Stainless steel structures – research, codification and practice

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ABSTRACT: This paper provides a summary of recent developments in research, codification and design practice surrounding the structural use of stainless steel, with an emphasis on structural stability. Stainless steel has a nonlinear material response and this directly influences the structural behaviour. Guidance on material nonlinearities in international design standards largely relates to the occurrence of traditional idealised plastic hinges, as seen in carbon steel structures. However, such hinges do not form in stainless steel structures. Design codes provide the design framework that structural engineers use to demonstrate structural safety. With stainless steel being a high value material, its efficient use is paramount, and this is a particular incentive to embrace the opportunities offered by advanced system-level analysis and design. The most accurate representation of the behaviour of a structure is achieved by directly allowing for instability, plasticity, residual stresses and initial geometric imperfections in a second order inelastic analysis. This paper presents an overview of recent developments in the stability design of stainless steel structures, due to be included in upcoming revisions of prEN 1993-1-4, AISC 370 and ASCE-8; inclusion of these new provisions for advanced analysis in these codes represents a huge opportunity for enabling more widespread application of stainless steel in structural applications. Recent applications of stainless steel in practice are also presented herein and, with a view to the future, opportunities for the application of advanced design and construction methods to stainless steel structures are explored.

1 INTRODUCTION

Stainless steel is a high-performance construction material that is synonymous with modern, resilient and sustainable construction. There is a wide variety of grades to suit a range of applications and demands. The initial cost of stainless steel is relatively high compared to carbon steel (typically 3-5 times); it is therefore particularly important that design codes make efficient use of the material properties based on a comprehensive understanding of the structural response and embrace advanced design methods that may deviate from traditional approaches. Increased availability and sophistication of advanced computational tools is transforming the landscape of structural design. Advanced analysis allows for more efficient, more accurate and safer design as well as the potential for more optimised design solutions.

Stainless steel has a nonlinear material response and this directly influences the structural behaviour. It is important that this is reflected in design methodologies to enable the safe and efficient assessment and design of stainless steel structures. Design codes provide the design framework that structural engineers use to demonstrate structural safety. As structural systems become more complex, the shortcomings of the current design standards become more marked, requiring an increased level of idealisation and design assumptions. Eurocode 3 is currently under revision and a new version of each part, including EN 1993-1-4 (prEN 1993-1-4, 2020) for the design of stainless steel structures, is due for publication in about 2023. Work is also currently ongoing on the preparation of the first American design standard for

hot-rolled and welded stainless steel construction, AISC 370 (AISC, 2020). These codes closely mirror their carbon steel counterparts (AISC, 2016; prEN 1993-1-1, 2020), but deviate where necessary to account for the differences in material behaviour between stainless steel and carbon steel and the resulting influence on structural behaviour (SCI, 2020; Baddoo & Francis, 2014; SCI, 2017). Additionally, new codified provisions for the design of steel and stainless steel structures by finite element (FE) analysis are in preparation for the upcoming prEN 1993-1-14 (prEN 1993-1-14, 2019); this will include detailed rules for FE analysis, material models, including a two-stage Ramberg-Osgood material model (Arrayago, Real & Gardner, 2015) for stainless steel and cold-formed steel (Gardner & Yun, 2018), equivalent bow imperfections for inelastic analysis (Walport, Gardner & Nethercot, 2020b) and CSM strain limits, enabling design by second order inelastic analysis to be carried out. The current absence of appropriate guidance on system-based design is a barrier to the optimal use of stainless steel in construction. These new international design provisions will enable the more accurate and sophisticated design of stainless steel structures, promoting their wider use in practice.

In this paper, the material stress-strain response of stainless steel is first discussed as this is fundamental to developing an understanding of the structural response at the cross-section, member and frame levels. With an emphasis on frame level design, Sections 3 and 4 provide an overview of the key developments introduced above for the stability design of stainless steel structures in prEN 1993-1-4 and AISC 370, respectively. In Section 5, recent applications of stainless steel in practice are presented and, with a view to the future, opportunities for the application of advanced design and construction methods to stainless steel structures are explored. Finally conclusions are drawn in Section 6.

2 MATERIAL NONLINEARITY

An accurate description of the material behaviour is crucial for the reliable assessment of a structural system. If material nonlinearity is considered in the global analysis of a frame, greater deflections ensue due to the loss of material stiffness (Walport *et al.*, 2019; Walport, Gardner & Nethercot, 2019; Gardner, 2019) and, neglecting the effects of material nonlinearity can result in over-predictions of system strength (Walport, Gardner & Nethercot, 2020b).

Stainless steel differs significantly from carbon steel in that its stress-strain curve is characterised by a rounded response with no sharply defined yield point. A number of material models have been developed to describe this nonlinear stress-strain behaviour, with the most widely used based on the basic Ramberg-Osgood formulation, as modified by Hill (Ramberg & Osgood, 1943; Hill, 1944). While this formulation provides an accurate representation for the degree of nonlinearity in certain regions of the stress-strain curve, depending on the choice of the strain hardening exponent, it does not, in general, provide an accurate representation of the full curve. This has led to the development of some two-stage Ramberg-Osgood models, notably by (Mirambell & Real, 2000; Rasmussen, 2003; Gardner & Nethercot, 2004; Gardner & Ashraf, 2006; Arrayago, Real & Gardner, 2015).

The modified Ramberg-Osgood model, given in Equations (1) and (2), where ε and σ are the strain and stress respectively, f_y is the yield (0.2% proof) stress, E is the Young's modulus, f_u is the ultimate stress, E_y is the tangent modulus at the yield (0.2% proof) stress, defined by Equation (3), ε_u is the ultimate strain, and n and m are the strain hardening exponents, is currently provided in EN 1993-1-4 (EN 1993-1-4:2006 + A1, 2015) and the Fourth Edition of the Design Manual for Structural Stainless Steel (SCI, 2017).

$$\varepsilon = \frac{\sigma}{E} + 0.002 \left(\frac{\sigma}{f_y} \right)^n \quad \text{for } \sigma \leq f_y \quad (1)$$

$$\varepsilon = 0.002 + \frac{f_y}{E} + \frac{\sigma - f_y}{E_y} + \varepsilon_u \left(\frac{\sigma - f_y}{f_u - f_y} \right)^m \quad \text{for } f_y < \sigma \leq f_u \quad (2)$$

$$E_y = \frac{E}{1 + 0.002n \frac{E}{f_y}} \quad (3)$$

An alternative form of the second stage is given by Equation (4), where $\varepsilon_{0.2}$ is the total strain at the 0.2% proof stress, equal to $0.002 + f_y/E$, which provides increased accuracy in the region of the ultimate stress, particularly for less ductile materials e.g. ferritic stainless steels; this is the model due to be included in the upcoming prEN 1993-1-14 (prEN 1993-1-14, 2019).

$$\varepsilon = \varepsilon_{0.2} + \frac{\sigma - f_y}{E_y} + \left(\varepsilon_u - \varepsilon_{0.2} - \frac{f_u - f_y}{E_y} \right) \left(\frac{\sigma - f_y}{f_u - f_y} \right)^m \quad \text{for } f_y < \sigma \leq f_u \quad (4)$$

The two-stage Ramberg-Osgood expression has also been applied to the modelling of stainless steel material behaviour at elevated temperature (Gardner *et al.*, 2010), where the second strain hardening exponent m (denoted m_θ at elevated temperature) can be defined explicitly with reference to the strength at 2% strain $\sigma_{2,\theta}$, the value of which is provided in structural fire design standards including EN 1993-1-2 (EN 1993-1-2, 2005). The elevated temperature stress-strain curve is forced to pass through $\sigma_{2,\theta}$ at 2% strain by defining m in the following manner (Liang *et al.*, 2019), where symbols are as previously defined with θ denoting elevated temperature:

$$m_\theta = \frac{\ln \left(\frac{\varepsilon_{u,\theta}}{0.02 - \varepsilon_{0.2,\theta} - \frac{\sigma_{2,\theta} - \sigma_{0.2,\theta}}{E_{0.2,\theta}}} \right)}{\ln \left(\frac{\sigma_{u,\theta} - \sigma_{0.2,\theta}}{\sigma_{2,\theta} - \sigma_{0.2,\theta}} \right)} \quad (5)$$

3 STABILITY DESIGN OF STAINLESS STEEL STRUCTURES IN EUROCODE 3

EN 1993-1-4 gives supplementary guidance for the design of stainless steel structures that complements the design rules given for carbon steel in EN 1993-1-1. The supplementary rules for stainless steel currently give no additional information for the global analysis of stainless steel structures. The guidance on material nonlinearities in EN 1993-1-1 largely relates to the occurrence of traditional idealised plastic hinges, as seen in carbon steel structures. However, such hinges do not form in stainless steel structures; instead zones of plasticity with gradually reducing stiffness, but often with peak capacities well in excess of the traditional plastic moment are exhibited (Theofanous *et al.*, 2014). Eurocode 3 is currently under revision; this section summarises the key changes to the codified provisions for the stability design of stainless steel structures due to be included in prEN 1993-1-4 (prEN 1993-1-4, 2020).

3.1 Material nonlinearity at frame level

Degradation of stiffness significantly affects the behaviour of a structural system and consequently, the distribution of internal forces and moments (Walport, Gardner & Nethercot, 2019). Figure 1 shows the ratio of moments from first order and second order plastic zone analyses using the two-stage Ramberg Osgood material model of an austenitic stainless steel portal frame plotted against the ratio of the secant modulus E_s to elastic modulus E of the most heavily stressed point of the frame for increasing load levels (Walport *et al.*, 2021). Depending on the location in the frame, ignoring material nonlinearity can result in both over-estimations and under-estimations of internal forces and moments. When the ratio of E_s/E is less than 0.2, these errors approach 10%; therefore, Equation 6, where E_s is the secant modulus as calculated using Equation 7, is proposed to define the limit beyond which members in the frame are deemed to remain predominantly elastic.

Figure 1. Ratio of internal moments from a plastic (SM_{MNA}) and elastic (SM_{LA}) analysis around the frame for an example 5×10 m austenitic stainless steel portal frame plotted against the ratio of secant modulus E_s to elastic modulus E at the most heavily stressed point in the frame.

$$\frac{E_s}{E} > 0.2 \text{ for elastic analysis} \quad (6)$$

$$E_s = \frac{E}{1 + 0.002 \frac{E}{\sigma} \left(\frac{\sigma}{f_y} \right)^n} \quad (7)$$

Considering geometric nonlinearities at the frame level, EN 1993-1-1 (prEN 1993-1-1, 2020) and other international steel design standards state that second order effects may be neglected provided $\alpha_{cr,sw}$ is greater than or equal to 10 when an elastic analysis is employed, with $\alpha_{cr,sw}$ being the factor by which the design loading on a frame has to be increased to cause overall buckling in a global sway mode. The nonlinear stress-strain behaviour of stainless steel results in greater deformations as the material loses its stiffness and, therefore, a greater susceptibility to second order effects (Walport *et al.*, 2019); a stricter requirement is therefore needed for plastic analysis. The influence of material nonlinearity on the sway stiffness of frames may be considered through the modified elastic buckling load factor $\alpha_{cr,sw,mod}$, as derived in (Walport *et al.*, 2019; Walport, Gardner & Nethercot, 2019; Walport *et al.*, 2021), and given by Equation 8, where K_s/K is the ratio of the secant lateral stiffness of the frame K_s at the design value of the load to the initial lateral stiffness K due to the influence of plasticity (i.e. as obtained from a first order plastic zone analysis) and Y is a factor that approximates the further loss of stiffness due to second order effects, taken from Table 1 (Walport *et al.*, 2021).

$$\alpha_{cr,sw,mod} = \frac{K_s}{K} Y \alpha_{cr,sw} \quad (8)$$

The method provides a consistent treatment of second order effects between elastic and plastic global analysis i.e. a limit of 10 on both α_{cr} and $\alpha_{cr,mod}$, deeming second order effects sufficiently small to be ignored if the amplification of the internal forces is no more than 10% of the original internal forces determined according to first order theory.

Table 1. Y factors.

	For single storey portal frames	For all other frames
Austenitic stainless steel	0.80	0.55
Duplex stainless steel	0.85	0.60
Ferritic stainless steel	0.90	0.65

3.2 Design by inelastic analysis

The most accurate representation of the behaviour of a structure is achieved through directly allowing for instability, plasticity, residual stresses and initial geometric imperfections through the use of second order (advanced) inelastic analysis with imperfections (i.e. GMNIA). The combined influence of residual stresses and geometric imperfections can be accounted for through the use of equivalent bow imperfections, as discussed in Section 3.2.1. Advanced analysis is commonly carried out using beam finite elements for ease of use and computational efficiency but these elements cannot capture local cross-section deformations. While shell finite elements are able to capture cross-section local buckling behaviour, they are computationally expensive. A practical solution is to use beam elements, with strain limits applied to simulate local buckling and control the level of plastic deformation and hence the capacity of the cross-section and ultimately the structure. In this approach (Fieber, Gardner & Macorini, 2020, 2019; Walport, Gardner & Nethercot, 2020a), the influence of material nonlinearity on the structural response is directly modelled through the definition of the full stress-strain curve of the material, while the strain limits, as defined by Equations (9) and (10), are taken from the continuous strength method (CSM), as further detailed in Section 3.2.2. Failure of the member/system is defined as the point at which either (1) the CSM strain limit is reached at any point in the system or (2) the peak load is attained during the analysis, whichever occurs first (Walport *et al.*, 2019).

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \frac{0.25}{\bar{\lambda}_p^{3.6}} + \frac{0.002}{\varepsilon_y} \quad \text{but} \quad \leq \Omega \quad \text{for} \quad \bar{\lambda}_p \leq 0.68 \quad (9)$$

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \left(1 - \frac{0.222}{\bar{\lambda}_p^{1.05}}\right) \frac{1}{\bar{\lambda}_p^{1.05}} + \frac{0.002(\sigma/f_y)^n}{\varepsilon_y} \quad \text{for} \quad 0.68 < \bar{\lambda}_p \leq 1.0 \quad (10)$$

where $\bar{\lambda}_p = \sqrt{f_y/\sigma_{cr,cs}}$ is the local slenderness of the full cross-section, where f_y is the yield (or 0.2% proof) stress and $\sigma_{cr,cs}$ is the elastic local buckling stress of the full cross-section (Gardner, Fieber & Macorini, 2019), ε_y is the yield strain, ε_u is the ultimate strain, σ is the maximum compressive stress at the considered cross-section, n is the strain hardening exponent and Ω is a project specific design parameter defining the permissible level of plastic deformation, with a recommended value of 15 to prevent excessive deformation (Fieber, Gardner & Macorini, 2019).

The design approach enables consistent and accurate exploitation of cross-section slenderness dependent levels of spread of plasticity, moment redistribution and strain hardening. Additionally, the continuous nature of the approach allows all cross-sections (i.e. those of all cross-section classes) to be designed in the same way. Design by inelastic analysis with strain limits allows for safe and reliable assessment of structures while maintaining computational efficiency and practicality for widespread use in industry.

3.2.1 Equivalent bow imperfections

Direct modelling of residual stresses in an analysis can present challenges to the designer and is not possible in many software packages; EN 1993-1-1 therefore provides ‘equivalent’ bow imperfections that implicitly account for the combined effects of geometric and material (i.e. residual stresses) imperfections. The equivalent bow imperfections given in prEN 1993-1-1 are for use with second order elastic analysis. These tabulated equivalent imperfections apply equally to second order elastic analysis of stainless steel members, provided the appropriate value of α is employed, though, due to the limiting slenderness in the stainless steel buckling curves not being a constant value, a revised equation, replacing the constant 0.2 with $\bar{\lambda}_0$, for back calculating the equivalent bow imperfection is needed, as given by Equation 11 (Walport, Gardner & Nethercot, 2020b), where α is the imperfection factor, $\bar{\lambda}$ is the relative slenderness of the member, $\bar{\lambda}_0$ is the limiting relative slenderness and M_{Rk} and N_{Rk} are the characteristic value of the moment and axial resistance, respectively.

$$e_0 = \alpha(\bar{\lambda} - \bar{\lambda}_0) \frac{M_{Rk}}{N_{Rk}} \quad (11)$$

It is generally not appropriate to use equivalent bow imperfections developed for elastic analysis in inelastic analyses, because the effects of plasticity can be double-counted and over-predictions (i.e. unconservative results) or under-predictions (i.e. conservative results) of buckling resistances can arise, depending on the form of the adopted material stress-strain curve (Walport, Gardner & Nethercot, 2020b). For design by second order inelastic analysis, equivalent bow imperfection magnitudes may be calculated using Equation 12, where L is the member length.

$$\frac{e_0}{L} = \alpha\beta = \frac{\alpha}{150} \quad \text{but} \quad \frac{e_0}{L} \geq \frac{1}{1000} \quad (12)$$

3.2.2 The continuous strength method

An elastic perfectly-plastic material model and the associated concept of cross-section classification underpin current metallic structural design codes. To overcome the shortcomings for application to materials that exhibit a high degree of nonlinearity and strain hardening, the continuous strength method was developed as a more rational alternative to cross-section classification (Gardner, 2008). The implications of the rounded stress-strain curve of stainless steel are that Class 4 cross-sections experience inelastic local buckling below the 0.2% proof stress while Class 1-3 cross-sections can benefit from the substantial strain hardening exhibited by stainless steel beyond the 0.2% proof stress. Depending on the type and proportions of the structural element or system, the nonlinear material response can lead to either a reduced or enhanced capacity relative to an equivalent component featuring an elastic, perfectly plastic material response (Gardner, 2019; Walport, Gardner & Nethercot, 2020b). In general, in strength governed scenarios, such as the in-plane bending resistance of stocky beams, the substantial strain hardening of stainless steel gives rise to capacity benefits, while in stability governed scenarios, the early onset of stiffness degradation results in reduced capacity.

The continuous strength method (CSM) is a deformation based design approach (Gardner, 2008; Gardner, Wang & Liew, 2011; Afshan & Gardner, 2013; Liew & Gardner, 2015; Buchanan, Real & Gardner, 2018; Zhao, Afshan & Gardner, 2017) that enables a rational exploitation of the spread of plasticity, strain hardening and element interaction in the design of stainless steel cross-sections. The benefit of considering element interaction is greatest for cross-sections where the slenderness of the flange and web differ most, while the capacity increase from strain hardening is of most benefit for more compact cross-sections and for the materials exhibiting the highest level of hardening i.e. the austenitic grades. The strain limits also control the level of moment redistribution, the benefit of which depends on the

proportions, loading and degree of indeterminacy in the structure. The magnitude of all these benefits will vary between structures but can be very significant and often up to 30%.

4 STABILITY DESIGN OF STAINLESS STEEL STRUCTURES IN AISC 370

The new AISC 370 Specification (AISC, 2020) will encompass the design, fabrication and erection of hot-rolled and welded austenitic and duplex stainless steel structures. The provisions closely mirror AISC 360 (AISC, 2016), but deviate where necessary to account for the differences in material behaviour between stainless steel and carbon steel and the resulting influence on structural behaviour (Baddoo & Francis, 2014; SCI, 2013). In AISC 370 (AISC, 2020), the influence of material nonlinearity and residual stresses can be accounted for in stability design by either (1) performing an elastic analysis but with reduced stiffness in the members or (2) performing an inelastic analysis. In the former case, the relative simplicity of elastic analysis is retained, while in the latter case, more accurate results are achieved.

4.1 Second order elastic analysis with stiffness reduction

Design by elastic analysis with stiffness reduction has been developed and widely used for carbon steel structures (Deierlein, 2003; Surovek-Maleck & White, 2004a, 2004b). More recently, this method of design has been extended to the design of stainless steel structures (Walport, Kucukler & Gardner, 2021) and is due to be incorporated into the upcoming AISC 370 (AISC, 2020) and ASCE-8 (ASCE, 2020) Specifications.

For design by second order elastic analysis, two stiffness reduction factors are defined: (1) τ_b , to account for the reduction in stiffness of members in compression due to the effects of yielding and residual stresses, and (2) τ_g , to account for the additional reduction in member stiffness due to combined compression plus bending. Additionally, for slender members, the τ_g factor results in a system design strength equal to τ_g times the elastic stability limit, which is equivalent to the margin of safety provided by the column buckling curves (Deierlein, 2003).

The proposed stiffness reduction factor τ_b for stainless steel, accounting for the combined effects of material nonlinearity and residual stresses, was derived directly from the Ramberg-Osgood expression, given by Equation 1. The stiffness reduction factor τ_b was taken as the ratio of the tangent modulus E_t to the elastic modulus E , where $E_t = d\sigma/d\epsilon$ and $\sigma = P_r/A$, where P_r is the axial compression and A is the cross-sectional area. To allow for the influence of residual stresses, the strain hardening exponent n was modified to an effective strain hardening exponent n_{eff} through calibration against FE results for the tangent flexural stiffness obtained from a W8×31 cross-section divided into 1440 monitoring areas, subjected to pure axial compression. The resulting stiffness reduction factor to account for the reduction in stiffness of stainless steel members in compression due to the effects of yielding and residual stresses is given by Equation 13, where P_r is the applied axial load, P_{ns} is the yield load and f_y is the yield stress, while the values of the effective strain hardening exponents n_{eff} are presented in Table 2.

$$\tau_b = \frac{1}{1 + 0.002n_{\text{eff}} \frac{E}{f_y} \left(\frac{P_r}{P_{ns}} \right)^{n_{\text{eff}} - 1}} \quad (13)$$

Since n_{eff} is a function of n , the varying degrees of roundedness of the stress-strain behaviour, as shown in Figure 2 for the typical grades of stainless steel (austenitic grade 301, duplex grade S32101 and ferritic grade 410S) is reflected in τ_b . The greatest reduction at low to moderate axial load levels occurs for austenitic stainless steel, mirroring the low limit of proportionality and the low value of the strain hardening exponent n .

Walport, Kucukler & Gardner (2021), calibrated τ_g against benchmark results from non-linear finite element analysis of pin-ended stainless steel columns, beams and beam-columns,

Table 2. Stiffness reduction factors τ for second order elastic analysis of stainless steel structures.

Code	Member type	$n_{\text{eff}} (\geq 2.5)$	τ_g	τ_b
AISC 370	Rolled or welded I-shaped sections buckling about the minor axis, and other sections not specified in this table	$0.45n$	0.7	$\tau_b = \frac{1}{1+0.002n_{\text{eff}}\left(\frac{E}{f_y}\left(\frac{P_r}{P_{\text{ult}}}\right)^{n_{\text{eff}}-1}\right)}$
	Rolled or welded I-shaped sections buckling about the major axis, welded box sections, and round HSS	$0.55n$		
	Rectangular HSS	n		
ASCE-8	All sections	$0.45n$	0.9

Figure 2. Stiffness reduction factor τ_b for typical austenitic, duplex and ferritic grades of stainless steel for I-shaped sections buckling about the minor axis in AISC 370 and for all sections in ASCE-8.

as well as sample frames. For AISC 370 and ASCE-8, values of 0.7 and 0.9 are required to sufficiently account for the influence of plasticity, respectively. The difference between the proposed value of τ_g for AISC 370 and ASCE-8 reflects the different cross-section force-moment interaction equation, the different column buckling curves and the different moment capacities between the two codes.

4.2 Second order inelastic analysis

As in prEN 1993-1-4, the most accurate representation of the behaviour of a structure is achieved through directly allowing for instability, plasticity, residual stresses and initial geometric imperfections through the use of second order (advanced) inelastic analysis with imperfections – see Section 3.2.

5 PRACTICE

With the growing need for sustainable and resilient structures, the use of high performance materials such as stainless steel, with its excellent durability characteristics, could help meet these future demands. Stainless steel has been used in construction since the 1920s; the earlier applications were predominantly chosen based on architectural appeal, for example in the cladding of the Chrysler Building in New York, while more contemporary applications take

Figure 3. Recent examples of the use of structural stainless steel in practice. a) Sanomatalo Building, Helsinki; b) MX3D bridge at Dutch Design Week. 2018.

advantage of the desirable durability and structural properties, two examples of which are shown in Figure 3, including the Sanomatalo Building in Helsinki, in which stainless steel was used to form an exterior bracing system (Figure 3a). With a drive for more sustainable construction, the opportunities for the increased use of stainless steel is clear and this is reflected in the annual consumption of stainless steel increasing at a compound growth rate far in excess of other materials (Baddoo, 2008). Further information of the use of stainless steel in construction and more examples of practical structural applications may be found in (Baddoo, 2008; Gardner, 2005; SCI, 2020).

Another emerging innovation in construction is metal 3D printing, also known as additive manufacturing (Gardner, 2019; Buchanan & Gardner, 2019). Additive manufacturing allows for highly optimised structures and engineered materials unlike those that can be achieved using traditional fabrication methods. Early research into the behaviour of metal 3D printed structural elements has been reported in (Kyvelou *et al.*, 2020) (Buchanan *et al.*, 2017), while a review of the methods, research, applications, opportunities and challenges for this technology has been presented in (Buchanan & Gardner, 2019). Wire and arc additive manufacturing (WAAM) has been used to create the world's first metal 3D printed bridge constructed by the Dutch company MX3D, as shown in Figure 3b (Gardner *et al.*, 2020). Additive manufacturing offers many opportunities for the construction sector, but there will also be fresh challenges and demands. The verification of the MX3D bridge has involved a significant amount of material, cross-section and structural testing, alongside advanced finite element simulations. This is a pioneering example of the potential of additive manufacturing and offers a new way of thinking for the design and verification of such structures.

Embracing the digital age will be fundamental for increased sustainability within the structural engineering industry and this will rely on greater emphasis on the use of advanced analysis, as well as inspection and load testing. Structural design is significantly dictated by the current state of design guidance; the current sparsity of appropriate guidance on system-based design and advanced analysis is a barrier to its use in achieving optimal design. It is essential to ensure that working in the digital world is comparable to the traditional well-established and widely accepted approaches in terms of accessibility, flexibility, complexity and productivity. This requires the novel system-level advanced analysis approaches to be embedded in the design environment through clear guidance. International codes of practice and design guides

form the core of that design guidance; inclusion of these new methods in the upcoming design codes presents a step-change for the future direction of structural design.

6 CONCLUSIONS

The nonlinear material stress-strain curve of stainless steel has a direct bearing on the structural response of stainless steel cross-sections, members and frames. With stainless steel being a high value material, its efficient use is paramount, and this is a particular incentive to embrace the opportunities offered by advanced system-level analysis and design. The most accurate representation of the behaviour of a structure is achieved by directly allowing for instability, plasticity, residual stresses and initial geometric imperfections in a second order inelastic analysis, but there is limited guidance to enable design on this basis in practice. As described in the present paper, the new provisions for design by advanced analysis in the upcoming second generation of the Eurocodes and the new American design standards will address this shortcoming, representing a huge opportunity for enabling more efficient and more widespread application of stainless steel in structural applications.

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