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The use of steel rbs to increase ductility of wooden beams

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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.

For this purpose, 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 present paper a rigorous analysis is developed in order to assure the protection of the beam to column connections and of all intermediate sections of the wooden beam when horizontal forces increase and steel "dog-bones" yield. In other words, the yielding of both "dog-bones" is promoted, leaving all the other sections in elastic range. The role played by vertical loads and by the amount of section reduction are accounted for, and in addition, the possibility that the beam to column connections are realized with a partial strength connection (a connection which is not able to transmit to column the whole plastic moment of the beam) is considered.

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 [6–16].

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 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 [17–29]. For this reason experimental and theoretical investigations [30–35] 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" (Fig. 1 – Photo taken in the

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

STRENGTH lab at the University of Salerno – Italy).

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 a double T 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 (Fig. 2). As an example, in Fig. 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.

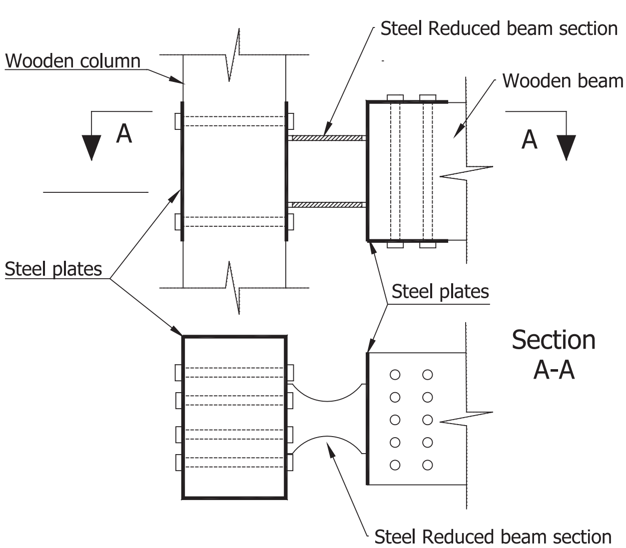

Fig. 2. The steel "Dog-Bone" for a wooden beam.

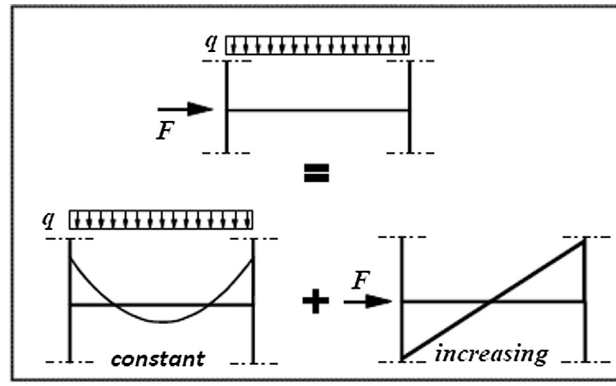

Fig. 3. Bending moment due to vertical loads and seismic forces.

Finally, due to the fact that steel has an higher yielding strength than wood, the steel part of the beam can have smaller section (Fig. 2) than the wooden part in order to accommodate architectural needs. In fact, the steel part could be completely covered by non-structural wooden elements.

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 Fig. 3 and the total bending moment diagram can be easily obtained as reported in Fig. 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 characterized 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 Sections 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

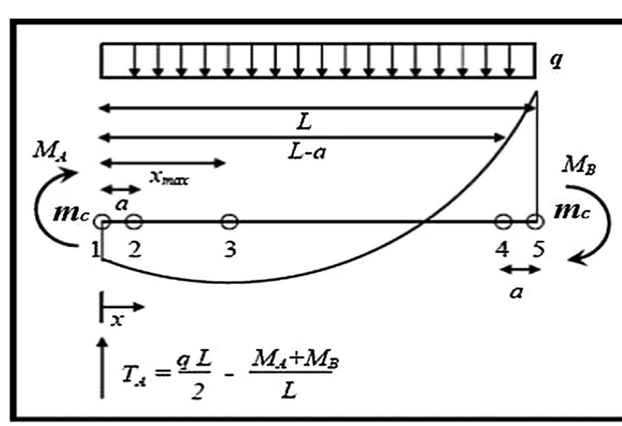

Fig. 4. Total beam bending moment diagram.