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SEISMIC DESIGN OF STEEL CONNECTIONS

BY

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Abstract. One of the main problems that occurre in the designing process of steel structure is the achieving of a proper connection. By the means of ductility, similar to the case of structures, connections are divided into ductility classes. The estimation of the ductility is based on the plastic rotation capacity of the connection. One of the most important aspects that one needs to take into account when designing a steel structure is the dissipative mechanism of the structure as well as the structural properties of the connection. The accurate estimation of connections structural properties is crucial in order to correctly choose the structural type and the structural analysis scenario to be applied.

Key Words: Seismic Design; Steel Connection; Ductility.

1. Introduction

In case of large seismic events the design of steel structures must be able to accurately approximate the response of the structure beyond the elastic range. As a consequence a mechanism must be supplied within some elements of the structural system so to accommodate the large displacement demand imposed by earthquake ground motions. In everyday applications, structural elements, such as walls, beams, braces and to a lesser extent columns and connections, are designed to undergo local deformations well beyond the elastic limit of the material without significant loss of capacity. Provisions of such large deformation capacity, known as ductility, are a fundamental tenet of seismic design.

In most cases, good seismic design practice has incorporated an approach that would provide for the ductility to occur in the members rather than the connections. This is especially the case for the steel frame structures, were the basic material has long been considered the most ductile of all materials used for building construction [1].

Another design philosophy encourages the contributions to the displacement ductility demand of connections through absorption of substantial energy quantities. In order to properly incorporate these elements into seismic design a much greater level of attention needs to be paid than for standard connection design or for moment connections to be subjected only to typical static loads. Besides typical strength requirements, such connections should take into account factors like

a) toughness of joining elements in the connections, including any weldments,

b) high level of understanding of the distribution of stresses and strains throughout the connection,

c) elimination of stress concentrations,

d) detailed consideration of the flow of forces and the expected path of yielding in the connection,

e) good understanding of the properties of the materials being joined at the connection,

f) the need for heightened quality control in fabrication erection, and inspection of the connection.

While these types of considerations are particularly critical for connections where inelastic response is anticipated, it also behooves the designers to take factors such as these into account for all connections of the seismic resisting system.

2. Structural Properties of Connections

2.1. Connection stiffness

The connections stiffness can be taken as the slope of the $M-\phi$ curve, (Fig. 1). Since the curves are nonlinear from the start, it is possible to define this stiffness based on tangent approach or on secant approach. A tangent approach is viable only if the analysis programs available can handle a continuous, nonlinear rotational spring. Even in this case, the computational overhead can be large and this option is recommended only for verification of the seismic performance of irregular structures. In most designs, for regular frames, a secant approach will probably yield a reasonable solution at a fraction of the calculation effort required by the tangent approach. In this case, the analysis can be carried out in two steps using linear springs. The stiffness of the connection is meaningful only when compared to the stiffness of the connected members.

A joint may be classified as rigid/fully restrained (FR), nominally pinned/simple or semi-rigid/partially restrained (PR) according to its rotational stiffness, by comparing its initial rotational stiffness. A nominally pinned joint shall be capable of transmitting the internal forces, without developing significant moments which might adversely affect the members or the structure as a whole. A nominally pinned joint shall be capable of accepting the resulting rotations under the design loads. Joints classified as rigid may be assumed to have sufficient rotational stiffness to justify analysis based on full continuity. A joint which does not meet the criteria for a rigid joint or a nominally pinned joint should be classified as a semi-rigid joint.

According to EN 1993-1-8 [2], the joints may be classified based on their rotational stiffness, by comparing its initial rotational stiffness with the bending stiffness of the connected members.

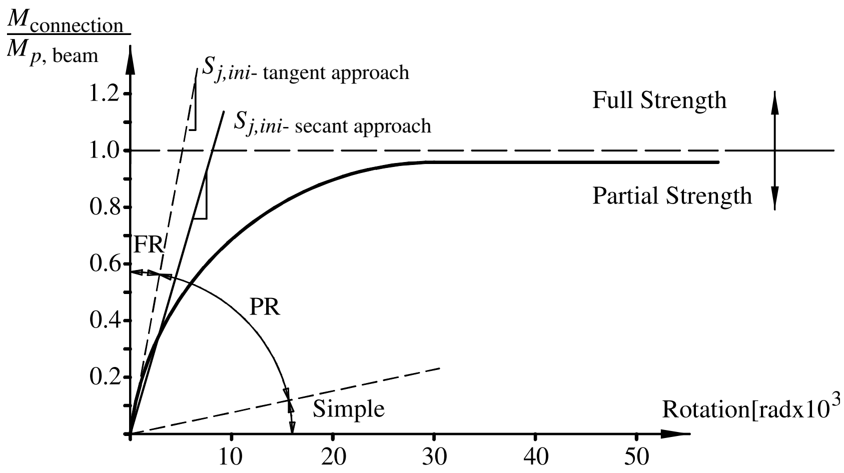

Fig. 1. – Connection classification by stiffness and strength.

The connections are classified as being rigid if

(1) $$S_{j,ini} \geq \frac{k_b E I_b}{L_b},$$

where $k_b = 8$ for frames where the bracing system reduces the horizontal displacement by at least 80% and $k_b = 25$ for other frames, provided that in every storey $k_b/k_c \geq 0.1$

The connections are classified as being nominally pinned if

(2) $$S_{j,ini} \leq \frac{0.5 E I_b}{L_b}$$

where: k_b – the mean value of I_b/L_b for all the beams at the top of that storey, k_c – the mean value of I_c/L_c for all the columns in that storey, I_b – the second moment of area of a beam, I_c – the second moment of area of a column, L_b – the span of the beam (centre-to-centre of columns) and L_c – the storey height of a column.

2.2. Connection Strength

A connection can be also be classified in terms of strength as either a full-strength, nominally pinned or partial-strength. The design resistance of a full strength joint shall be not less than that of the connected members, while a partial-strength connection can only develop a portion of it. A nominally pinned joint shall be capable of transmitting the internal forces, without developing significant moments which might adversely affect the members or the structure as a whole. For classifying connection according to strength, it is common to nondimensionalize the vertical axis of the $M-\phi$ curve by the beam plastic moment capacity, $M_{p,\text{beam}}$, [1] as is shown in Fig. 1. Connections not capable of transmitting at least 25% of the design resistance for full strength connections are classified as nominally pinned. A nominally pinned joint shall be capable of accepting the resulting rotations under the design loads. A joint which does not meet the criteria for a full-strength joint or a nominally pinned joint should be classified as a partial-strength joint.

2.3. Connection Ductility

Connection ductility is a key parameter either when the deformations are concentrated in the connection elements, as is the typical case in semi-rigid connections, or when large rotations are expected adjacent to the connections, as in the case of ductile moment frames with welded connections. The required ductility will depend on the flexibility of the connections and the particular application (for example, a braced frame in a nonseismic area *versus* an unbraced frame in high seismic area). A connection can be classified as ductile based on both its absolute and its relative rotation capacity.

The design code EN 1998-1 [3] has introduced three levels of structural ductility class connections with design concepts and range of reference values of the behavior factors: low ductility class (DCL), medium ductility class (DCM) and high ductility class (DCH). For medium and high ductility classes, specific requirements are introduced concerning structural ductility (behavior factor q), element ductility (cross sectional classes), material (yield strength and toughness) and joint ductility (rotation capacity).

Dissipative semi-rigid and/or partial strength connections are permitted, provided that all of the following conditions are satisfied:

a) the connections have a rotation capacity consistent with the deformations;

b) members framing into the connections are demonstrated to be stable at ultimate limit state (ULS);

c) the effect of connections deformation on global drift is taken into account using nonlinear static (pushover) global analysis or nonlinear time history analysis.

The overstrength condition for connections need do not apply necessarily if the connections are designed in a manner enabling them to contribute significantly to the energy dissipation necessary to achieve the chosen q-factor.

The moment frame connections design should be such that the plastic rotation capacity in the plastic hinge is not less than 35 mrad for structures of high ductility class and 25 mrad for structures of medium ductility class with $q > 2$. When assessing the rotation capacity in the plastic hinge the following factors must be taken into consideration: deformation of the connection, including column web panel deformation; plastic hinge rotation and elastic deformation of the beam [4]. The column elastic deformation should not be included in the evaluation of plastic rotation capacity of the plastic hinge [3].

Even with the limits mentioned above for the rotation capacity of joints, EN 1998-1 does not specify any formula for this evaluation, except testing and design experience.

Moment frames, defined as a building frame systems in which seismic shear forces are resisted by shear and flexure in members and connections of the frame are divided as follows (according to AISC 341-05 [5]): special moment frame (SMF), intermediate moment frame (IMF) and ordinary moment frame (OMF). According to the same regulations, the moment frame connections must be designed so that the plastic rotation capacity in the plastic hinge to be at least 40 mrad for SMF and 20 mrad for IMF This values include the elastic rotation of the column, which equal to 10 mrad for most of the moment frames [5].

Even if the rotation capacity of the beam-to-column joints is connected with the classification of frames, the AISC code is not providing any formula for the evaluation of this very important characteristic.

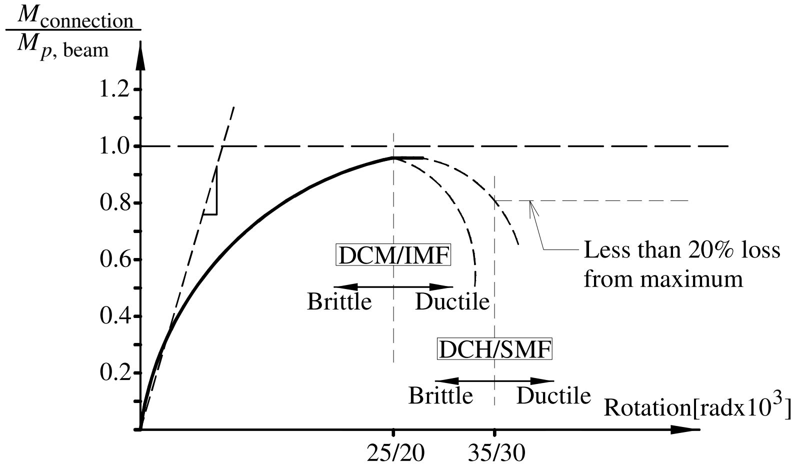

Fig. 2. – Connection classification by absolute ductility.

The American document FEMA 350 [6], in chapter 3: "Connection Qualification", provides pre-qualification data and design procedures for alternative types of welded, fully restrained, steel moment-frame connections, suitable for use in new constructions. This pre-qualification is extremely important, because a designer is able to choose a type of connection and then to follow the instructions concerning the cross sectional dimensions, in order to obtain a certain ductility class of the joint. These SMF connections, described in FEMA 350 design document, are designed to sustain a total rotation of 40 mrad before significant strength degradation and 60 mrad before complete loss of resistance.

Both, the absolute and relative rotation capacities, however, need to take into account any strength degradation that may occur as a result of local buckling or slip, particularly under cyclic loading. Both EN 1998-1 and AISC 341-05 require that the strength degradation in ductile connections subjected to cyclic loads to be limited to 20% of the maximum capacity when the relative or absolute rotation limits are reached.

Fig. 2 presents the classification boundaries based on the absolute rotation for moment frames with high and medium ductility according to EN 1998-1 and special moment frames and intermediate moment frames according to AISC 341-05.

For comparing the rotation capacity of connections with similar moment-rotation capacity a relative ductility index, μ, equal to the ratio of ultimate rotation to the yield rotation can be defined. Usually, relative ductilities of 6 or more have been associated with ductile connections [1].

3. Seismic Design Requirements for Connections

Proper system selection is a critical element in successful seismic design. Various systems, such as fully and partially restrained moment-resisting frames, concentrically braced frames and eccentrically braced frames, are addressed in the EN 1998-1 and AISC 341-05 seismic provisions. These provisions have specific requirements for the different structural system that address connection design.

Table 1
Type of Joint Model

Method of global analysis	Classification of joint		
Elastic	Nominally pinned	Rigid	Semi-rigid
Rigid–Plastic	Nominally pinned	Full-strength	Partial-strength
Elastic–Plastic	Nominally pinned	Rigid and full-strength	Semi-rigid and partial-strength, Semi-rigid and full-strength, Rigid and partial-strength
Type of joint model	Simple	Continuous	Semi-continuous

Connection design depends very much on the designer's decision regarding the method by which the structure is analysed. Eurocode 3 gives four approaches for the design of a structure in which the behavior of the connection is fundamental. These design methods are defined as simple design, semi-continuous design, continuous design and experimental verification. Elastic, plastic and elastic–plastic methods of global analysis can be used with any of the first three approaches, and Table 1 shows how the joint classification, the type of framing and the method of global analysis are related [2].

For moment-frames with high and medium ductility classes connections have specified values for both inelastic deformation and strength capacities, since it expected that these connections will absorb substantial energy during the design earthquake. Deformation capacities are to be demonstrated by qualified cyclic testing of the selected connection type. At the minimum acceptable inelastic deformation level, the provisions require that the nominal beam plastic moment, Mp, be reached unless local buckling or a reduced beam approach is followed, in which the value is reduced to 0.8 of the nominal beam plastic moment. The minimum beam shear connection capacity is defined as resisting a combination of full-factored dead load, a portion of live and snow load, and the shear that would be generated by the expected moment capacity (including material overstrength) of the beam due to seismic actions. For moment frames with low ductility class, the strength requirement is similar and the deformation limit is reduced.

The design requirements for partial-strength in frames with high and medium ductility are similar to those required for full-strength connections as described previously. For structures with low ductility, a set of requirements are provide to ensure a minimum capacity level of 50% of the weaker connected member, and that connection flexibility is considered in the determination of the overall frames drifts.

4. Conclusions

A very important aspect that needs to be taken into consideration, when performing the seismic design of steel structures, is the dissipative mechanism of the structure. In the case of moment frames the seismic energy dissipative mechanisms ensure the consumption of energy by the plastic deformation of certain parts of the structure. As shown previously, the plastic hinge can be directed to the beams or even to the connection's elements, in the case of moment frames.

Due to the fact that the structural properties of connection subjected to cyclic loads can be determined only through experimental testing the global analysis as well as the design of the constructive details becomes very laborious when one chooses to direct the plastic hinges to the connection's elements. The provisions regarding the ductile connection pre-qualification provided in the American FEMA 350 document facilitate the design of such structures. In order to make life easier for the designers a similar standard should be elaborated, that must provide pre-qualification guidelines for connections.

An important aspect that must be taken into consideration when dealing with structures that are designed with connections having a dissipative behavior is the difficulties faced when retrofitting/replacing the damaged components, due to a strong seismic action, of that connection.

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PROIECTAREA SEISMICĂ A ÎMBINĂRILOR METALICE

(Rezumat)

Una din principalele probleme ce apar în proiectarea structurilor metalice o constituie asigurarea unei ductilităţi satisfăcătoare a îmbinărilor. Din punct de vedere al ductilităţii, similar structurilor, îmbinările sunt clasificate în clase de ductilitate. Modul de evaluare a ductilităţii îmbinărilor se realizează pe baza capacităţii de rotire plastică a acestora. Unul din cele mai importante aspecte ce trebuie luate în considerare atunci cand se proiectează seismic structurile metalice este mecanismul de disipare a energiei seismice precum şi proprietăţile structurale ale îmbinărilor. Determinarea corectă a proprietăţilor structurale ale îmbinărilor este esenţială pentru a alege corect tipul şi metoda de calcul a structurii.