Connection characteristics in seismic proof steel construction

P. Brož
Professional Association for Science, Research and Consultancy, Czech Republic

Abstract

The evaluation of steel structures damaged by earthquakes in the last decade (e.g., in Northridge, CA, U.S.A. (1994) and in Kobe, Japan (1995)) has contributed considerably to the improvement of the design of structures and especially details relating to their ductility and to the dissipation of energy. An analysis of research results led to significant corrections of codes, guidelines and instructions. It was necessary to re-evaluate and extend the content of corresponding codes currently based on Allowable Stress Design (ASD), plus the Load and Resistance Factor Design (LRFD) method. This paper is a brief comment on the substance of the reliability assessment of selected structural details with respect to the dissipation of energy corresponding to the repeated elastoplastic deformation caused by earthquakes. The application of the probabilistic methods is also indicated.

In research increasing attention is paid to the structural steel components exposed in case of earthquake to elastoplastic reversals. Such components serve as energy devices and their design and reliability assessment requires a special approach. The paper gives substance to the evaluation of the structural response to the earthquake, the actual behaviour of exposed details and the principles of safety assessment.
Keywords: beam-to-column connections, ductile damage, energy dissipation, joint characteristics, moment-rotation response, quasistatic failure, response damping.

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1 Introduction

Securing the reliability of load-bearing systems of buildings against earthquake effects is an important problem of design and the assessment of safety, serviceability and durability of structures. Ways are sought continuously to find an ever truer expression of earthquake “load”, the determination of the response of the structure to this load and an optimum arrangement of load-bearing systems including their details, which would assure the reliability of building structures in the broadest meaning of the term. Analyses of earthquake consequences (see e.g. [2]) and research results make it possible to improve to the quality of the overall understanding of these problems. The new concept forms the object of detailed systematic studies and the basis for important improvements of standards – see [3]. In the design and assessment of steel structures exposed to earthquakes, stress is laid on the optimum selection of details enabling effective energy dissipation. With regard to the relatively very small number of cycles and the character of failure, these problems do not belong to the framework of high- or low-cycle fatigue; it is advisable to consider them as a specific category for which an adequate name (such as “high-frequency fatigue” of “quasi-strength assessment”, etc.) is yet to be found.

So far the components of steel structures exposed to alternative plastification have been assessed with considerable simplification, mostly on the basis of experimental results with the “prescribed” application of the partial coefficients method. It is obvious that in this field the way to a truthful reliability assessment will be very difficult, due in particular to the great variability and character of input quantities, complex transformation models, evaluation of damage cumulation and unclear reference values.

Elastoplastic deformation of steel members, structures and systems depends on material characteristics, cross section geometry, temperature, etc. The number of serious factors also includes the velocity of response to load, which comes into consideration e.g. in the assessment of the response to earthquake – see Fig.1. The diagram shows the area roughly corresponding to the velocities of the response to earthquake load. It is obvious that energy dissipation will depend on the interaction of several random variable quantities that must be granted particular attention in the preparation of the probabilistic approach to reliability assessment. In this process experimental examination of both individual quantities and of their interaction, including damage accumulation and origin of failure under repeated elastoplastic loading, are a great help.

Energy dissipation and origin of failure of an alternately plasticized structural bar are related to the magnitude, shape and number of hysteresis loops – see Fig. 2. The loop area corresponds to the work performed during one cycle. Principal data on the shapes of hysteresis loops corresponding to the given history of earthquake effects can be obtained experimentally by laboratory tests of samples in which, naturally, it is necessary to assure that the steel of the tested sample and the velocity of load application correspond with the steel used for the manufacture of the assessed detail and the loading data.

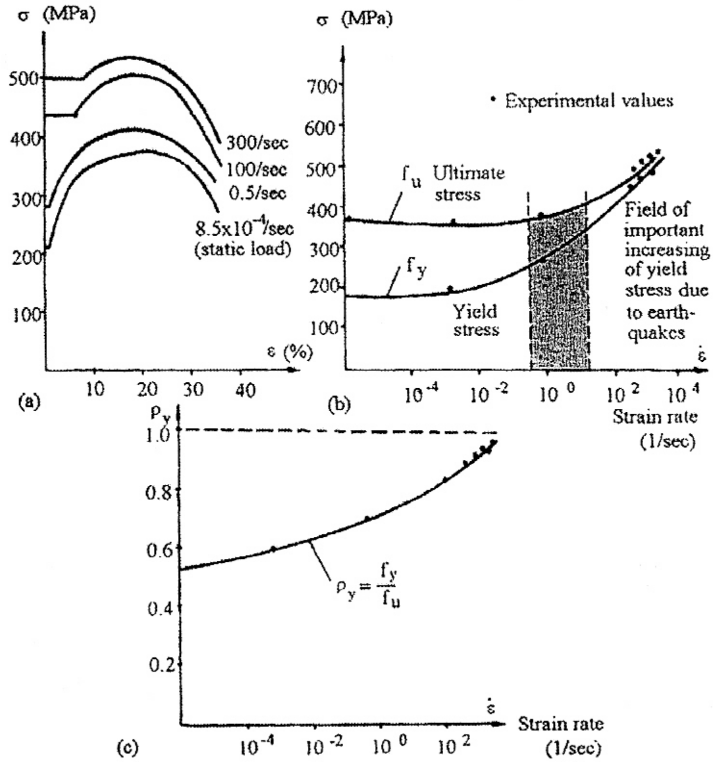


Figure 1: Influence of response velocity on the yield limit of steel.

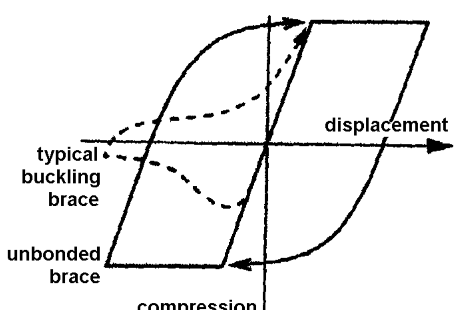


Figure 2: Hysteresis loop of a bar axially loaded alternately in tension and compression.

It is possible to consider five basic types of exploitation of steel structures:

a) material ductility, e.g. straight steel bar alternately loaded in tension and compression;
b) cross section ductility, e.g. the curvature of a steel beam loaded in flexure in the elastoplastic region;
c) steel beam deformation: under flexural load with reference to ductility according to the diagram partly in the elastic and partly in the elastoplastic region;
d) framework ductility – loading of the bars of the system by a combination of flexure, axial forces and shearing forces in the elastoplastic region;
e) ductility with reference to energy and hysteresis.

During experimental examination of a slender straight steel bar loaded alternately in tension and compression, it is possible to exploit the great scope of plastic strain. The number of principal problems in this particular case includes securing the slender bar against buckling during compressive loading in elastic and particularly in the plastic region, the consideration of the scope and velocity of plastic strain accordance with earthquake effects, the application of an adequate model of defect accumulation for definite failure – all that also with regard to the dispersal of random variable quantities forming the input quantities of reliability assessment according to the methodology of probabilistic methods.

The preparation of the monitoring of deformations and energy dissipation of a (simply supported and continuous) steel beam loaded in the elastoplastic region (including the consideration of the influence of residual stresses) can be found in extensive studies published as early as thirty years ago. Computer development should make it possible to supplement the results of the examined problems of elastoplastic deformation of the bar loaded in compression and flexure (including the consideration of the influence of residual stress) with the monitoring of the history of the elastoplastic response and energy dissipation in the framework of reliability assessment.

The conscious ductility exploration will be based doubtlessly primarily on the results of experimental research of structural details and members. Of particular importance will be the design of joints, connections, and similar details giving rise to the origin of a complex stress and strain state.

Physically significant for the given problem is the ability of the structure to transfer plastic deformations so as to dissipate the input energy by the hysteresis action of the material. To prevent failure the damage due to plastic deformation must be limited in every case. This requires chiefly the monitoring of the plasticity redistribution capacity of the structure in the framework of which the differentiation of structures into standard and regular damage distribution is applied, as a rule, but sometimes the actual damage distribution may be highly irregular due to random yield limit values in individual structural members even if the structure has been so designed on the basis of nominal yield limit. In this way the random character of the yield limit may be considered as an “irregularity” of a certain type.

As an aspect of growth of fatigue cracks in the assessment of the performance risks of the structure the quantification of failure probability as a function of time has been recommended. In this process both the material and the geometric variability of the strength of structural components and experimental samples exercise influence on the probabilistic distribution of time until failure occurrence. According to [4] e.g. the method using the deterministically defined crack growth trajectories for the a posteriori selection of probability distribution requires a vast number of failure data for the estimate of the parameters of the afore mentioned distribution. Therefore, recent probabilistic models reduce data dependence in two ways:

a) By the selection of a set of relevant distribution functions,
b) By the derivation of functional correspondence between probabilistic distribution and crack growth parameters.

The development has brought about the definition of the fatigue crack as the crack initiation and its growth to certain dimension, i.e. of the order of a millimetre in tough materials or failure of brittle material. The studies concentrated first on the proximity of threshold occurrence of fatigue, when the best part of service life is taken up by crack initiation. In this process the data dispersal of fatigue tests in numerical studies was approximated by exponential, normal, log-normal, Weilbull and Gumbal distributions (of extreme values in the last mentioned case).

As soon as the crack exceeds microscale or detectable macrocrack dimension ($a \geq 1$ mm), the prediction of the velocity of crack propagation and dispersal around its central direction become fundamental problems. Fundamental studies of probabilistic models enable the determination of the relations of crack growth with central tendency and probabilistic distribution around it. The methodology of linear or nonlinear fracture mechanics may be used subsequently for the derivation of residual strength of structural members during their service life. Using the strength and load changes derived from the aforementioned simulation studies it is subsequently possible to determine the reliability of the assessed member.

The results of experimental research and the calculations of the stress and strain state under different load conditions make it possible to estimate the bearing capacity of structural members. The basic data to be used in this process are the results of appropriate specimen tests under soft and hard load application as well as the date computed on the basis of theory of elasticity and plasticity and those obtained by experimental examination of stress and strain in elastic and elastoplastic region in structural members or their models under service loads. If no experimental data are available the bearing capacity calculations may use the curves of resistance to low-cycle failure derived by computation and approximate local maximum stress and strain values.

With the use of ductility the results of analysis of steel structures during alternate plastification may contribute significantly to the rigorous and effective assessment of their reliability.

The stochastic crack growth model makes use of the set of distribution functions as well as the function correspondence between the crack growth parameters and the probabilistic distributions.

It will be imperative to intensify and accelerate the progress towards the reliability analysis of factory-made members of steel structures to derive a more detailed and general results of the analysis of the key problem of the strength drop with time.

To develop the problems of major plastic deformations and the improvement of their application it will be desirable to enrich the probabilistic methodology in accordance with the development of stochastic theory. This applies particularly to the further development of the evolution damage prediction of discrete and continuous elastoplastic structures loaded by random pulses. In this connection further thematic points appear desirable, namely optimization of service reliability, procedures of analysis of the response of structures, advanced simulations of reliability systems, nonlinearity and random fluctuation.

2 Design of joints

Steel structures possess excellent performance as regards strength and ductility. In particular, the mechanical characteristics of materials and constructional elements influence the implementation the design requirements set by seismic codes. Dissipative structures are designed by permitting the yielding of some zones of their members. In the course of a catastrophic earthquake these regions have to dissipate the input energy by dint of hysteretic ductile characteristics in the plastic range. The forming of dissipative mechanisms is in relation to the segmentation of structures. Moment – resisting frames have a large number of dissipative zones placed near by the beams – to – column connections that dissipate energy by means of cyclic bending behaviour. Consequently, the ductility of seismic resistant steel frames is utterly affected by the characteristics of their joints.

The first approach of beam-to column joints is based, in compliance with [6], on the position of the dissipative zones at the beam ends, and so the earthquake input energy is dissipated through the cyclic plastic bending of the ends concerned. The second concept is dependent on the energy dissipation by dint of the cyclic plastic bending of the joints. It is obvious that, in the first case, connections should have sufficient strength so that plastic hinges can be shaped at the beam ends rendering to the frame. In the second case, the parameters of cardinal importance of the joint characteristics are their ductility and energy dissipation capacity under cyclic loading.

Although participation by the connections in the seismic energy dissipation, according to the second approach, is not forbidden, it is strongly limited in everyday practice, because experimental control of the effectiveness of such connections under cyclic loading is required.

In the first design approach, the connection parameters affecting the global characteristics of the frame are the rotational stiffness and the bending resistance. In the other approach, two additional parameters – both capacities must be contemplated: the joint rotation and its energy dissipation.

3 Prediction of cyclic effects

Simulating cyclic characteristics of the connection generalized typological models that are more extensively applied in seismic – resistant steel structures has a primary meaning, particularly for low-cycle fatigue phenomena.

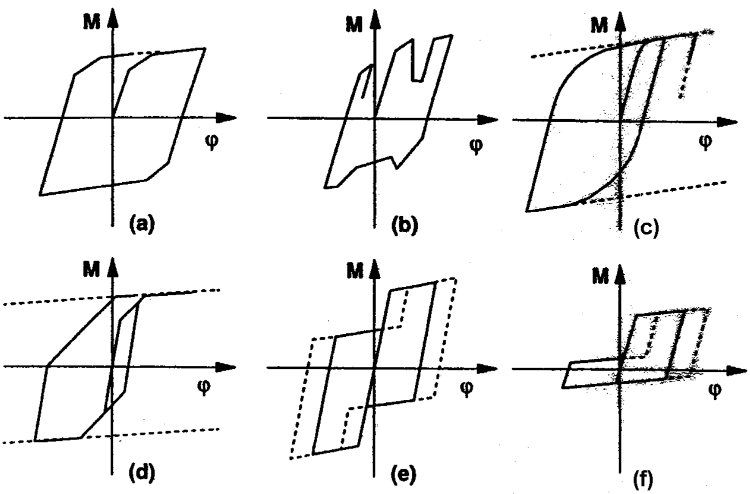


Figure 3: Models for cyclic behaviour of beam-to-column connections (according to [6]).

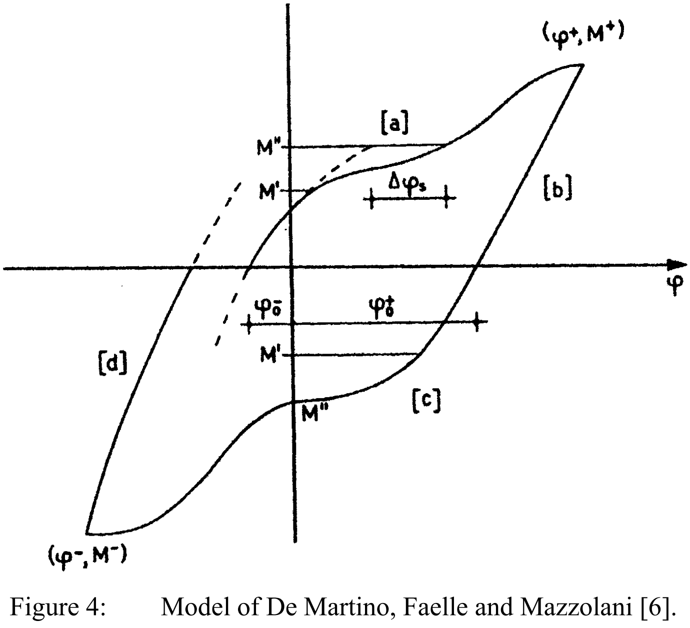
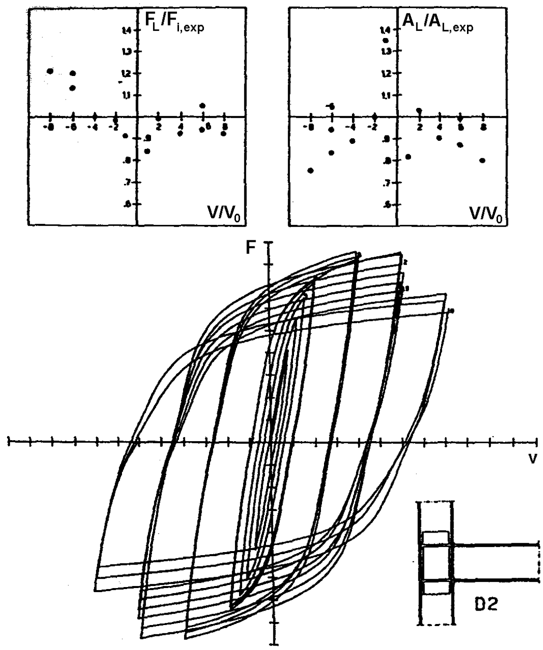
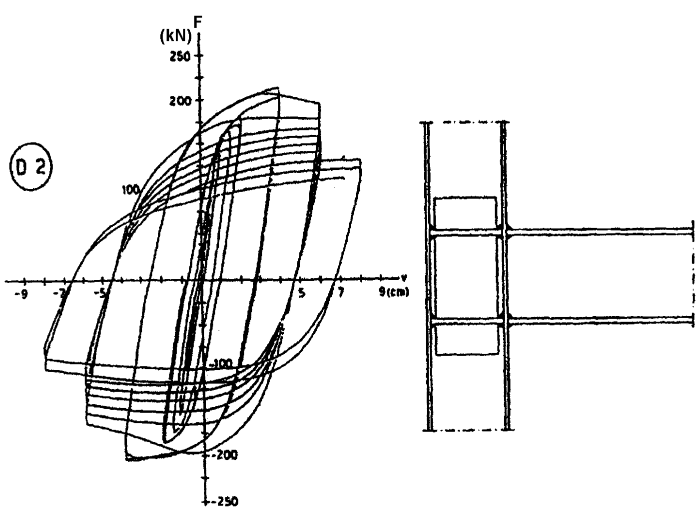


Figure 4: Model of De Martino, Faelle and Mazzolani [6].



(a)



(b)

Figure 5: Comparison between mathematical and experimental results [6].

The proposed models are described by varying degrees of accuracy (fig. 3). Some of them are multilinear (fig. 3 (a), (b), (d), (f)) and further suppose a continuous and stable behaviour that simulates the joint (fig. 3(c)). Others simplify even complex behaviour, with possible discontinuity owing to the bolthole clearance (fig. 3(b)) and non-symmetry of the expression in the tensile and compressive zone (fig. 3(f)).

The model according to fig. 4 permits representation of the load history for non-specific connection through a moment-rotation relationship, which may be divided into four branches:

a. load increasing from O to M^+
b. load decreasing from M^+ to O
c. load increasing in the negative direction from O to M^-
d. load decreasing from M^- to O

The comparison between the results of tests and the numerical model can be carried out on the basis of the main behavioural parameters. An example is given in fig. 5 that refers to the joint type D2.

The simulated hysteresis curve is plotted in the lower part of the figure. The upper diagrams give the ratios between simulated and experimental values for the strength F and the area A of a half-cycle at each value of the imposed displacement υ. The corresponding scatters are usually less than 15%.

Mathematical models are, in principle, able to closely fit any shape of M-φ curve, but they suffer from the disadvantage that they cannot be extended outside the range of calibration data.

The main problem in predicting cyclic moment-rotation behaviour is represented by the anisotropy of the angle slice behaviour. In fact, the gap between the outstanding leg and the column flange alternately opens and closes giving rise to significant boundary non-linearity. This behavioural feature has been accounted for by introducing gap elements, which are activated when the gap between the outstanding leg and the column flange closes (fig. 6).

In [7], the kinematic hardening model was employed for the cyclic plasticity of the material.

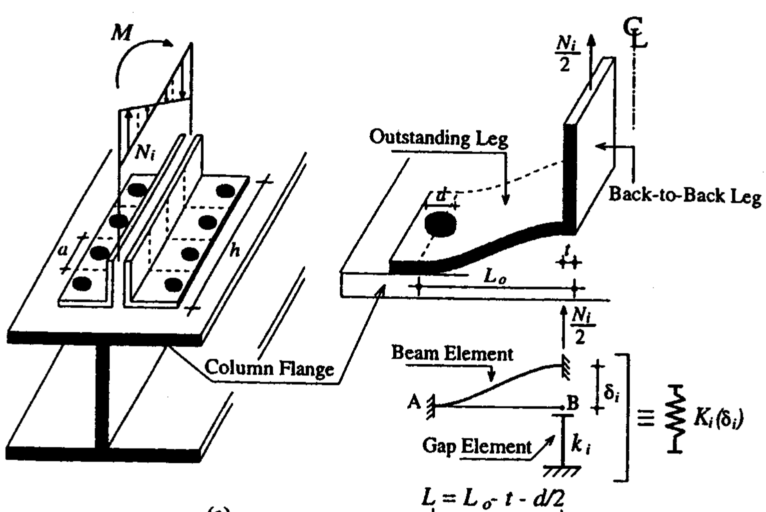


Figure 6: Modelling angle behaviour [6].

4 Cyclic response simulation of double-angle joints

The simulation introduced generates the cyclic moment-rotation curve of a double-angle joint by interlock of the load-deformation response in pure tension and compression of double-angle sections. The connection is simplified in the capacity of two rigid rods linked together by an assembly of nonlinear springs being an example of the axial response of the double angle segments. Especially, the rigid rods AB and CD (fig. 7) stand for the column and the web of the beam.

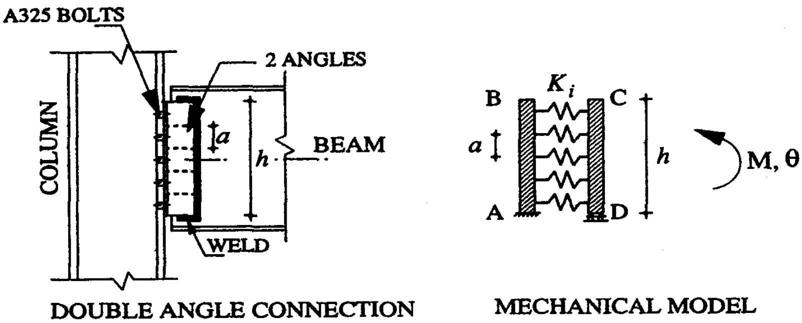


Figure 7: Model of joint tested [7].

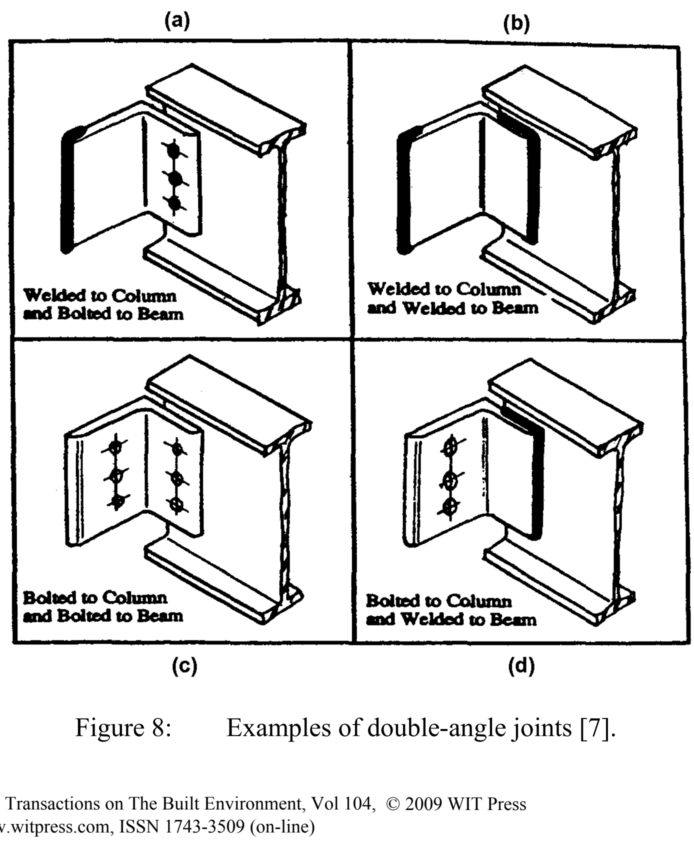


Figure 8: Examples of double-angle joints [7].

The column and the web of the girder are supposed to be undeformable in relation to double angles.

The anisotropy of the characteristics of the double-angle sections when subject to tension and compression makes up the key issue when predicting the cyclic moment-rotation behaviour. Actually, the gap between the obtrusive leg and the column flange that opens and closes, by turns, at the top and bottom of the angle, creates substantial boundary nonlinearity. This attribute is justified by introducing space elements that are activated if the gap between outstanding leg and column flange closes. The discussion is concentrated on the characteristics of bolted-welded and bolted-bolted joints of figs 8(c) and 8(d). In the present case, double angle segments are defined by cutting the web cleats mid-distance between the two adjacent bolts joining the angle to the column. On top of that, owing to the symmetry, it is supposed that both angle sections conduct themselves in a like manner. And so, the analysis of the response of one angle section solely is adequate to limit the total axial characteristics. It is possible to put on the development of the model in the following steps: (i) studying in the axial pull-push behaviour of an angle segment; and (ii) joining together the axial response of the angle sections to get moment-rotation response of the rigid rod CD that acts for the joint response.

5 Conclusion

Research results of the problems of reliability of structures exposed to earthquake have proved that many of the hitherto recommended rules and conventional principles of the design of details of steel structures are not satisfactory and, therefore, make it necessary to reassess the substance of their function with regard to energy dissipation and their incorporation in the load-bearing system. Attention must be afforded especially to the problems of the damping of dynamic response of steel structures during earthquake and the reliability of structural members exposed to repeated elestoplastic deformations. It will be also necessary to supplement the failure criterion (see e.g. [5]) with the criterion of work accumulation, expressed informatively by the number and area of hysteresis loops.

Lately nonlinear dynamic analysis of structures is gently moving from research public to design offices. One of the most important issues in nonlinear dynamic analysis is the accuracy of the computational model used to represent nonlinear response of components of the structure. However, there has to be a balance between accuracy and simplicity of a model. Often, very accurate models are too complex or time consuming to be used in most design offices owing to the large number and variations of components of the structure. The model proposed in this paper is sufficiently powerful to be used by structural engineers as well as researchers and provides enough accuracy for the design applications. The natural use of the simulation by designers may be in investigating the response of current construction to earthquakes.

The model given appears to express the leading physical effects studied in experiments, without bringing in any empirical curve-fitting parameter.

The model can be employed in predicting cyclic response to other types of joints, such as top-and seat angle connections, tea shear and end-plate joints. A good correlation is gained as regards initial stiffness, strength, ductility, and capacity for energy dissipation comparing the simulation predictions with the test outcomes.

In the Czech Republic, steel structures are relatively little endangered by earthquakes. Therefore, the assessment of their reliability with particular reference to this criterion is not in the centre attention during their design, as a rule. Serious achievements in the field of design and reliability of parts exposed to alternate plastifications, however, need not be applied with reference to earthquake only. They may be applied also in other situations when energy dissipation and conscious use of knowledge of high-frequency fatigue or quasi-strength assessment may contribute significantly to an effective assessment of reliability of steel load-bearing structure and so to the prevention of its defects.

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