Steel and Composite Structures, *Vol. 25, No. 1 (2017) 45-55*
DOI: https://doi.org/10.12989/scs.2017.25.1.045

Reliability analysis of the nonlinear behaviour of stainless steel cover-plate joints

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(*Received March 07, 2016, Revised May 30, 2017, Accepted June 08, 2017*)

Abstract. Stainless steel exhibits high ductility and strain hardening capacity in comparison with carbon steel widely used in constructions. To analyze the particular behaviour of stainless steel cover-plate joints, an experimental study was conducted. It showed large ductility and complex failure modes of the joints. A non-linear finite element model was developed to predict the main parameters influencing the behaviour of these joints. The results of this deterministic model allow us to built a meta-model by using the quadratic response surface method, in order to allow for efficient reliability analysis. This analysis is then applied to the assessment of design formulae in the currently used codes of practice. The reliability analysis has shown that the stainless steel joint design according to Eurocodes leads to much lower failure probabilities than the Eurocodes target reliability for carbon steel, which incites revising the resisting model evaluation and consequently reducing stainless steel joint costs. This approach can be used as a basis to evaluate a wide range of steel joints involving complex failure modes, particularly bearing failure.

Keywords: stainless steel joints; Eurocode 3; finite element modelling; reliability index; elastic-plastic analysis; bearing failure mode

1. Introduction

Stainless steel for structural applications in civil engineering is less common than other construction materials, but the number of applications is steadily increasing due to its numerous advantages in the framework of life cycle cost approaches (Gardner 2005). Particularly, the stainless steel shows high ductility and strain hardening allowing for significant energy dissipation under cyclic loading or shocks (Euroinox 2006). The large deformation of the stainless steel before failure allows the possibility of large load redistribution in structural joints and members (Rasmussen 2003). It also exhibits an excellent resistance to corrosion, a pleasant aspect and a better fire resistance (Gardner and Baddoo 2006). These characteristics make this material very interesting for structural applications. However, a particular attention must be given to the analysis of joints considering the stiffness, the strength and the deformation capacity (Bouchair *et al*. 2008, Salih *et al*. 2010, 2011).

In steel structures, joints constitute singular discontinuities as they ensure the transmission of forces and stresses between elements. For practical reasons, bolted connections are mainly used because they allow a quicker and easier building process on site. In steel structural joints, bolts can be loaded in tension, shear or a combination of both. The fracture patterns are variables. When a bolt is loaded in shear, the failure can occur either in the threaded part or in the smoothed part of the bolt shank. In tension, the fracture occurs in the threaded portion. Furthermore, in T-stub connections, it is necessary to take account of the amplifying effect of prying forces (Alkhatab and Bouchaïr 2007). In general, stainless steel bolts show a more ductile behaviour than carbon steel ones, although it remains less important than that of the structural stainless steel material constituting the attached plates. In the case of T-stubs, stainless steel show large advantages given by the strain hardening of the material instead of being limited by the conventional yield strength (Bouchaïr *et al*. 2008).

Among various types, bolted cover-plate connections are commonly used to transmit direct tension or compression forces between plates using shear in bolts. Cover-plates are also used in more complex joints such as beam-to-beam or beam-to-column where the bending moment is transmitted by different cover-plate joints in tension and compression (See Fig. 1(a)). The knowledge of cover-plate behaviour can thus be generalized to a wide variety of joint configurations supporting normal and shear forces, with or without bending moments. Even in usual simple configurations, the global load-displacement behaviour of cover-plate joints involves complex pattern of deformations in gross cross-section, net cross-section, bolts in shear and bearing zone under the bolts (see Fig. 1(b)).

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http://www.techno-press.org/?journal=scs&subpage=6
ISSN: 1229-9367 (Print), 1598-6233 (Online)

The equations used in Eurocode 3 (EC3) to check the resistance of steel joints regarding the bearing failure mode combines various failure modes (shear-out or tear-out and bearing). The approach developed in this paper can be generalized to other configurations of bolted connections with one or more bolts-rows using various materials with high or low ductility. The block-shear combining shear-out, bearing and tension can be covered by the approach developed. This approach can be applied by considering other analytical equations representing the complex behaviour called bearing failure such as AISC specification's equations and their adaptation (Lip and Mehmet 2015).

Besides, many available studies were developed to take into account the actual joint characteristics to be included in the global analysis of structures targeting a high level of reliability. Thus, the basic components of connections, such as cover-plates, can be used to identify a moment-rotation behaviour (resistance, stiffness and rotation capacity) for various configurations of connections (Kozlowski 2016). Bearing failure modes can also be observed in plate connections to circular hollow steel sections with more complex loading and behaviour (Hassan *et al*. 2015) or in thin steel plate shear walls attached to the boundary frame members (Vatansever and Berman 2015). The bearing failure modes combined to curling due to the thickness of the steel sheets need to be more analyzed to reach optimal structural performance of cold-formed steel connections to support the demand increase for cold-formed systems (Qin and Chen 2016).

The usual factors involved in the complex behaviour of bolted connections come from the material, the dimensions of the plate and the relative position of the hole. Though, for cases like joints composed of highly slender bolts or thin elements, out-of-plane displacements of the plates (Kim *et al*. 2008) and bending of the bolts are also to be considered in the models, at least at the ultimate state. Indeed, large bending of the bolts generates a high level of axial force that can mobilize a non-negligible second order tie effect. In this study bolts are considered with low slenderness and plates with sufficient thickness. Thus, the out-of-plane bending displacements of the plate (curling) and its effects can be neglected. However, the common approach, developed in this study, combining mechanical modelling and reliability analysis can be easily generalized to other configurations of joints, including thin plates, through performing adequate mechanical numerical model.

The present paper is focused on the reliability analysis of basic stainless steel cover-plate joints, in order to show the interest of this kind of material in this particular kind of joints by performing a methodology of assessing the actual design code provisions. This evaluation can be considered as an example to be generalized to various configurations of connections before obtaining practical design use of the results. The specificities of stainless steel material and the design analytical formulae used to check the ultimate resistance of stainless steel bolted cover-plate joints are introduced. The mechanical behaviour of this type of joints involve large nonlinear elastic-plastic strains. Therefore the numerical model developed by Bouchaïr *et al*. (2007) is presented and validated with experimental results. This model is then applied to define a response surface of the joint, according to different limit state criteria. Based on material and geometrical uncertainties, the reliability analysis is performed to evaluate the failure probability and the influence of the design variables on the joint carrying capacity. The numerical application is performed on a joint configuration involving complex nonlinear behaviour associated to bearing failure mode.

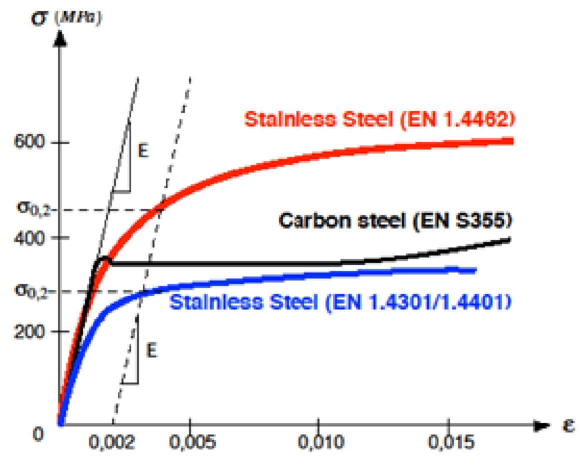
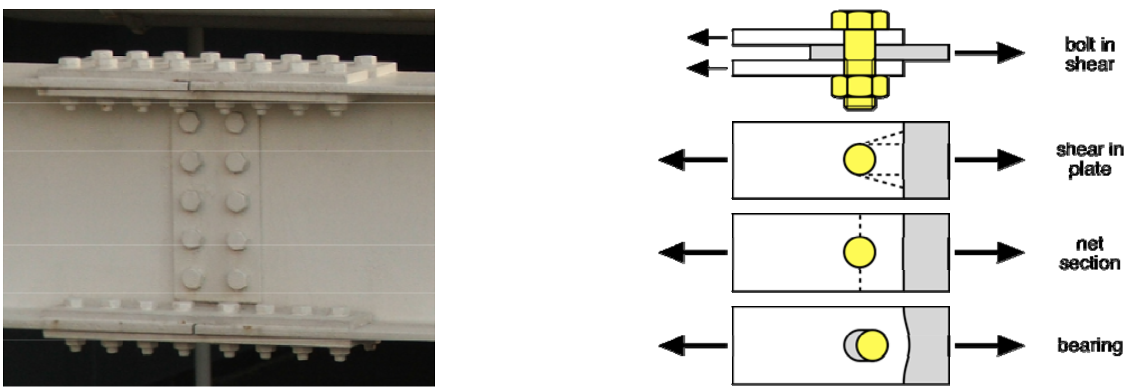


Fig. 2 Typical stress-strain curves of carbon and stainless steel (European grades)



(a) (b)

Fig. 1 Example of moment resisting cover-plate joint (a); and typical failure modes (b)

2. Stainless steel joint design

The behaviour of stainless steel is different from that of carbon steel as shown through the tensile curve (see Fig. 2). While carbon steel exhibits linear elastic behaviour followed by plastic deformation before strain-hardening, stainless steel does not show a well marked yield strength and it is nonlinear even for low load levels. As a consequence, conventional yield strength is usually defined at 0.2% of the plastic strain. Furthermore, the ultimate state is reached at a relatively high level of load and a large deformation in comparison with that reached at the yield strength. That means the existence of a large reserve of resistance and deformation capacity after the yield strengths. Stainless steel also exhibits a non-symmetrical behaviour in tension and compression (Euroinox 2006) and has a general trend to display anisotropic behaviour.

Several studies on the mechanical behaviour of stainless steel structural members (Burgan *et al.* 2000, Kouhi *et al.* 2000, Van Den Berg 2000) brought elements of comparison with EN 1993-1-4 requirements (EN1993-1-4 2006). For cover-plate joints, research works have been conducted for thin walled structures. The main concerns of these studies are either to propose an analytical model representing the structural behaviour, in order to evaluate the design rules (Fan *et al.* 1997), or to propose alternative design provisions (Kim *et al.* 2008, Moze *et al.* 2007).

For the moment, the design methods for stainless steel are often based on those for carbon steel with allowance made for the high ductility and deformability exhibited by this material (Bouchaïr *et al.* 2008). Actually, the use of stainless steel for structural applications is covered by European standard EN 1993-1-4 (2006). As thin elements are the main applications for stainless steel, the sections used in connections concern mainly cover-plate types. However, for beam-to-column joints, an approach based on the component method can be used with some adaptations.

The design of stainless steel bolted joints is usually given by straightforward application of carbon steel rules, covered by EN-1993-1-8 (2005). However, the design of carbon steel joints is evaluated at the Ultimate Limit State (ULS), where the deformation criterion at the Serviceability Limit State (SLS) is considered to be implicitly satisfied. This is justified by the linear behaviour of carbon steel before yielding and by the low ratio between ultimate and yield strengths of the basic material (generally between 1.1 and 1.5). In the case of stainless steel, the stress-strain curve is fully nonlinear and the ratio between the ultimate strength and the yield limit may exceed 2 particularly for austenitic stainless steel (Kim and Yura 1999). So, a design for the ULS may not guarantee that no excessive deformations will occur at the SLS, since the ratio between the ULS and the SLS loads is usually between 1.35 and 1.5. Another possible verification should concern the deformation of the hole in bearing, which is not easy to predict. EN 1993-1-4 (2006) considers that the ULS verification is sufficient if a reduced ultimate limit is used in bearing.

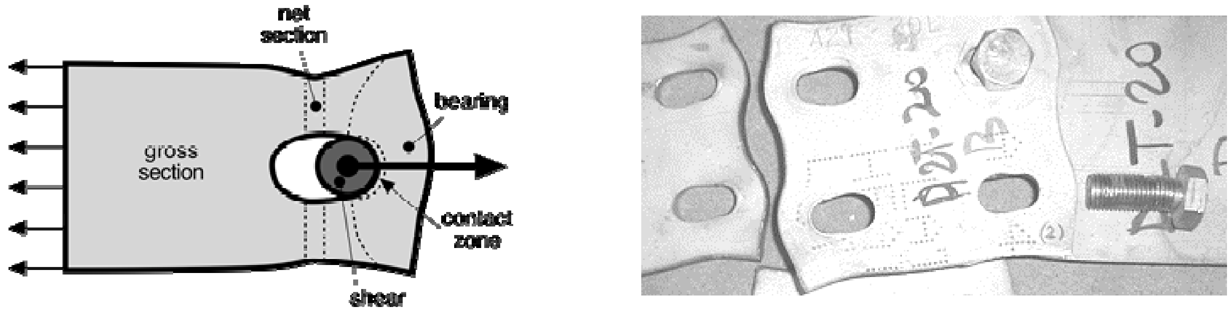
The design formulae of joints concern mainly the bolts in tension and shear, and the cover-plate strength (net section, gross section and bearing failure modes). For the moment resisting connections, the provisions of EN 1993-1-8 (2005) are applicable to the various components. In the following, the design formulae are described for various failure modes: bolt shear, net section, gross section and bearing. The partial safety factors to be used in design formulae are $\gamma_{M0} = \gamma_{M1} = 1.1$ and $\gamma_{M2} = 1.25$. The main failure modes are shown on Fig. 3.

2.1 Bolts resistance in shear (BS)

The shear strength of carbon steel bolts, for one shear plane, depends on the position of the shear plane with regard to the threading and the ductility of bolts. EN 1993-1-4 considers that stainless steel bolts behave like carbon steel ductile bolts of class 4.6, 5.6 or 8.8. Thus, the coefficient α_V used for shear is equal to 0.6 independently of the shear plane position (in the threaded part of the shank or not). The use of only one coefficient is realistic regarding the ductile character of stainless steel bolts and the experimental tensile test results (Bouchaïr *et al.* 2007). The design value of the shear resistance of bolts $F_{v,Rd}$ can be calculated according to EN 1993-1-8 (2005) using Eq. (1).

$$F_{v,Rd} = \alpha_V f_{ub} A(\text{or } A_S)/\gamma_{M2} \tag{1}$$

where f_{ub} is the ultimate tensile strength of the bolt, A (or As) is the gross cross-section area of the bolt (when the shear plane passes through unthreaded portion of the bolt); or the tensile stress area of the bolt (if the shear plane passes through the threaded portion of the bolt). In practice, the shear test in tension, which gives lower mean values of resistance, is more conservative and this is the most



(a) (b)

Fig. 3 Zones of deformation (a); joint with two bolts in shear showing large hole elongation after test (b) common configuration in practice. This trend is comparable to that of carbon steel bolts (Kulak *et al*. 2001).

2.2 Net cross-section resistance (*NS*)

The net cross-section resistance in a plate loaded in tension is given by Eq. (2).

$$N_{u,Rd} = k_r A_{net} f_u / \gamma_{M2} \tag{2}$$

where $k_r = 1 + 3r(d_0/u - 0.3) \leq 1$, r being the ratio of the number of bolts at the cross-section in tension to the total number of bolts in the joint ; $u = 2e_2$, with $u \leq p_2$; A_{net} is the net cross-section area; d_0 is the nominal diameter of the bolt hole; e_2 is the edge distance from the centre of the bolt hole to the adjacent edge (in the direction perpendicular to the direction of the load transfer); and p_2 is the spacing between the bolt holes (in the direction perpendicular to the direction of load transfer). The reduction factor k_r seems to be used to take into account the eccentricity of the load transferred to the holes by the bolts (in comparison with the plates with non-loaded holes).

2.3 Gross cross-section resistance (*GS*)

The plastic resistance limits the resistance of the gross cross-section A, which should be determined by using the yield strength of stainless steel f_y in Eq. (3).

$$N_{pl,Rd} = A f_y / \gamma_{M0} \tag{3}$$

This can be a limitation for stainless steel members in tension because the nominal value of the conventional yield stress is relatively low. The use of the real characteristics can be a more practical and efficient solution.

2.4 Bearing resistance (*Be*)

The bearing resistance is complex to describe because it includes the shear-out (or tear-out) and the local compression of the plate under the bolt hole. In addition, it can include the local compression of the bolt on its surface that has generally a higher resistance than the plate material. To check the resistance of the joint, the use of the ultimate capacity could not be sufficient (Bouchaïr *et al*. 2008). In fact, the bearing failure mode can also be governed, for stainless steel, by the need to limit the hole elongation under serviceability loads, which is not easy to determine. However, a reduced value $f_{u,red}$ given by the combination of the yield strength f_y and the ultimate tensile strength f_u can be used instead of the real ultimate tensile strength f_u as follows: $f_{u,red} = 0.5f_y + 0.6f_u$. Thus, it can be considered that the hole elongation is limited and the requirement of limited deformation may be avoided at the ultimate limit state. In EN 1993-1-4 (2006), the definition of bearing resistance uses only the reduced ultimate limit of the material. Thus, bearing resistance is given by a formula similar to that used for carbon steel as given by Eq. (4).

$$F_{b,Rd} = k_1 \alpha_b f_u d t / \gamma_{M2} \tag{4}$$

with, in the direction parallel to the load transfer:

- for end bolts : $\alpha_b = \min\{e_1/(3d_0); f_{ub}/f_u; 1\}$
- for inner bolts: $\alpha_b = \min\{p_1/(3d_0) - 1/4; f_{ub}/f_u; 1\}$

and, in the direction perpendicular to the load transfer:

- for edge bolts: $k_1 = \min\{2.8e_2/d_0 - 1.7; 2.5\}$
- for inner bolts : $k_1 = \min\{1.4e_2/d_0 - 1.7; 2.5\}$

3. Modelling and test results

A numerical model has been developed (Bouchaïr *et al*. 2007) in order to analyze the behaviour of cover-plate joints and the influence of many parameters. The joint is composed of one central plate and two lateral plates connected by two bolts. The model is at first applied to a configuration named 2T, which represents the case of symmetrical joints composed of two bolts in double shear and austenitic stainless steel (EN 1.4307, AISI 304L) plates. During tests, the failure mode observed for this configuration is the bearing failure as expected. For this reason, this configuration is chosen to develop the reliability analysis in the present study.

3.1 Finite element model

In the finite element model, the joint symmetry is considered in order to save the computing time, basic components are modelled and combined through contact elements (see Fig. 4) to represent the global behaviour of the analysed joint. The models need to be accurate and simple to perform a large number of calculations as required by the reliability analysis.

The components C0 and C1 are quasi similar representing two plates with loaded holes. They represent the elementary case of plates with hole loaded by a cylindrical bolt in contact with the internal surface of the hole. A third component C2 is added to represent the deformation of the threaded zone of the bolt in contact with the plate hole. The threads are characterized by large local deformations that influence the overall behaviour of the joint. The finite element model is implemented in the software Cast3m developed by CEA (Cast3M 2015) using TET3 linear 3D tetrahedral elements. The loading is introduced by applying monotonic displacement at the bolt ends. Indeed, for the central component C0, the bolt is in double shear, and for the lateral plate C1 it is in single shear. Considering the symmetry of the joint, only a quarter of the joint is modelled (see Fig. 4). The contact is considered between the surface of the bolt and the internal cylindrical surface of the hole, without friction. The stress-strain curves of the materials are described through the tabulated data obtained from tensile tests. The nonlinear character of the problem is due to the elastic-plastic behaviour of materials, the contact between the bolt and the hole and the large displacement conditions. It is solved using iterative Newton-Raphson scheme.

3.2 Experimental validation and comparison with EC3

Tests were conducted (Bouchaïr 2005, Bouchaïr *et al*.

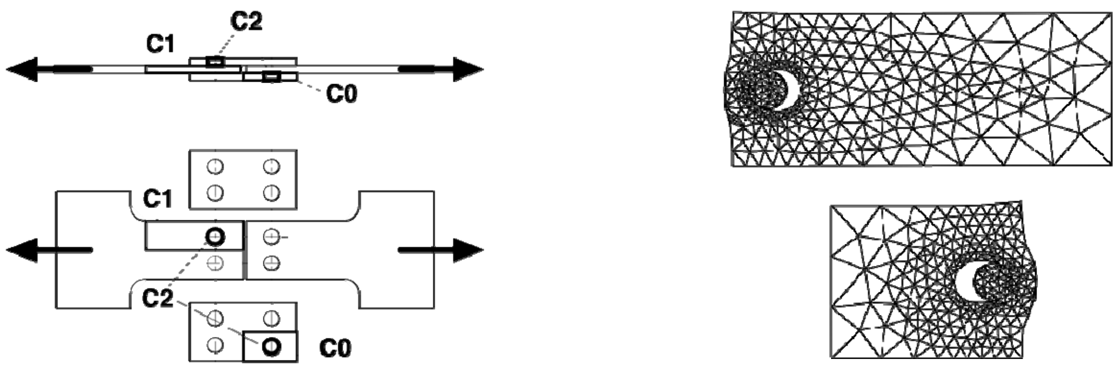


Fig. 4 Cover-plate joint components for modeling (a); deformed components C0 and C1 (b)

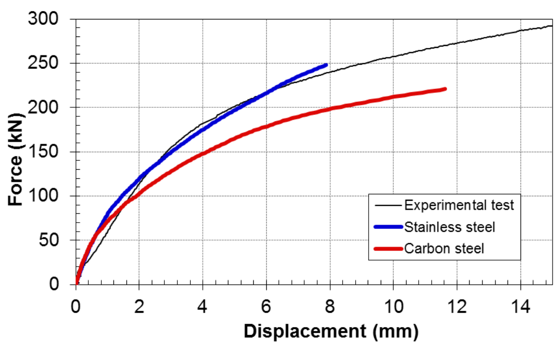


Fig. 5 Load displacement result for 2T-20 joint

2007, Averseng and Bouchaïr 2009) to observe the failure modes of cover-plate joints, considering twelve configurations with three bolt diameters and four arrangements. In this work, only the results of 2T- connections, that exhibit bearing as failure mode, are presented. In fact, this failure mode is the most complex to represent. The other configurations exhibit two common failure modes which are the bolt shear and the plate net section. The thicknesses of the lateral and central plates were respectively 5 and 10 mm in all the tests. Three diameters of bolts are considered (12, 16 and 20 mm). Displacement cycles were imposed and it was shown that the slopes of the load-displacement curve during the regular unloading-reloading phases were identical, which is the sign of a stable connection rigidity, even though high values of global displacement (Bouchaïr 2005).

The global force-displacement curves for all the joints are in good accordance with results obtained from the numerical model. To illustrate this comparison, Fig. 5 shows the case of the joint 2T-20 with two bolts and two shear planes. The experimental tests (Bouchaïr 2005) show a significant difference (see Table 1) between the ultimate loads from experiments and the design values given by EN 1993-1-4 rules with a partial safety factor taken equal to 1.0.

Given its geometry, the 2T-20 joint exhibits a large ductility with a bearing failure mode. However, most of the failure modes predicted by calculation according to EN 1993-1-4 formulae are associated with the gross section mainly when the nominal characteristics of materials are used. Actually, this limit is reached before the net section resistance because the ratio between the areas of the net and gross cross-sections is close to 0.7 (0.67 to 0.69), while the ratio between the steel yield strength and its ultimate tensile strength is less than 0.5 (Kim and Yura 1999). In addition, to obtain a basis of comparison, the same calculations are done with the nominal values of the mechanical characteristics of materials.

4. Structural reliability analysis

The structural safety consists in verifying the probability of violating the design rules, taking into account the uncertainties in data, design, construction and operation. Each design rule represents a failure mode, defined in terms of the basic variables, for which uncertainties and fluctuations can be modelled by random variables. The objective of reliability analysis is to evaluate the probability of occurrence of a specific failure scenario.

Table 1 Comparison of analytical and experimental results

Test results			Calculation according to EC3* (kN)					
Joint type	F_u-test (kN)	Mode	Bolt shear (BS)	Bearing (Be) using f_u	Bearing (Be) using $f_{u.red}$	Net section (NS)	Gross section (GS)	F_u-test/ F_u-EC3
2T-12	**179.4**	BS	174.6	187.6	**157.8**	359.0	251.6	1.14
2T-16	**341.9**	Be/BS	359.8	236.2	**198.8**	428.5	307.5	1.72
2T-20	**444.5**	Be	501.0	306.9	**258.2**	555.8	391.3	1.72

*$\gamma_M = 1$, f_u and f_y obtained from tests

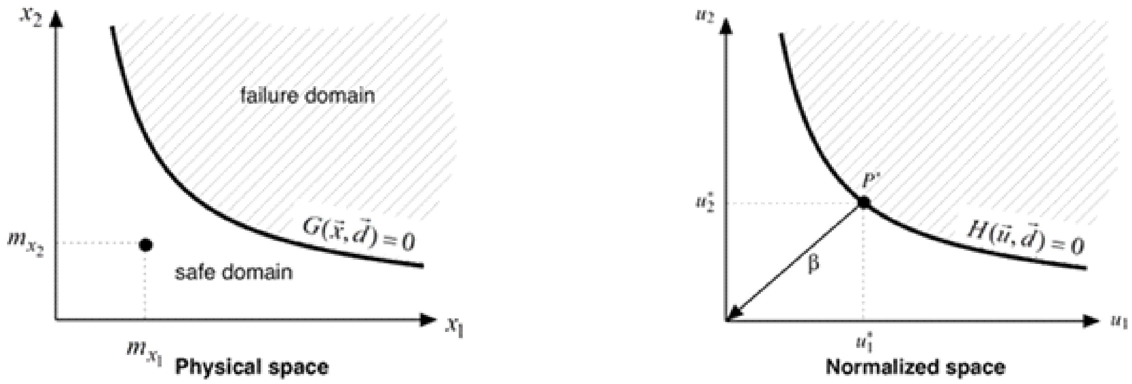


Fig. 6 Physical and normalized spaces for reliability analysis

4.1 Basic concepts of reliability analysis

To evaluate the failure probability with respect to a given failure scenario, a limit state function $G(x_i, d_k)$ is defined such as $G(x_i, d_k) > 0$ indicates the state of safety and $G(x_i, d_k) \leq 0$ indicates the state of failure (see Fig. 6), where x_i are the realizations of the random variables X_i and d_k are the deterministic design variables. The failure probability is then calculated using Eq. (5).

$$P_f = P_r\left[G(x_i,d_k)\leq 0\right] = \int_{G(x_i,d_k)\leq 0} f_{X_i}(x_i)\, dx_1 \cdots dx_n \quad (5)$$

where P_f is the failure probability, $f_{Xi}(xi)$ is the joint density function of the random variables Xi and $Pr[...]$ is the probability operator. The evaluation of the integral in Eq. (5) is not easy, because it represents a multi-dimensional integral with very small quantity to evaluate, and also because all the necessary information for the joint density function are not available in practice (Lemaire *et al*. 2009). For these reasons, the First Order Reliability Method (FORM) (Ditlevsen and Madsen 1996) has been developed by introducing the reliability index β to represent the normalized margin between the median point and the most probable failure point (noted P^*); this index can be evaluated by solving the constrained optimization problem using Eq. (6).

$$\beta = \min\sqrt{\sum_j u_j^2} \quad \text{under the constraint: } G(x_i, d_k) \leq 0 \quad (6)$$

where u_j are the normalized random variables, which are given by standard probabilistic transformation of the physical variables x_i: $u_j = T_j(x_i, d_k)$. In *FORM* approximation, the failure probability is given by Eq. (7).

$$P_f \approx \Phi(-\beta) \quad (7)$$

where $\Phi(.)$ is the standard Gaussian cumulated distribution function.

4.2 Response surface method

As in our case, the joint resistance, and consequently the limit state function G, cannot be given as an explicit function in terms of the random variables, the reliability problem of Eq. (6) can be efficiently solved by using a meta-model rather than full nonlinear finite element models, involving large computation time. The meta-model can be obtained by defining a response surface (RS) based on a limited number of finite element analyses. The concept of Response Surface Methods (RSM) consists in building a polynomial expansion of the limit state function $G(x_i, d_k)$, where the polynomial coefficients are computed by least square fitting (Soares *et al*. 2002, Neves *et al*. 2006). To build the whole response surface, the choice of several realizations can be performed either by the experiment design techniques leading to regular points in the design space, or by Monte Carlo simulations leading to randomly distributed points. To build a complete quadratic polynomial, it is required to perform at least a number of $(n+1)(n+2)/2$ finite element analyses, where n is the number of variables. In our case, a multi-level experiment plan is

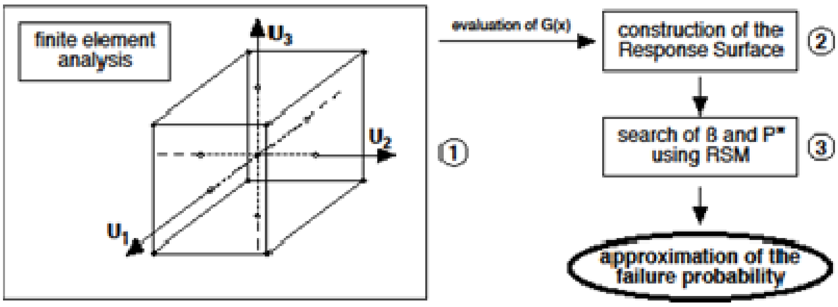


Fig. 7 Analysis procedure in the reliability evaluation

adopted to cover the whole design space with good precision.

The reliability evaluation, using response surface methods, can be divided into three steps (see Fig. 7). In the first step, the mechanical model is solved at the points pre-defined by the experiment plan; in the second step, the finite element analysis results are used to approximate the joint response by polynomial function; the third step consists in solving the reliability problem Eq. (6) on the response surface in order to provide the reliability index and the failure point. The failure probability can then be calculated on the basis of FORM approximation Eq. (7).

Based on the finite element model described in section 3, the response surface model gives us the ultimate load as a function of the joint parameters $P_U(e_1, d, t,...)$. A good state of operation implies that the joint resists the applied load P_A. Therefore, the limit state function is defined by Eq. (8).

$$G(x_i, d_k) = P_U(e_1, d, t, ...) - P_A(G, Q) \quad (8)$$

where G and Q are respectively the dead and live loads. The applied load P_A may be considered according to one of the following situations:

- P_A is considered as deterministic design load, specified by the design codes of practice, such as the Eurocode 3 (EN1993-1-4 and EN1993-1-8). In this case, the reliability is assessed with respect to the safety provided by the design code. This approach is suitable to focus the study on the strength variability. The deterministic load P_A is given by: $P_A = 1.35G_k + 1.5Q_k$; the partial factors 1.35 and 1.5 are meant to ensure that the probability for the structure to meet a load higher than P_A is very low all over the structural lifetime.
- P_A is computed from the real applied loads which are also random. In this case, the reliability assessment is relative to the joint failure, as it includes load and resistance uncertainties.

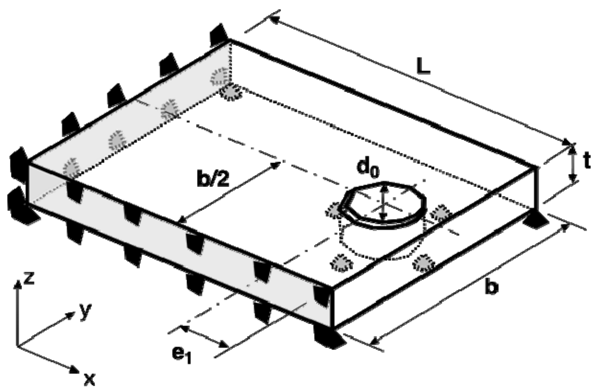


Fig. 8 Geometrical parameters of the plate used in reliability analysis

5. Reliability analysis of the joint

5.1 Statistical data

The part of cover-plate joint considered in the present study is a basic component representing a plate loaded by a bolt as explained in section 3 (see Fig. 8). The stress-strain curves of the materials, based on the yield strength, are considered as main parameters and their influence is taken into account through the factor k applied to scale the stress-strain curve: $\sigma(\varepsilon) = k\sigma_{ref}(\varepsilon)$, where $\sigma_{ref}(\varepsilon)$ is chosen equal to 235 MPa. Preliminary study based on finite element analysis is performed to define the main parameters influencing the behaviour of cover-plate joints. Thus, the random variables considered are the plate half-width b with thickness t and the end-distance edge e_1, in addition to the scale factor k (see Table 2). The standard deviations for geometrical parameters are derived from fabrication tolerances, and the standard deviation for the factor k is derived from the known coefficient of variation of steel industry, which is equal to 7% (Nowak 1994, Melchers 2007). This coefficient of variation allows defining the mean value of k by the percentile of 5%, leading to the value of 1.151.

Unlike carbon steel, stainless steel material and joints does not present clear plastic load limits, as the force-displacement curve is fully non-linear and does not have a

Table 2 Cover-plate joint parameters

Variable	Symbol	Unit	Type	Mean	St-dev.
Joint length	L	mm	Deterministic	100	
Bolt diameter	d	mm	Deterministic	20	
Hole diameter	d_0	mm	Deterministic	22	
Longitudinal pitch	p_1	mm	Deterministic	60	
Transversal pitch	p_2	mm	Deterministic	60	
Plate half-width	b	mm	Lognormal	60	1.0
Plate thickness	t	mm	Lognormal	10	0.5
End-distance (load direction)	e_1	mm	Lognormal	30	1.0
Constitutive law parameter	k	-	Lognormal	1.151	0.092
Reference yield stress	f_y	MPa	Deterministic	235	
Reference ultimate stress	f_u	MPa	Deterministic	622.39	
Bolt ultimate stress (A4-80)	f_{ub}	MPa	Deterministic	800	

plateau. It is therefore necessary to specify a conventional ultimate load. In this study, four possibilities are chosen and compared:

- P_{U2}^{C} corresponding to the curve load at a displacement of 2 mm,
- P_{U5}^{C} corresponding to the curve load at a displacement of 5 mm,
- P_{U2}^{X} corresponding to the intersection of tangents at origin and at a displacement of 2 mm,
- P_{U5}^{X} corresponding to the intersection of tangents at origin and at a displacement of 5 mm.

The displacement of 2 mm is considered as the lower level corresponding to the gap between the bolt and the hole diameters; this displacement has generally negligible effects on the load distribution in the whole steel structure. The displacement of 5 mm corresponds to the radius of the bolt, indicating that excessive plastic deformations are achieved at the hole. Naturally, these two reference displacements are conventional, and other values could be chosen when justified.

Table 3 Parameters used as variables in the model

Variable	Unit	Values in the design of experiments
b	mm	30, 40, 50, 60, 70, 80
e_1	mm	50, 40, 35, 30, 25, 15
t	mm	8, 10, 12
k		0.848, 1.00, 1.06, 1.272, 1.484

5.2 Response surface construction

In order to define an explicit relationship between input and output parameters, the response surface method described in section 4.2 is now applied with the four resisting loads P_{U2}^{C}, P_{U5}^{C}, P_{U2}^{X} and P_{U5}^{X}, defined as functions of the random variables (b, e_1, t, k). The geometrical parameters are chosen to vary in the range that covers large varieties of the most used configurations in engineering structures. It is to emphasize that the values of these parameters influence significantly the failure mode. Therefore, the finite element analysis gives the envelope of the various modes rather than the effect of each mode separately. The scale factor for the constitutive law of the material is chosen to vary from 0.848 to 1.484, corresponding to a variation of the yield stress from 200 to 350 MPa. Table 3 gives the values provided by the design of experiments approach, leading to 540 combinations (i.e. nonlinear finite element analyses).

For the whole set of finite element outputs (see Fig. 9), a quadratic response surface is fitted in order to give the resisting load of the cover-plate joint. For example, the fitted expression giving the resisting load corresponding to the intersection of tangents at the origin and at the displacement of 5 mm (P_{U5}^{X}) is given by Eq. (9).

The fitting of this expression for all the finite element results gives a maximum error of 5% on the load capacity. The fitted response surface is therefore considered to be sufficiently accurate, in order to be used in the reliability analysis. It is to remind that this expression is only valid between the limits of variation of the parameters; in other

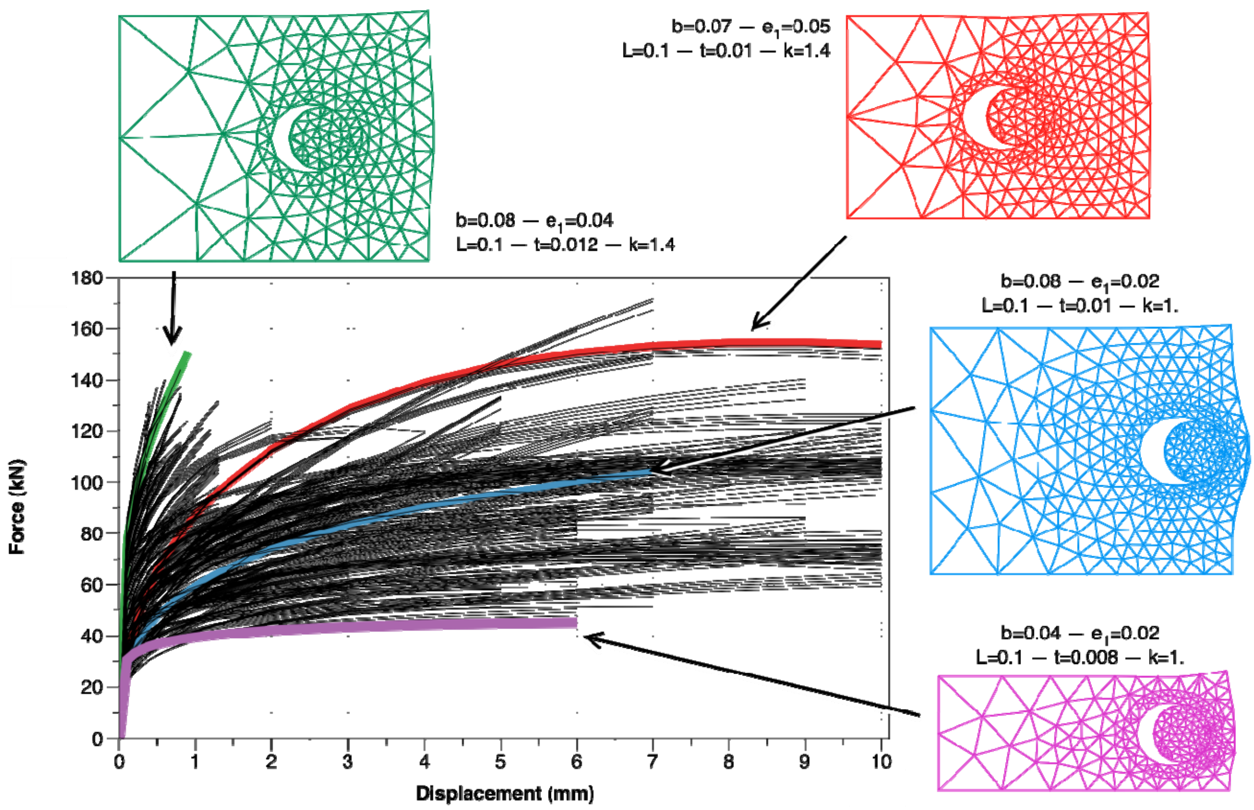


Fig. 9 Whole set of computed finite elements response curves and examples of corresponding deformed meshes

Table 4 Statistical moments and failure probabilities for the cover-plate joint resistances

Ultimate capacity	P_{U2}^{C}	P_{U5}^{C}	P_{U2}^{X}	P_{U5}^{X}
Mean	186.0	234.5	140.4	197.3
Standard deviation	16.22	21.14	12.40	17.62
Failure probability under EC3 load (141 kN)	8.6×10^{-4}	2.5×10^{-9}	5.2×10^{-1}	5.7×10^{-5}

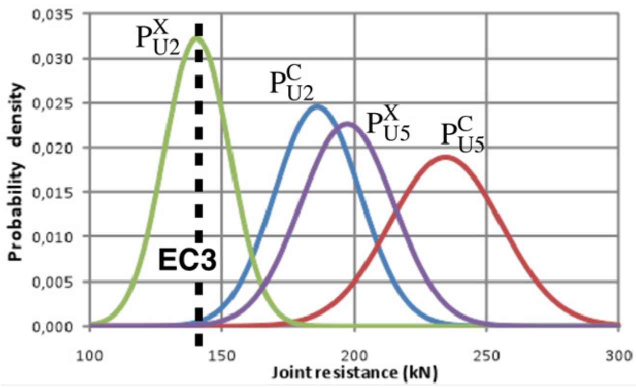


Fig. 10 Probability distributions of the cover-plate joint resistances

words, only interpolation is allowed for this approach of design of experiments, in order to preserve the required precision.

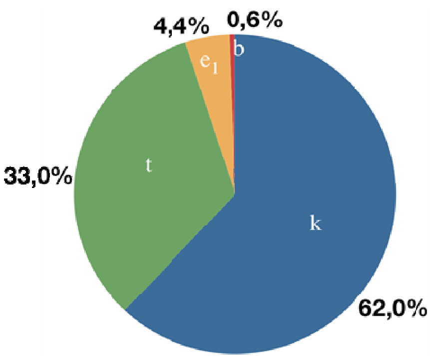
$$
\begin{aligned}
P_{U5}^{X}(b,e_1,t,k) = & -217.77 + 36.01b - 22.26e_1 + 178.77t \\
& -16.36k - 6.30b^2 - 6.36e_1^2 - 140.05t^2 \\
& -21.06k^2 + 8.54be_1 + 17.14bt + 11.21bk \\
& +20.28e_1t + 15.08e_1k + 97.21tk
\end{aligned}
\tag{9}
$$

5.3 Joint reliability regarding Eurocode capacity

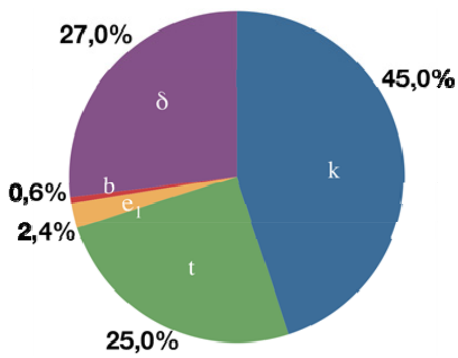
The statistical parameters of the cover-plate joint resistance according to the four definitions of resistances (see Table 4) are calculated from the response surface. For all cases, the coefficient of variation is about 9%. Normal distribution assumptions are acceptable as the statistical moments are less than 0.2 for the skewness and close to 3 for the kurtosis. The resistances P_{U2}^{C} and P_{U5}^{X} give comparable capacities for the cover-plate joint, the resistance P_{U5}^{C} is largely above the other values (see Fig. 10). On the opposite, the resistance P_{U2}^{X} is clearly low as it represents the most pessimistic situation; it does not usefully use the ductile properties of the stainless steel.

The Eurocode 3 (EC3) (EN1993-1-4 2006, EN1993-1-8 2005) is based on the use of the characteristic values of the yield strength f_{y_k} = 235 MPa and the ultimate tensile strength f_{u_k} = 622.4 MPa. For the defined cover-plate joint (see Table 2), the design according to EC3 leads to: $f_{v,Rd}$ = 301.4 kN (bolt shear with two shear planes and unthreaded section), $N_{u,Rd}$ = 236.5 kN (net cross-section), $N_{pl,Rd}$ = 141.0 kN (gross cross-section) and $F_{b,Rd}$ = 141.3 kN (bearing with ultimate tensile strength); the maximum allowed ultimate load is therefore F_{Ed} = 141 kN. If we consider that the EC3 load corresponds to the characteristic value at 95% and by assuming normal distribution with 9% of coefficient of variation, the mean ultimate load should be at 165 kN. Except for the resistance P_{U2}^{X}, the three other load capacities are largely above the value specified by the EC3. As indicated (see Table 4), the probability of failure when the EC3 load is applied (i.e.,141 kN) is 8.6×10^{-4} for P_{U2}^{C} and 5.7×10^{-5} for P_{U5}^{X} (it is to note that additional safety margin is provided by the characteristic values of loads and partial load factors). The importance of the resistance variables regarding the reliability under this load is depicted in Fig. 11. The material strength is the most important variable with 62%, followed by the plate thickness with 33%; the parameters b and e_1 has negligible importance for the considered joint dimensions. These results emphasize that special attention should be paid for the way of modelling the material stainless steel behaviour.

In order to take account for the random errors in the numerical model, the output load capacity is multiplied by a random parameter δ that is normally distributed with mean equal to 1 and standard deviation equal to 5%. For the case P_{U5}^{X}, the probability of failure under the EC3 load is increased to 3.9×10^{-4} (the standard deviation of this resistance slightly increases to 18.6 kN); this probability remains very low in practical engineering, as 141 kN is a characteristic load. When taken into account (see Fig. 11(b)), the model error δ grabs a significant amount of sensitivity (27%) but still remains less than the material strength.



(a) Without model error



(b) With model error

Fig. 11 Importance of the various joint parameters on the strength

Table 6 Failure probabilities of the cover-plate joint

Ultimate capacity	P_{U2}^{C}	P_{U5}^{C}	P_{U2}^{X}	P_{U5}^{X}
Probability of failure excluding model error	4.5×10^{-11}	9.8×10^{-18}	4.2×10^{-5}	2.3×10^{-12}
Probability of failure considering model error	8.4×10^{-10}	7.8×10^{-15}	3.1×10^{-5}	4.8×10^{-11}

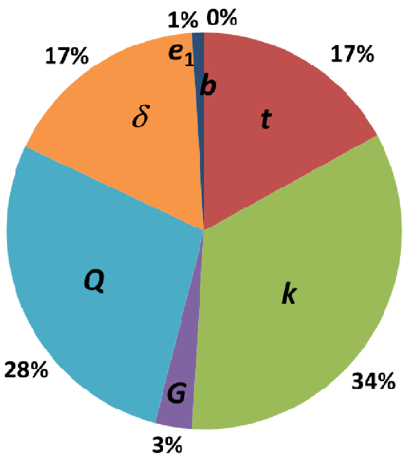


Fig. 12 Importance factors for the design variables

5.4 Joint reliability regarding applied loads

According to EC3, the design values of loading corresponding to the fundamental combination of dead and live loads at the ultimate limit state is given by Eq. (10).

$$F_{Ed} = 1.35G_k + 1.5Q_k = 141kN \tag{10}$$

The characteristic values G_k and Q_k are given for percentiles of 95%. In other words, the design load F_{Ed} has very low probability to be met in practical structures, as two levels of safety are included, namely the characteristic values and the partial load factors. By assuming that the dead load is about 35% of the total load for steel structures and the coefficients of variation (COV) are 10% for dead load and 20% for live load, it can be possible to calculate the mean values for the allowed dead and live loads (see Table 5). When all these safety measures are included, the probability that the applied load exceeds the level of 141 kN is equal to 3.9×10^{-11}. Given the failure probability of the joint (see Table 6) and knowing that the reliability target for the Eurocodes is 10^{-4} for ultimate limit state and that stainless steel degradation (especially corrosion) is lower than for carbon steels, it could be reasonably suggested to allow higher design load for this type of joints. In addition, the live load and the material strength are the major parameters influencing the reliability (see Fig. 12), followed by the model error (δ) and the plate thickness (t).

6. Conclusions

In this work, the reliability of stainless steel joints is investigated. The Eurocode 3 design formulae are compared to the finite element model involving nonlinear material behaviour and contact in the bolt thread. This model is validated by experimental testing of cover-plate joints. It is clearly seen that stainless steel exhibits larger load capacities than that given by the analytical formulae of Eurocode 3.

In order to perform the reliability analysis, a design of experiments approach is performed to provide explicit response surface for joint resistance. The reliability analysis shows rather large safety provided by the stainless steel joint. It also indicates the importance of material behaviour and model errors on the overall reliability of the joint. This preliminary reliability investigation suggests to increase the design load on stainless steel, with respect to Eurocode formulae.

This conclusion should be confirmed by further reliability analyses on other geometrical and load configurations. In future works, the approach developed in this study can be applied to various configurations of joints with different material characteristics considering carbon and stainless steel with different configurations of bolts positions and diameters. It will consider the number of bolts and the eccentricities due to the hole positions defects and load direction, with the aim to calibrate design formulae for practice.

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