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# STRESZCZENIA

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## STRESS AND STRAIN CONCENTRATIONS IN STEEL ANGLE TENSION MEMBERS CONNECTED BY ONE LEG

The paper presents the numerical simulations results of net section failure in tensioned angles. Angles are made of structural steel with nominal grade S235. Simulation takes into account ductile fracture initiation, by application of Gurson-Tvergaard-Needleman (GTN) material model. Parametrical analysis of ultimate resistance was carried out. The finite elements analyses were conducted by ABAQUS computer program. Shear lag effect in considered joint was observed, as a non uniform tensile stress distribution in angles in the vicinity of a connection. Stress concentration areas and stress concentration factors have been predicted, both in elastic and ultimate behaviour of joint. Especially change of non-uniform stress distribution in net cross-section was observed, during increase of loading, until the ultimate resistance was reached.

**Keywords:** lap bolted connections, shear lag effect, net section fracture, numerical simulations, stress and strain concentration

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## RESISTANCE OF STEEL FASTENERS SUBJECTED TO SHEAR AT PUBLIC ARENAS IN NORMAL AND FIRE TEMPERATURES – PROBABILISTIC APPROACH

The buildings with great grandstands are the public places where consequences of failure are very high. For this reason, according to EN 1990 they belong to CC3 class consequence of failure. The reliability class RC3 is associated with the consequences class CC3 [7, 8] and is defined by the $\beta = 4.3$ reliability index with probability of failure $p_f \approx 8.54 \cdot 10^{-6}$. Shear connections have to transfer forces between structural members – steel body and bolts with adequate degree of safety. The load-carrying mechanism of bolted shear connections is complex and analytical methods for predicting the shear resistance are not applicable. Instead the resistance of the connections may be determined using empirical formulas. The distributions of horizontal and shear resistance within steel body – bolts will be described depending on material characteristics of steel body and bolts components. The characteristic resistance of steel shear connection is obtained as minimum of two variables: bolds resistance and steel body resistance. Probability function of this minima will be defined and described in this paper. Laboratory tests provide the only practicable basis for specifying safety margins for ultimate strength connections. The determination of partial safety factors within shear connections will be presented according to EN1990.

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Design value of such resistance is specified as suitable fractile of log- normal probability distribution, calculated with the assumption that the acceptable probability of down-crossing is not greater than $p_{f,ult} \approx 2.91 \cdot 10^{-4}$. It means that the target reliability index, defined for the resistance, is taken as $\beta_{R,req} = 3.44$, in accordance with the European recommendations (EN 1990***)***.

**Keywords:** steel structures, structural safety, steel bolt connections, fire safety

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# TEMPERATURE DISTRIBUTION IN A STEEL BEAM-TO-COLUMN JOINT WHEN EXPOSED TO FIRE. PART 1: END-PLATE JOINT

Temperature distribution usually observed in steel beam-to-column end-plate joint after 15 minutes of its standard fire exposure is presented and discussed in detail. Two types of joints are analysed for comparative purposes. The first one is a pure steel connection while the other is covered by a reinforced concrete slab. Numerical simulation of the considered joint heating scenario was performed using the 3D model in the ANSYS environment. Some results were additionally verified by simpler calculations carried out on 2D models using the SAFIR computer program. It is emphasized that due to the local accumulation of many massive steel plates the representative temperature values identified in particular joint components are significantly lower than those which at the same time are measured in beam and column outside the connection. This means that the classic assumption of even temperature over the entire length of all the structural elements of a frame load-bearing structure exposed to fire at any time during such fire, without distinguishing in the formal model any cooler nodal elements, is always safe but very conservative. In addition, as the fire develops the differentiation between the temperature values relating to the beam web and to the beam flanges becomes more visible. This effect is particularly significant in the presence of a massive floor slab adjacent the upper flange of a frame I-beam which effectively cools it.

**Keywords:** beam-to-column steel end-plate joint, fire, temperature distribution, joint components, numerical simulation

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# TECHNOLOGICAL ASPECTS OF EXECUTION OF WELDED JOINTS IN HOLLOW SECTIONS

Steel structures designed according to Eurocode 3 are executed in accordance with the provisions of the standard PN-EN 1090-2, which is referred to in Eurocode 3. In addition, the standard PN-EN 1090-2 refers to in its content a number of welding standards, e.g. PN-EN ISO 9692-1. These standards provide guidelines for welded connections, which should be applied in the case of joints connecting steel hollow sections. Analysis of above-mentioned provisions revealed that for fillet welds they are simultaneously fulfilled only if the inclination angles of the elements are in the range of 70°–100°. According to recommendations of PN-EN 1993-1-8 and EN 1090-2, the same weld type around the perimeter of the element connected to the chord of lattice structure is possible to execute only for inclination angle lower than 60°. Discrepancies between these standards also exist with regard to the interpretation of the dimension of the flare groove welds in connections of rectangular hollow sections with the same width. In addition, analyses of the recommendations for welding in cold-formed zones indicate that, for steel grades currently used for the production of cold-formed rectangular hollow sections, welding in these zones is not permitted only for profiles with wall thickness equal to 12.5 and 16 mm. The above-mentioned issues point out the need for mutual unification of standards for the design and execution of steel joints in hollow sections.

**Keywords:** steel structures, lattice structures, welded joints, hollow sections

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# TELESCOPIC JOINTS IN STEEL TUBE TOWERS

Modeling problems of new generation of steel shell towers supporting overhead power transmission lines and having telescopic joints are presented in this paper. The towers of such structure, designed according to the European standard EN 50341 [8], ensure high reliability even when subjected to high technological and climate loads. In this paper elements of differentiated reliability requirements are verified, and special attention is paid to the values of variable loads coefficients for different reliability classes of the considered structure. Using computer modeling tools and linear, elastic shell theory, the modeling error of telescopic joints in a sample tower supporting overhead power transmission line rated at 110 kV is estimated.

**Keywords:** columns, shells, power lines, bearing capacity, reliability, telescopic joint

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# DAMAGE ANALYSIS OF THE BLADE TO THE ROTOR HUB CONNECTION IN THE WIND TURBINE

The paper analyzes possible causes of bolts fracture in the connection of the blade to the hub of the wind turbine rotor. This failure has been growth shortly after the wind turbine was started, a few days after the storm. During the storm, electric power was turned off in the whole surrounding area for many hours, therefore it was impossible to control the device. After an initial analysis of failure modes in other wind turbines, it was found, that the bolts failure in the analysed case may be regarded as quite rare. Due to the fact that a significant part of bolts was broken brittle (flat fracture surfaces and lack of permanent elongation), it was decided to analyse the structure of the material in the direction of stress corrosion and hydrogen embrittlement. Among others, microscope analysis of fracture surfaces after their proper cleaning from corrosion products was carried out. The broken bolts were made as undersized, what is mainly used in fasteners intended for hot-dip galvanizing. On the basis of the purchase documentation analysis, it was presumed that the bolts were not galvanized by their manufacturer, which is obliged to perform the routine inspection in production process preventing the introduction of the hydrogenated bolts into the market. Besides the bolts examination, the scenarios of connection loads in extreme situations, in particular the impact of wind with high energy in the most unfavourable direction relative to the blade and nacelle, as well as the force resulting from the impact of the rotating blade on the tower that preceded the collapse of the whole structure, were considered. Conclusions from the performed tests and calculations by use of two calculation models of the connection were presented.

**Keywords:** undersized bolts, mechanical properties of bolts, wind turbine, blade failure, model of joint

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# EXECUTION AND INSPECTION OF STEEL HOLLOW SECTIONS WELDED JOINTS

The hollow section welded joints require a number of actions before starting welding and appropriate supervision during this operation to achieve joints with adequate quality level, which should be confirmed by the post-completion tests. The execution of hollow section joints is associated not only with welding, but also with cutting and additional machining of edges. In some cases, weld surfacing is also applied to correct sections fit-up. Weld surfacing and thermal cutting can cause local hardening of connected elements. The welding has to be preceded by an assessment of the previous technological processes. The welded joints can be only executed on the basis of detailed Welding Procedure Specifications (WPS). It is advisable to manufacture – in accordance with previously prepared WPS – pre-production joints for testing, proving the ability of the welding personnel to execute welded joints with specified quality, using the available equipment. The quality of welded joints is proved by testing.

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A type of conducted tests, thus scope and type of welding defects possible to detect, depends on the weld type, wall thickness of connected elements and joint geometry. The authors' experience indicates that the proper execution of the welded joint of hollow sections is difficult task, which often requires pre-production quality testing of the joints.

**Keywords:** steel structures, lattice structures, hollow sections, welded joints, welds

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# FIRE RESISTANCE OF TIMBER JOINTS WITH STEEL FASTENERS

Fire safety is a major concern in the design of timber construction. Wood is combustible material. The thermal response of timber connections is usually the main factor in evaluating the overall load-bearing capacity of wood structures exposed to fire. The analysis of timber joints under fire conditions is difficult and complex. Finite element model is developed to predict the thermal behavior of bolted wood-to-wood joints exposed to fire. In fire, the material characteristic depend on the temperature. The thermal model is continuous, taking into account the thermal continuity between the joint components. Also, the thermal model is used to predict the evolution of the temperature field inside the connection.
The paper presents a summary of results from a numerical studies of the fire behavior of wood-to-wood timber connections with steel bolt. As a result of computer simulations the temperature distribution was obtained. During fire exposure, the timber section is reduced and steel bolt reduces strength. Load-carrying capacity per shear plane in fire conditions was calculated using two methods: design methods according to EN 1995-1-1 [5] and reduced load method according to EN 1995-1-2 [6]. In the first approach, the timber section loss and steel strength reduction during the fire were taken into account.

**Keywords:** thermal conductivity, fire safety, connections, elevated temperatures

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# CRITICAL TEMPERATURE EVALUATION FOR A STEEL FRAME WITH JOINT STIFFNESS DECREASING IN FIRE

A procedure to determine the critical temperature of a selected steel frame bearing structure is presented and discussed. This temperature, in case of fully developed fire, when the temperature of the exhaust gasses enveloping the structural members is equalized within the whole fire zone, may be considered as an impartial measure of safety. The obtained result does not depend on the heating progress but only on the static scheme and the load level in the considered structure. The quantitative and qualitative evaluation of the influence the joint stiffness decreasing in fire exerts on the resultant critical temperature constitutes the basic objective of the authors. It has been shown, that proceeding according to the recommended computational procedure does not necessarily result in an estimate fully unambiguous in interpretation. The critical temperature specified in a global mode, for the whole considered frame, is usually associated with a specific component of such frame, interpreted as the so called "weakest link". Thus local loss of bearing capacity in such element is in this approach equivalent to the total destruction of the whole bearing structure. Indication, which of the components present in the considered frame should be treated as the critical one, because of its behaviour under fire conditions, seems to be a key to the forecast safety level warranted to the users of the structure. The authors show, that this association changes depending on the selected computational method, and this in turn substantially limits the reliability of the obtained estimate.

**Keywords:** steel bearing frame, fire, critical temperature, joint stiffness, moment – rotation relationship, weakest link

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# TEMPERATURE DISTRIBUTION IN A STEEL BEAM-TO-COLUMN JOINT WHEN EXPOSED TO FIRE. PART 2: FLANGE-PLATES AND WEB-CLEATS JOINT

In the second part of this paper the temperature distribution is analysed for a thermally uninsulated steel beam-to-column flange-plates and web-cleats joint after 15 minutes of its exposure to a fully developed fire. Two types of such a joint are considered separately, firstly the pure steel connection with a beam and a column evenly heated on all four sides and then the analogous one, but with a massive reinforced concrete floor slab lying on the upper beam flange. In the latter case the joint beam is heated only on three sides. In addition, in each of the analysed joint the beams of two sizes are analysed independently for comparative

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purposes. Those that are made of the bigger I-section have a more slender web, while the smaller ones are more stocky. However, the smaller I-section heats faster than the bigger one because the section factor calculated for it has a greater value. In general, it can be concluded that in all the joints considered by the authors the steel temperature turned out to be much lower than that measured outside these joints. Moreover, a significant difference is observed in the temperature values identified in the beam web and in the beam flanges. Finally, the temperature distribution obtained from a numerical simulation and identified in the selected cross-sections of the joint beam in the case of a joint with adjacent floor slab is referred to the analogous distribution recommended for use in such circumstances in the standard EN 1993-1-2.

**Keywords:** beam-to-column joint, flange-plates joint, web-cleats joint, temperature distribution, section factor, numerical simulation

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# DESIGN RESISTANCE OF WELDED KNEES IN STEEL FRAMES

The paper presents issues related to calculations of welded knee joints, in which the interconnected load-bearing elements, beams and columns, can be made from plate girders with slender webs.
At the beginning, a typical knee joint of portal steel frame was characterized, along with presentation of calculations for the internal forces in characteristic zones of the knee, i.e. in tension, shear, and compression zone. Then, the checking procedures of resistance were presented in detail for each designated knee zone, paying particular attention to the influence of complex stresses state and loss of stability in the shear and the compression part of knee joint.
The work also presents a comprehensive calculation example, which illustrates the described method usage in practical design of welded knees of steel frames.

**Keywords:** steel frame, knee joint, transverse stiffeners, diagonal stiffeners, resistance

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# SHAPING OF ARCHITECTURALLY EXPOSED STEEL STRUCTURES

Architecturally Exposed Structural Steel (AESS) is steel that must be designed to be structurally sufficient to support the primary needs of the structure and – at the same time – remains exposed to view, being a significant part of architectural language of the building [4, 6]. The quality requirements of AESS typically exceeds the requirements of Standard Structural Steel (SSS), what increases the time and costs of the design and execution of AESS. Currently used classification of AESS distinguishes 5 categories of execution quality. This categorization has a hierarchical structure, each higher category of structure execution contains all the properties of lower category. The basis of presented classification is the degree of human visual perception of the structure. It is mainly related to the distance of the potential observer from the structure, which allows in varying degrees to see the details of structure execution. Joints and connections are the main means of architectonic expression in architecturally exposed steel structures. The principles of joints and connections shaping in AESS are the same as for SSS but additionally some requirements to the expected aesthetic are formulated.

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This additional requirements cause that AESS can be significantly (even a few hundred percent) more expensive than SSS with exactly the same functionality and durability. However, PN-EN 1090-2 [7] gives no provisions about AESS executions, which may impede mutual understanding between architect, structural engineer, contractor and investor.

**Keywords:** exposition of steel structure, classification of AESS, higher quality requirements, higher costs of AESS

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# INFLUENCE OF THE END-PLATE THICKNESS ON THE STEEL BEAM-TO-COLUMN JOINT STIFFNESS WHEN SUBJECT TO BENDING

Based on the numerical simulation performed within the Abaqus computational environment for a typical end-plate beam-to-column joint the influence of the end-plate thickness on the effective joint rigidity has been verified. The initial joint rigidity at first determined for 20 mm thick end-plate has been compared with rigidity of the joint constructed with substantially more flexible end-plates 10, 8 and 6 mm thick. In all the considered cases the column was equipped with horizontal ribs stiffening the web at the height of beam top and bottom flange. No diagonal ribs were applied. In addition the column flange at the zone directly adjacent to the beam end-plate in all the analyzed cases has been set to 30 mm. This way it did not affect the computationally determined rigidity of considered joints. Juxtaposition of $M$-$\varphi$ curves characterizing the considered joints and depicting the relationship between the applied bending moment and relative change of the initial angle between undeformed axes of beam and column in the analyzed frame indicates qualitatively different modes of destruction of the considered joints, and thus different computational models determining their bearing capacity. In the first case obtained parameters seem to indicate that the joint is nominally rigid but in all the remaining cases the bearing capacity seems to be exhausted by the increasing deformation of the more and more flexible end-plate.

**Keywords:** beam-to-column steel end-plate joint, end-plate thickness, joint flexibility, initial stiffness, numerical simulation

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# FATIGUE TESTS OF WELDED JOINTS IN STEEL ORTHOTROPIC BRIDGE DECK

The procedure supported by testing has been used for the fatigue assessment of the existing steel bridge with the orthotropic deck, built in the early 80's of the XX century. The main goal was to check if the remaining fatigue life of the existing steel deck is at least 25 years, without the need of extensive repair or strengthening. The assessment comprised the fatigue calculation according to European codes with the use of updated values for material resistance. In order to obtain the updated information on material resistance the fatigue testing was carried out and the actual fatigue resistance $\Delta\sigma_C$ was applied in damage accumulation calculation. The fatigue tests were carried out for two most critical deck details. Test specimens were cut out of the existing deck at the locations where the preliminary analysis showed the possible highest stress ranges. The assessment

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based on the actual material resistance revealed that the bridge deck had got very long service life. The main results of the fatigue assessment of orthotropic steel deck supported by testing have been presented in the paper.

**Keywords:** welded connections, orthotropic deck, fatigue testing, service life

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# RESISTANCE OF THE WELDS IN CHS JOINTS WITH THE RIB PLATES

With regard to the calculation of welds' resistance in connections between hollow sections in EN 1993-1-8, very general information is given without any indication of specific calculation procedures. These recommendations are basically as follows: the resistance of the welds must have the value of the cross-section and assessment of the welds' resistance based on the effective lengths is allowed in cases when forces in the braces are smaller than the resistance of the joint, but the detailed method is not specified. The objective of this paper is to present the most up-to-date information for the design of welds for overlap joints with reinforcing rib plates. The article presents the FEM analysis of the welds in the intermediate joint with the rib in a truss made of circular hollow sections. The conclusions from the analysis were presented.

**Keywords:** trusses, hollow sections, K joint with the ribs, welds, FEM analysis

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# RESISTANCE OF TENSION BRACE IN PLUG & PLAY N SHAPE RHS TRUSS CONNECTION

The paper presents concept of RHS truss with N-shape connections in the plug & play form, made in non-welded technology. The procedure of calculating the resistance of tension brace truss connection was presented, using the component method. The results of obtained resistance of joint was compared with the tension bracing RHS strut resistance and the usefulness of the concept considered was determined.

**Keywords:** steel truss N shape RHS joints, non-welded plug &play connections

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# ON THE DESIGN OF A STEEL END-PLATE BEAM-TO-COLUMN BOLTED JOINT ACCORDING TO PN-EN 1993-1-8

Considering joints with unstiffened columns, the load capacity of an inner bolt-row being a part of bolts group defined by a flange capacity is directly proportional to a distance between bolts. In turn, a flexibility of the column flange in the inner bolt-row area depends not only on that distance but also on a flexibility of other basic joint components. Hence, that situation may occur, when internal forces in inner bolt-row will be greater than its capacity estimated as an equivalent of T-stub. This possibility has been taken into account in the standard [3] – see the rule in the point 6.2.4.2 (3). In practice, this rule is not implemented in calculations of this kind of joints. In this work, a simplified algorithm of these joints calculation as well as an example, where the need for force reduction in the inner bolt-row to the value of bolt resistance has occurred, were presented. Moreover, the influence of the aforementioned reduction on the joint stiffness was estimated.

**Keywords:** component method, equivalent T-stub, joint capacity and stiffness

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# APPROXIMATED METHOD FOR DETERMINING MOMENT RESISTANCE AND STIFFNESS OF BOLTED BEAM TO COLUMN JOINTS MADE WITH ANGLE WEB AND FLANGE CLEATS

Bolted beam to column joints with angle cleats are often used in braced and unbraced steel frame structures. It is related to their simple technology which does not require expensive welding process. Works on the estimation of moment resistance and stiffness of such connections were already carried out in the 1930s. However, the lack of appropriate computational tools forced researchers to introduce simplifications and some assumptions in determining the strength of a joint in a complex load condition. Currently, computing techniques allow for taking into account the actual resistance and stiffness of connections at the stage of global static analysis of the structure. Advanced computer methods as well as applied analytical models allow for a fairly precise determination of the parameters of this type of joints. However, these methods, due to their complexity, are quite time-consuming and labor-intensive, and they are suitable for verification of resistance of the connection or analysis of the structure in the final design phase. The paper presents simplified formulas for calculating the moment resistance and rotational stiffness of the beams to column joints with the use of angle sections connecting both the flanges and the web of the beam. An outline of the component method for determining the moment resistance and stiffness of such connections is also presented. The analysis of the influence of individual components of the joint on its global resistance and stiffness was conducted. The presented formulas, developed on the basis of the component method, preferred by Eurocode 3, can be used in the preliminary determination of the characteristics of the joints, used in the global structure analysis.

**Keywords:** bolted joint with angle web and flange cleats, moment resistance, stiffness, joint characteristics, simplified formula, component method

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# CREDIBILITY OF FEM ANALYSIS IN THE T-STUB MODELLING

The paper presents the results of a comparative analysis between numerical calculations of T-stubs of the 3rd stage of FEM models hierarchical validation and the results of laboratory tests. The procedure for the development of the material characteristics used in numerical calculations of FEM models is presented. The scope of this article allows determining the non-linear characteristics of the T-stub which maps the work of the end-plate joint of the beam to the column in the tensile zone. The results of laboratory tests of a series of T-stubs made of rolled profiles (HEB240, HEA240) and of welded profiles (thickness of end-plate: $t_p$ = 12 mm and $t_p$ = 20 mm) have been presented. The principles of shaping the geometrical features of the FEM model of end-plate joints of the T-stub type are given, with particular emphasis on the shaping of the bolt with a thread. The impact of the bolt thread on the accuracy of the obtained results was assessed. The criterion of reliability of the obtained results with respect to the maximum force in the bolt obtained on the basis of laboratory tests in the axial tensile test of the bolts in the configuration: bolt - washer - nut was formulated.

**Keywords:** T-stub, rotation capacity, material ductility, multistage hierarchical validation, FEM modelling

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