Imperial College London

S2

BEHAVIOUR OF STAINLESS STEEL FRAMES

Fiona Walport **Supervisor: Professor Leroy Gardner**

Department of Civil and Environmental Engineering – Imperial College London

INTRODUCTION

The current stainless steel design provisions have been developed following, and in conjunction with, the carbon steel design codes. Stainless steel however has significant differences in its material characteristics, and the primary aim of this study is to assess whether these differences are appropriately accounted for in the codes, to allow for the safe and efficient design of stainless steel structures.

A parametric study has been carried out with a range of frame geometries to assess the current design provisions and the assumption that it is sufficient to assume a linear-elastic material behaviour for the design of stainless steel structures.

STAINLESS STEEL

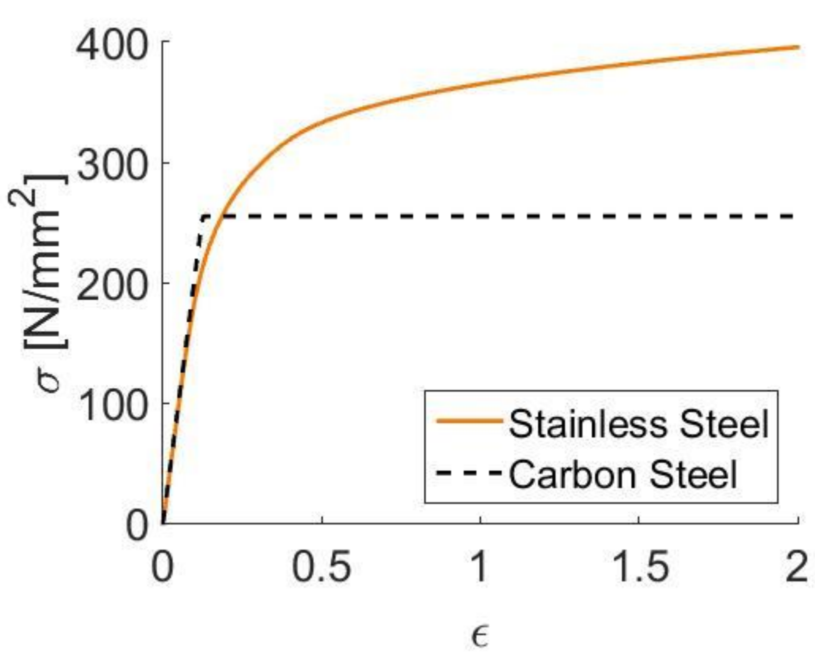

Fig. 1: Representative stress-strain curves.

Structural carbon steel can be modelled using an idealised bi-linear material stress-strain curve. Stainless steel however differs significantly in its response and is characterised by a non-linear rounded stress-strain response with no sharply defined yield point. The two-stage Rasmussen Ramberg-Osgood material model as stated in Annex C of EN 1993-1-4 (2006), was used to model the non-linear stress-strain relationship of Grade 1.4301 austenitic stainless steel in the numerical models.

NUMERICAL MODELLING

Fig. 2 shows the frame configuration modelled in this project. For each comparison, the width of the frame and ratio of vertical and horizontal loads remained the same with only the height of the frame altered. In changing the height, both the member slenderness and the critical load factor α_{cr} of the frame varied.

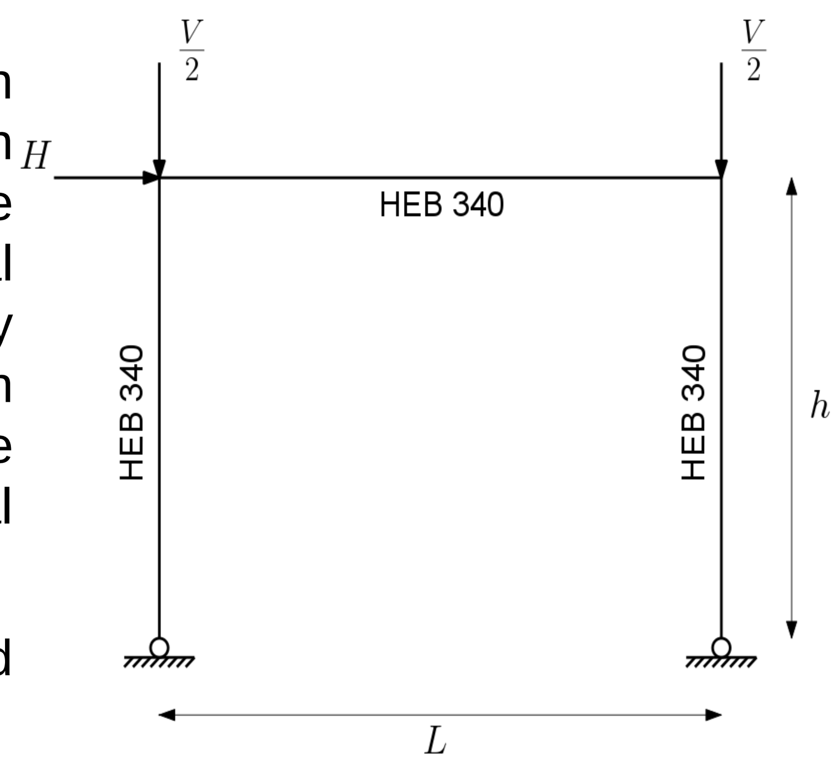

Fig. 2: Details of portal frame modelled.

The following analyses were carried out:

- First-order (LA)
- First-order with an amplification factor applied (LA + k_{amp})
- First-order; materially non-linear (MNA)
- Second-order (GNA)
- Second-order; geometrically and materially non-linear with imperfections (GMNIA)

BEAM-COLUMN MEMBER DESIGN

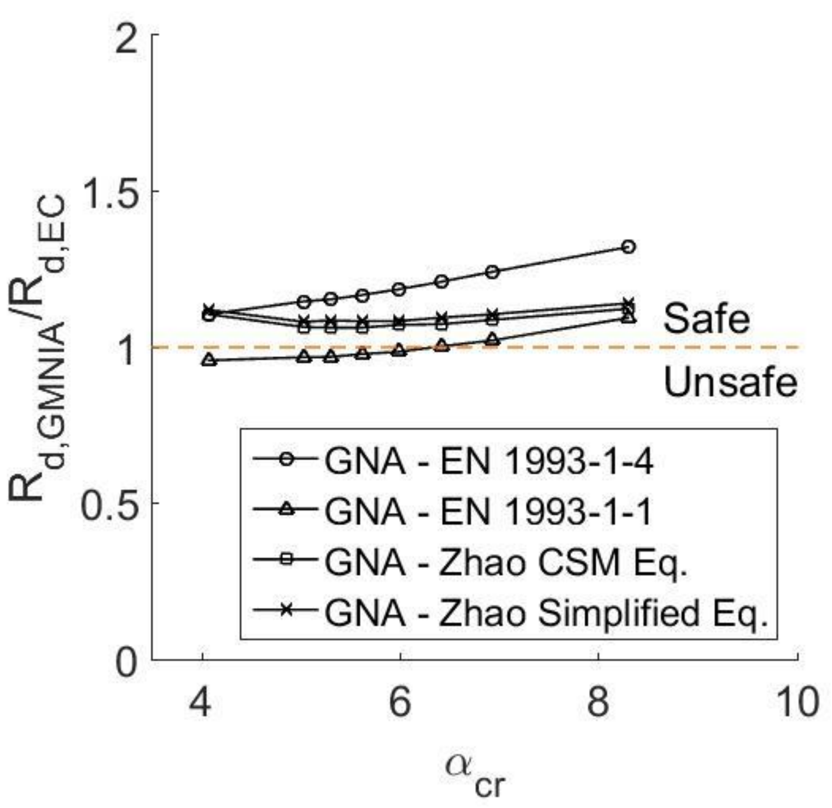

Fig. 3: GNA normalised by GMNIA for the pinned frame set with H=0.2V.

Using EN 1993-1-4 (2006) the member checks from the linear-elastic analyses can be seen to predict highly conservative results. This is due to the interaction factor k being restricted to a minimum value of 1.2 and the under-prediction of moment and axial capacities.

The comparison equations from EN 1993-1-1 (2005) and Zhao (2016) result in safe yet less conservative results than the stainless steel beam-column equations. Whilst EN 1993-1-4 (2006) has the minimum restriction for k, this is not the case with the alternative equations and therefore they result in less conservative strength capacities when moment governs.

ACKNOWLEDGEMENTS

I would like to thank Professor Leroy Gardner and Andreas Fieber for their invaluable help, support and guidance throughout this project.

SECOND-ORDER EFFECTS

The code states that a first-order linear analysis may be carried out when $\alpha_{\mathrm{cr}} \geq 10$. When $\alpha_{\mathrm{cr}} < 10$ the influence of the deformation of the structure must be taken into account through a second-order analysis. This can be confirmed in Fig. 4 by observing the increase in ratio of GNA to LA moments as α_{cr} decreases. In Fig. 5 the importance of second-order effects for low values of α_{cr} and in Fig. 6 the insignificance for high values of α_{cr} may be seen.

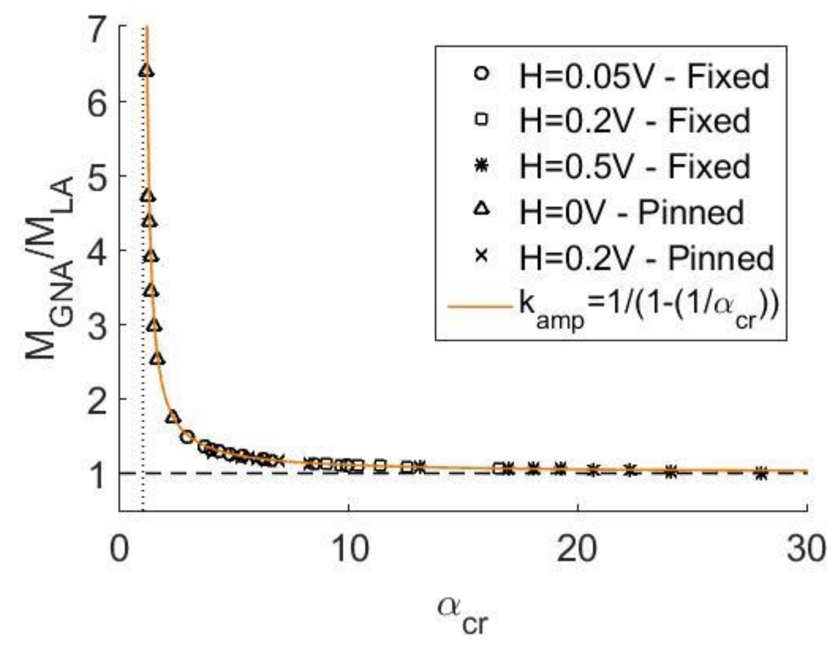

Fig. 4: Ratio of LA and GNA moments at the GMNIA collapse load.

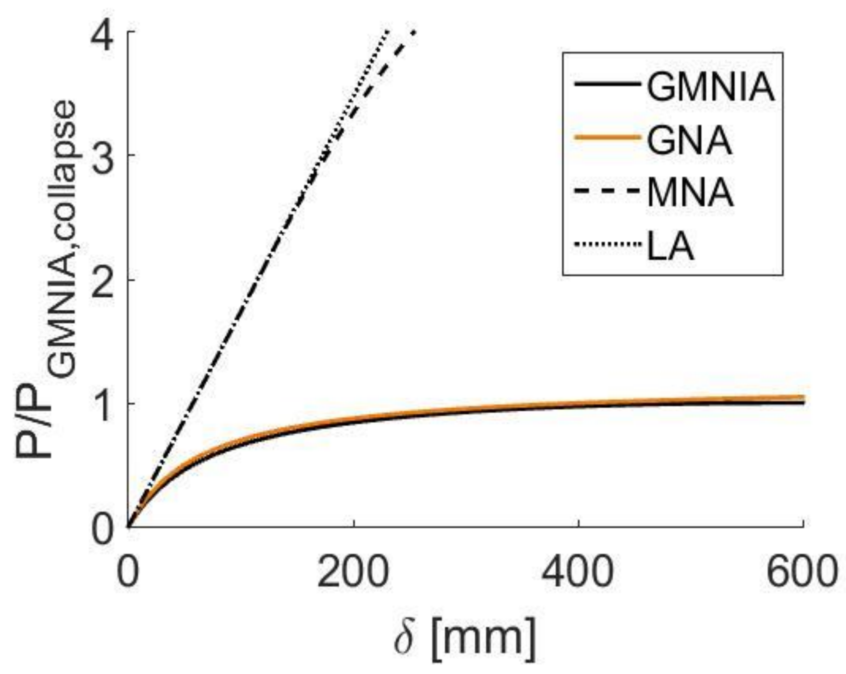

Fig. 5: 15x10 m frame with H=0, pinned boundary conditions and $\alpha_{\mathrm{cr}} = 1.16$.

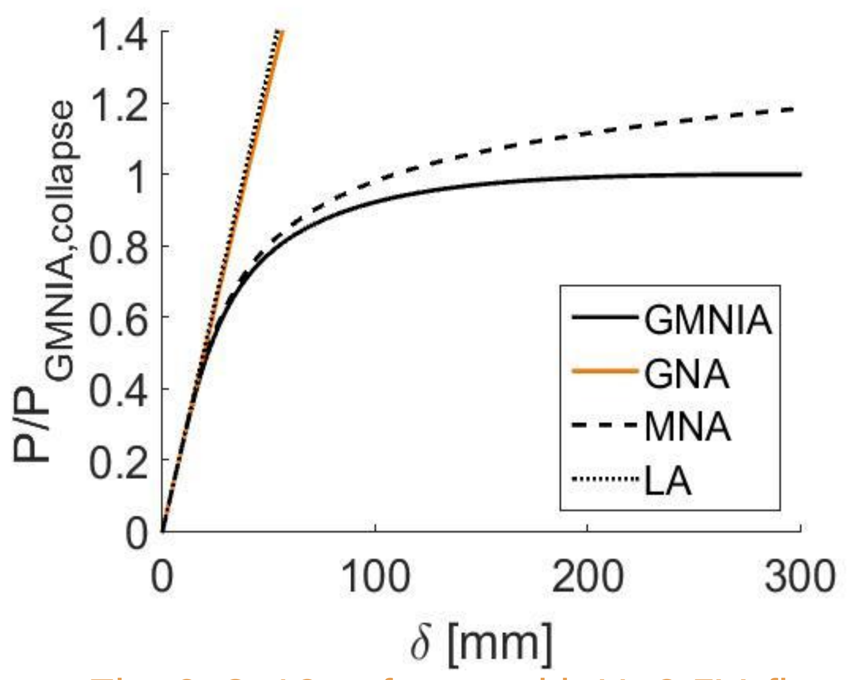

Fig. 6: 3x10 m frame with H=0.5V, fixed boundary conditions and $\alpha_{\mathrm{cr}} = 28.0$.

MATERIAL NON-LINEARITY

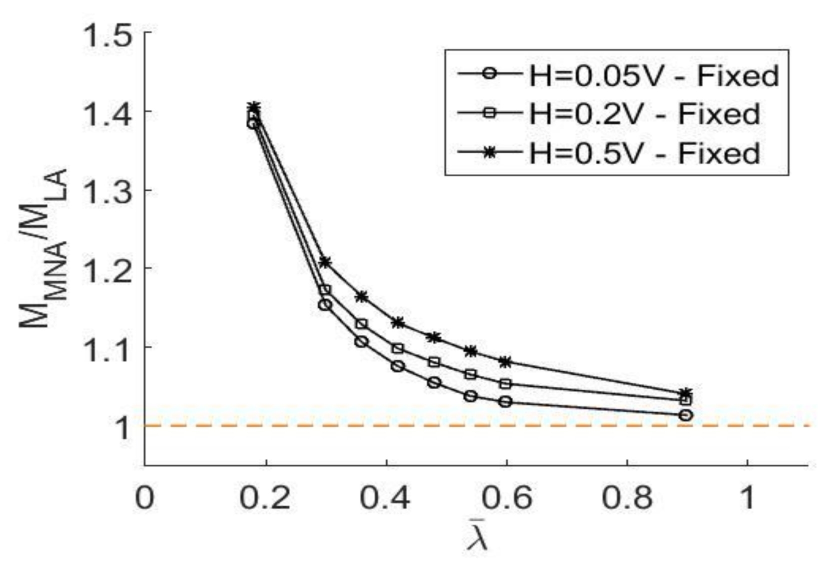

Fig. 7: Ratio of LA and MNA moments at the GMNIA collapse load.

The material non-linearity for stainless steel leads to significant amplification of the moments, and so load capacities, for low values of member slenderness $\bar{\lambda}$, with little effect at higher values.

This contradicts the results that suggest it is safe to assume a linear-elastic material model for stainless steel design.

CONCLUSIONS

The numerical modelling, undertaken using the finite element software Abaqus, confirms some key characteristics of frame behaviour, however shows that whilst assuming a linear-elastic analysis leads to safe strength predictions using EN 1993-1-4 (2006), the current beam-column design equations lead to highly conservative member capacities. The results are also compared with beam-column equations provided in EN 1993-1-1 (2005) and Zhao (2016), which are shown to be more efficient. Material non-linearity is shown to have a large influences as member slenderness decreases, however with the current codified equations the over prediction of the interaction factor leads to the predicted strength capacities, assuming a linear-elastic analysis, remaining safe.

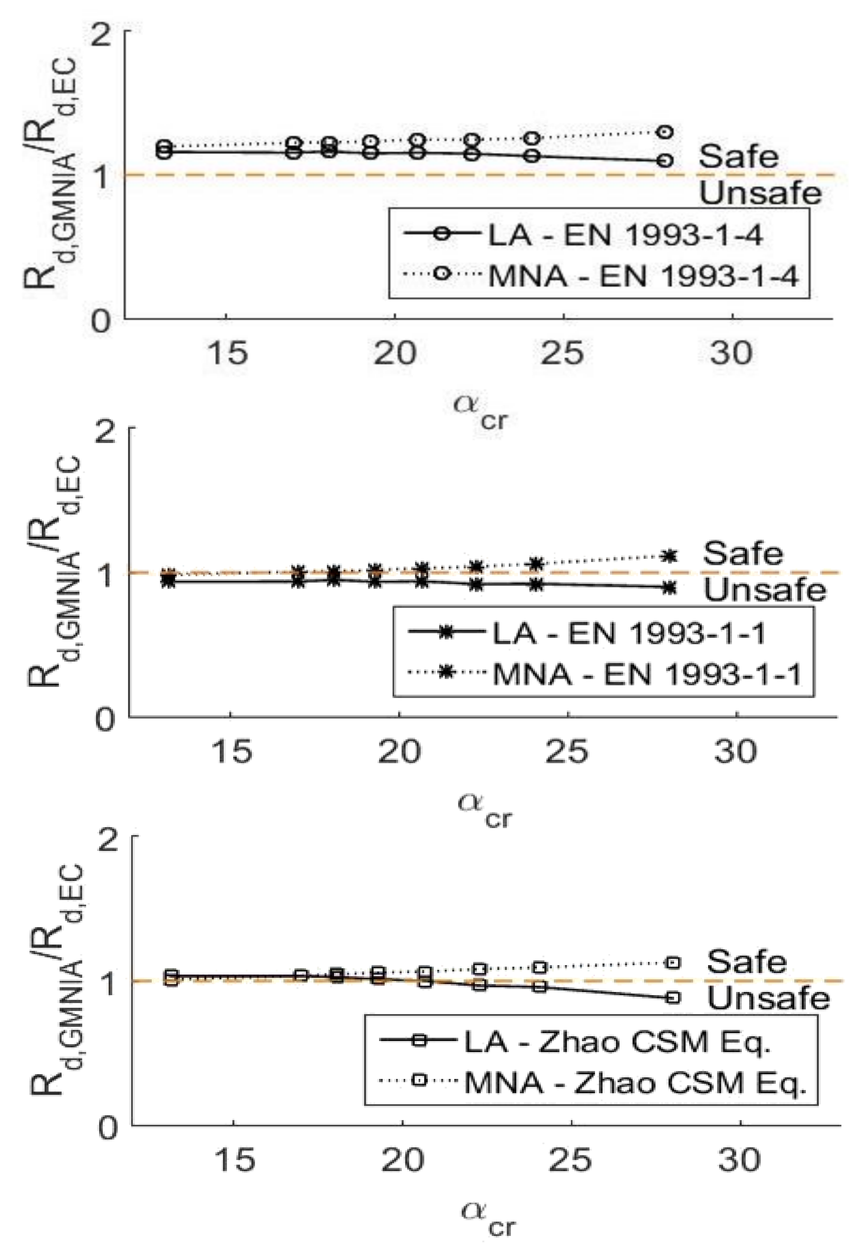

Fig. 8: : LA and MNA normalised by GMNIA for the fixed frame set with H=0.5V.

REFERENCES

EN 1993-1-1. 2005. *Eurocode 3 – Design of steel structures – Part 1-1: General rules and rules for buildings.* Brussels: CEN.

EN 1993-1-4. 2006. *Eurocode 3 – Design of steel structures – Part 1-4: General rules – Supplementary rules for stainless steels.* Brussels: CEN.

Zhao, O., Gardner, L., & Young, B. 2016. Behaviour and design of stainless steel SHS and RHS beam-columns. *Thin-Walled Structures,* 106, 330-345.