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Direct buckling resistance check of complex steel plated structures

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Abstract
Finite element analysis-based design of steel structures is increasingly prevalent in applied engineering. However, comprehensive standardization was lacking in the past. To address this gap, a new Eurocode part – EN 1993-1-14 – has been developed to support the application of the numerical model-based design. The design rules are divided into two categories: (1) numerical design calculations and (2) numerical simulations. The first category addresses practical design scenarios, while the second allows for supplementing physical experiments. Within numerical design calculations, there are two sub-tracks: (a) analyses requiring subsequent design checks and (b) direct resistance check. The current paper focuses on the direct buckling resistance checks, where the ultimate buckling resistance is obtained directly from the numerical analysis, eliminating the need for additional analytical or empirical calculations. This analysis method is widely employed for buckling assessments of members, and segments, but not for complex structures (e.g. full bridges). The paper demonstrates its application on complex structures, specifically a steel railway arch bridge with a span of 100 meters (328 feet), where arch buckling is the critical failure mode. The study presents the methodology for direct buckling resistance check and its application to complex bridge structures. It includes details on development, verification, and validation of the numerical model, application and combination of equivalent geometric imperfections, as well as results from the buckling resistance check.

1. Introduction
The application of design by finite element analysis (FEA) is becoming increasingly prevalent in the daily design praxis; however, standardized design guidelines were limited in the past. The Eurocode 3 framework lacked a comprehensive design rule specifically tailored for FEA-based design of steel structures. Existing guidance were scattered across various sections of the code, such as Annex C of EN 1993-1-5 and EN 1993-1-6, which restricts their applicability and may lead to contradictions in several cases. In response to this gap, CEN/TC 250/SC 3 envisioned the consolidation of FEA-based design rules into a single, comprehensive document. This new standard aims to encompass all aspects of structural steel design, providing rules for utilizing FEA and other numerical methods to verify ultimate limit states, serviceability limit states, and fatigue.

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To achieve these goals, at first an Ad-Hoc Group (AHGFE) was established in 2016 with the general scope to further develop Annex C of EN 1993-1-5 into a more general and comprehensive Annex or an individual part on design assisted by finite element analysis. The main requirement was that this new document should cover the needs of all existing parts of Eurocode 3: Design of steel structures. In 2019 a new Working Group (CEN/TC 250/SC 3/WG22: "EN 1993-1-14 - Design assisted by FEM") was established replacing the AHGFE with the aim to continue the development process. Two documents were developed by the committee in the last years: (i) the new code EN 1993-1-14 and (ii) a Technical Report. The aim of the Technical Report is to give detailed background information and design guidance along with benchmark examples of the new design rules. In particular, the Technical Report contains guidelines and explanations for numerical modelling, selection of analysis type, model validation and verification and design checks of steel structures using finite element analysis. It also summarizes the research that underpins the design rules and provides a starting point for further development. Additionally, it features benchmark cases and worked examples to facilitate the proper application of the EN 1993-1-14 design rules and ensure model validation possibility to designers.

In the current paper, a unique case study is presented on an arch bridge, which demonstrates the application of the numerical model-based buckling resistance check following the design rules of the EN 1993-1-14. In the presented example the direct resistance check is used, which was commonly applied on structural members (e.g. buckling check of beams / columns or on separate joints) in the past, but not on complex structures (e.g. on full bridge models). The aim of the paper is to interpret and demonstrate the design possibilities provided by the new EN 1993-1-14 to designers. The background information and results of numerical modelling and the calculation results are presented here.

2. Design methodology of EN 1993-1-14

The scope of EN 1993-1-14 is to give rules on the application of finite element analysis and other numerical methods for verifying ultimate limit states, serviceability limit states and fatigue of steel structures. It contains rules on the following topics:

a. the modelling of structural components and boundary conditions,
b. the use of imperfections (geometrical imperfections and residual stresses),
c. the modelling of material properties,
d. the modelling of loads,
e. types of analysis,
f. verification and validation of FE models,
g. modelling of limit state criteria,
h. the harmonization of limit state criteria and the chosen modelling level and type of analysis,
i. the partial factors to be applied,
j. the choice of software and documentation.

The provisions of the new code consider the following design criteria: (i) plastic failure, (ii) fracture, (iii) buckling, (iv) fatigue, (v) serviceability. In the current paper only the basic design strategies and rules related to numerical model-based buckling resistance check are introduced.

Designers have several choices to develop numerical models and perform buckling resistance check. Harmonization of limit state criteria and the chosen modelling level and type of analysis is a crucial point of the correct design process. Therefore, EN 1993-1-14 introduces different design tracks, which gives the possibility to perform the analysis on different model and analysis levels. Design rules are separated into two main tracks based on the analysis purpose:

- numerical design calculations,
- numerical simulations.

Figure 1 gives a general overview of these tracks and the key components of each. For the application of the design rules given in EN 1993-1-14, it is important to determine first which corresponding track is required for the FE analysis purpose.

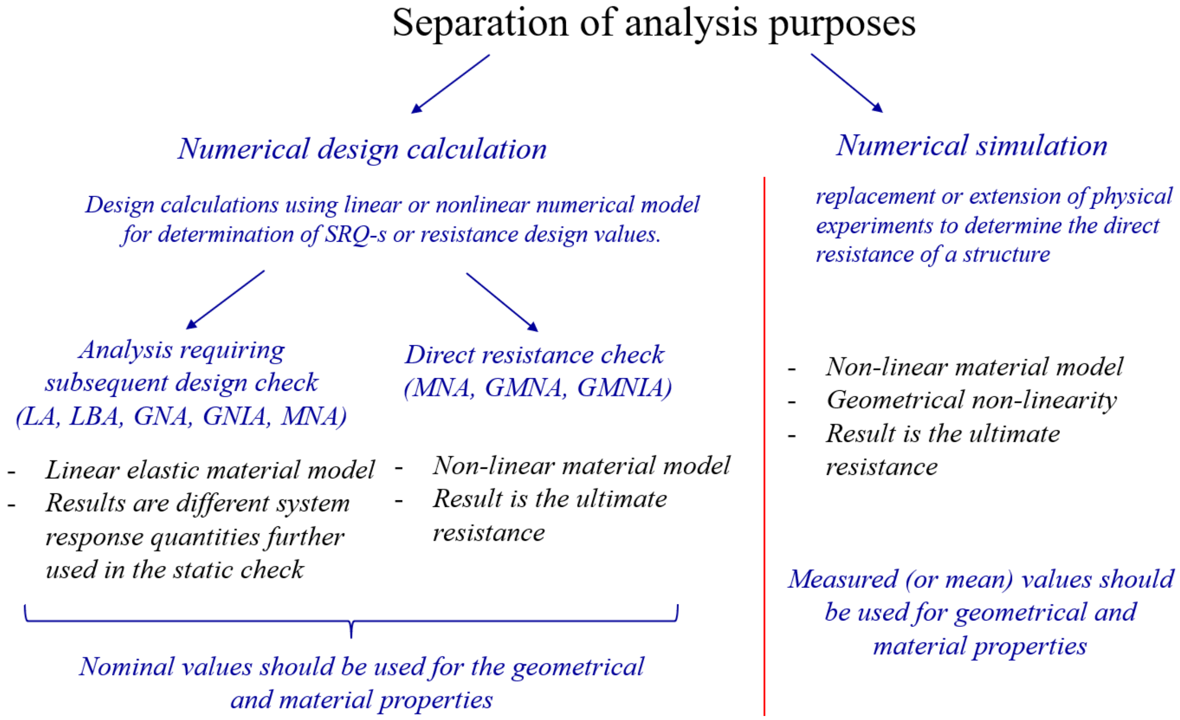

Figure 1: Numerical model and design tracks covered by EN 1993-1-14.

The first track, numerical design calculations, addresses practical design cases and is divided into two sub-tracks: (1) indirect resistance check - analysis requiring subsequent design check and (2) direct resistance check. In the indirect resistance check, the numerical model calculates System Response Quantities (SRQs) such as internal forces, deformations, critical load factors, and stresses. These SRQs are then incorporated into analytical design formulas to determine the structure's resistance, meaning the numerical model provides design parameters for further processing. This sub-track commonly employs analysis types like LA (linear analysis), LBA (linear bifurcation analysis), GNA (geometrically non-linear analysis), GNIA (geometrically non-linear analysis with imperfections), MNA (materially non-linear analysis).

The second sub-track is the direct resistance check, which directly determines the ultimate resistance of the structure through numerical simulation, eliminating the need for additional analytical or empirical calculations. This approach mandates the use of materially non-linear analyses, including MNA, GMNA, or GMNIA, to ascertain structural resistance. The selection of the analysis type hinges on the significance of second-order effects and imperfections on the structure's non-linear behavior and resistance. For instance, Geometrically and Materially Non-linear Analysis (GMNA) is suitable when imperfections have a negligible effect, as observed in tension members or plastically designed components without stability issues. Geometrically and Materially Non-linear Analysis with Imperfections (GMNIA), which includes both imperfections and non-linear analysis, is broadly applicable for determining structural resistance. Although these advanced analysis types are not yet common in daily practice, the EN 1993-1-14 standard aims to facilitate their wider adoption by practicing designers and R&D engineers. For numerical design calculations, model's geometric and material properties should use nominal values, and applied loads should reflect design values, including the relevant load factors and load combination factors, if relevant.

The second track, numerical simulation, primarily serves to supplement physical experiments and validate numerical models. Endorsed by EN 1993-1-14, this approach enables product development and R&D engineers to expand experimental datasets using accurate geometrically and materially non-linear numerical models and analyses. It is particularly beneficial for scenarios difficult to investigate through physical testing alone. For these simulations, geometrical and material properties should be derived from measured or statistically predicted mean values. Applied loads must also be defined by their mean (actual) values. Simulation results are to be treated identically to physical test results, with the same design rules applied for their evaluation. Their inherent uncertainties are managed through statistical evaluation of the test (or virtual FE test) results via partial safety factor evaluation or design resistance calculation.

In the current paper the application of the direct resistance check is desmonstrated.

3. Buckling resistance check methods of EN 1993-1-14

For buckling resistance check the following alternative methods are given by the EN 1993-1-14:

a) design by LA (or MNA, GMNA) and LBA,
b) design by GNIA in combination with LBA,
c) design by GNIA in combination with cross-sectional resistance, and
d) design by GMNIA.

These approaches are termed as alternatives, but the appropriateness and applicability of each approach should be checked by the designer for the specific case. In the following a short description of the different methods is enclosed.

Design by LA or MNA and LBA

Linear elastic analysis (LA) or material non-linear analysis (MNA) and linear elastic bifurcation analysis (LBA) may be used together to determine the relative slenderness of the investigated structure, that is related to the particular load case combination and loading situation. The LBA may be used to calculate the elastic critical buckling load (R_{cr}) of the structure. The LA or MNA, may be used to calculate the characteristic elastic or plastic resistance of the structure, depending on which one is appropriate. The calculated relative slenderness may be used to determine the design buckling resistance according to the rules of the appropriate buckling check.

Design by GNIA in combination with LBA

Geometrically non-linear elastic analysis with imperfections (GNIA) and linear elastic bifurcation analysis (LBA) may be used in conjunction to determine internal forces, stresses and the relative slenderness of the analysed structure related to the relevant load case combination. This method is applicable if several different buckling modes on the same structure should be checked. The GNIA is applicable only for in-plane buckling modes in accordance with the provisions of EN 1993-1-1. By using equivalent geometric and performing geometrically non-linear analysis, one or more buckling modes to be checked may be substituted. In this case the buckling check should not be executed based on the calculation of the overall relative slenderness and the relevant buckling reduction factor, but it may be undertaken by verification of the cross-sectional resistance. The principle underlying this method is the assumption that the stability check can be substituted by a geometrically non-linear imperfect analysis that calculates the second order internal forces or stresses. If the geometrically non-linear – second order – effects in individual members or certain individual member imperfections are not fully included in the global analysis, the stability of individual members should be further checked according to the relevant design rules.

Design by GNIA in combination with cross-sectional resistance

Design using GNIA in combination with cross-sectional resistance is an extended version of the previous design check. In this case, all possible imperfections associated with different buckling modes (in-plane and out-of-plane buckling together with the torsional modes) should be included in the numerical model leading to a complex model geometry with various imperfections (e.g. imperfection combinations). When equivalent geometric imperfections associated with all possible failure modes are defined in the model and GNIA is performed, the buckling check may be substituted by a simple stress check.

Design by GMNIA

Geometrically and materially non-linear analysis with imperfections (GMNIA) is employed to characterize a structure's behavior through its load-displacement path. This advanced analysis must account for both geometric imperfections and structural imperfections, such as residual stresses. These imperfections can be applied either separately or as equivalent geometric imperfections, which are then superimposed onto the perfect structure to represent all potential failure modes and deviations. When combining various equivalent geometric imperfections, all possible combinations, including different directions, must be evaluated, and the combination yielding the minimum resistance should be selected. All relevant load case combinations that induce compressive or shear membrane stresses also require consideration for buckling resistance, with a separate GMNIA performed for each. To determine the structure's ultimate resistance, the design values of applied load case combinations are increased by a load amplification factor, yielding an elastic-plastic load-deformation curve. The buckling resistance (R_{GMNIA}) is then derived directly from the calculated load-deformation path. For structures requiring a check against first yield (e.g. steel bridges), GMNIA or GMNA can still be used, but only the elastic portion of the load-displacement path is considered. This elastic part is identified by comparing results from GNIA (or GNA) with GMNIA (or GMNA). Finally, the characteristic buckling resistance ($R_{\mathrm{b,k}}$) is obtained by adjusting R_{GMNIA} with the model factor (γ_{FE}), as defined by Eq. 1:

$$R_{\mathrm{b,k}} = \frac{R_{\mathrm{GMNIA}}}{\gamma_{\mathrm{FE}}} \tag{1}$$

where γ_{FE} is the model factor considering uncertainties of the numerical model and the executed analysis type,

R_{GMNIA} is the calculated buckling resistance based on GMNIA.

The design buckling resistance ($R_{b,d}$) may be determined on the basis of the characteristic buckling resistance ($R_{b,k}$) divided by partial factor related to the relevant failure mode.
In the case of direct resistance check, the consideration of the model uncertainties has a special importance. Following the rules of EN 1993-1-14, the inaccuracies and uncertainties of the numerical model and the applied analysis method are generally accounted for by using the model factor (γ_{FE}). The model factor covers both the uncertainties of the numerical model and the executed analysis type. However, it does not override the application of any other partial safety factors, which are specified for the given buckling check. The application of the model factor is also linked to the analysis purpose, as summarized in Figure 2.

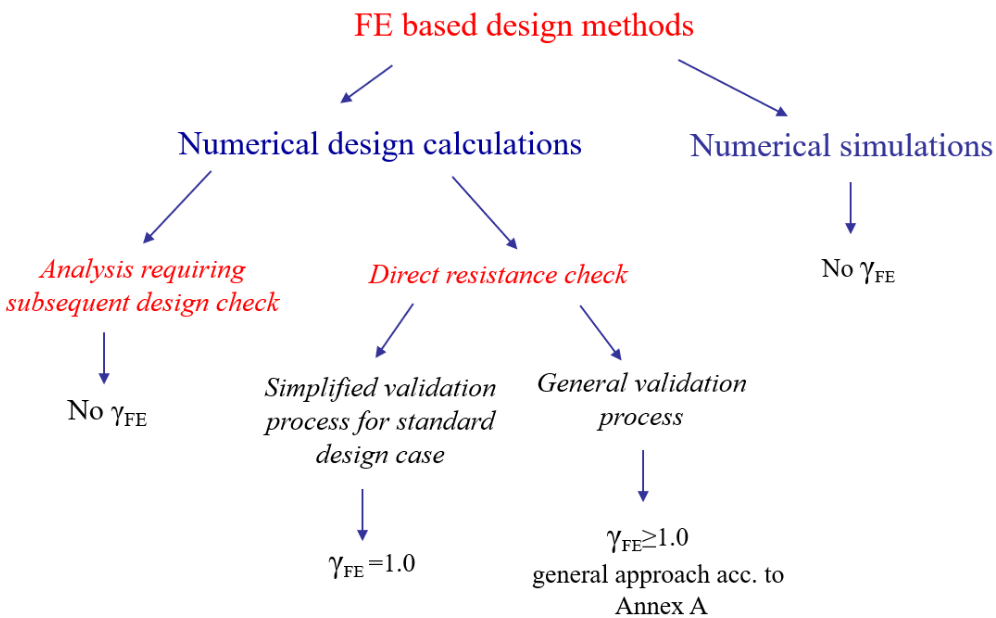

Figure 2: Application of the model factor (γ_{FE}).

When a numerical model is used for numerical design calculations requiring subsequent design checks, the uncertainty of structural resistance is solely covered by the partial safety factors defined in EN 1990: Basis of design, and all parts of EN 1993; thus, the model factor (γ_{FE}) should not be applied.

However, if the numerical model is employed for design calculations involving a direct resistance check – as in this paper – design uncertainty must be assessed using a combination of EN 1993 partial factors and the model factor (γ_{FE}). Similarly, for numerical simulations that use statistically distributed input parameters to determine test-based resistance (actual partial factors) according to EN 1990, the model factor is not applied. In this case the uncertainty of design by numerical simulation is treated in the same way as physical experiments according to EN 1990.

The model factor (γ_{FE}) can be determined by comparing numerical calculation results (R_{check}) with either known test results ($R_{test,known}$) or resistances obtained from established calculation methods ($R_{k,known}$). Its derivation method – general or simplified model validation – depends on the numerical model's purpose. For standard resistance checks, particularly when failure modes are verified against existing Eurocode-based resistance models using a verified numerical model, a simplified validation allows for a predefined γ_{FE} value without statistical evaluation. In such instances, including the current paper's example, the recommended value for γ_{FE} is 1.0. Current research efforts are focused on more precisely defining these simplified model factor values, recognizing that they can vary depending on the applied model level and studied failure mode.

4. Case study – direct resistance check

4.1 Arch bridge geometry and numerical model

The bridge under examination is a single-span, steel arch bridge with an orthotropic steel deck suspended by cables to the arch in a network layout. The bridge designed to carry 2x2 railway tracks, each superstructure carries the traffic load of two railway tracks. Notably, the right (northern) superstructure is 1 meter narrower than its left (southern) counterpart, resulting in a smaller corresponding substructure. The bridge features a 93.0 m span, with a width varying between 14.0 and 15.0 m at the two superstructures. The stiffening girder is a closed-section steel box girder; its dimensions are 1.5x2.2 m. The arch is a steel box-section, non-walkable, hermetically sealed. The two arches are connected and laterally stiffened at 6 places along the length of the bridge with D914x16 section bars. The arch is connected to the deck by cables, which are placed in a network layout. The deck is an orthotropic steel structure, the thickness of the deck plate is 22 mm, which is supported by flat steel stiffening ribs at every 400 mm. The distance between the crossbeams is typically 2650 mm, except for the two side crossbeams, where their distance is 2775 mm. The load-bearing steel elements of the structure are mainly made of S355 steel grade with characteristic yield strength (f_y) of 355 MPa and ultimate strength (f_u) of 490 MPa. The joint between the arch and the deck system is strengthened; the two sides of the arch is manufactured by using S460 steel grade with characteristic yield strength (f_y) of 460 MPa and ultimate strength (f_u) of 540 MPa. Figures 3-4 shows the typical cross-section and the side view of the structure. The developed numerical model and its details with the applied FE mesh is shown in Figures 5-6. The numerical model is developed in the Finite Element program Ansys 19.0 to perform the static check.

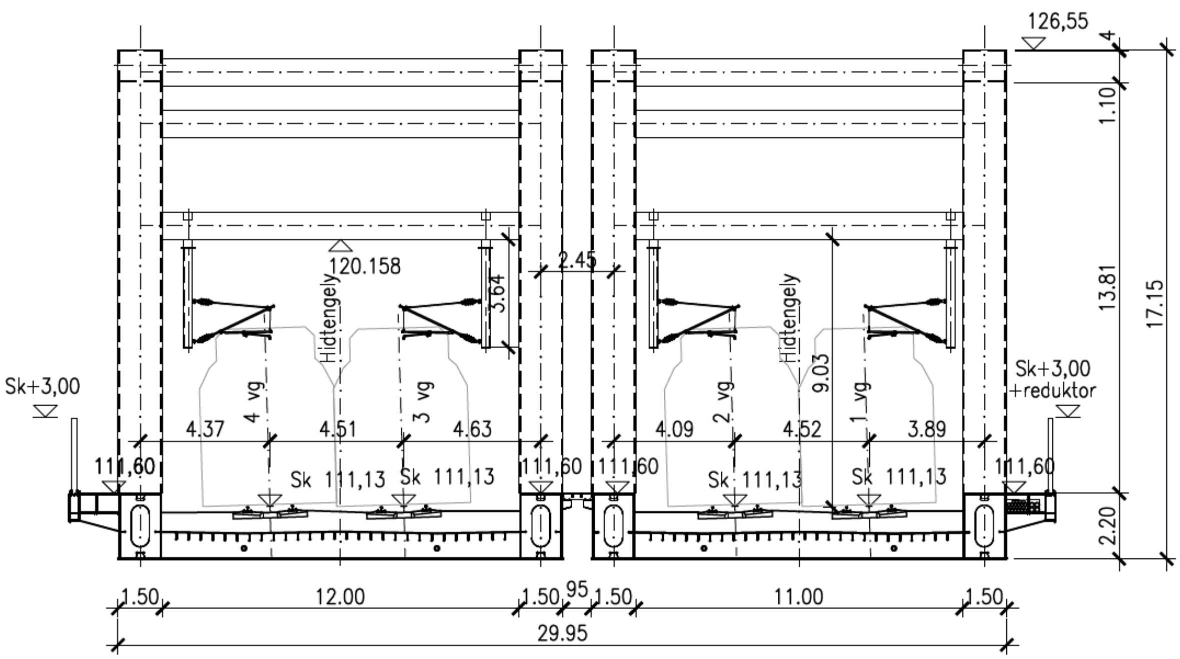

Figure 3: Typical cross-section of the bridge (designer: UVATERV Zrt.).

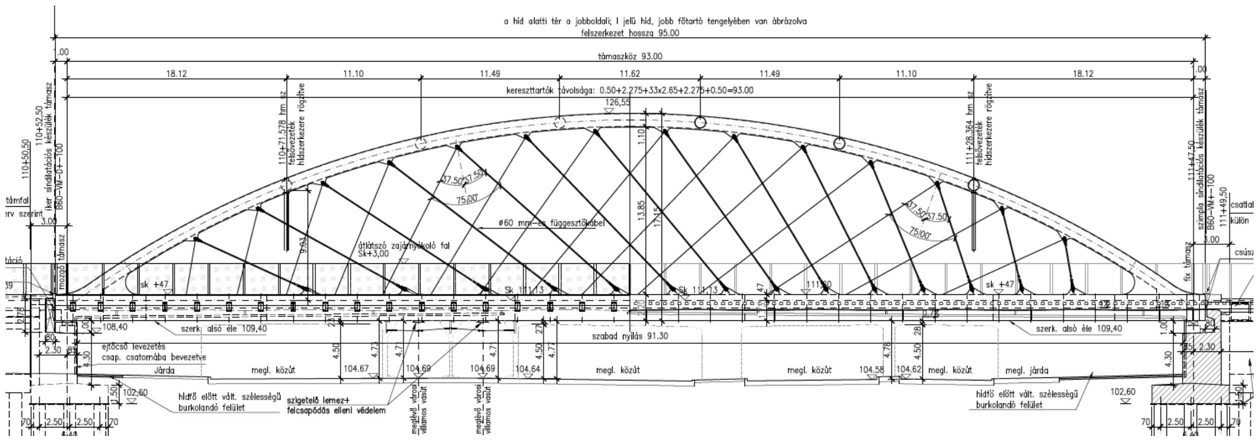

Figure 4: Side view of the bridge (designer: UVATERV Zrt.).

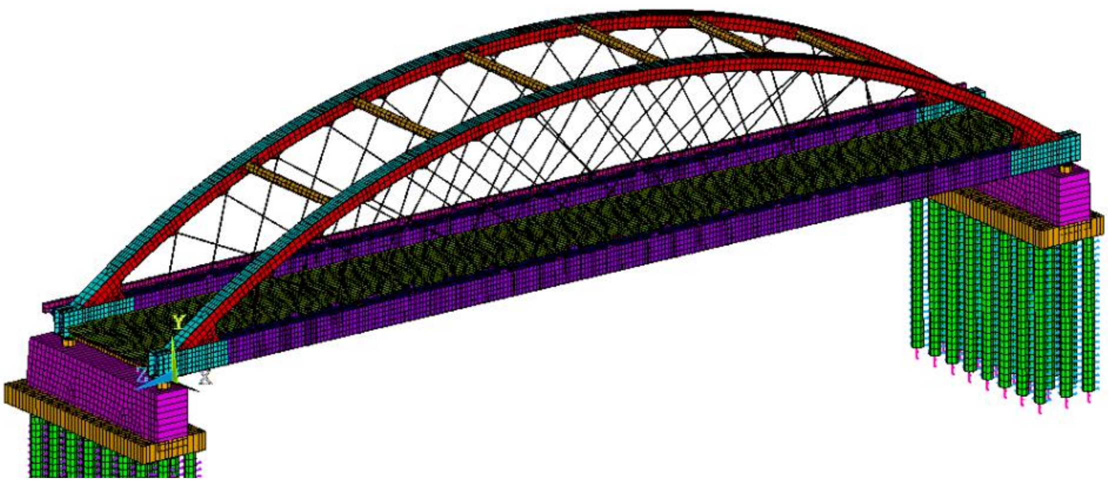

Figure 5: Global model used for numerical analysis.

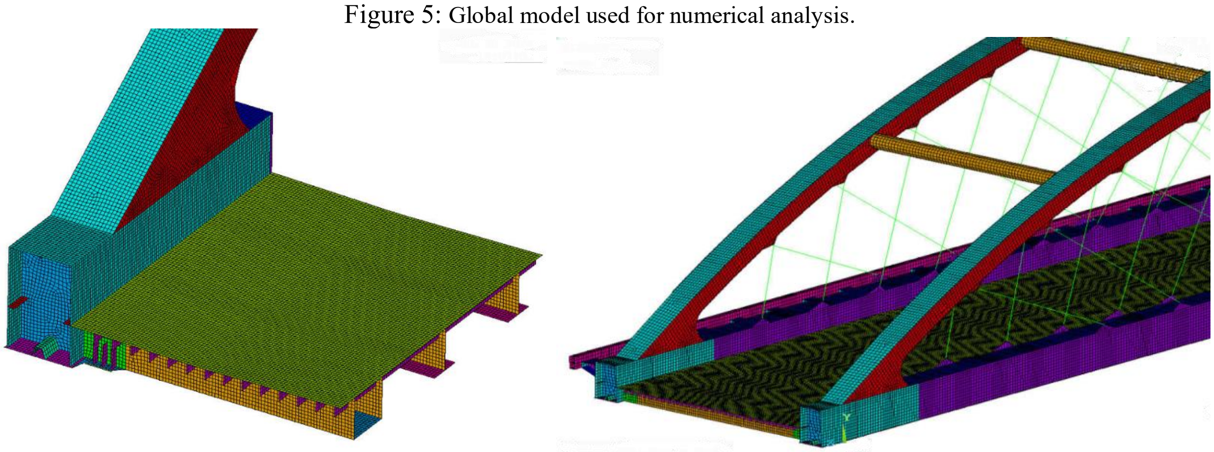

Figure 6: Details of the numerical model.

The numerical model is a combined shell and beam element model. For the superstructure, each plate element included in the shop drawings and participating in the global structural behavior of the bridge are modelled by shell elements. The hollow section arch, the stiffening girder (serving as tie bar), and the orthotropic deck are all modeled with shell elements. The suspension cables are modeled with tension-only beam elements.

4.2 Load and support conditions

The bridge has been loaded by the following loads according to EN 1991:

self-weight and dead load,
railway traffic load,
meteorological loads – wind and temperature.

By applying these loads, all the relevant load case combinations are created, which serve as the basis for stress check. Based on the results of the linear analysis and stress check for all relevant load case combinations, the ultimate one is selected and further considered in the buckling analysis. The results of only one load case combination will be presented here for demonstration purposes.

The primary goal when modeling support conditions is to accurately and conservatively reflect the behavior of physical supports and internal forces in the numerical model. The bridge's superstructure connects to the reinforced concrete substructure via bearings, which typically provide localized pinned support. Accurate modeling of these bridge bearings is critical for direct resistance checks. Localized load introduction points, if not handled correctly, can lead to significant stress concentrations and numerical singularities, potentially overestimating capacity reduction and yielding erroneous results. To counteract this, all stiffeners within the bridge bearing zone must be explicitly modeled to effectively distribute concentrated forces throughout the superstructure. Furthermore, implementing true pinned boundary conditions, free from unintended clamping effects, is essential, ensuring support acts through a single node. To prevent localized pinned supports from artificially reducing resistance, rigid links should be utilized within the bearing zone. These links connect all nodes in this area to the simply supported node. This comprehensive modeling technique effectively distributes loads, accurately simulates pinned behavior, and successfully avoids numerical singularities.

4.3 Material model

The material model applied follows a quad-linear model to capture the non-linear behavior of the steel in accordance with EN 1993-1-14. Figure 7 shows the material model graphically, while its mathematical formulation is given by Eqs. (2)-(6). This model is defined by three input parameters: elastic modulus (E), yield strength (f_y), and tensile strength (f_u). All the other measures of the material model are calculated from these input values. A standard elastic modulus of 210000 MPa and Poisson’s ratio of 0.3 are assumed for all analyses.

$$\varepsilon_{sh} = 0.1\frac{f_y}{f_u} - 0.055, \quad \text{but } 0.015 \le \varepsilon_{sh} \le 0.03 \tag{2}$$

$$\varepsilon_u = 0.6\left(1 - \frac{f_y}{f_u}\right), \quad \text{but } 0.06 \le \varepsilon_u \le \text{A} \tag{3}$$

$$C_1 = \frac{\varepsilon_{sh} + 0.25(\varepsilon_u - \varepsilon_{sh})}{\varepsilon_u} \tag{4}$$

$$C_2 = \frac{\varepsilon_{sh} + 0.4(\varepsilon_u - \varepsilon_{sh})}{\varepsilon_u} \tag{5}$$

$$E_{sh} = \frac{f_u - f_y}{C_2\varepsilon_u - \varepsilon_{sh}} \quad (6)$$

where: $\varepsilon_y = f_y/E$ is the yield strain,
ε_{sh} is the strain hardening strain,
E_{sh} is the strain hardening modulus,
A is the elongation after fracture (0.2 is used in the current analysis),
C_1 and C_2 are material coefficients.

For S355 steel grade the following values are applied: $\varepsilon_{sh} = 0.015$; $\varepsilon_u = 0.182$; C_1=0.31; C_2=0.448; $C_1 \cdot \varepsilon_u = 0.057$; $C_2 \cdot \varepsilon_u = 0.082$; E_{sh}=2310 MPa.

For S460 steel grade the calculated values are the followings: $\varepsilon_{sh} = 0.030$; $\varepsilon_u = 0.089$; C_1=0.505; C_2=0.604; $C_1 \cdot \varepsilon_u = 0.045$; $C_2 \cdot \varepsilon_u = 0.054$; E_{sh}=3407 MPa.

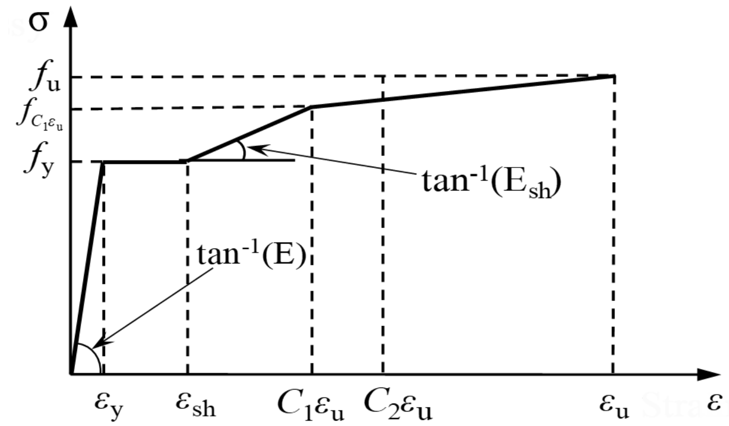

Figure 7: Material model according to EN 1993-1-14.

4.4 Imperfections and imperfection combinations

In the current analysis, equivalent geometric imperfections are applied due to the impractical complexity of incorporating residual stresses in a full bridge model. When employing equivalent geometric imperfections in a non-linear analysis, it is crucial to adopt imperfection forms corresponding to each investigated failure mode. In the current study buckling of the arch can have the following possible failure modes: (i) out-of-plane buckling in one sine wave form; (ii) out-of-plane buckling in two sine waves form; (iii) in-plane buckling in two sine waves form; (iv) local plate buckling of the compressed plates of the arch cross-section. To comprehensively represent these, four distinct imperfections, as illustrated in Figures 8-10, are incorporated into the numerical model. For each potential failure mode, the most detrimental imperfection, representing the realistic worst-case scenario, must be selected. The direction(s) of these imperfections (or imperfection combinations) should be chosen to identify the lowest structural resistance. If the critical imperfection direction is not self-evident or prescribed by regulations, various directions must be investigated. When multiple forms of equivalent geometric imperfections from different sub-groups (e.g. frame, member, and cross-section imperfections) are used, each imperfection is applied at its maximum amplitude and combined through linear addition, as is the case in this study. For equivalent cross-section imperfections in plated structures, their combination may be necessary. The leading imperfection is applied at its full amplitude, while accompanying imperfections are applied at 70% of their defined value. However, this specific rule is not applied in the current example. In the present example hand-defined imperfections with the following amplitudes are applied:

- global out-of-plane imperfection (maximum amplitude: $L_1/250$ – acc. to EN 1993-2, where $L_1=\sqrt{20 \cdot L[m]}$, L is the span length of the arch),
- global in-plane imperfection (maximum amplitude: L/500 – acc. to EN 1993-2),
- local plate buckling-type imperfection (maximum amplitude: min(a/200, b/200) – acc. to EN 1993-1-14; where a and b are the relevant length and width of the analyzed plate).

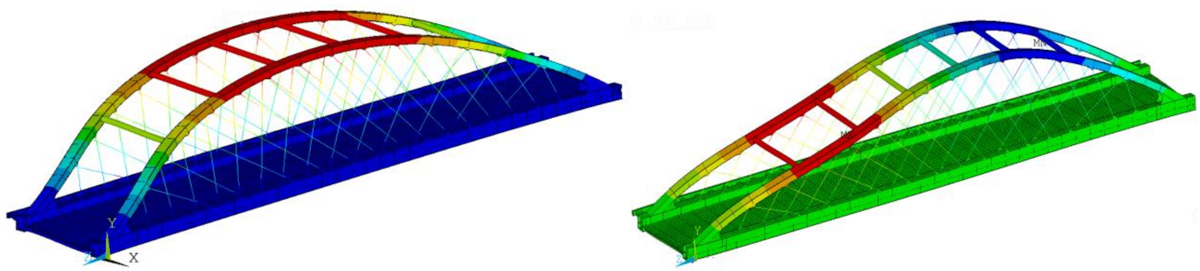

Figure 8: Out-of-plane global imperfections – a) one sine wave, b) two sine waves.

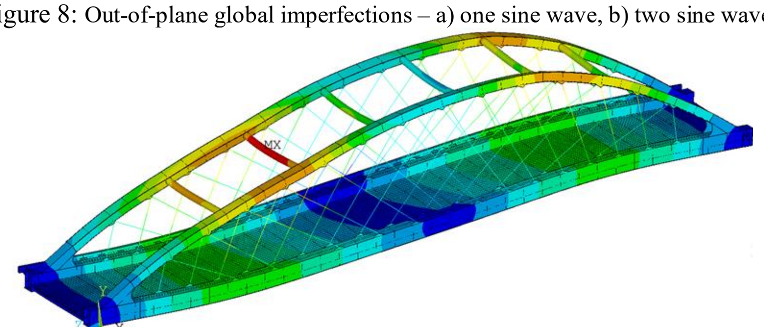

Figure 9: In-plane global imperfection.

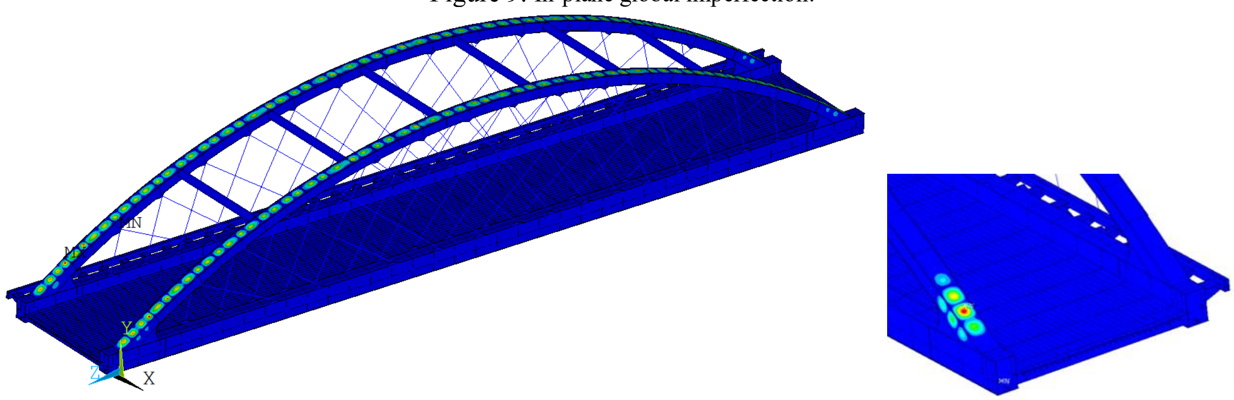

Figure 10: Local plate buckling-type imperfections.

4.5 Analysis type and evaluation strategy

Geometrical and material non-linear analysis with imperfections (GMNIA) is executed to determine the characteristic buckling resistance of the structure. This non-linear analysis generates a load-displacement path that illustrates the structure's behavior under specific boundary conditions, load case combinations, and accounting for elasto-plastic instability. A separate GMNIA is performed for each load case combination; this paper presents the analysis of one combination. To determine the complete load-deformation path, the structure is subjected to amplified design values of the applied load case combinations. The non-linear analysis utilizes a Full Newton-Raphson approach with a 0.1% convergence tolerance based on the Euclidean norm of residual forces. Prior to detailed evaluation, the numerical model's ultimate failure mode is thoroughly investigated to confirm the appropriateness of the applied load, the failure mode representation, and the comprehensive inclusion of all necessary imperfections. The observed failure mode, as shown in Figure 11, is a coupled buckling of the arch, integrating global out-of-plane buckling with local plate buckling, consistent with both expectations and the introduced imperfections.

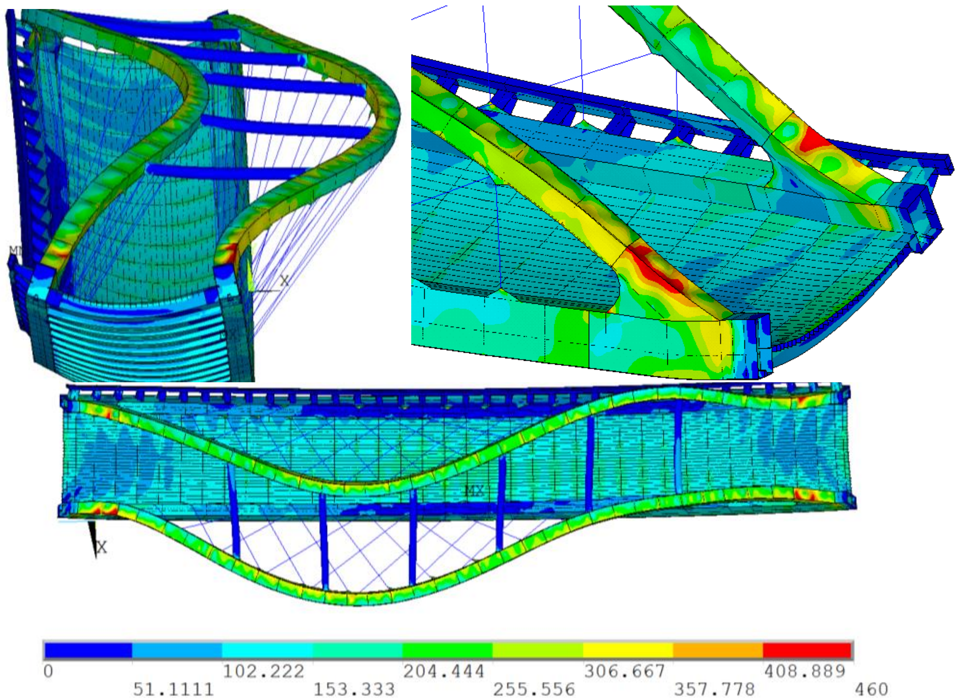

Figure 11: Obtained failure mode: deformed shape and von Mises stress distribution at ultimate load level [MPa].

4.6 Model verification and validation

The verification process encompasses a discretization error check (focusing on mesh density and convergence) and an engineering judgment of the calculation results. Figure 12 illustrates the mesh sensitivity study, where the horizontal axis represents applied mesh sizes from 100 to 500 mm. The blue dots, corresponding to the left vertical axis, show the ultimate load factors calculated by GMNIA, reflecting the highest load on the load-displacement path where convergence was achieved. In contrast, the red marks, referenced by the right vertical axis, indicate the critical load factors computed by LBA. Both sets of results demonstrate that reducing the mesh size leads to a decrease in the ultimate or critical load. The diagram also presents the converged GMNIA value, with dashed lines denoting mesh sizes that yield 1% and 5% differences from this value. Based on the discretization error check, a minimum FE mesh size of 150 mm is recommended, which introduces approximately a 1% resistance difference from the mathematically most accurately treated value. Notably, larger mesh sizes (around 400-500 mm) show a decreasing or equal ultimate load factor. This trend is attributed to the dimensions aligning with the sub-panel width, implying that larger elements no longer significantly influence the ultimate load as mesh size becomes dictated by the panel geometry. Despite this, such coarse calculations can still lead to inaccurate results. Diagrams also show the difference between the calculated and the mathematically most accurately treated solution can be large in case of the ultimate and critical load as well. In the analyzed case the maximum difference in the ultimate load is around 22%, and 31% in the critical load factor. These results prove the importance of the discretization error check, because both the ultimate and the critical loads are usually on the unsafe side by applying too large FE mesh sizes.

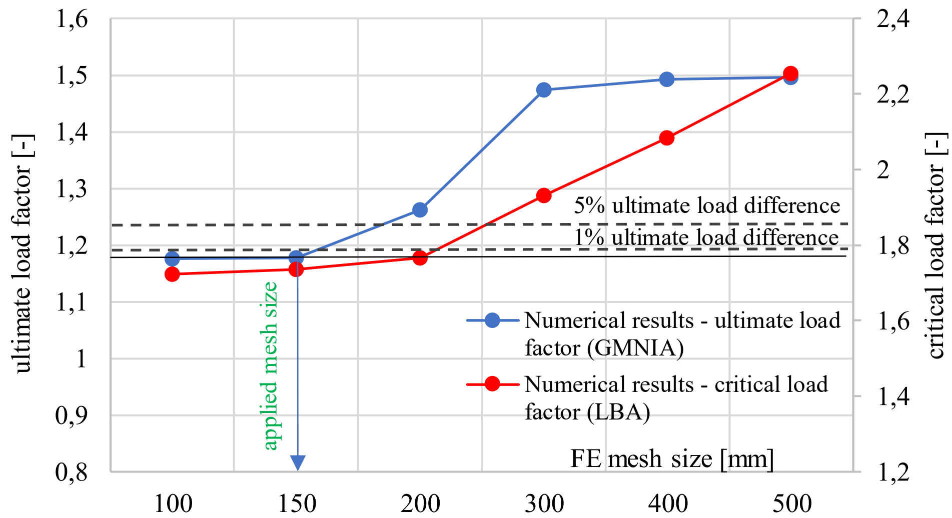

Figure 12: Result of the discretization error check.

Validation is the second step after verification, where benchmark cases should be adopted as a reference to check the accuracy of the numerical model and its application in the particular application field. Due to the inherent complexity of the full bridge structure investigated here, direct test-based validation is not feasible. However, the authors previously conducted comprehensive research on welded box-section columns, which demonstrated the same ultimate failure mode of local and global buckling interaction. In that prior work, the modeling technique, including finite elements, solver settings, and convergence criteria, was rigorously validated against experimental results. This earlier validation confirmed the numerical model's ability to accurately simulate column buckling behavior, ultimate forces, and axial deformations by utilizing measured material and geometrical properties. These validation efforts involved four distinct research programs, with further details available in (Radwan and Kövesdi 2022). After the numerical modeling technique is validated the determination of the model factor is the following step. If direct resistance check is applied for standardized failure mode (check of failure modes with existing Eurocode-based design resistance model) using standardized imperfections and verified numerical model as presented here above, simplified model validation can be applied using a predefined value for model factor. The proposed predefined value is equal to $\gamma_{FE} = 1.0$.

4.7 Analysis results

The imperfect shape of the analyzed structure is defined by superimposing equivalent geometric imperfections onto the perfect geometry, thereby capturing all potential failure modes and geometric deviations. This necessitates investigating multiple imperfection combinations, with a non-linear analysis performed for each. The minimum resistance derived from these analyses is subsequently chosen as the final design resistance. Imperfections, as detailed in Section 4.4, are combined following established rules. Since local and global buckling imperfections belong to distinct imperfection classes, they are handled separately and linearly added to form these combinations. To identify the minimum resistance, imperfection directions are also varied across the load case combinations. A total of 16 imperfection combinations, covering all physically possible scenarios, were studied. Table 1 summarizes these applied combinations, and Figure 13 presents their corresponding ultimate load amplifiers.

Table 1: Ultimate load factors using different imperfection combinations.

	Global imperfection			Local imp.	Load ampl. factor [-]
	Out-of-plane - 1 sine wave	Out-of-plane - 2 sine waves	In-plane - 2 sine waves	plate buckling	
1	1	-	1	1	1.232
2	1	-	-1	1	1.274
3	-1	-	1	1	1.284
4	-1	-	-1	1	1.329
5	-	1	1	1	1.178
6	-	1	-1	1	1.272
7	-	-1	1	1	1.247
8	-	-1	-1	1	1.199
9	1	-	1	-1	1.233
10	1	-	-1	-1	1.276
11	-1	-	1	-1	1.283
12	-1	-	-1	-1	1.330
13	-	1	1	-1	1.179
14	-	1	-1	-1	1.273
15	-	-1	1	-1	1.246
16	-	-1	-1	-1	1.199

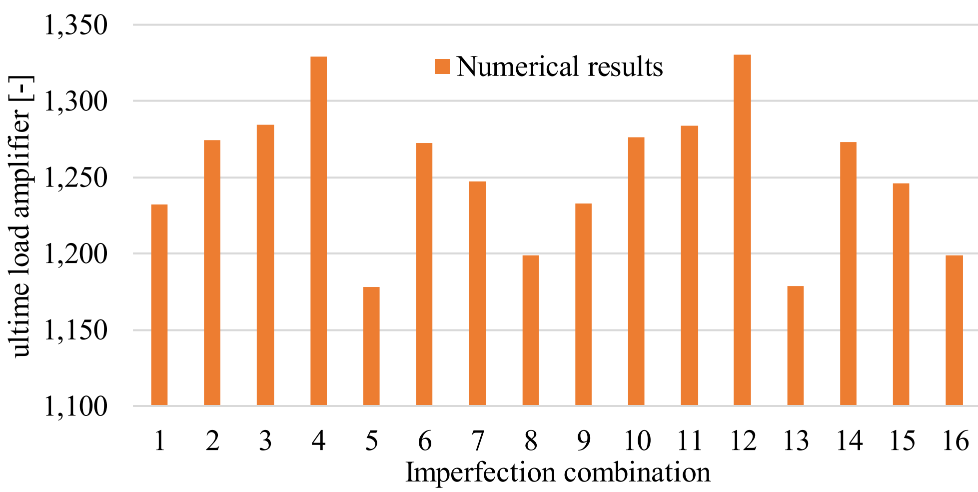

Figure 13: Ultimate load factors using different imperfection combinations.

Results unequivocally demonstrate the importance of analyzing diverse imperfection combinations; in this particular case, the difference between minimum and maximum ultimate load factors reached 13%. The minimum load factor, identified as 1.178, is crucial for subsequent evaluation. Furthermore, the direction of global imperfections significantly impacts the ultimate load, underscoring the necessity of performing simulations for all relevant imperfection directions.

Resistance is found to be lowest when a two-wave out-of-plane imperfection is combined with a positive in-plane global imperfection. Conversely, the direction of local imperfections has a negligible effect on the ultimate load. The non-linear analysis generates a load-displacement path, as illustrated in Figure 14, which plots the midspan vertical deflection (horizontal axis) against the applied load factor (vertical axis). In this model, a load factor of 1.0 represents the design value of internal forces, incorporating partial safety factors. The load-deformation path obtained from GMNIA reached a peak at a load factor of 1.179. Importantly, the maximum plastic strains at this ultimate load level remained below the 5% limit, which is the predefined allowable value for welded plated structures. To determine the characteristic buckling resistance ($R_{b,k}$), the resistance derived from GMNIA (R_{GMNIA}) is adjusted using the model factor according to Eq. (7).

$$R_{\mathrm{b,k}} = \frac{R_{\mathrm{GMNIA}}}{\gamma_{\mathrm{FE}}} = \frac{1{,}178}{1{,}0} = 1.178 \tag{7}$$

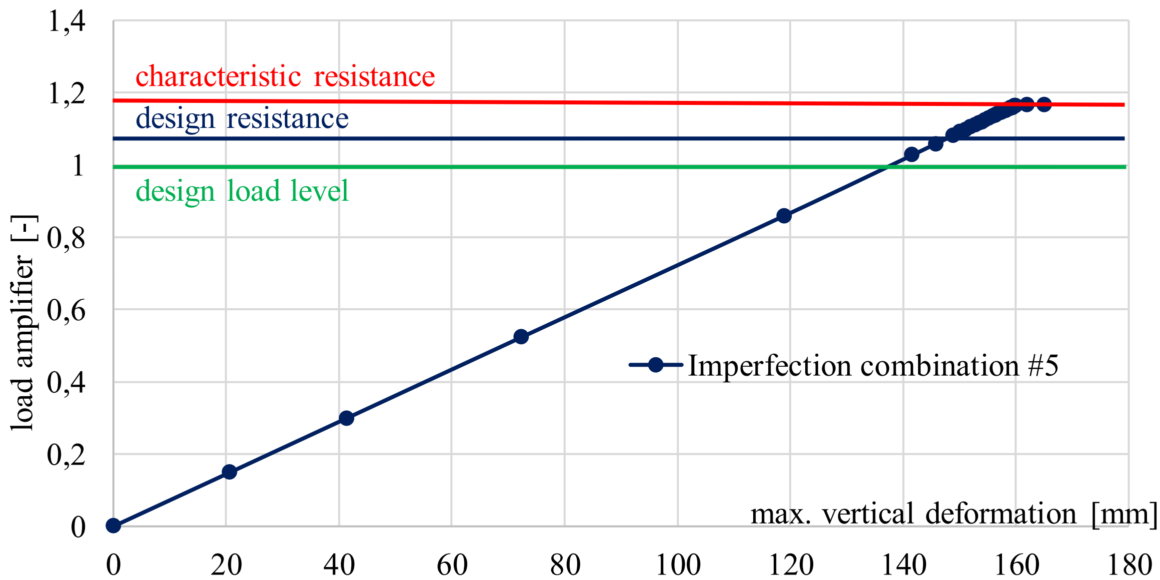

Figure 14: Calculated load-deformation path.

The design buckling resistance ($R_{b,d}$) can be determined on the basis of the characteristic buckling resistance ($R_{b,k}$) divided by the partial safety factor (γ_{M1}) according to EN 1993-2 as given by Eq. (8).

$$R_{\mathrm{b,d}} = \frac{R_{\mathrm{b,k}}}{\gamma_{\mathrm{M1}}} = \frac{1{,}178}{1{,}10} = 1.071 \tag{8}$$

The design buckling resistance should be compared to the design value of the internal forces represented here by load factor equal to 1.0. This means the utilization ratio of the analyzed structure is 93% in the critical loading situation.

7. Conclusions

This paper addresses the application of numerical model-based design for steel structures, specifically focusing on the direct buckling resistance check. It introduces the new Eurocode part EN1993-1-14, which standardizes numerical model-based design rules for steel structures, offering different modeling levels for design calculations and numerical simulations. The study focuses on the direct resistance check, where the ultimate buckling resistance is determined directly from numerical model, eliminating the need for further analytical expressions.

A case study of a single-span, steel railway arch bridge is presented to demonstrate the correct application of the numerical model-based design, where arch buckling is identified as the critical failure mode. Due to the complexity of full bridge model, equivalent geometric imperfections are applied, covering global out-of-plane, in-plane, and local plate buckling modes. The numerical model, combining shell and beam elements with a quad-linear material model, comprehensively accounts for elasto-plastic instability and various imperfection combinations. Rigorous verification and validation process, including mesh sensitivity studies and comparison to benchmark cases, confirms the model's accuracy, highlighting the necessity of proper discretization of the model for reliable load predictions. The analysis of different imperfection combinations reveals their significant impact on the ultimate load factor, necessitating a thorough evaluation of all possible scenarios. The study also demonstrates how the new EN1993-1-14 design framework can be applied to complex bridge structures and how to determine the characteristic and design buckling resistance considering different model levels and analysis types.

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