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System reliability-based Direct Design Method for space frames with cold–formed steel hollow sections

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ABSTRACT

Design-by-analysis methods for steel structures are receiving considerable attention from professional engineers, researchers and standard-writing groups. Designing by analysis, termed as the Direct Design Method (DDM), is premised on the use of geometric nonlinear inelastic finite element analysis to determine the ultimate strength of steel structural frames and subsequently incorporating a *system* resistance factor (ϕ_s) to account for the effects of uncertainties in geometric parameters, stiffness and strength. This paper outlines the DDM in the context of cold-formed compact Hollow Steel Sections (HSS), including the reliability analysis framework at system level underpinning the Method. The system resistance factors for a series of representative 3D frames with hollow locally stable cross-sections are derived.

1. Introduction

Steel structures are traditionally designed by a combination of a frame analysis that provides the internal actions and a design specification which provides rules for calculating the strength of members and connections for the set of internal actions (forces and moments) determined from the structural analysis. While in the past 40 years the structural frame analysis has shifted from hand-calculated based analysis to second-order elastic analysis, the two-step design approach has been utilized extensively for more than one hundred years. With the advances in structural engineering during the past two decades, the behaviour of a highly redundant structural steel system can now be precisely determined by nonlinear finite element analysis, which may be a beam element-based plastic zone analysis for compact sections, or shell element-based large deformation inelastic analysis for structures with thin-walled members prone to distortional as well as local instabilities [1–7]. The advanced nonlinear finite element analysis provides engineers an opportunity to move from the two step member-based design method to a system-based approach.

In this paper, the direct design by analysis approach is referred to as "Direct Design Method" (DDM). The DDM provides the unique feature such that system failure (ultimate frame strength) rather than member limit state is regarded as the design criterion. The frame analysis in the DDM shall incorporate all sources of important nonlinear actions affecting the structural behaviour, notably second-order effects, plasticity, semi-rigidity of connections, residual stress, initial geometric imperfection, and be able to detect all the relevant limit states (e.g., sectional yielding and member buckling) covered by the specification equations. The essence of the DDM is that the structure is modelled as realistically as possible, to the point of accurately simulating the structural response that one would achieve in physical tests of the structure. Such an analysis is termed "advanced analysis" in AS4100 [8] and AS4600 [9], "inelastic method" in AISC360-10 [10] and AISI-S100 [11], and "geometrical and material nonlinear analysis with imperfections" in European terminology. Modern finite element (FE) analysis software such as ANSYS, Strand 7, ABAQUS [12], and the open-source software OpenSees increasingly feature material and geometric nonlinear analyses which may employ beam elements and/or shell elements. Technically, the application of advanced structural analysis for system-based design has diminished considerably, especially seeing that the performance of standard desktop computers now allows sizeable structures to be analysed by advanced analysis sufficiently quickly to be a practical proposition.

The DDM has significant practical advantages over the conventional member-based design methods. By using the advanced analysis, the failures of member and connection can be explicitly assessed within a structural frame system and subsequently, the capacities of the member and connection can be directly checked without the use of design strength provisions of a structural standard. Apart from generally leading to lighter frames, the DDM provides a conceptually simple method of design with more uniform reliability of a wide spectrum of steel frames [13,14]. It also informs the designer of the failure mode and the complete path from elastic to ultimate and post-ultimate, enabling the designer to consider the consequences of failure, thus

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Table 1
Statistics of the basic random variables.

	Mean	COV	Distribution	Reference
E	E_n	0.06	Normal	[29]
F_y	$1.1F_{yn}$	0.1	Lognormal	[29]
t	$0.964t_n$	0.039	Normal	[40]
D	$1.05D_n$	0.10	Normal	[29]
L	L_n	0.25	Extreme Type 1	[29]
L_{apt}	$0.25L_n$	0.60	Gamma	[29]
W	$0.96W_n$	0.37	Extreme Type 1	[29]

COV = coefficient of variation.

Table 2
Statistical data of geometric imperfection scale factors for HSS.

	a_1	a_2	a_3
μ	$1.26 \times 10^{-4}\,L$	$4.08 \times 10^{-5}\,L$	$2.2 \times 10^{-5}\,L$
σ	$1.62 \times 10^{-4}\,L$	$5.16 \times 10^{-5}\,L$	$2.81 \times 10^{-5}\,L$

providing further incentive for using the Method.

The approach for design based on advanced analysis of the overall structural frame behaviour have been incorporated into several steel design standards, e.g., [8,10], except for earthquake design. However, even with sophisticated nonlinear finite element analysis, the true behaviour of a steel structure cannot be evaluated with certainty because numerous sources of uncertainty exist in structural loads, strength and stiffness of system, members and connections. These uncertainties give rise to risk and introduce a probability of failure. In *load and resistance factor design* (LRFD), member reliability requirements are achieved through the resistance factors using a reliability calibration process [15,16]. However, the system-based DDM has yet to address the problem of satisfying the minimum system-based reliability requirement.

A limited number of research studies have considered the system reliability implications of steel frames design by DDM, mostly for hot-rolled steel frames. A group of 16 planar low-rise steel moment frames subjected to gravity loads were considered in [17]. The study suggested that a system resistance factor about 0.86 to 0.91 is needed for the DDM to achieve a target reliability index of 3.0 on system collapse. As only the structural loads and the steel yield strength were modelled as random variables, the derived system resistance factors are likely higher than warranted. In [18,19], a reliability framework based on First-Order Reliability Method (FORM) was established for evaluating the system resistance factors for the DDM of low- to mid-rise planar braced and moment frames. The study showed that a system resistance factor of 0.80–0.85 would be required to achieve a target system reliability index of 3.0–3.25 under gravity loads. For cold-formed steel structures, the system resistance factors for storage rack structures and portal frames were derived in [20], in which the DDM is based on shell element-based structural analysis. In this paper, the procedures of deriving the system resistance factors for the DDM are presented with particular emphasis on spatial cold-formed steel frames with compact hollow sections. Sixteen baseline frames (eight moment frames and eight braced frames) with different configurations and failure modes are investigated. The system resistance factors corresponding to different levels of target reliability are derived.

2. Direct design method

In the DDM, the frame analysis and design check are achieved in one integrated step, taking the LRFD format,

$$\phi_s R_n \geqslant \sum \gamma_i Q_{ni} \tag{1}$$

where R_n is the nominal ultimate strength of the frame predicted by advanced analysis using the nominal geometric and material properties while ϕ_s is a system resistance factor that considers the effects of uncertainties in frame strength, geometric properties and stiffness, Q_{ni} are applied nominal loads (gravity, wind loads) to the whole frame, and γ_i are load factors from the loading standards (e.g., ASCE 7 [21]).

When conducting a nonlinear finite element analysis, the frame's ultimate strength is predicted using a static pushdown analysis for gravity loads, or a static pushover analysis for combined wind and gravity loads. The loads are increased proportionally and incrementally in the analysis until system failure. The peak point of the load-displacement response is determined as the frames' ultimate strength. In case of there is no descending branch in the load-deflection response, the frame's ultimate strength is chosen as the point where the gradient of the load-displacement response is decreased to 5% of its initial gradient [22]. The applied load is often expressed as the product of a scaling load factor λ and a reference applied load. Thus, Eq. (1) is equivalent to $\lambda_u \geq 1/\phi_s$, in which λ_u is denoted as the ultimate frame load factor.

3. System reliability assessment

The system resistance factor ϕ_s in Eq. (1) needs to be determined using concepts of probabilistic limit state design accounting for the uncertainties in loads and system resistance to achieve a design with the predefined level of system reliability [16]. In the structural reliability theory, the safety of a structure is quantified by its probability of failure, P_f, i.e., the likelihood of reaching the limit state(s) during its lifetime. A common approach to evaluate structural reliability is through the Monte Carlo Simulation (MCS) [23]. Computing the probability of failure of a structure by MCS involves the following steps: (1) in each simulation, randomly generate samples of the uncertain properties of a structure (e.g., elastic modulus, yield stress, residual stress, initial geometric imperfections, strain hardening etc.) and (2) perform an advanced analysis to check if the ultimate strength of the (random) frame is equal to or greater than the applied loads: (3) repeat a sufficient number of simulations, and estimate the probability of failures as $P_f = n/N$, in which N is denoted as the total number of simulations performed, and n represents the number of simulations where the structure cannot withstand the applied loads. More advanced simulation-based methods using variance-reduction techniques are also available for system reliability evaluations [24–27]. In probability-based structural design, the so-called reliability index, β, is customarily used as an alternative of reliability to P_f, with the relationship $\beta = \Phi^{-1}(1 - P_f)$, where Φ^{-1} is denoted as the inverse of the distribution function of a standard normal distribution [16,23].

For a cold-formed steel structure, the important random variables that need to be accounted for in reliability analyses include: structural loads, yield stress, Young's modulus, residual stress, cross-sectional properties, and initial geometric imperfections. Furthermore, the inherent randomness in material properties and loads, the model uncertainty of advanced analysis also needs to be considered. The probabilistic characteristics for these uncertainties will be discussed in detail in Section 4.

The computational cost of the aforementioned direct Monte Carlo method can be very intensive, because each simulation requires a nonlinear structural analysis. The direct Monte Carlo method is impractical for code-development which involves system reliability evaluations of a large number of steel frames. To overcome this difficulty, the First-Order Reliability Method (FORM) [23,28] is adopted for system reliability assessment in this study. In this approach, the stochastic characteristics of a steel frame are first estimated using a relatively small number of simulations, and subsequently combined with the statistics of applied structural loads to estimate the probability of failure.

Consider a steel frame at its limit under the gravity load combination $1.2\,D_n + 1.6\,L_n$ specified in ASCE 7 [21], in which D_n and L_n are the nominal dead and live loads, respectively. The design equation of Eq.