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DESIGN OF STEEL FRAME WITH VARIABLE CROSS-SECTION CONSIDERING STABILITY USING GENERAL METHOD ACCORDING TO EN 1993-1-1

Summary: In this paper, the advantages of the general method for verification of the lateral torsional buckling are demonstrated through the calculation of a real steel frame with variable cross-section in Germany. According to the Eurocode 3, the minimum load amplifier of the design loads and the minimum amplifier for the in plane design loads to reach the elastic critical load with regards to lateral or lateral torsional buckling are determined with the linear buckling analysis and the geometrically and materially nonlinear analysis using finite element method. The results show that shell models can create arbitrarily complex cross-sections and obtain more accurate results comparing to beam elements. Finally, the practical implementation of the reinforcement of steel frame is shown.

Keywords: steel frame, stability, variable cross-section, general method, numerical simulation.

1. INTRODUCTION

Welded frames with variable cross-section are commonly used in some steel structures, to save material and reduce costs. However, on the other hand, due to its complicated changes of cross-section along the length direction, it is difficult to accurately calculate the buckling resistance of frames using conventional methods (according to EN 1993–1–1 [1], Section 6.3.3) with hand calculation. In other words, the calculated buckling resistance of frames is an approximate result obtained by many simplifications. Of course, it is also possible to calculate the frame held by purlins and wall beams using the **G**eometrically and **M**aterially **N**onlinear **A**nalysis with **I**mperfections included (GMNIA). But this requires a wider knowledge of **F**inite **E**lement **M**ethod (FEM). An early example is given in [2].

The so-called “general method” is provided in EN 1993–1–1, Section 6.3.4 to carry out load-bearing capacity verifications against overall spatial failure, namely lateral torsional buckling. This approach can be used to analyze the spatial stability of steel frames with complicated thin-walled, predominantly open profiles and to determine the global individual solutions of overall frames and structures at risk of bending torsion [3]. Since there are no analytical solutions available for the exact calculation of elastic critical load and associated eigenmodes, an FE-based method of stability analysis and verification is optimal and easy to use in practice.

2. GENERAL DESCRIPTION OF REAL CASE STUDY

There is an industrial single-story steel building with a dimension of 48×36×6 m^3 [L×B×H], which built at 1990s in Berlin, Germany. Recently, according to the owner's request, this steel hall should be demolished as a whole and rebuilt in northern Germany. However, the new site is located in the "Norddeutsches Tiefland" area of Germany, which requires calculation of exceptional snow (accidental) loads with a partial safety factor 2.3. Therefore, the entire steel building needs to be re-calculated. Fig. 1 shows a sketch of one of the frames.

3. NUMERICAL MODEL

In this case study, the numerical analysis of steel frame with variable section made from steel S335 was conducted using the general-propose commercial finite element software Abaqus® [4]. First, each part of the frame (such as flange and web) is built using the shell model, and then all the parts are merged into a whole frame model in 3-dimension, as shown in Fig. 2. The bottom of the two frame columns is coupled with a central reference point with an all degrees of freedom fixed coupling connection. Then these two reference points are defined as fixed bearings. To simplify the influence of the connection and support between the frames, the out-of-plane direction support is defined at every 6-meter at the upper flange of the rafter, as displayed in Fig. 3.

In this example, the snow load and the roof weight are defined as linear load loading with a value 11.2 kN/m at the middle of upper flange of the frame rafter. In order to add linear load to the shell element, a circular beam element with a diameter of 1 mm is defined at the middle of the upper flange, as shown in Fig. 2. To consider the self-weight of the frame, a gravitational field along the opposite direction of Y is defined as 1.0×9800, where 1.0 is safety factor for permanent load and 9800 is the product of the earth's gravitational constant 9.8 and the dimensional conversion factor. In this model, mm, N, ton and s are used as the unit of consider the self-weight of the frame, a gravitational field along the opposite direction of Y is defined as 1.0×9800, where 1.0 is safety factor for permanent load and 9800 is the product of the earth's gravitational constant 9.8 and the dimensional conversion factor. In this model, mm, N, ton and s are used as the unit of length, force, mass and time respectively. Because the steel building is relatively low, the influence of wind load and welding residual stress is ignored in this study.

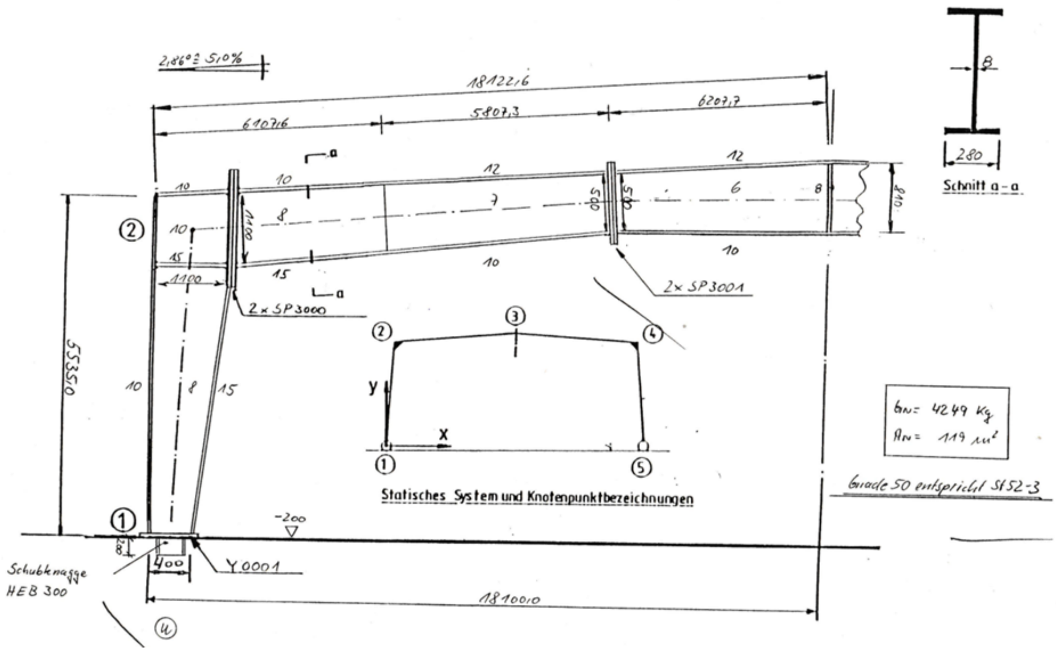

Figure 1. Sketch of a steel frame

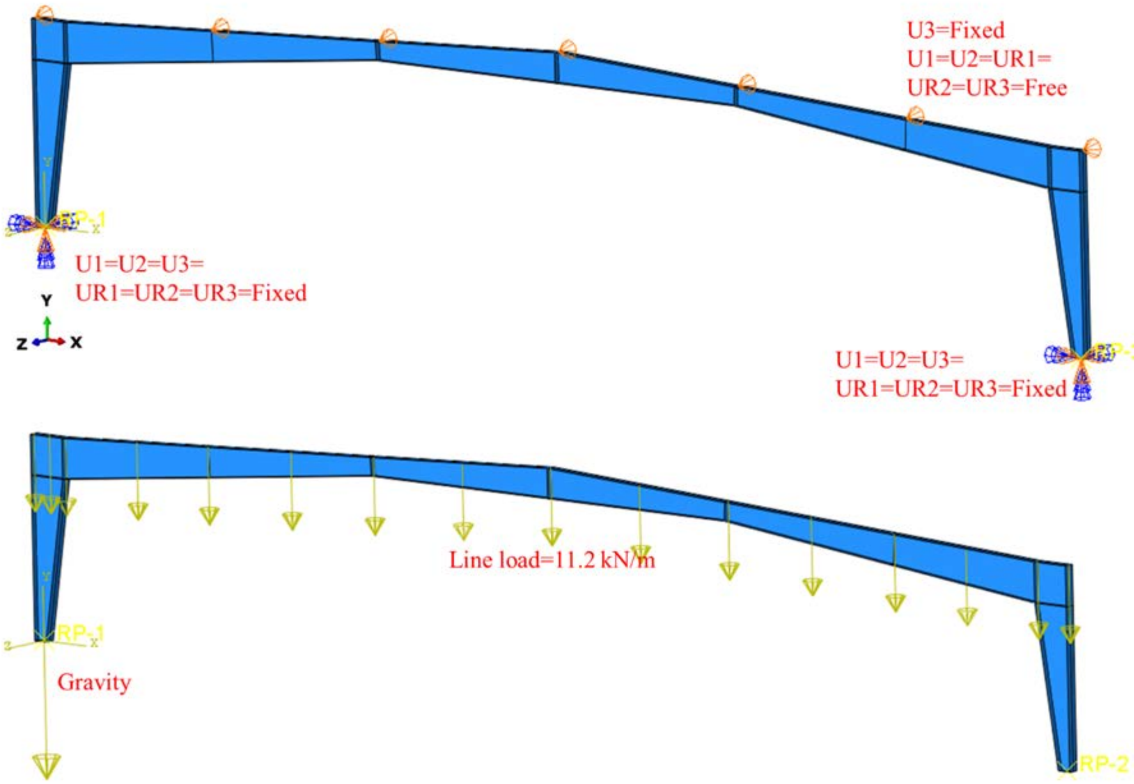

Figure 2. Numerical model and boundary conditions of the steel frame

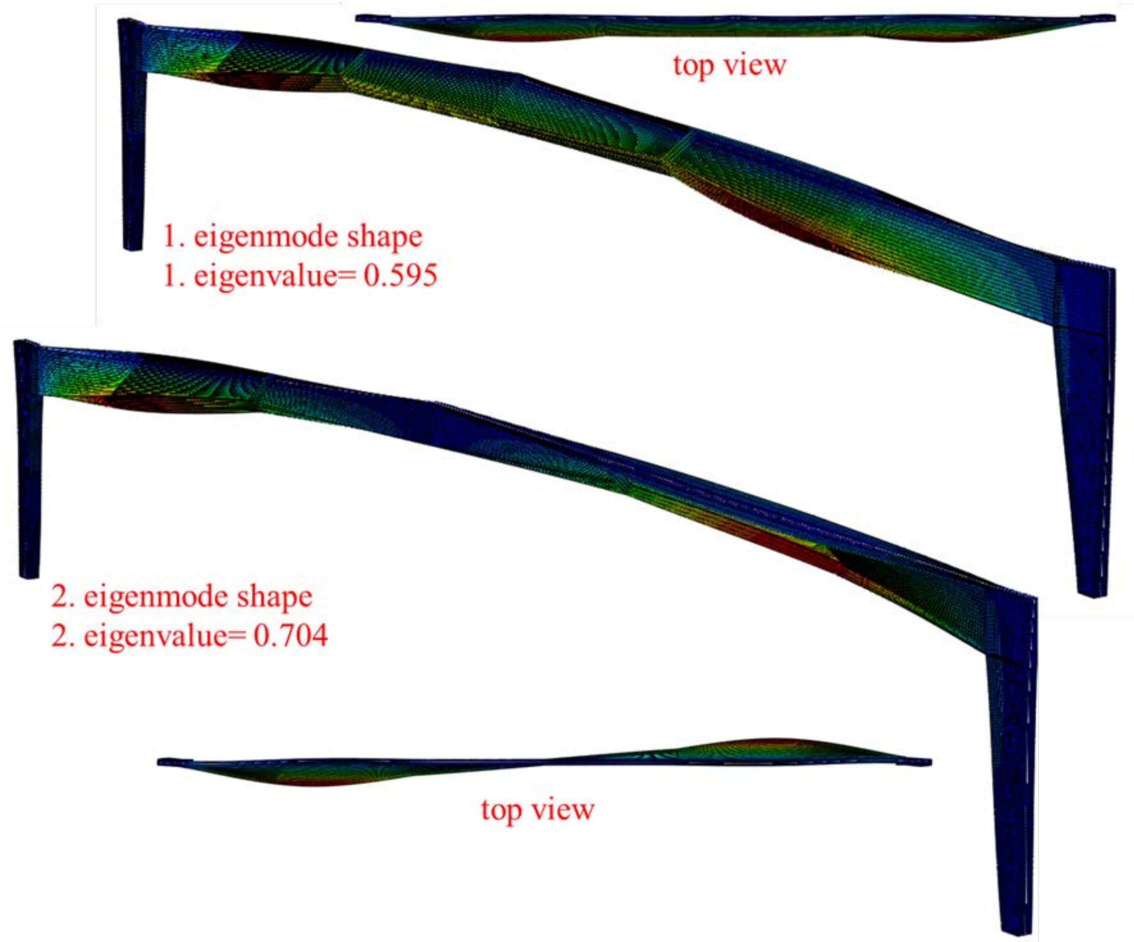

Figure 4. First two eigenvalues and eigenmode shape

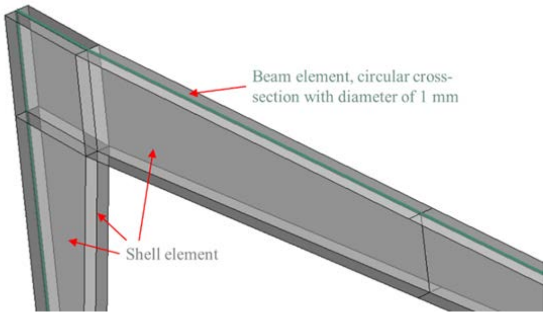

Figure 3. Element type of the steel frame

The non-linear elasto-plastic material model is employed and defined according to the corresponding design standard with a Young's modulus of 210 GPa and a yield stress of 355 MPa The hardening property of the steel is not considered, or in other words it is simplified as zero. According to the convergence study, the element size of 50 mm was used and the total number of shell element for whole frame is about 28 thousand.

In this study, the subspace iteration eigensolver is employed to analyze the eigenmodes and eigenvalues for Linear Bifurcation Analysis (LBA), as shown in Fig. 4. The nonlinear static problem is solved for geometrically and materially nonlinear analysis using Riks (arc-length) Method. Since the Riks approach allows you to find static equilibrium states during the unstable phase of the response. The minimum load amplifier α_(ult,k) of the design loads to reach the characteristic resistance of the most critical cross section without taking lateral or lateral torsional buckling and the minimum amplifier α_(cr,op) for the in plane design loads to reach the elastic critical load with regards to lateral or lateral torsional buckling can be determined with the LBA and GMNA. In this study, the min. eigenvalue or the 1. Eigenvalue is used as the load amplifier α_(cr,op) and the ratio of max. moment of the frames based on GMNA and the design bending moment is defined as load factor α_(ult,k), as shown in Fig. 5. It is worth mentioning that local imperfections are not considered here, as it is not a decisive factor. Hence, these two values are determined with numerical approach as α_(ult,k)=1.990 and α_(cr,op)=0.595, respectively.

4. DESIGN USING GENERAL METHOD

According to the regulations of general method in EN 1993-1-1, section 6.3.4, this approach can be used for the verification of the resistance to lateral and lateral torsional buckling where the conventional methods are not applicable.

Base on the Formula 6.64 in [1], the global non-dimensional slenderness λ ̅_op can be calculated as following:

$$\bar{\lambda}_{op} = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr,op}}} = \sqrt{\frac{1.990}{0.595}} = 1.829 \quad (1)$$

Therefore, the corresponding reduction factor for the non-dimensional slenderness χ_op can be determined with the formula of Lateral torsional buckling curves base on λ ̅_op=1.78,

$$\chi_{op} = \frac{1}{\Phi_{op} + \sqrt{\Phi_{op}^2 - \bar{\lambda}_{op}^2}} \quad (2)$$

where, $\Phi_{op} = 0.5 * \left[1 + 0.76 * \left(\bar{\lambda}_{op} - 0.2\right) + \ \bar{\lambda}_{op}^2\right]$
It can be determined that $\chi_{op} = 0.204$.

Hence, the overall resistance to out-of-plane buckling of these frames can be verified by ensuring with the formula 6.63 in EN 1993-1-1,

$$\frac{\chi_{op}\alpha_{ult,k}}{\Upsilon_{M1}} = \frac{0.204 * 1.990}{1.1} = 0.37 < 1.0 \quad (3)$$

where Ɣ_M1 means partial factor for stability, which is equal to 1.1 according to German National Annex [5]. Therefore, the steel building needs to be reinforced during the reconstruction at the new location.

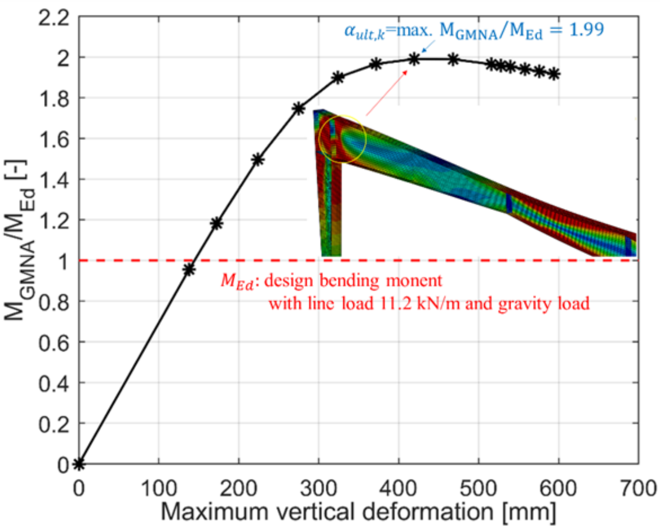

Figure 5. Moment vs max. deformation curve with GMNA

5. CONCLUSION AND DISCUSSION

The examples show that the steel frames with variable cross-section can be ensured by the verifications according to the general method for lateral torsional buckling with numerical approach. The shell model used can not only easily create complex arbitrary cross-sections, but also provide more accurate results

Figure 6. Reinforcement of steel frames

compared to the beam element. Such as, when the flange width of the frame beam is increased from 280 mm to 440 mm without changing other conditions in this example, local buckling will occur according to the first eigenmode and the 1. branch point load is not suitable for the subsequent overall buckling analysis of the steel frame. While using beam elements may ignore these issues.

Moreover, for the practical engineering problem, the flexural capacity of the frame columns is strengthened by the use of triangular additional structural members. Fig. 6 shows the implementation of the reinforcement.

Acknowledgements

The calculations were carried out by the authors. The client arranged the reinforcement of the structure.

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