BEHAVIOUR AND DESIGN OF STAINLESS STEEL BOLTED CONNECTIONS

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ABSTRACT

Using previously developed and validated finite element (FE) models, an investigation into the behaviour of stainless steel bolted connections has been carried out. Both net section rupture and bearing failure were studied. The main parameters affecting net section rupture were found to be edge distance e_2 and bolt configuration. It was shown that the ductility of stainless steel is sufficient to ensure a failure at the full fracture load (net area $\times$ ultimate tensile strength) for both thick and thin material and hence no reduction factor is required in the design equation. This was confirmed by reliability analysis and a suitable partial safety factor has been derived. The investigation into bearing behaviour showed that the deformation response of stainless steel connections is different from that of carbon steel connections, with stainless steel exhibiting pronounced strain hardening. The locations of fracture initiation obtained from the numerical models match those observed during experimental studies of carbon steel and stainless steel connections. The results of the parametric study were utilised as the basis for the development of design provisions for bearing failure in stainless steel bolted connections, covering both cases in which deformation controls and those for which ultimate strength is the only criterion.

KEYWORDS

Bearing failure, Bolted connections, Finite element modeling, Joints, Net section failure, Stainless steel.

1. INTRODUCTION

The use of stainless steel in construction is steadily growing due the attractiveness of its mechanical properties, durability, appearance and fire resistance. A comprehensive exploration of the benefits offered by the material and its recent usage in structures has been reported by Gardner (2005). Interestingly, although high initial cost is its major drawback, recent developments suggest that low nickel content stainless steel can provide many of the durability related benefits of more traditional grades but at around half the cost – thereby further encouraging its adoption in appropriate circumstances (Theofanous & Gardner, 2009, 2010)). Unlike carbon steel, the stress-strain relationship of stainless steel is rounded without a well-defined yield stress and the material exhibits significant strain hardening. Despite these fundamental differences, design provisions for bolted connections between stainless steel structural members in current international standards are essentially based on the rules for carbon steel with only some very limited modifications (EN 1993-1-4, 2006; SCI/Euro Inox, 2006; ASCE, 2002; AS/NZS, 2001). This is attributed to the lack of comprehensive information on the behaviour of these particular structural components.

Few studies have been carried out to investigate the behaviour of stainless steel bolted connections. The first experimental work was conducted a quarter of a century ago by Errera et al. (1974). They investigated the behaviour of bolted and welded connections in order to assess the validity for stainless steel of the design equations for cold-formed carbon steel bolted connections, which had been proposed by Winter (1956). Identical failure modes were observed leading to the conclusion that Winter's design equations for the net section and bearing resistance are valid, without modifications for stainless steel connections. An experimental study of bolted connections in ferritic stainless steel was carried out by Van Der Merwe (1987). Most recently Kim et al. (2008) performed tests together with a numerical study that formed the basis for proposed design equations for net section and bearing resistance of connections between thin gauge stainless steel sheets susceptible to out-of-plane deformations (curling).

The experimental investigation reported by Ryan (1999) was the first study of stainless steel connections using thicker material. Laboratory tests on bolted connection made from three types of stainless steel (austenitic, ferritic and duplex) formed the basis for revising the design provisions in ENV 1993-1-4 (1996). Although Ryan's results suggested removal of the 0.9 reduction factor in the net section design formula, he recommended maintaining the factor k_r until such time that the number of tests provided full justification. Bouchair et al. (2008) developed numerical models to replicate the behaviour of these experimental specimens, and recommended that the reduction factor k_r in the net section design equation and the influence of the end and edge distances on bearing behaviour be further investigated.

Owing to the limited number of test results presently available, further investigation is necessary to understand the behaviour of these structural components so as to improve the current design provisions. In this paper, an approach is adopted in which the available test data is first used to validate numerical models which are then utilised to study a much

wider coverage of key design variables. On the basis of the experimental and numerical results, improved design equations for net section and bearing capacities of stainless steel connections are proposed.

2. FINITE ELEMENT (FE) MODELLING

2.1 General

The finite element analysis software ABAQUS 6.7-1 (2007) was used to develop numerical models for austenitic and ferritic stainless steel bolted connections to examine their response under static shear loading. The configurations of these specimens, which matched the test arrangements utilised by Ryan (1999), are presented in Figure 1. In order to reduce the size of the model and, consequently, the computational cost, only one quarter of the connection was modelled by applying appropriate boundary conditions. The 3D solid (brick) element with full integration – C3D8 – which has proved to be suitable to simulate lap bolted connections in many previous investigations (Chung & Ip, 2000; Ju et al., 2004), was employed in this study to model both the plates and bolts. Loading was applied by means of uniform displacement-control at the end of the central plate. Figure 1 illustrates the boundary conditions applied to the FE models.

The compound Ramberg-Osgood stress-strain model developed by Mirambell & Real (2000) and Rasmussen (2003) was adopted to represent the non-linear material response of the investigated stainless steels – austenitic grade 1.4306 and the ferritic grade 1.4016. Numerical analyses of bolted connections are expected to involve large inelastic strains, therefore, nominal stresses and strains were converted to the corresponding true values, which take into account the change in geometry under load. Contact between all components that are expected to interact with each other was defined using the surface-to-surface contact feature in ABAQUS. Frictional effects were taken into account by using the classical isotropic Coulomb friction model. Bolts were located centrally into the holes with a uniform clearance of 1.0 mm. Different levels of bolt preload were applied. Loads were carried initially by friction until the occurrence of slippage, after which direct bearing was the primary means of load transfer.

2.2 Failure criteria for FE models

To verify the ability of the FE models to replicate the experimental results, including the full deformation characteristics and failure modes, precise material modelling and failure criteria are required. In numerical investigations of structural members such as beams and columns, the ultimate capacity is, simply, the peak load from the load-deformation curve. In the case of bolted connections however, this curve usually exhibits no clear peak, and the failure mode must be identified in order to determine the load carrying capacity. For this reason, employing suitable failure criteria is vital when studying bolted connections numerically. Three failure modes were considered in this study – net section fracture, bolt shear failure and bearing failure.

Figure 1: FE models, (a) 2 bolts longitudinally (b) 2 bolts transversely
(c) 3 bolts (d) 4 bolts

2.2.1 Net section failure

The existence of holes in a plate causes a discontinuity in geometry and a disruption of the stress trajectories. As a result, stress concentrations occur in the region of the hole. As the applied load increases the magnitude of the maximum stress increases until fracture initiates at the edge of the bolt hole where the maximum stress concentration develops. The crack then propagates transversely across the net section resulting in the net section failure. The form of this failure is greatly affected by the ductility of the material. If the material is ductile, local yielding at the hole edge as a result of stress concentration allows for stress redistribution across the net section, followed by necking and finally a ductile failure of the connection. In contrast, if the material possesses little ductility, the stress concentration in the vicinity of the hole cannot be redistributed leading to immediate cracking, resulting in a brittle failure. Net section fracture is critical in connections with relatively narrow plate widths. To model net section rupture in the FE models, reference was made to the aforementioned point at the edge of the bolt hole; when the maximum equivalent plastic strain at this point reached the true localized fracture strain of the material, taken as 100% (Salih et al. (2010)), it was assumed that a crack initiates, and the ultimate load is said to be reached.

2.2.2 Bolt shear failure

In bearing-type lap connections the load is transferred between the connected plates by means of shear across the bolt cross-section. When the shear load in the bolt exceeds its

capacity brittle failure occurs called “bolt shear”. Kulak et al. (1987) concluded that the shear strength of a single bolt is about 62% of its ultimate tensile strength. Ju et al. (2004) investigated bolt shear failure for lap bolted connections similar to those in this study using numerical models. It was proposed that a bolt fails in shear when the critical cross-section of the bolt becomes fully plastic. In the current study, this criterion has also been adopted. Hence, when the equivalent principal plastic strain over the full critical cross-section reaches the true yield strain, the cross-section of the bolt is fully plastic, and is deemed to have failed.

2.2.3 Bearing failure

The bearing resistance of bolted connections may be defined either on the basis of strength or deformation criteria. For the former, the bearing resistance is taken as the maximum load measured in tests regardless of the associated extension. In contrast, adopting a deformation criterion, the ultimate capacity of a connection is defined as the load corresponding to a specified extension. For carbon steel, for which the load-deformation behaviour flattens once significant bearing deformations occur, the exact choice of deformation limit has relatively little effect on the limiting load. However, stainless steel specimens, owing to the hardening characteristics of the material, generally exhibit a rising relationship, with no clear flattening. Thus, the load level corresponding to bearing failure will depend on the deformation limit selected. Figure 8 demonstrates the difference in the deformation response of carbon steel and stainless steel bolted connections. In the experimental study by Ryan (1999), three specimens were reported to have failed in a bearing mode. Since, these connections were not loaded until the occurrence of rupture, the reported maximum loads for these connections do not correspond to an actual fracture. Thus, applying arbitrary deformation limits will give inconsistent results. For this reason, the validation of the FE models for these connections will be on the basis of the load-deformation curves only. The bearing behaviour of stainless steel connections is investigated in detail in Section 3.

2.3 FE model validation

The overall deformation behaviour of the connections was validated by comparing the numerically obtained load-deformation curves with those obtained from 24 previously reported laboratory tests (Ryan, 1999). It is clear from the typical curves shown in Figure 2 that the predicted load-deformation curves are in good agreement with the tests. In addition, the ultimate capacities obtained from the FE models using the proposed failure criteria were compared with those obtained in the tests. The comparison (Salih et al. (2010)) showed that the developed models are able to predict the observed ultimate capacities very accurately, with a mean FE/test ratio of 0.99 and a standard deviation of 0.03. Comparisons between the deformed test specimens and the corresponding FE models are shown in Figure 3. It can be concluded that the developed numerical models are able to replicate the experimental behaviour and can, therefore, be employed to increase the pool of structural performance data by means of parametric studies.

(a) Typical type 'a' austenitic stainless steel connection

(b) Typical type 'b' ferritic stainless steel connection

Figure 2: Typical load-deformation curves from tests and numerical models

Figure 3: Comparison between deformed test specimen and numerical model

3. PARAMETRIC STUDIES

The number of available test results on stainless steel bolted connections is insufficient to fully investigate the behaviour of these components. In order to study the net section rupture and bearing behaviour in detail, the data set has been extended by generating further results using the validated FE models. The same two types of stainless steel considered in Section 2 were investigated – austenitic grade 1.4306 and the ferritic grade 1.4016 – though these grades may be considered to be representative of the wider austenitic and ferritic families of stainless steel, respectively. The employed material properties have been described by Salih et al. (2010). Eurocode 3 Part 1.3 (2006) provides design expressions for bolted connections between thin carbon steel plates with thicknesses of 4.0 mm or less. This definition of thickness range is adopted in the present parametric study where 10.0 mm and 8.0 mm thick plates were investigated to represent thick plate connections, while 4.0 mm and 2.0 mm thick plates were considered to investigate thin plate connections.

3.1 Investigation of net section rupture

To examine the effect of the key parameters (the edge distance e_2 and the ratio r) on the net section capacity of lap connections, six configurations of double shear connections with three plates of the same thickness were adopted; these correspond to four values of r (1/2, 1/3, 1/4 and 2/5) with a range of edge distance ratio e_2/d_0 up to 4.0. Single-row connections were adopted for the FE models comprising thin plates (2.0 mm and 4.0 mm), while both single-row and double-row connections were used for the thick plate models (8.0 mm and 10.0 mm). Figures 4 and 5 show the results of the parametric studies for the thin and thick plate connections respectively. It can be seen that for stainless steel bolted connections failing in net section rupture, the nominal stress (ultimate load / net section area) at the critical net section at failure is greater than or equal to the material ultimate tensile strength.

Figure 4: Comparison between FE results and design code equations for thin plate bolted connections

(a): Two bolts in one row

(b): Three bolts in one row

(c): Four bolts in one row

(d): Six bolts in two rows

(e): Eight bolts in two rows

(f): Five bolts in an 'X' arrangement

Figure 5: Comparison between FE results and design code equations for thick plate bolted connections

3.2 Investigation of bearing failure

In the parametric studies, lap connections with bolts in double shear were investigated as shown in Figure 6 with plate thicknesses of 8.0 and 10.0 mm. The two main parameters that were investigated are the end distance ratio e_1/d_0 , varying from 0.8 to 4.0, and the edge distance ratio e_2/d_0 with four values: 1.5, 2.0, 3.0 and 4.0.

Figure 6: Configuration of FE models adopted to investigate bearing behaviour

Bearing failure in thin sheet connections is characterized by two additional phenomena known as curling and pulling into line. While the former occurs for plates in single shear connections and in the external plates of double shear connections, the latter takes place only in single shear connections. Curling is defined as the out-of-plane deformation of the sheet in front of the end bolt. This deformation is fundamentally a buckling of the plate subjected to compressive stresses. An investigation into the behaviour of thin gauge sheet connections is currently underway.

In an arrangement where bearing is critical, the distribution of plastic strain in the plate in front of the bolt exhibits peaks at two symmetrical locations at about $\theta = 45^\circ$ and 135° as shown in Figure 7. This strain distribution matches observations during testing ((Errera et al., 1974; Winter, 1956; Kim & Kuwamura, 2008; Dhalla et al., 1972; Rogers & Hancock, 1999; Puthli & Fleischer, 2001) and confirms that bearing fracture occurs at these locations. This conclusion will be adopted to determine the bearing capacity using the strength criterion: when the peak plastic strain in the plate material in front of the bolt reaches the localized fracture strain of the material (Salih et al. (2010)), fracture occurs and the maximum load is said to have been reached.

In order to investigate the difference in the deformation behaviour of carbon steel and stainless steel bolted connections, an FE model was developed with the material behaviour of austenitic stainless steel and the geometry of a carbon steel connection tested by Kim (1996). Figure 8 shows that, for the carbon steel connection, the load-deflection response attained a relatively flat state once significant bearing deformations had occurred, and therefore, the exact deformation limit has relatively little effect on the “failure load”. However, for the stainless steel connection, owing to the rounded stress strain nature of the material and the pronounced strain hardening, a rising relationship, without significant

flattening off was obtained. Thus, selection of different deformation limits will significantly affect the load corresponding to bearing failure.

Figure 7: Plastic strain distribution in front of the bolt

Figure 9 shows the stiffness at three stages of loading of a stainless steel connection from the parametric study. The stiffness at the point of bearing fracture is almost equal to the stiffness at 6.35 mm deformation and is about 50% of the stiffness at 3.0 mm deformation; this loss of stiffness may be considered to be relatively modest. It may be concluded that defining bearing failure on the basis of deformation limits (3.0 mm or 6.35 mm which are commonly adopted for carbon steel), underestimates the true bearing resistance of stainless steel connections.

Figure 8: Comparison between carbon and stainless steel connection behaviour

Figure 9: Stiffness of typical stainless steel connection

3.3 Determination of bearing capacity of bolted lap connections

The bearing resistance of bolted connections in shear has been determined in previous studies either on the basis of a strength or deformation criterion. These two criteria are discussed in this section.

3.3.1 Strength criterion

For this criterion, the bearing capacity of a bolted connection is taken as the maximum load attained in the test regardless of the associated deformation. Many researchers ((Errera et al., 1974; Winter, 1956; Kim & Kuwamura, 2008; Dhalla et al., 1972; Rogers & Hancock, 1999; Puthli & Fleischer, 2001) who conducted experimental studies adopted this criterion to develop bearing design equations even though large deformations were observed at the ultimate load. For instance, Rogers and Hancock (1999) developed a bearing design equation for cold-formed carbon steel bolted connections by adopting the maximum loads from their tests, although in many specimens a level of deformation as large as 15 mm was reached. In the above-mentioned studies and in Kim's (1996) experimental work, the researchers observed that the failure was initiated by fracture at the edge of the elongated bolt hole at two symmetrical locations oriented at approximately $\theta = 45^\circ$ and 135° (see Figure 7) which confirms the occurrence of peak strains at these locations.

3.3.2 Deformation criterion

The bearing resistance of a connection according to this criterion is taken as the applied load measured at a pre-specified acceptable deformation depending on the usage of the connections. This limit does not correspond to the maximum load attained in the test and hence, no rupture takes place in the material. The definition of bearing failure in bolted shear connections by limiting deformations has been adopted by many researchers, but, there is no consensus about whether to limit the permanent or the total elongation and what is a suitable deformation limit. Perry (1981) investigated carbon steel bolted connections, and recommended that the failure load be the load corresponding to a deformation of 6.35 mm, since beyond this level, the load-deflection curves of typical connections becomes virtually flat. Perry's definition has been adopted in developing design guidance for carbon steel connections in the AISC (2005). The Euro Inox (2006) design provisions for stainless steel connections were developed on the basis of a 3.0 mm deformation limit at ultimate loading conditions. By imposing this limit, it was suggested that the deformation at the service loads would be of the order of 1.0 mm. Eurocode 3 Part 1.4 (2006) adopted the same design provisions.

4. CURRENT DESIGN PROVISIONS

4.1 Design rules for net section failure

The design rules for cold-formed carbon steel connections covering net section failure in Eurocode 3 Part 1.3 (2006) have also been adopted in Eurocode 3 Part 1.4 (2006) and the SCI/Euro Inox Design Manual for structural stainless steel (2006) with no distinction being made between thick and thin plated connections. Resistance to net section fracture $N_{u,EC3}$ is given by Eqn. 1:

$$N_{u,EC3} = \frac{k_t A_n f_u}{\gamma_{M2}} \quad (1)$$

where k_r is a reduction factor defined by Eqn. 2, A_n is the net cross sectional area at the critical section, f_u is the material ultimate tensile strength and γ_{M2} is a partial safety factor

$$k_r = 1 + 3r \left(\frac{d_0}{u} - 0.3 \right) \leq 1.0 \quad (2)$$

where d_0 is the bolt hole diameter, r is the ratio between the number of bolts at the cross-section considered to the total number of bolts in the connection and $u = 2e_2$ but $\leq p_2$, where e_2 is the edge distance and p_2 is the spacing between bolts perpendicular to the loading.

The ASCE (2002) and AS/NZ (2001) Standards provide design rules for stainless steel bolted connections composed of thin cold-formed sheets which are similar to those recommended for carbon steel in the AISI Specification (1997). The nominal net section resistance is given by Eqn. 3, where symbols have been harmonised with those used in Eurocode 3.

$$N_{u,ASCE} = A_n f_t \quad (3)$$

where, for a single shear connection

$$f_t = \left(1.0 - r + 2.5r \frac{d}{p_2} \right) f_u \leq f_u \quad (4)$$

for a double shear connection

$$f_t = \left(1.0 - 0.9r + 3r \frac{d}{p_2} \right) f_u \leq f_u \quad (5)$$

in which d is the nominal bolt diameter. In the case of single-row bolted connections, p_2 is the width of the sheet.

Eqns 2 and 5 are based on Winter's formula, as modified by Popowich (1969).

4.2 Design rules for bearing failure

The ultimate bearing resistance of carbon steel connections in Eurocode 3 Part 1.8 (2006) is given by the following equation:

$$N_{b,EC3} = \frac{k_l \alpha_b t d f_u}{\gamma_{M2}} \quad (6)$$

where α_b is the smallest of α_d , f_{ub}/f_u or 1.0, γ_{M2} is a partial safety factor = 1.25. In the direction of load transfer, $\alpha_d = e_1/3d_0$ for end bolts and $\alpha_d = e_1/3d_0 - 1/4$ for inner bolts. In the direction perpendicular to load transfer, k_1 is the smaller of $(2.8e_2/d_0 - 1.7)$ or 2.5 for edge bolts and $(1.4p_2/d_0 - 1.7)$ or 2.5 for inner bolts, t is the plate thickness, d is the nominal bolt diameter, d_0 is the bolt hole diameter, e_1 is the end distance, e_2 is the edge distance and f_u and f_{ub} are the ultimate tensile strengths of the plate and bolt material respectively.

The SCI/EuroInox Design Manual and Eurocode 3 Part 1.4 adopt Eqn. 6 for stainless steel connections with a slight modification: a reduced ultimate tensile strength of the plate material $f_{u,red}$ obtained from Eqn. 7 is used in Eqn. 6 in place of f_u . This modification was proposed to limit bearing deformations at the ultimate and service loads to acceptable levels, while maintaining the format of the resistance equation and the bearing coefficients for carbon steel. These standards apply these provisions for both thick hot-rolled and thin cold-formed connections.

$$f_{u,red} = 0.5f_y + 0.6f_u \quad (7)$$

The ASCE (2002) and AS/NZS (2001) Standards provide design rules for bolted connections composed of thin cold-formed stainless steel but do not have rules for those made of thick hot-rolled stainless steel. These standards adopt the design provisions in the AISI (1997) for cold-formed carbon steel bolted connections given by the following equations:

$$\text{for single shear connections:} \quad N_{b,ASCE} = \left(\frac{e_1}{d} \right) t d f_u \leq 2.0 t d f_u \quad (8)$$

$$\text{for double shear connections:} \quad N_{b,ASCE} = \left(\frac{e_1}{d} \right) t d f_u \leq 2.75 t d f_u \quad (9)$$

5. PROPOSED DESIGN RULES

5.1 Net section capacity

Figures 4 and 5 show the results of the parametric studies for the thick and thin plate connections respectively. It is clear that for stainless steel bolted connections composed of either thick or thin plates failing in net section rupture, the nominal stress at the critical net section at failure is consistently greater than or equal to the material ultimate tensile strength. This suggests that the ultimate material tensile strength can be used without including a reduction factor, and hence the ultimate net section capacity $N_{u,prop}$ is then given by:

$$N_{u,prop} = A_n f_u \quad (10)$$

In order to verify the net section fracture prediction of Eqn. 10 and to determine an appropriate partial safety factor γ_{M2} , statistical analyses in accordance with Annex D of EN 1990 (2002) were performed (Salih et al. (2010)). Note that γ_{M2} is the partial safety factor applied in Eurocode 3: Part 1.1 to fracture resistance.

For the proposed nominal resistance of the net section, γ_{M2} was found to be about 1.1, which is less than the recommended value of 1.25 in EN 1993-1-1 but equal to the value applicable to hot-rolled carbon steel in the UK National Annex to EN 1993-1-1. Therefore, the ultimate design capacity for net section failure for hot-rolled and cold-formed stainless steel bolted connections can be taken as:

$$N_{u,prop} = \frac{A_n f_u}{\gamma_{M2}} \quad (11)$$

where $\gamma_{M2} = 1.1$. Note that, to ensure a ductile failure of a tension member, the ultimate resistance of the net cross section (Eqn. 11) should be greater than the yield resistance (gross area x yield strength) of the gross section.

5.2 Bearing capacity

In order to exploit the high ductility and strain hardening characteristics of stainless steel, the concept in the AISC Standard (2005) for bearing design will be adopted. Two bearing design equations will be proposed based on the requirement (or not) for a deformation limit under service loads. The first bearing design equation is for bolted connections where the deformation under service loads is not a design consideration. This equation will be developed by adopting the strength criterion to define the ultimate bearing capacity, as controlled by fracture. The second equation is for connections where the deformation under service loads is a design consideration, and therefore the equation will be developed by considering the deformation criterion to define the service load and consequently the corresponding ultimate bearing capacity. The ultimate strength of the stainless steel material will be adopted in the proposed design equations with a format similar to that of Eqn. 6.

5.2.1 Bearing capacity when deformation under service loads is not a design consideration

In this scenario the ultimate bearing capacity is the load that corresponds to bearing fracture $N_{b,frac}$ as discussed in Section 3. The corresponding bearing coefficients $\beta_{b,frac}$ defined by Eqn. 12, obtained from the parametric studies are plotted in Figure 10 – $\sigma_{b,frac}$ is the nominal bearing stress at fracture. It can be seen that for $e_2/d_0 > 1.5$ the bearing stress factor for all FE models is greater than β_1 as defined by Eqn. 13.

$$\beta_{b,frac} = \frac{N_{b,frac}}{t d f_u} = \frac{\sigma_{b,frac}}{f_u} \quad (12)$$

$$\beta_1 = 2.5 \left(\frac{e_1}{3d_0} \right) \leq 2.5 \quad (13)$$

Thus for $e_2/d_0 > 1.5$, the proposed bearing design equation is given by Eqn. 14. For values of $e_2/d_0 \leq 1.5$ further investigation is required.

$$N_{b, \text{frac}, \text{prop}} = \frac{2.5\beta_1 t d f_u}{\gamma_{M2}} \quad (14)$$

Figure 10: Bearing coefficient from parametric studies for strength criterion

5.2.2 Bearing capacity when deformation under service loads is a design consideration

In this second scenario, the ultimate bearing capacity of the bolted connection should be defined such that the deformation at the serviceability limit state is kept within an acceptable limit. Examining the load-deformation curves for all FE models, it was found that at 1.0 mm deformation the connections remain essentially elastic. Therefore, a service load corresponding to 1.0 mm deformation was adopted, and the corresponding ultimate bearing capacity $N_{b, \text{def}}$ (here controlled by deformation) was then obtained by assuming an average ratio of ultimate to service loads of 1.45. The bearing coefficients $\beta_{b, \text{def}}$ obtained from the parametric studies are given by Eqn. 15.

Figure 11 shows the resulting bearing coefficient $\beta_{b,def}$ from Eqn. 15 – $\sigma_{b,def}$ is the corresponding nominal bearing stress. The factor β_2 given by Eqn. 16 is proposed to provide a lower bound to the finite element data and plotted in Figure 11. For comparison, the bearing coefficient of Eqn. 13 is also plotted.

$$\beta_{b,def} = \frac{N_{b,def}}{tdf_u} = \frac{\sigma_{b,def}}{f_u} \quad (15)$$

$$\beta_2 = 1.25 \left(\frac{e_1}{2d_0} \right) \leq 1.25 \quad (16)$$

The ultimate bearing capacity for this category is therefore given by:

$$N_{b,def,prop} = \frac{2.5\beta_2 t d f_u}{\gamma_{M2}} \quad (17)$$

Eqn. 17 will ensure that the deformation at serviceability will be acceptable; there is, therefore, no need to conduct a separate check.

Figure 11: Bearing coefficient from parametric studies for deformation criterion

6. CONCLUSIONS

Validated FE models have been used to perform parametric studies to investigate the net section fracture and bearing behaviour of stainless steel connections under static shear load. The parametric studies, together with the test data, showed that the reduction factor k_r employed in Eurocode 3 Part 1.4 and the SCI/Euro Inox Design Manual is unnecessary. A design equation for the net section capacity of stainless steel connections, applicable to both thick and thin material was proposed, and a suitable partial safety factor was derived following reliability analyses in accordance with EN 1990.

The parametric studies showed that the fundamental difference in the bearing behaviour of stainless steel and carbon steel connections is that, while the load-deformation curve for carbon steel connections flattens off after the initiation and spreading of yielding, for stainless steel connections this curve continues to rise significantly owing to strain hardening. For this reason, the limiting deformations used to define the bearing capacity of carbon steel connections were found to be unsuitable for stainless steel connections. Different failure definitions have therefore been devised for stainless steel connections, and bearing design equations that cover two cases – one restricting and one ignoring serviceability deformations – have been proposed. These equations define the bearing capacity in terms of the material ultimate strength f_u instead of the so-called reduced ultimate strength $f_{u,red}$, and therefore, are consistent with the provisions for carbon steel connections.

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