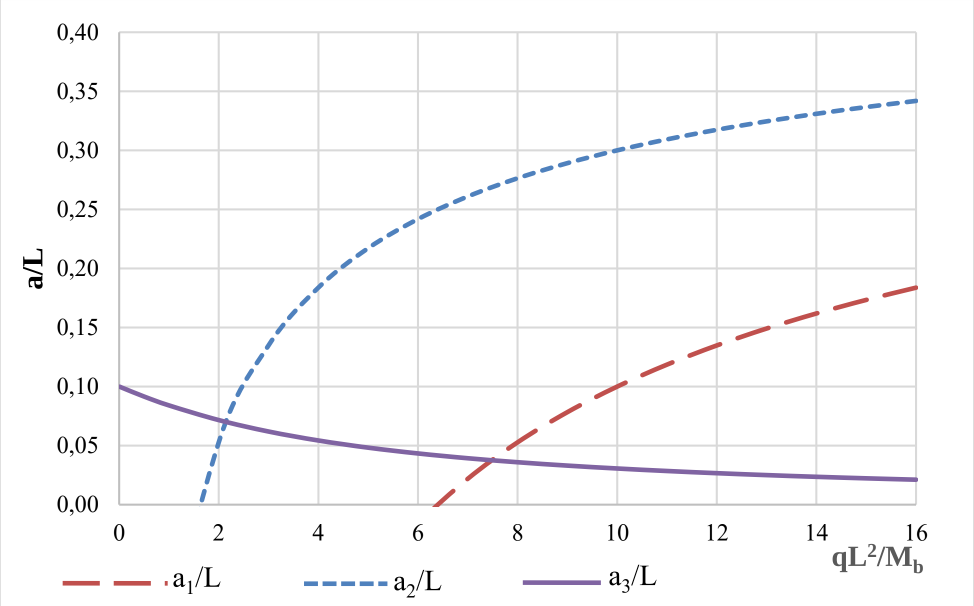

Figure 7: Limit values of a_1/L, a_2/L and a_3/L for m_{db} = 0.80 and m_c=1.00.

4. Design abachi and comparison regarding the current codes provisions

Different design abachi, accounting for all the parameters involved in the design process, can be derived starting from the result obtained in the previous section.

In particular, by considering a given value of m_c, equations providing the limit values a_1/L and a_3/L can be determined for all the possible values of m_{db} and qL^2/M_b. In such a way, the abachi reported in Figure 8 (for m_c equal to 1), Figure 9 (for m_c equal to 0.9), Figure 10 (for m_c equal to 0.8), and Figure 11 (for m_c equal to 0.7), are obtained.

As already noted, if the value of m_c is equal or smaller than the value of m_{db} no solution can be found, because if the resistance of the beam to column connections is smaller than the adjacent RBS, the yielding of the latter section cannot precede the yielding of the former one. Furthermore, it is evident that by increasing the value of non-dimensional vertical load the available range of a/L decreases.

In fact, from Figure 8, it can be observed that for a value of non-dimensional load lower than six, only an upper limit is effective, because the lower one becomes negative, so that it does not provide any condition.

From the same figure it can be noted that by decreasing the value of non-dimensional vertical load from five to zero the upper limit increases.

On the contrary, when the value of the vertical load is equal or greater than six, also a lower bound is obtained. Consequently, in this case a range for a/L is provided.

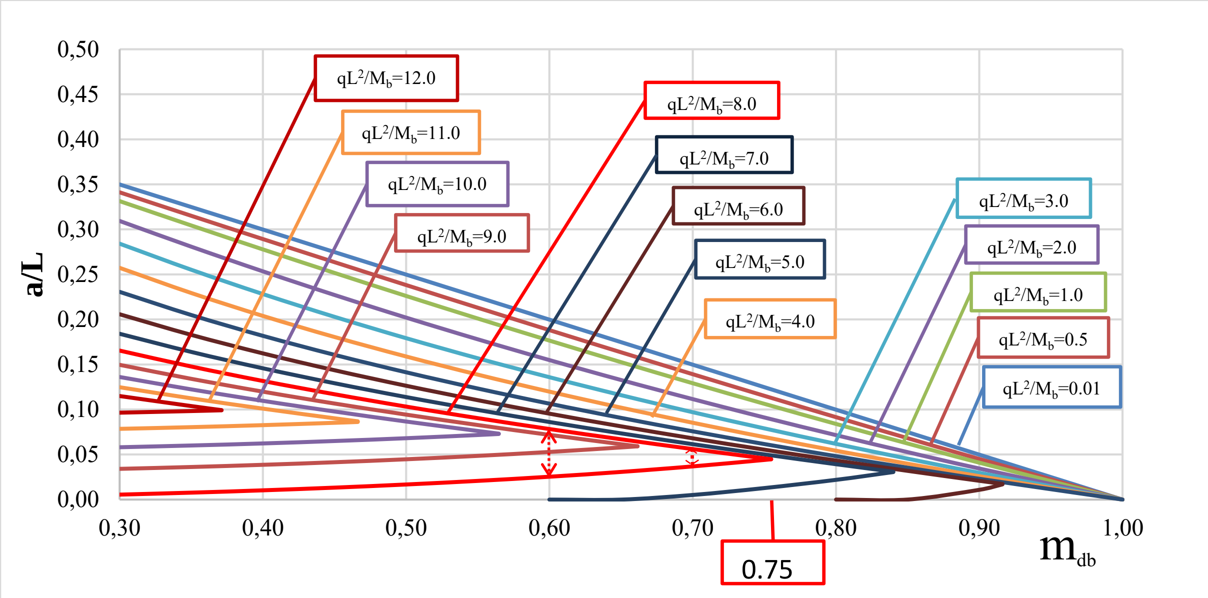

Figure 8: *Design abacus for m_c =1.00*

As an example, considering the abacus given in Figure 8, if a vertical load qL^2/M_b equal to 8 is assumed, no solution is available for values of m_{db} greater than 0.75. On the contrary, a range, which increases as m_{db} decreases, can be found for values of m_{db} lower than 0.75. As it can be observed from the figure, if a bigger value of vertical load is considered, a smaller value of the range is obtained. In fact, with the increase of vertical load, there is a decrease of the upper limit and an increase of the lower limit of *a/L*. As an example for $m_c=1$ if qL^2/M_b=8 and m_{db} = 0.70 the available range for *a/L* is *[0.037-0.056]*, while if m_{db} = 0.60, for the same value of vertical load, the available range increases, because it is given by *[0.025-0.072]* Figure 8.

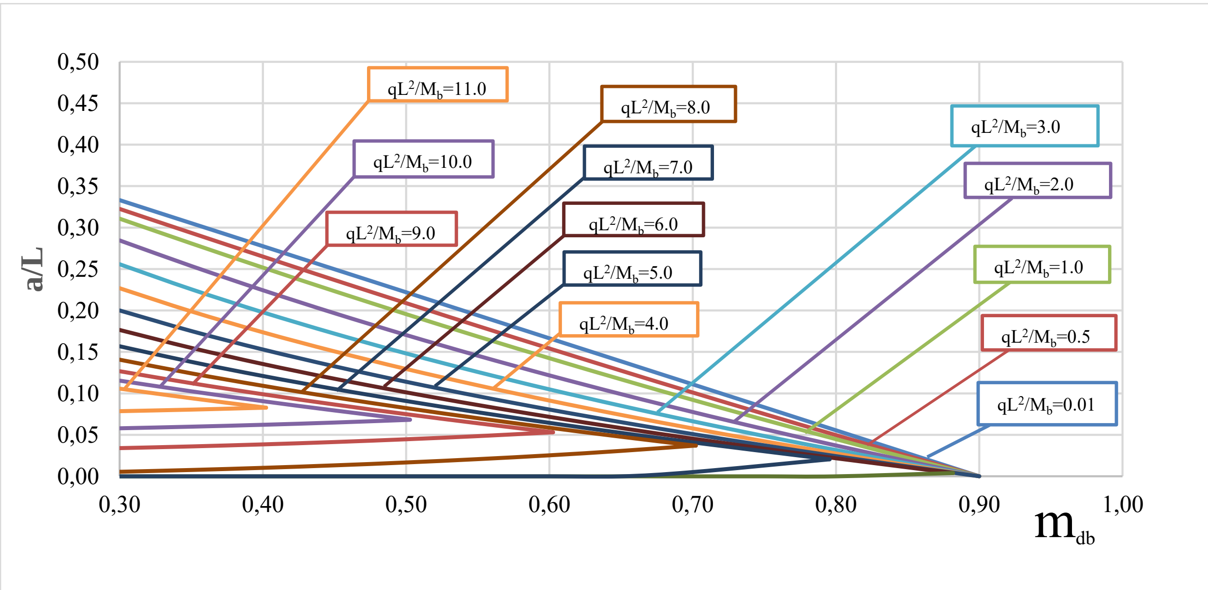

Figure 9: *Design abacus for m_c =0.90*

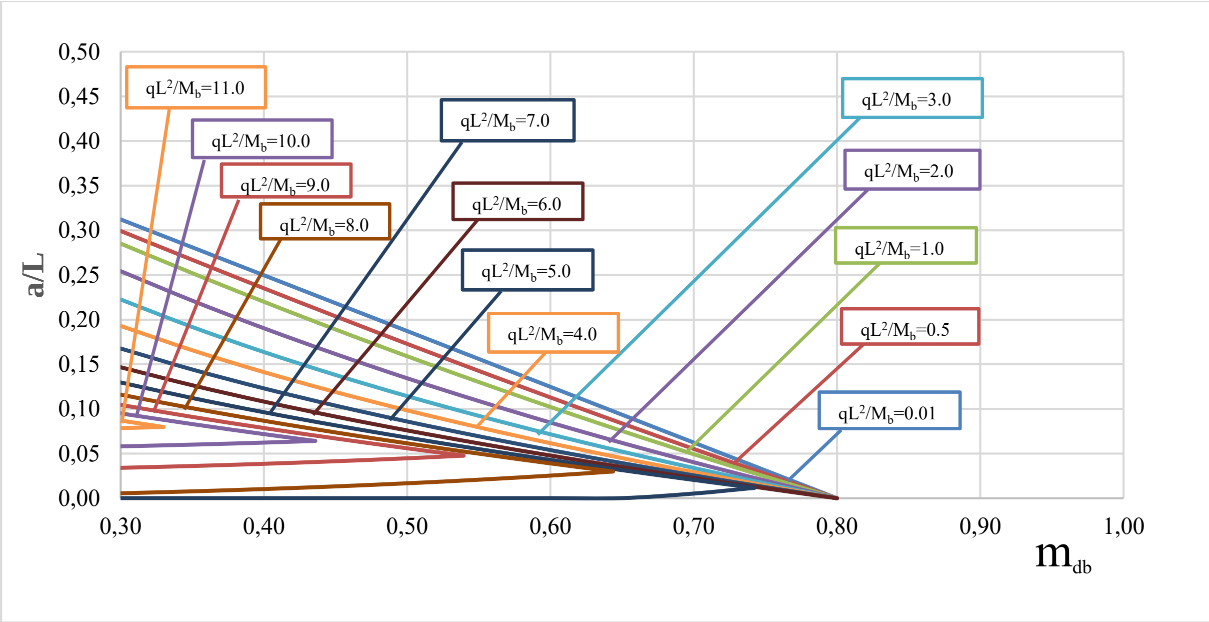

Figure 10: *Design abacus for m_c =0.80*

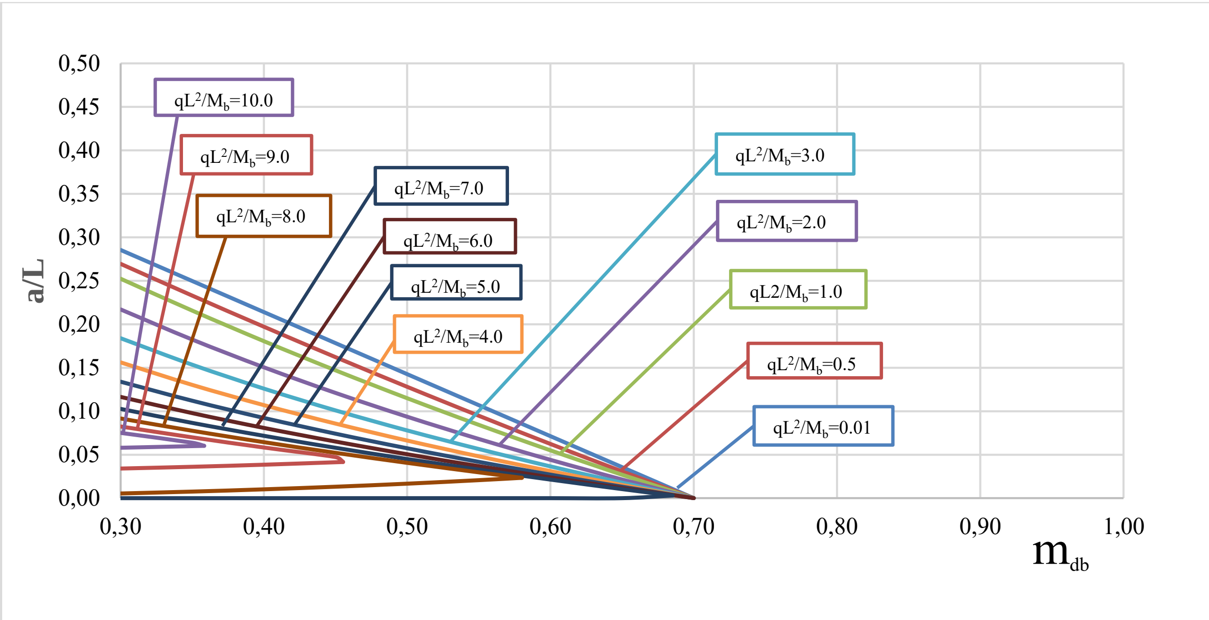

Figure 11: *Design abacus for m_c =0.70*

At the aim of applying in a correct way the obtained abaci it is of fundamental importance to clarify the meaning of parameters *a* and *L*. In fact, *a* is the distance between the beam to column connections and the middle point of the RBS, while *L* is the distance between the two connections as depicted in Figure 12, where the length of the reduced beam section has been reported and indicated with *b*.

Consequently *L* is a quantity which is different from the bay span L_i. Obviously the relation between *L* and L_i can be obtained form Figure 12 as:

$$L = L_i - H_{c1}/2 - H_{c2}/2 \tag{38}$$

being H_{c1} and H_{c2} the section heights of the column. This observation is very important, because in the design phase of beams and “dog-bones”, column sections are still unknown, and, as a consequence, L is still unknown. So that, in this phase, only the value of L_i is available. From a practical point of view, even if it is an important aspect, this fact does not represent a big problem, because a value for H_{c1} and H_{c2} can be assumed as first attempt. What is important to underline is that at the end of the frame design, the value of L which is to be used in the abacus is the one given in eq.(38) and represented in Figure 12.

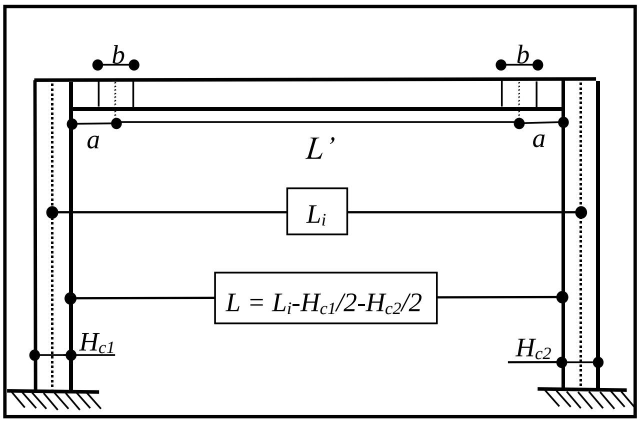

Figure 12: Difference between L and L_i

Another important consideration regarding the utility of the above abaci concerns the design provision nowadays available in current codes ([35] [36]) for the design of RBS. In fact, the only available suggestion is the maximum bending moment expected at the beam-to-column connection, which is obtained by a free body diagram as:

$$M_{max,con} = m_{db} M_b + V_{db} a \tag{39}$$

where V_{db} is the maximum shear strength in the reduced beam section given by:

$$V_{db} = \frac{2 m_{db} M_b}{L - 2a} + q \frac{(L - 2a)}{2} \tag{40}$$

The design requirement provided by codes is

$$M_{max,con} \le M_{p,con} \quad \Rightarrow \quad m_{db} M_b + V_{db} a \le M_{p,con} \Rightarrow \ a \le \frac{M_{p,con} + m_{db} M_b}{V_{db}} \tag{41}$$

So that an upper limit for the location of RBS is determined, but no lower bound is given. This is not correct, because when a lower limit is given in the abaci above reported, it means that the *condition A* is not satisfied if $a<a_1$. In these cases, if a is lower than a_1, the second plastic hinge will not develop in the second dog bones (section 2), but in the section of the beam where the maximum bending moment is reached (section 3). So that the yielding of a wooden section is not avoided. In other words, the limit value of a provided in equation (41) could be not enough. In fact, by satisfying the equations (39), we can be sure that (with reference to Figure 4) the yielding of right “dog bone”(section 4) will precede the yielding of connection B (section 5),

but then we have no information regarding the formation of second plastic hinge. So we cannot be sure that the second plastic hinge will develop in the second “dog-bone” rather than in one of the connections (section 1 or 5) or in the section where the maximum moment is achieved when horizontal forces increase. This information can be cached only by solving the equations provided in Session 3. In addition, the upper limit provided by equation could be different from the actual upper limit because it is obtained on a final structural scheme with two plastic hinges in the two RBSs, but the intermediate scheme characterized by only one plastic hinge is completely neglected. It can be concluded that the current codes provisions for the design of RBSs are not able to avoid the yielding of a connection or the yielding of the wooden section where the maximum bending moment is achieved. The desired design goal can be reached only if the limits provided by the obtained abaci are respected.

4. Conclusions

In this paper the use of steel reduced beam section in a wooden beam have been proposed and analysed. The goal of the proposed design method is the protection of beam to column connections and of all intermediate sections of the wooden beam when horizontal forces increase. In particular, the yielding of both “dog-bones” is to be promoted, leaving all the other sections in elastic range.

The procedure takes into account the possibility that the beam to column connection is a partial strength connection, because, as often happens for wooden structures, it is not able to transmit to the column the entire plastic moment of the beam.

From the obtained results it can be observed that the smallest m_{db} value provides the widest range for the realization of the “dog-bones”; in fact, increasing m_{db} the range identified by a reduces up to zero for $m_{db} = m_c$.

The provided design abachi constitute an easy way to understand if the beam-to-column connections and all the wooden beam sections are protected or not by the realization of a “dog-bone” when horizontal forces increase. In fact, all the parameters involved in the design process are necessary in order to use the abaci; in particular, the non-dimensional resistance of the “dog-bones” m_{db}, the non-dimensional resistance of the beam to column connection m_c, the non-dimensional distance of the “dog-bone” from the beam to column connection a/L, and the non-dimensional value of vertical load qL^2/M_P.

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