Proceedings of the
Annual Stability Conference
Structural Stability Research Council
Louisville, Kentucky, April 1-4, 2025

System-based design methods: Stability and reliability of benchmark structural steel frames

Damir Akchurin[1], Sándor Ádány[2], Ronald D. Ziemian[3], Kim J. R. Rasmussen[4], Benjamin W. Schafer[5]

Abstract
The current design practice for steel structures is largely governed by component-based methods. However, with recent advances and the increasing accessibility of nonlinear structural modeling and analysis tools, there has been growing interest in developing and adopting system-based design methods, such as the Direct Analysis Method specified in the *American Institute of Steel Construction Specification for Structural Steel Buildings* and the Direct Design Method currently being implemented in the Australian and New Zealand design specification. These methods explicitly capture most, if not all, component-level limit states directly in the analysis, simplifying or eliminating separate component-level limit state checks while ensuring overall stability of a structure and accounting for beneficial inelastic load redistribution effects. A key practical challenge with these methods has been the lack of a formal framework to ensure adequate system-level reliability. The Direct Analysis Method, for example, relies on component-level resistance factors and relatively ad hoc cross-sectional stiffness reduction factors to achieve target levels of system-level reliability, while the Direct Design Method addresses this issue more formally using rigorously calibrated system-level resistance factors. To evaluate and quantify the realized system-level reliabilities of these design methods under varying conditions of uncertainty, a series of benchmark structural steel frames were designed according to both methods using the Genetic Algorithm and then analyzed under uncertainty. The effects of two key nonlinearities that dominate system response were considered: stability-driven geometric nonlinearity and yielding-driven material nonlinearity, both of which significantly influence system-level reliability outcomes. The findings of this study highlight the importance of incorporating system-based design approaches to better capture the complexities of the actual structural response, while providing valuable insights into improving the robustness of current design methods through an investigation of the system-level reliabilities achieved by these methods.

1. Introduction
The assessment of the stability of steel structures remains one of the most challenging aspects of design due to the complex interactions between structural members that influence the overall system-level response. To appropriately account for geometric and material nonlinearities, which

[1] Graduate Research Assistant, Department of Civil and Systems Engineering, Johns Hopkins University, akchurd1@jhu.edu
[2] Associate Research Scientist, Department of Civil and Systems Engineering, Johns Hopkins University, asandor2@jhu.edu
Professor, Department of Structural Mechanics, Budapest University of Technology and Economics, adany.sandor@emk.bme.hu
[3] Professor, Department of Civil and Environmental Engineering, Bucknell University, ziemian@bucknell.edu
[4] Professor, School of Civil Engineering, University of Sydney, kim.rasmussen@sydney.edu.au
[5] Professor, Department of Civil and Systems Engineering, Johns Hopkins University, schafer@jhu.edu

directly influence a structure's response, it is necessary to incorporate these nonlinearities into structural analyses and design provisions. Accordingly, most design specifications worldwide permit the use of various forms of nonlinear structural analyses, combined with appropriate design provisions, to directly account for beneficial system-level load redistribution effects – an approach that is fundamental to achieving materially and economically efficient designs.

1.1. Direct Analysis Method

In the United States, starting from 1986, the *American Institute of Steel Construction (AISC) Load and Resistance Factor Design (LRFD) Specification for Structural Steel Buildings* (AISC, 1986) required, at a minimum, the use of 2^{nd}-order elastic analysis (accounting for geometric nonlinearities while assuming material linearity) to compute the required axial P_r and flexural M_r strengths for each structural member. However, because this approach could not account for the spread of plasticity in structural members, it still necessitated the use of the Effective Length Method (ELM), which required an additional set of elastic or inelastic buckling analyses to determine the effective length factors K for structural members under compression to compute their available compressive design strengths, thereby complicating the design process.

In the early 2000s, the Structural Stability Research Council formed a task committee to develop improved design provisions for steel structures that could directly account for geometric nonlinearities and indirectly account for material nonlinearities while simplifying the design process by eliminating the need for the ELM. Through a series of comprehensive studies (Deierlein et al., 2002; Maleck, 2001; Maleck and White, 2003; Surovek-Maleck and White, 2004a, 2004b; Martinez-Garcia, 2006), the task committee developed a new design method, termed the Direct Analysis Method (DAM), which was formally introduced in the 2005 edition of the *AISC Specification for Structural Steel Buildings* (AISC, 2005) and remains largely unchanged in the most recent 2022 edition of the *Specification*[6] (AISC, 2022).

The key benefit of the DAM is that it eliminates the need to compute effective length factors by allowing them to be set to unity ($K = 1$) when determining the available compressive design strengths of structural members under compression. This significantly simplifies the design process by eliminating the need for additional buckling analyses. However, in exchange, the DAM requires the use of reductions to cross-sectional stiffnesses of structural members, which implicitly account for the effect of partial yielding due to the spread of plasticity accentuated by the presence of the initial residual stresses and also attempt to account for component-level uncertainties. For instance, a reduction factor of 0.80 must be applied to the stiffness of all structural members, and an additional factor τ_b must be applied to their flexural stiffness. The stiffness reduction factor τ_b is given by

$$\tau_b = \begin{cases} 1, & \text{if } \dfrac{P_r}{P_{ns}} \leq 0.5 \\ 4\dfrac{P_r}{P_{ns}}\left(1 - \dfrac{P_r}{P_{ns}}\right), & \text{if } \dfrac{P_r}{P_{ns}} > 0.5 \end{cases}, \tag{1}$$

where P_{ns} is the cross-sectional compressive strength of a structural member. For structural members with nonslender cross-sections, which are used throughout this study, the cross-sectional

[6] For brevity, the *AISC Specification for Structural Steel Buildings* will be referred to as the *Specification* henceforth in this paper.

compressive strength is given by $P_{ns} = F_y A_g$, where F_y is yield stress and A_g is gross cross-sectional area. Taken together, the DAM requires one to apply a factor of 0.80 to the axial cross-sectional stiffnesses ($EA \rightarrow 0.80EA$) and a factor of $0.80\tau_b$ to the flexural cross-sectional stiffnesses ($EI_{zz} \rightarrow 0.80\tau_b EI_{zz}$) to implicitly account for partial yielding of structural members in the design process. Additionally, DAM requires one to take into the account the initial frame out-of-plumbness imperfections, either through direct modeling or the use of notional loads.

Although the DAM can technically be classified as a system-based design method, because it can account for system-level effects of a structures by directly considering geometric nonlinearities and implicitly considering material nonlinearities, it remains largely component-based as it still requires, albeit simplified, component-level limit state checks to ensure the overall stability of a structure. If the analysis according to the requirements of the DAM method is denoted by $f_{\text{DAM}}(\cdot)$, the strength criterion for each structural member can expressed mathematically in a familiar LRFD format as

$$\phi_c R_{nc} \geq f_{\text{DAM}}\left(\sum_i \gamma_i q_{ni}\right) = R_{rc}, \tag{2}$$

where ϕ_c is the component-level resistance factor related to the limit state governing its response, R_{nc} is the nominal strength against this limit state, R_{rc} is the required strength of the structural member found from the analysis, γ_i represents load factors from load combinations that are considered during the design of the structure, and q_{ni} represents nominal loads acting on the structure.

Because the limit state checks are performed at the component level, ensuring uniform reliabilities for each structural members, the DAM, in effect, tries to ensure adequate system-level reliability by indirectly accounting for the interactions between structural members through the incorporation of geometric and material nonlinearities in the analysis.

1.2. Direct Design Method

Due to advances, and the increasing accessibility of nonlinear structural modeling and analysis tools, in 2005, the *Specification* also permitted the design of steel structures as actual systems by using rigorous 2$^{\text{nd}}$-order inelastic analysis, which fully accounts for complex interactions between structural members as well as initial geometric and material imperfections. This approach, referred to here as the Advanced Inelastic Analysis Method (AIAM), allows for a more holistic understanding of system-level response. It is also worth noting that several earlier studies further reinforced this decision by demonstrating the potential for significant weight savings with the AIAM (Miller, 1995; Ziemian, 1990; Ziemian and Miller, 1997). For example, Ziemian, McGuire, and Deierlein (1992) showed that weight reductions of up to 15% could be achieved compared to simpler design methods based on the ELM, which cannot fully capture the beneficial effects of load redistribution.

The provisions of the current version of the AIAM, outlined in Section 1.3 of Appendix 1 of the most recent edition of the *Specification*, require that the modulus of elasticity E and yield stress F_y of all structural members be reduced by a factor of 0.90. However, as noted in the Commentary to Appendix 1, this reduction factor is conservative and is not derived by rigorous system reliability analysis. Consequently, AIAM does not guarantee that a specific level of system reliability is achieved when used. To address this limitation, in 2016, Zhang et al. proposed a new design method, termed the Direct Design Method (DDM)[7]. Similar to the AIAM, the DDM relies on rigorous 2nd-order inelastic analysis but ensures a target level of system reliability by applying a system-level resistance factor ϕ_s to the entire structure, directly accounting for any potential risks associated with geometric and material uncertainties that influence the stability and strength of a system.

If the analysis according to the requirements of the DDM method is denoted by $f_{\mathrm{DDM}}(\cdot)$, the strength criterion for the entire structure can expressed mathematically as

$$\phi_s R_{ns} \geq f_{\mathrm{DDM}}\left(\sum_i \gamma_i q_{ni}\right) = R_{rs}, \tag{3}$$

where ϕ_s the system-level resistance factor, independent of any specific system-level limit state, R_{ns} is its nominal ultimate strength against the applied loads, and R_{rs} is its required strength. In conducting the 2nd-order inelastic analyses, when the applied loads are increased incrementally by using a load proportionality factor λ, the ultimate nominal strength of the entire structure can alternatively be denoted using λ_{un} and the strength criterion in Eq. (3) can be rewritten into a more convenient form:

$$\phi_s \lambda_{un} \geq 1. \tag{4}$$

In the DDM, any target level of system reliability can be, in principle, achieved by calibrating ϕ_s in Eq. (4) through rigorous system-level reliability analysis. Although the DDM has not been yet formally adopted in the Australian and New Zealand design specifications for structural steel buildings, the studies by Zhang et al. (2016) recommend using ϕ_s of 0.85 for low- and mid-rise structural steel frames. This calibration corresponds to a system reliability index β_s of 2.9 for frames under gravity loads only and 2.7 for frames subjected to combined gravity and wind loads. Consequently, ϕ_s of 0.85 is used throughout the study presented herein.

2. Benchmark Structural Steel Frames

To investigate the realized system-level reliabilities achieved by the DAM and DDM, the first 12 structural steel frames presented in Ziemian and Ziemian (2021) are selected for comparison. These frames represent a wide range of geometric configurations, showcasing varying sensitivities to nonlinear geometric and material effects. In all frames, the structural members are oriented to experience flexure about their major axes, with the exception of frames #8 and #10. The frames are also assumed to be fully braced out-of-plane, allowing for fully planar analyses. While the load magnitudes provided by Ziemian and Ziemian (2021) are used for further design according to the DAM and DDM in this study, the elastic moduli E and yield stresses F_y are assumed to be 200 GPa (29000 ksi) and 345 MPa (50 ksi), respectively.

Instead of considering only one load combination per frame, as was done in Ziemian and Ziemian (2021), three load combinations of interest from the *ASCE 7-22: Minimum Design Loads and*

---

[7] Note that the referenced studies use the term "Direct Analysis Method" as an umbrella term for all design method based on the rigorous 2nd-order inelastic analysis. However, in this study, this term is specifically used to refer to the system-based design method proposed in the referenced studies which ensures adequate system-level reliability using the system-level resistance factor ϕ_s.

Associated Criteria for Buildings and Other Structures (ASCE, 2021) are considered for during the design of frames subjected to combined gravity and wind loads:

$$\begin{cases} 1.4D_n \\ 1.2D_n + 1.6L_n \\ 1.2D_n + 0.5L_n + 1.0W_n \end{cases}. \quad (5)$$

For the frames subjected to gravity loads only (frames #4, #9, and #10), the three load combinations in Eq. (5) reduce to two as the wind load does not need to be considered:

$$\begin{cases} 1.4D_n \\ 1.2D_n + 1.6L_n \end{cases}. \quad (6)$$

As discussed later, the analyses of the selected structural steel frames are conducted using traditional line finite elements, which are incapable of capturing limit states associated with local section instabilities. Therefore, the selection of sections was restricted to those classified as compact in flexure about both the major and minor axes and nonslender in axial compression. This classification ensures that the chosen sections can reach their full plastic capacity before any local instabilities occur, thereby justifying the use of line finite elements in the conducted analyses.

The W-shaped sections listed in the 16th edition of the *AISC Steel Construction Manual* (AISC, 2023) were used as the base set for designing the structural steel frames in accordance with the DAM and DDM. These W-shaped sections were classified as compact and nonslender based on the requirements stipulated in Chapter B of the *Specification*. Out of the 289 available W-shaped sections, 168 were found to be compact in flexure about both the major and minor axes and nonslender in axial compression. Following standard practice, the selection of sections for columns was further restricted to W8X… through W14X… sections. As a result, 168 sections were available for selection for beams and braces, and 72 sections were available for selection for columns.

3. Methodology

3.1. Structural Design Optimization Scheme

For a robust comparison of the realized system-level reliabilities achieved by the DAM and DDM, it is important to obtain the most optimal structural steel frame designs, with the least possible weight, that satisfy all the requirements of each design method. While it is possible to achieve this manually through a trial-and-error approach for frames with only a few structural members, this approach quickly becomes impractical for larger, more complex frames with larger number of members. To address this challenge, optimization techniques are employed to systematically identify the lightest structural configurations that meet all applicable design constraints. In fact, there exists an entire field of structural design optimization dedicated to studying and developing methodologies for achieving materially and economically efficient designs while satisfying the performance requirements – either linear or nonlinear constraints – prescribed by the provisions of the chosen design method.

In general, the process of structural design can be formulated as a well-posed mathematical optimization problem, where the weight $W(\vec{p})$ of a structure is minimized with respect to the parameters $\vec{p}$ – *design variables* – that describe the structure. In the case of this study, the parameters $\vec{p}$ are the cross-sectional dimensions and properties of the W-shaped sections permitted for selection in the design. Given that the selection of sections in this study is restricted only to those listed in the *AISC Steel Construction Manual*, the parameters $\vec{p}$ are discrete, thereby making the optimization problem at hand inherently discrete in nature. For this reason, it is convenient to introduce a new set of N_M integer design variables $\vec{\alpha}$, where N_M is the number of structural members in a structure of interest, with respect to which the optimization will be performed, such that $\vec{p} = \vec{p}(\vec{\alpha})$ and, thus, $W = W(\vec{\alpha})$. In effect, the i^{th} component of the design vector $\vec{\alpha}$ defines the section that will be prescribed to the i^{th} structural member of a structure from the list of available sections: 168 sections if the member is a beam or a brace, and 72 sections if the member is a column.

3.1.1. Strength Constraints

As mentioned before, the structural design optimization scheme should not only yield designs with the least possible weight but also satisfy the performance requirements – *strength constraints* – imposed by the provisions of the chosen design method.

The DAM, for example, dictates that the strength criterion given in Eq. (2) must be satisfied for each structural member. In this study, three possible component-level limit states are considered:

1. If a structural member is under *pure axial tensile load* ($P_r > 0$ and $M_r = 0$), the Eq. (2) is rewritten as
$$\phi_{ct} P_{nt} \geq |P_r|, \tag{7}$$
where ϕ_{ct} is the component-level resistance factor related to the tensile yielding in the gross cross-section of the member and is equal to 0.90, and P_{nt} is the nominal tensile strength of the member determined in accordance with Chapter D of the *Specification*.
2. If a structural member is under *pure axial compressive load* ($P_r < 0$ and $M_r = 0$), the Eq. (2) is rewritten as
$$\phi_{cc} P_{nc} \geq |P_r|, \tag{8}$$
where ϕ_{cc} is the component-level resistance factor related to the flexural buckling of the member and is equal to 0.90, and P_{nc} is the nominal compressive strength of the member determined in accordance with Chapter E of the *Specification*.
3. If a structural member is under combined axial and flexural load ($P_r \neq 0$ and $M_r \neq 0$), the Eq. (2) is rewritten as
$$\begin{cases} \dfrac{|P_r|}{P_c} + \dfrac{8}{9}\dfrac{|M_r|}{M_c} \leq 1, & \dfrac{|P_r|}{P_c} \geq 0.2 \\ \dfrac{1}{2}\dfrac{|P_r|}{P_c} + \dfrac{|M_r|}{M_c} \leq 1, & \dfrac{|P_r|}{P_c} < 0.2 \end{cases}, \tag{9}$$
where $P_c = \phi_c P_n$ is available axial design strength determined in accordance with Chapter D if $P_r > 0$ and Chapter E if $P_r < 0$, and $M_c = \phi_{cb} M_n$ is the available flexural design strength, where ϕ_{cb} is the component-level resistance factor related to the yielding due to flexure of the member and is equal to 0.90, and M_n is the nominal flexural strength determined in accordance with Chapter F of the *Specification*. Eq. (9) is commonly referred to as the beam-column interaction equation.

Taken together, the strength criteria given in Eq. (7), (8), and (9) constitute the strength constraints, denoted by $g_i^j(\vec{\alpha})$, that must be satisfied (i.e., take values less than or equal to 0) for each structural member i for each load combination j of interest within the structural design optimization scheme:

$$g_i^j(\vec{\alpha}) = \begin{cases} \left|P_{ri}^j\right| - \phi_{ct} P_{nti}^j, & \text{if } P_{ri}^j > 0 \text{ and } M_{ri}^j = 0 \\ \left|P_{ri}^j\right| - \phi_{cc} P_{nci}^j, & \text{if } P_{ri}^j < 0 \text{ and } M_{ri}^j = 0 \\ \frac{\left|P_{ri}^j\right|}{P_{ci}^j} + \frac{8}{9}\frac{\left|M_{ri}^j\right|}{M_{ci}^j} - 1, & \text{if } \left|P_{ri}^j\right|/P_{ci}^j \geq 0.2 \text{ and } M_{ri}^j \neq 0 \\ \frac{1}{2}\frac{\left|P_{ri}^j\right|}{P_{ci}^j} + \frac{\left|M_{ri}^j\right|}{M_{ci}^j} - 1, & \text{if } \left|P_{ri}^j\right|/P_{ci}^j < 0.2 \text{ and } M_{ri}^j \neq 0 \end{cases} \leq 0, \tag{10}$$

noting that the required and nominal axial and flexural strengths P_r, M_r, P_n, and M_n of the structural members are dependent on the design vector $\vec{\alpha}$ with respect to which the optimization will be performed.

For the DDM, the strength constraints, denoted by $g^j(\vec{\alpha})$, is simply given by Eq. (4) and is only checked for the entire structure for each load combination j of interest within the structural design optimization scheme:

$$g^j(\vec{\alpha}) = 1 - \phi_s \lambda_{un}^j \leq 0, \tag{11}$$

noting that the ultimate nominal strength (load proportionally factor) λ_{un} of a structure, obtained from 2nd-order inelastic analysis, is dependent on the design vector $\vec{\alpha}$ with respect to which the optimization will be performed.

3.1.2. Constructability Constraints

While this study ignores serviceability requirements, as they typically dominate the design process by imposing, albeit implicitly, much stricter constraints on the overall stability of a structure than strength requirements, it does incorporate *constructability constraints* to ensure geometric compatibility between structural members at connection locations. In this study, three types of constructability constraints are considered.

1. At the beam-to-column connections, if the column is oriented to bend about its major axis, the flange width of the column $b_{f,c}$ must be greater than or equal to the flange width of the beam $b_{f,b}$ as shown in Figure 1 (a).
2. At the beam-to-column connections, if the column is oriented to bend about its minor axis, the web depth of the column $d_c - 2t_{f,c}$ must be greater than or equal to the flange width of the beam $b_{f,b}$ as shown in Figure 1 (b).
3. At the column-to-column connections, the both the depth d_{bc} and flange width $b_{f,bc}$ of the bottom column must be greater than or equal to the depth d_{tc} and flange width $b_{f,tc}$ of the top column as shown in Figure 1 (c).

If there are N_{BCC} beam-to-column connections in a structure, the constructability constraints at the k^{th} beam-to-column connection, denoted by $u_k(\vec{\alpha})$, can be expressed as:

$$u_k(\vec{\alpha}) = \begin{cases} b_{f,b} - b_{f,c}, & \text{if column is oriented to bend about its major axis} \\ b_{f,b} - (d_c - 2t_{f,c}), & \text{if column is oriented to bend about its minor axis} \end{cases} \leq 0. \tag{12}$$

Similarly, if there are N_{CCC} column-to-column connections in a structure, the constructability constraints at the l^{th} beam-to-column connection, denoted by $\vec{v}_l(\vec{\alpha})$, can be expressed as:

$$\vec{v}_l(\vec{\alpha}) = \begin{bmatrix} v_{l1}(\vec{\alpha}) \leq 0 \\ v_{l2}(\vec{\alpha}) \leq 0 \end{bmatrix} = \begin{bmatrix} d_{tc} - d_{bc} \leq 0 \\ b_{f,tc} - b_{f,bc} \leq 0 \end{bmatrix}. \tag{13}$$

To summarize, the structural design optimization problem can then be formulated as:

minimize $W(\vec{\alpha}) = \rho \sum_{i=1}^{N_M} A_{gi} L_i$ with respect to $\vec{\alpha}$, (14)

subject to *Box constraints*

If i^{th} member is a beam or a brace:
$1 \leq \alpha_i \leq 168$,
If i^{th} member is a column:
$1 \leq \alpha_i \leq 72$,
$i \in \{1, \dots, N_M\}$,

Strength constraints

For the DAM: $g_i^j(\vec{\alpha}) \leq 0$ in Eq. (10), $i \in \{1, \dots, N_M\}, j \in \{1, \dots, N_{LC}\}$,

For the DDM: $g^j(\vec{\alpha}) \leq 0$ in Eq. (11), $j \in \{1, \dots, N_{LC}\}$,

Constructability constraints

$u_k(\vec{\alpha}) \leq 0$ in Eq. (12), $k \in \{1, \dots, N_{BCC}\}$,

$v_{l1}(\vec{\alpha}) \leq 0$ and $v_{l2}(\vec{\alpha}) \leq 0$ in Eq. (13), $l \in \{1, \dots, N_{CCC}\}$,

where ρ is the density of steel assumed be 8000 kg/m^3 (0.290 lb/in.^{3}) throughout this study, L_i is the length of the i^{th} structural member, and N_{LC} is the number of load combinations considered in the design process.

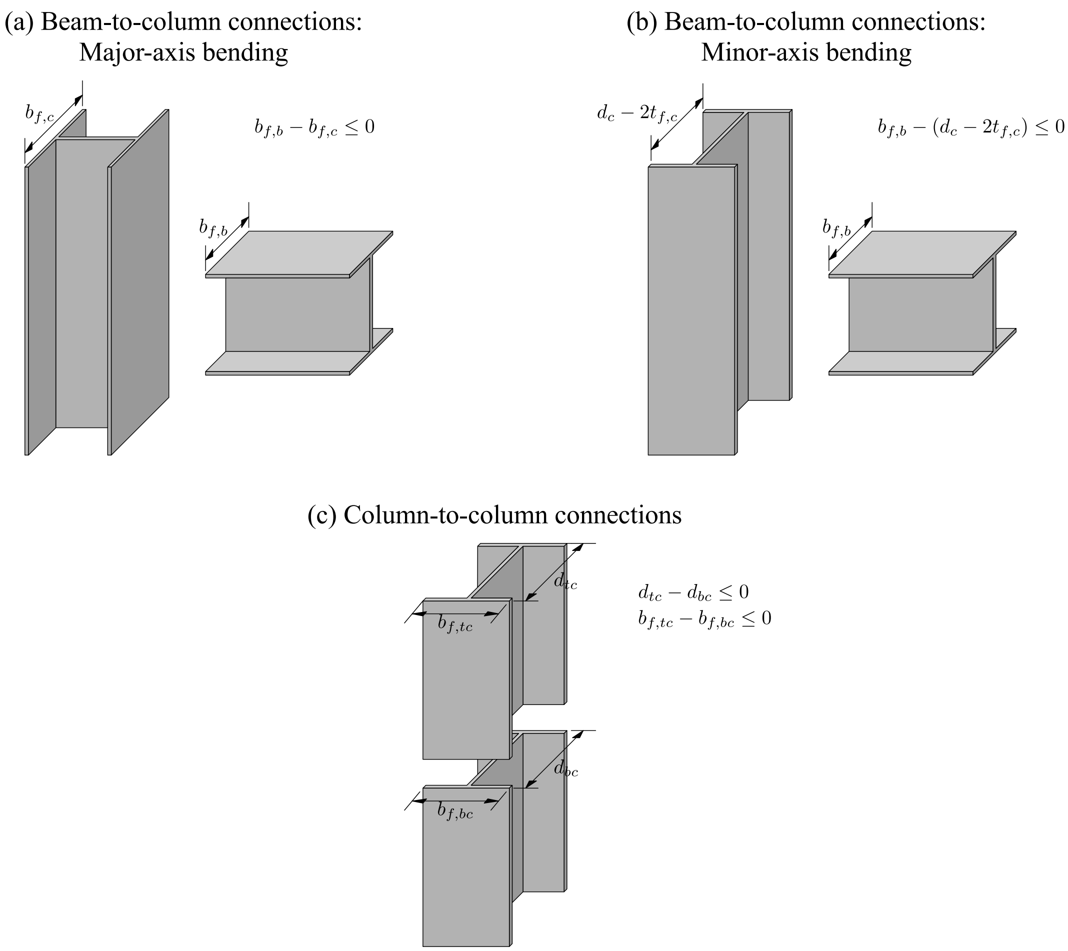

Figure 1: Graphical representation of the constructability constraints used in the developed structural design optimization scheme.

3.1.3. Genetic Algorithm

The discrete optimization problem with nonlinear constraints formulated in Eq. (14) was solved using the Genetic Algorithm (GA) implemented in the Metaheuristics.jl package (Mejía-de-Dios and Mezura-Montes, 2022), an open-source package written in the Julia programming language (Bezanson et al., 2017) that implements several global optimization metaheuristic algorithms for solving single- and multi-objective optimization problems. A key distinction of the implementation of the GA in the Metaheuristics.jl package, compared to other implementations that typically rely on the Penalty Method to guide the algorithm toward the constrained optimum, is its use of the Constrained Violation Rule (Deb, 2000). Unlike the Penalty Method, which penalizes the objective function for constraint violations, the Constrained Violation Rule computes the average constraint violation to perform pairwise comparisons of individuals in the population during the selection process; thus, enabling the algorithm to focus on feasibility during the optimization process without relying on arbitrary penalty parameters. Moreover, the Constrained Violation Rule ensures that the entire population will reach the feasible region if at least one feasible solution is identified or provided.

In the used optimization scheme, the Binary Tournament Selection was used as the selection operator to ensure that fitter individuals had a higher probability of being chosen while maintaining diversity within the population, the Simulated Binomial Crossover (Deb and Agrawal, 1995) was used as the crossover operator to generate offspring by combining the genetic material of parent solutions in a way that encourages exploration of the design space, and the Polynomial Mutation (Deb and Deb, 2014) was used as the mutation operator to introduce small and controlled variations to individual solutions, improving the algorithm's ability to escape local optima. Additionally, an elitist strategy was used to ensure that the fittest individuals from the current generation were carried over to the next, thereby preserving high-quality solutions throughout the optimization process. The described setup had demonstrated the best convergence rates.

For each benchmark structural steel frame, the optimization process was repeated 10 times, with an initial population of 100 individuals in each run. Among these runs, the individual with the least weight that satisfied all strength and constructability constraints was selected as the optimal design.

3.1.4. Finite Element Modeling and Analysis

The 2nd-order elastic and inelastic analyses required by the DAM and DDM were performed using OpenSeesPy, an open-source Python package for finite element analysis based the OpenSees framework (McKenna et al., 2010). For structural steel frame designs according to the DAM, elastic beam-column finite elements based on the Euler-Bernoulli Beam Theory were used to model the structural members, and for frame designs according to the DDM, which explicitly considers material nonlinearities, displacement-based finite elements with fiber-type sections were employed. For the DDM analyses, similar to the approach used by Zhang et al. (2016), the European Convention for Constructional Steelwork's (ECCS) self-equilibrating residual stress pattern was adopted. In both methods, each structural member was discretized into 4 finite elements. Initial nominal frame out-of-plumbness imperfections ψ_n with magnitude of 1/500 were directly modeled to avoid the development of fictitious shear forces and moments during the analyses. For simplicity, and in accordance with standard design practices, loads were applied proportionally, meaning that gravity and wind loads were applied simultaneously. To appropriately account for potentially large deflections, the corotational geometric transformation was used in analysis according to both methods. For the DAM, the Load Control Method was employed to incrementally apply the loads, while for the DDM, the Arclength Control Method was used.

It is also important to address how the stiffness reduction factor τ_b is computed for the frame designs according to the DAM in the formulated structural optimization scheme. Ideally, τ_b must be updated at each increment of the finite element solver; however, this approach requires one to update the cross-sectional stiffnesses of all structural components, which cannot be done simply without rebuilding the finite element model, at each increment and is, therefore, computationally expensive. To simplify the process, this study adopts an alternative approach:

1. A 1st-order elastic analysis is performed under the assumption that $\tau_b = 1$ for all structural members. Note that the other stiffness reduction factor of 0.80 is still applied.
2. The required axial strengths P_r are extracted from the results of performed 1st-order elastic analysis and the approximate stiffness reduction factor $\hat{\tau}_b$ is computed using Eq. (1) for each structural member.
3. The finite element model is rebuilt with properly reduced axial and flexural cross-sectional stiffnesses of all structural members and the 2nd-order elastic analysis is performed.

The results of the last performed 2nd-order elastic analysis are then used to compute the strength constraints given by Eq. (10) within the developed structural design optimization scheme.

3.2. System Reliability Analysis

The failure of a structure is defined as an event where the ultimate random (actual) strength of the structure, determined from the 2nd-order inelastic analysis and represented by $\boldsymbol{\lambda_u}$[8], is insufficient to resist the random load acting on it. Mathematically, this occurs when $\boldsymbol{\lambda_u}$ does not exceed unity, meaning the structure is unable to support the applied load fully without collapsing. If N_{RLC} random load combinations are considered in the system reliability analysis of a structure, then the limit state function for the structure can be expressed as:

$$\boldsymbol{G}(\vec{\boldsymbol{X}}) = \min \begin{Bmatrix} \boldsymbol{\lambda}_{\boldsymbol{u}}^{1} - 1 \\ \vdots \\ \boldsymbol{\lambda}_{\boldsymbol{u}}^{N_{RLC}} - 1 \end{Bmatrix}, \tag{15}$$

where $\vec{\boldsymbol{X}}$ is used to represent all random variables that contribute to the response of that structure and are described in the following sections.

The system-level probabilities of failure $P_{f,sys}$ were estimated using Monte Carlo simulations due to the simplicity and robustness of this approach. In each simulation, a random sample of a frame was generated by randomly assigning values to the geometric and material properties based on their respective probability distributions. Then, a 2nd-order inelastic analysis of the frame was performed to determine whether it would collapse under each considered random load combination by evaluating the limit state function in Eq. (15). Depending on the magnitude of $P_{f,sys}$, between 10,000 and 1,000,000 simulations were conducted for each frame design, obtained in accordance with the DAM and DDM. To improve the convergence of the Monte Carlo simulations, the sampling of random variables was performed using the Latin Hypercube Sampling technique, as

[8] In the study presented herein, bold symbols are used to denote random variables.

implemented in the Fortuna.jl, an open-source Julia package for structural and system reliability analysis (Akchurin, 2024).

The estimated system-level probabilities of failure $P_{f,sys}$ were then converted into system-level reliability indices β_{sys}, which serve as a more familiar and interpretable metric of reliability, using the relationship:

$$\beta_{sys} = \Phi^{-1}(1 - P_{f,sys}), \quad (16)$$

where $\Phi^{-1}(\cdot)$ is the inverse cumulative density function of the standard normal distribution.

3.2.1. Random Loads and Load Combinations

Random load combinations were modeled following the methodology proposed by Akchurin et al. (2024) based on the Turksta's rule (Turkstra and Madsen, 1980). For structural steel frames subjected to combined gravity and wind loads, the following three random load combinations were considered:

$$\begin{cases} \boldsymbol{D} + \boldsymbol{L_{apt}} \\ \boldsymbol{D} + \boldsymbol{L_{max}} \\ \boldsymbol{D} + \boldsymbol{L_{apt}} + \boldsymbol{W_{max}} \end{cases} = \begin{cases} \boldsymbol{X_D} D_n + \boldsymbol{X_{L_{apt}}} L_n \\ \boldsymbol{X_D} D_n + \boldsymbol{X_{L_{max}}} L_n \\ \boldsymbol{X_D} D_n + \boldsymbol{X_{L_{apt}}} L_n + \boldsymbol{X_{W_{max}}} W_n \end{cases}, \quad (17)$$

where $\boldsymbol{D}$ is the random dead load, $\boldsymbol{L_{apt}}$ is the random arbitrary point-in-time live load, $\boldsymbol{L_{max}}$ is the random maximum lifetime live load, and $\boldsymbol{W_{max}}$ is the random maximum lifetime wind load. For the frames subjected to gravity loads only (frames #4, #9, and #10), the three random load combinations in Eq. (17) reduce to two as the random maximum lifetime wind load $\boldsymbol{W_{max}}$ does not need to be considered:

$$\begin{cases} \boldsymbol{D} + \boldsymbol{L_{apt}} \\ \boldsymbol{D} + \boldsymbol{L_{max}} \end{cases} = \begin{cases} \boldsymbol{X_D} D_n + \boldsymbol{X_{L_{apt}}} L_n \\ \boldsymbol{X_D} D_n + \boldsymbol{X_{L_{max}}} L_n \end{cases}. \quad (18)$$

The statistics normalized random variables $\boldsymbol{X_D}$, $\boldsymbol{X_{L_{apt}}}$, $\boldsymbol{X_{L_{max}}}$, and $\boldsymbol{X_{W_{max}}}$, which are used to scale the nominal loads D_n, L_n, and W_n acting on the structural steel frames of interest in the system reliability analysis, used in this study are based on the statistics used in Akchurin et al. (2024) and are presented in Table 1.

Table 1: Statistics of the normalized random variables patinating to the loads.

Random variable	Distribution	Mean, μ	COV, V
$\boldsymbol{X_D}$	Normal	1.05	0.10
$\boldsymbol{X_{L_{apt}}}$	Gamma	0.22	0.54
$\boldsymbol{X_{L_{max}}}$	Gumbel	1.10	0.19
$\boldsymbol{X_{W_{max}}}$	Gumbel	0.47	0.35

3.2.2. Random Geometric and Material Properties

The statistics of the random variables representing the uncertainties present in the geometric and material properties of a structure, as used in this study, are based on the data provided in Zhang et al. (2016).

The normalized random variable associated with the elastic modulus $\boldsymbol{X_E}$ follows a normal distribution with a mean μ of 1.00 and a coefficient of variation V of 0.06, and the normalized random variable associated with the yield stress $\boldsymbol{X_{F_y}}$ follows a lognormal distribution with μ of 1.05 and V of 0.10. In the conducted system reliability analyses, it was assumed that the material properties of all structural members were perfectly correlated. However, the ECCS residual stress pattern used for each structural member was individually scaled by a normally distributed random scaling factor $\boldsymbol{\chi}$ with μ of 1.05 and V of 0.21.

The random variable representing the initial frame out-of-plumbness imperfections $\boldsymbol{\psi}$ follows a lognormal distribution with μ of 1/770 and V of 1/880. The initial member out-of-straightness imperfections were modeled as a linear superposition of the first 3 randomly scaled buckling modes of a single column under compression:

$$\boldsymbol{\delta}(x) = \sum_{m=1}^{3} \boldsymbol{a_m} \sin\left(\frac{m\pi x}{L}\right), \tag{19}$$

where $x \in [0, L]$ is the coordinate along the longitudinal axis of a structural member and $\boldsymbol{a_m}$ are normally distributed random scaling factor with a random sign – either negative or positive – for the m^{th} buckling mode with the statistics provided in Table 3 of Zhang et al. (2016). Lastly, the variations in the cross-sectional dimensions of each structural member were considered in the system reliability analyses. The statistics and the correlation matrix of the random variables associated with the cross-sectional dimensions of W-shaped sections are provided in Tables 1 and 2 of Zhang et al. (2016).

4. Results

The results of the performed structural design optimization in accordance with both the DAM and DDM are summarized in Table 2. As can be observed, for all structural steel frames, the developed structural design optimization scheme successfully found DDM designs with weights that are either lower than or equal to those of the corresponding DAM designs. A maximum weight reduction of 39% was achieved for frame #10, which has 2 bays and 2 stories, and is subjected to gravity loads only, and features columns oriented to bend about their minor axes. On average, the DDM produced designs that used 13% less steel weight compared to the DAM designs.

Table 2: Weights W of the optimal structural steel frame designs obtained using the developed structural design optimization scheme.

Frame #	$\boldsymbol{W_{DAM}}$ (kg)	$\boldsymbol{W_{DDM}}$ (kg)	$\boldsymbol{W_{DDM}/W_{DAM}}$
1	1,556	1,452	0.93
2	6,170	5,903	0.96
3	964	964	1.00
4	6,149	4,836	0.79
5	1,934	1,823	0.94
6	1,969	1,969	1.00
7	1,841	1,841	1.00
8	2,747	2,350	0.86
9	14,653	11,326	0.77
10	15,880	9,634	0.61
11	5,024	4,141	0.82
12	9,045	7,256	0.80
		Mean, $\boldsymbol{\mu}$	0.87
		COV, $\boldsymbol{V}$	0.14

The system-level reliability indices for all structural steel frames designed in accordance with both the DAM and DDM are summarized in Table 3. As can be observed, for most structural steel frames, the DAM designs exhibit higher levels of system reliability compared to the corresponding DDM designs. This fact reflects the inherent conservatism of the DAM, which applies stiffness reductions, leading to designs with higher safety margins but often at the expense of increased weight. However, in some cases (frames #5 and #8), the DDM designs outperform the corresponding DAM designs in terms of system reliability while still achieving lower weights. This result highlights the DDM's ability to better capture the actual structural response, due to the explicit consideration of nonlinear geometric and material effects, leading to more materially and economically efficient designs without compromising safety. Furthermore, the DDM provides a significantly more uniform set of reliabilities than the DAM.

It can also be observed that for frames #4, #9, and #10 subjected to gravity loads only, the DAM designs exhibit much higher reliabilities compared to the corresponding DDM designs, likely due to the degree of conservatism of the DAM's stiffness reduction factors for designs governed by gravity loads.

Table 3: System-level probabilities of failure $P_{f,sys}$ and reliability indices β_{sys} of the optimal structural steel frame designs obtained using the developed structural design optimization scheme.

Frame #	**DAM**		**DDM**		$\boldsymbol{\beta_{sys}^{DAM}/\beta_{sys}^{DDM}}$
	$\boldsymbol{P_{f,sys}}$	$\boldsymbol{\beta_{sys}}$	$\boldsymbol{P_{f,sys}}$	$\boldsymbol{\beta_{sys}}$	
1	2.8×10^{-4}	3.45	6.8×10^{-4}	3.20	1.08
2	6.0×10^{-5}	3.85	3.4×10^{-4}	3.40	1.13
3	4.8×10^{-5}	3.90	4.8×10^{-5}	3.90	1.00
4	$1.0\times10^{-6*}$	4.75*	3.4×10^{-4}	3.40	1.40
5	3.0×10^{-4}	3.43	1.2×10^{-4}	3.67	0.93
6	2.0×10^{-4}	3.54	2.0×10^{-4}	3.54	1.00
7	1.6×10^{-3}	2.95	1.6×10^{-3}	2.95	1.00
8	1.7×10^{-3}	2.93	1.1×10^{-3}	3.06	0.96
9	5.9×10^{-6}	4.38	2.0×10^{-3}	2.88	1.52
10	$1.0\times10^{-6*}$	4.75*	1.1×10^{-3}	3.06	1.56
11	6.4×10^{-5}	3.83	2.7×10^{-3}	2.78	1.38
12	1.1×10^{-4}	3.69	2.6×10^{-3}	2.79	1.32

Notes:
*These frames did not fail once within 1,000,000 Monte Carlo simulations and $P_{f,sys} = 1.0 \times 10^{-6}$, corresponding to $\beta_{sys} = 4.75$, was used as the lower bound on their system-level reliabilities.

Since the difference between component-level and system-level reliabilities depends on the extent to which the structural system permits load redistribution following first yield, it is useful to compare the levels of achieved system reliability against a metric representing a structure's inelastic load redistribution capabilities. Following the approach used in Zhang et al. (2018), the difference between the nominal ultimate load proportionality factors achieved by the DAM and DDM designs, $\lambda_{DAM} - \lambda_{DDM}$, is used. The resulting comparison is presented in Figure 2.

As can be observed, for frames with only a few members and, thus, smaller inelastic load redistribution capabilities, the DDM is still capable of producing designs with lower weights and higher system reliabilities (frames #5 and #8). For larger frames, the DAM results in frame designs with approximately linearly increasing reliability for increasing inelastic load redistribution capability metric, while the DDM results in frame designs with more uniform β_{sys} between 2.75 and 3.00, which is consistent with the calibration performed in Zhang et al. (2016).

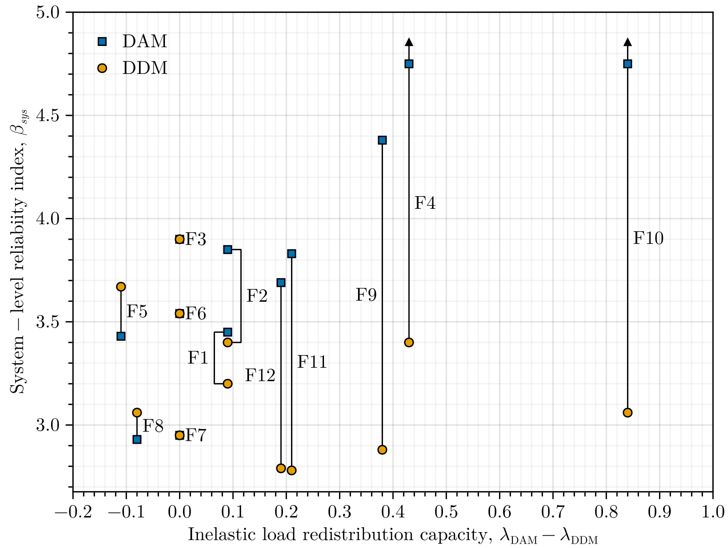

Figure 2: Comparison of system-level reliability indices β_{sys} achieved the DAM and DDM against the inelastic load redistribution $\lambda_{DAM} - \lambda_{DDM}$ capacity of each benchmark structural steel frame.

5. Conclusions

This study presents a comprehensive comparison of structural steel frame designs optimized using the Genetic Algorithm according to the provisions of the Direct Analysis Method (DAM) and the Direct Design Method (DDM). It also provides a comparison of the system-level reliabilities achieved by these design methods, demonstrating that while the DAM generally results in designs with higher system-level reliabilities, the DDM offers more materially and economically efficient designs with more uniform reliabilities, often achieving weights up to 39% lighter without compromising safety. Moreover, the system-level reliabilities and inelastic load redistribution capabilities further highlight the potential advantages of design methods based on 2^{nd}-order inelastic analyses, such as the DDM, particularly in structures with many structural members that benefit most from the explicit consideration of nonlinear geometric and material effects. The findings of this study emphasize the importance of incorporating system-based design approaches in the current design specifications to better capture the complexities of the actual structural response, while providing valuable insights into improving the robustness of current design methods through an investigation of the system-level reliabilities achieved by these methods.

Acknowledgments

The authors would like to thank the American Institute of Steel Construction for the financial support of the *"System Reliability of Steel Structures"* project. The authors would also like to thank Dr. Jesús-Adolfo Mejía-de-Dios and Dr. Efrén Mezura-Montes from the University of Veracruz for developing the open-source package Metaheuristics.jl and providing invaluable support in its application. The first author would also like to personally thank fellow graduate students Saroj Khanal and Tomás Tapia from the Johns Hopkins University for insightful discussions on the discrete optimization problems with nonlinear constraints.

References

AISC, 2023. AISC: Steel Construction Manual.

AISC, 2022. ANSI/AISC 360-22: Specification for Structural Steel Buildings.

AISC, 2005. ANSI/AISC 360-05: Specification for Structural Steel Buildings.

AISC, 1986. AISC: Load and Resistance Factor Design Specification for Structural Steel Buildings.

Akchurin, D., 2024. Fortuna.jl: Structural and System Reliability Analysis in Julia. JOSS 9, 6967. https://doi.org/10.21105/joss.06967

Akchurin, D., Sabelli, R., Ziemian, R.D., Schafer, B.W., 2024. ASD and LRFD: Reliability Comparison for Designs Subjected to Wind Loads. Journal of Constructional Steel Research 213, 108327. https://doi.org/10.1016/j.jcsr.2023.108327

ASCE, 2021. ASCE 7-22: Minimum Design Loads and Associated Criteria for Buildings and Other Structures.

Bezanson, J., Edelman, A., Karpinski, S., Shah, V.B., 2017. Julia: A Fresh Approach to Numerical Computing. SIAM Rev. 59, 65–98. https://doi.org/10.1137/141000671

Deb, K., 2000. An Efficient Constraint Handling Method for Genetic Algorithms. Computer Methods in Applied Mechanics and Engineering 186, 311–338. https://doi.org/10.1016/S0045-7825(99)00389-8

Deb, K., Agrawal, R.B., 1995. Simulated Binary Crossover for Continuous Search Space. Complex Systems 9, 115–148.

Deb, K., Deb, A., 2014. Analysing Mutation Schemes for Real-Parameter Genetic Algorithms. IJAISC 4, 1. https://doi.org/10.1504/IJAISC.2014.059280

Deierlein, G.G., Hajjar, J.F., Yura, J.A., White, D.W., Baker, W.F., 2002. Proposed New Provisions for Frame Stability Using Second-Order Analysis, in: Proceedings of the Annual Stability Conference. Presented at the Annual Stability Conference, Structural Stability Research Council, Seattle, WA.

Maleck, A.E., 2001. Second-Order Inelastic and Modified Elastic Analysis and Design Evaluation of Planar Steel Frames. Georgia Institute of Technology, Altlanta, GA.

Maleck, A.E., White, D.W., 2003. Direct Analysis Approach for the Assessment of Frame Stability: Verification Studies, in: Proceedings of the Annual Stability Conference. Presented at the Annual Stability Conference, Structural Stability Research Council, Baltimore, MD.

Martinez-Garcia, J.M., 2006. Benchmark Studies to Compare Frame Stability Provisions, in: Proceedings of the Annual Stability Conference. Presented at the Annual Stability Conference, Structural Stability Research Council, San Antonio, TX.

Mejía-de-Dios, J.-A., Mezura-Montes, E., 2022. Metaheuristics: A Julia Package for Single- and Multi-Objective Optimization. JOSS 7, 4723. https://doi.org/10.21105/joss.04723

Miller, A.R., 1995. Advanced Second-Order Inealstic Analysis of Steel Structures with Columns Experiencing Minor-Axis Bending Subject to Strength Limit State Requirements. Bucknell University, Lewisburg, PA.

Surovek-Maleck, A.E., White, D.W., 2004a. Alternative Approaches for Elastic Analysis and Design of Steel Frames. I: Overview. J. Struct. Eng. 130, 1186–1196. https://doi.org/10.1061/(ASCE)0733-9445(2004)130:8(1186)

Surovek-Maleck, A.E., White, D.W., 2004b. Alternative Approaches for Elastic Analysis and Design of Steel Frames. II: Verification Studies. J. Struct. Eng. 130, 1197–1205. https://doi.org/10.1061/(ASCE)0733-9445(2004)130:8(1197)

Turkstra, C.J., Madsen, H.O., 1980. Load Combinations in Codified Structural Design. J. Struct. Div. 106, 2527–2543. https://doi.org/10.1061/JSDEAG.0005599
Zhang, H., Liu, H., Ellingwood, B.R., Rasmussen, K.J.R., 2018. System Reliabilities of Planar Gravity Steel Frames Designed by the Inelastic Method in AISC 360-10. J. Struct. Eng. 144, 04018011. https://doi.org/10.1061/(ASCE)ST.1943-541X.0001991
Zhang, H., Shayan, S., Rasmussen, K.J.R., Ellingwood, B.R., 2016a. System-Based Design of Planar Steel Frames, I: Reliability Framework. Journal of Constructional Steel Research 123, 135–143. https://doi.org/10.1016/j.jcsr.2016.05.004
Zhang, H., Shayan, S., Rasmussen, K.J.R., Ellingwood, B.R., 2016b. System-Based Design of Planar Steel Frames, II: Reliability Results and Design Recommendations. Journal of Constructional Steel Research 123, 154–161. https://doi.org/10.1016/j.jcsr.2016.05.005
Ziemian, C.W., Ziemian, R.D., 2021. Efficient Geometric Nonlinear Elastic Analysis for Design of Steel Structures: Benchmark Studies. Journal of Constructional Steel Research 186, 106870. https://doi.org/10.1016/j.jcsr.2021.106870
Ziemian, R.D., 1990. Advanced Methods of Inelastic Analysis in the Limit States Design of Steel Structures. Cornell University, Ithaca, NY.
Ziemian, R.D., McGuire, W., Deierlein, G.G., 1992a. Inelastic Limit States Design. Part I: Planar Frame Studies. J. Struct. Eng. 118, 2532–2549. https://doi.org/10.1061/(ASCE)0733-9445(1992)118:9(2532)
Ziemian, R.D., McGuire, W., Dierlein, G.G., 1992b. Inelastic Limit States Design Part II: Three-Dimensional Frame Study. J. Struct. Eng. 118, 2550–2568. https://doi.org/10.1061/(ASCE)0733-9445(1992)118:9(2550)
Ziemian, R.D., Miller, A.R., 1997. Inelastic Analysis and Design: Frames with Members in Minor-Axis Bending. J. Struct. Eng. 123, 151–156. https://doi.org/10.1061/(ASCE)0733-9445(1997)123:2(151)