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Reliability of stainless steel frames designed using the Direct Design Method in serviceability limit states

Itsaso Arrayago [a,*], Kim J.R. Rasmussen [b]

[a] *Universitat Politècnica de Catalunya (UPC), Dept. of Civil and Environmental Engineering, Spain*
[b] *The University of Sydney, School of Civil Engineering, Australia*

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ABSTRACT

Steel structures can be consistently and efficiently designed using system-based *design-by-analysis* approaches such as the Direct Design Method. However, since direct design approaches lead to potentially lighter structural configurations, they can also result in larger deformations under service loads. Thus, greater attention may be required to serviceability limit states in structures designed using *design-by-analysis* approaches than for structures designed elastically at their ultimate limit state following current *two-stage* approaches, especially for materials showing highly nonlinear stress *vs* strain responses such as stainless steel alloys. With the aim of investigating the influence of allowing larger deformations in the ultimate limit state design of stainless steel structures, this paper presents an explicit analysis framework for assessing serviceability reliability at system level. Using this framework, the paper investigates the serviceability reliability of cold-formed stainless steel portal frames designed using the Direct Design Method for different load cases, including the gravity load and the combined gravity plus wind load combinations. The study considers six baseline frames covering the most common stainless steel families and international design frameworks (i.e., Eurocode, US and Australian frameworks), for which the reliability of vertical deflection and lateral drift serviceability limit states is investigated using advanced numerical simulations and First-Order Reliability Methods. From the comparison of the calculated average annual reliability indices and the relevant target reliabilities for the different design frameworks, it was found that the reliability of stainless steel frames appears to be adequate for the serviceability limit states investigated for the Eurocode, US and Australian frameworks.

1. Introduction

Serviceability limit states are conditions in which the normal use of structures is compromised due to excessive deformations of components, local damage, damage to services or machinery and occupant discomfort. The serviceability limit state criteria include deflections and vibrations, and although they do not generally result in safety issues, they can have notable economic consequences and should be carefully addressed in structural engineering practice. The main types of serviceability non-compliances are related to the excessive deflection of horizontal members or lateral displacement of structures, in which cases the limit state function can be written as per in Eq. (1), where δ_a is the allowable deflection limit and $\Delta(t)$ is the deflection of the structure at time t due to the applied loading [1].

$$g = \delta_a - \Delta(t) \quad (1)$$

Allowable deflection limits δ_a are usually specified in design codes or can be defined by the engineer or the building authorities, and can be either constant (deterministic) or considered as uncertain variables [1]. On the other hand, the actual deflection of the structure $\Delta(t)$ can be estimated directly from suitable structural analyses. Until now, elastic analyses were customarily sufficient to determine $\Delta(t)$ when the design of conventional steel structures was based on the traditional *two-step* approach adopted in current international structural codes [2–4]. However, most relevant design codes for carbon steel and stainless steel structures in the Australian, US and Eurocode frameworks (i.e., AS/NZS 4100 [2], AISC 360 [3], AISC 370 [5], prEN 1993-1-14 [6]) already incorporate preliminary versions of new direct system-based *design-by-analysis* approaches that have been developed over the last decade in response to the rapid advances in design software and exponential growth in the computational power of desktop computers. The Direct Design Method (DDM) is one such *design-by-analysis* methods and is

* Corresponding author at: Jordi Girona 1-3, Building C1, 08034 Barcelona, Spain.
E-mail address: itsaso.arrayago@upc.edu (I. Arrayago).

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Table 1
Summary of system safety factors $\gamma_{M,s}$ and resistance factors ϕ_s for the direct ultimate limit state design of cold-formed stainless steel frames [19,20].

Loading type	Eurocode framework	US framework	Australian framework
Gravity loads	$\gamma_{M,s} = 1.15$	$\phi_s = 0.95$	$\phi_s = 0.90$
Wind loads	$\gamma_{M,s} = 1.20$	$\phi_s = 0.90$	$\phi_s = 0.90$

based on geometric and material nonlinear (advanced) finite element analyses that incorporate the effect of actual material properties, geometric imperfections and residual stresses. Design recommendations for the direct design of hot-rolled carbon steel frames [7–10], cold-formed steel frames [11,12], steel storage rack frames [13] and steel scaffolds [14] using the DDM have been proposed in recent years, and the extension of the Method to new materials and structural types is currently underway. Recommendations for the direct design of stainless steel frames have been recently proposed as a result of the research works carried out in the frame of the NewGeneSS project [15], which accounted for the pronounced nonlinearity of the stress-strain response, the significant strain-hardening properties exhibited by stainless steel alloys, and the consequent need for independent reliability calibrations [16,17]. Using the variability models specifically calibrated for stainless steel member and material properties reported in [18], the reliability of cold-formed stainless steel portal frames subjected to gravity loads (i.e., dead loads and imposed (live) loads) and combinations of dead and wind loads at their ultimate limit state was investigated in [19,20], from which the system safety factors $\gamma_{M,s}$ and resistance factors ϕ_s reported in Table 1 were proposed for the three main international design frameworks, i.e., the Eurocode, US and Australian frameworks, based on suitable target reliability indices.

Although key advantages of the structural performance of stainless steel alloys such as the considerable strength reserve due to strain-hardening and the capacity to sustain large deformations before reaching collapse can be fully exploited through advanced system-based design approaches, these methods generally lead to potentially lighter structures and result in larger ultimate limit state deformations [7]. When combined with the nonlinear behaviour exhibited by stainless steel alloys even for moderate levels of strain, the larger deformations and loss of material stiffness result in more significant destabilising effects and thus, greater attention is required to serviceability limit states for stainless steel structures than for carbon steel structures. In fact, international design standards for stainless steel structures (prEN 1993-1-4 [21], ASCE 8 [22] and AS/NZS 4673 [23]) require that the effect of the nonlinear stress-strain behaviour is taken into account when estimating deflections by using a reduced modulus of elasticity E_r, which is based on the secant modulus corresponding to the design stress, instead of the initial Young's modulus E. This effect is implicitly taken into account in *design-by-analysis* approaches, but does require a separate serviceability limit state check, the reliability of which is investigated herein, in addition to resistance considerations.

This paper sets out a general framework for assessing the serviceability limit state reliability at system level and presents the reliability analysis of stainless steel portal frames under gravity loads and combined dead and wind load configurations. The paper introduces six baseline frames used in the analysis and briefly describes the finite element models developed in Section 2, while in Section 3 target reliability indices required for serviceability limit states for the three main international design frameworks are presented, and suitable load combinations and stochastic models for different loading types and reference periods are described. Section 4 presents the input information necessary to derive reliability analyses, including statistics of the uncertain variables affecting the behaviour of stainless steel frames and the resulting frame stiffness characteristics. Section 4 also describes the methodology adopted in the analysis and provides an overview of the serviceability limit state criteria for portal frames available in the literature. Finally, Section 5 presents and discusses the results of the serviceability reliability calculations for vertical deflections under gravity loads and for lateral drifts under wind loads for the three design frameworks.

2. Description of baseline frames and finite element models

This Section briefly describes the main characteristics of the baseline frames considered in this study, and provides a summary of the key features of the finite element models developed and the validation thereof against experimental results.

2.1. Baseline cold-formed stainless steel frames

The study presented in this paper is based on six portal frames made from cold-formed stainless steel rectangular hollow section (RHS) tubes, including the three most common stainless steel families. As indicated in Table 2, the austenitic stainless steel grade EN 1.4301 (ASTM 304) was adopted for Frames 1 and 2, while the duplex grade EN 1.4462 (ASTM 2205) was chosen for Frames 3 and 4 and the ferritic grade EN 1.4003 (ASTM UNS S40977) was used for Frames 5 and 6. Frames 1, 3 and 5 featured RHS 150×100×4 tubes, while the cross-section adopted for Frames 2, 4 and 6 was RHS 250×150×4, where the H×B×t notation indicates the cross-section height H, width B and thickness t. The nominal overall frame dimensions for each frame are also reported in Table 2, including the span length and the heights at eaves and at roof ridges. These baseline frames are identical to those used in previous publications on ultimate limit state reliability calibrations under gravity loads [19] and combined gravity plus wind load [20] cases, and the full details of the frames and their failure modes can be found in these publications. For the joints at the base, apex and eave sections the stiffened welded connections suitable for rigid portal frames recommended in [24] were adopted, typical examples of which are shown in Fig. 1. The moment–rotation curves of this type of connections can be reasonably described by a bi-linear model defined by two stiffness parameters and two moment capacities [12,19], the details of which are given in [19,20] for the baseline frames. Table 3 reports the nominal initial stiffness values adopted at the base, apex and eave welded connections, while the values of the remaining parameters can be found in [19,20]. It should be also noted that the study presented in this paper assumed that the ultimate moment and ductility capacities of the joints were not reached at the failure states of the frames, and thus did not consider joint failure.

Table 2
Key parameters defining baseline stainless steel portal frames (adapted from [19,20]).

Portal frame	Stainless steel family and grade	Cross-section	Span [m]	Height at eaves [m]	Height at roof ridge [m]
Frame 1	Austenitic	150×100×4	8.0	4.8	6.0
Frame 2	(1.4301–304)	250×150×4	10.0	6.4	7.0
Frame 3	Duplex	150×100×4	8.0	4.8	6.0
Frame 4	(1.4462–2205)	250×150×4	12.0	6.6	7.4
Frame 5	Ferritic	150×100×4	8.0	4.8	6.0
Frame 6	(1.4003-S40977)	250×150×4	11.0	6.5	7.2

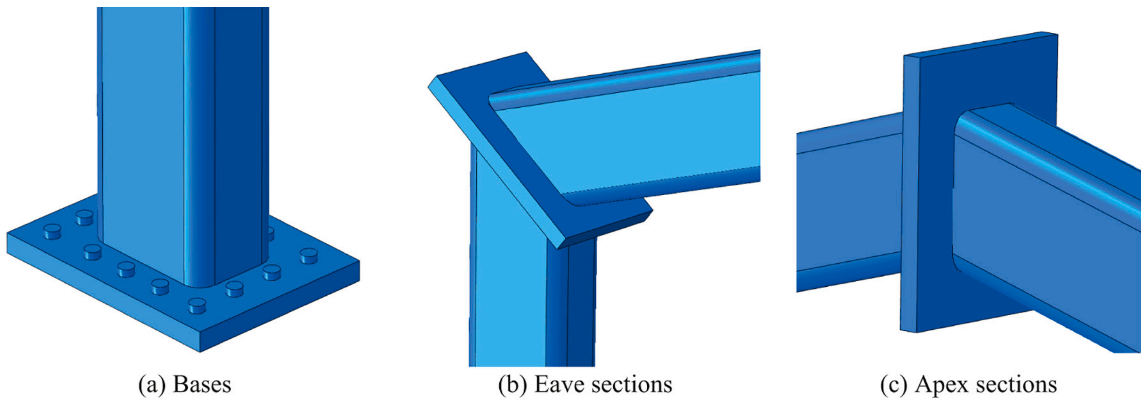

Fig. 1. Typical joint configurations considered for the baseline stainless steel portal frames.

Table 3
Nominal initial stiffness values adopted at the base, apex and eave connections (from [19,20]).

Portal frame	Nominal initial stiffness [kNm/rad]		
	Base joints	Apex joint	Eave joints
Frame 1	4000	6600	6400
Frame 2	5000*	7500	7000
Frame 3	4000	6600	6400
Frame 4	5000*	9500	9000
Frame 5	4000	6600	6400
Frame 6	5000*	8500	8000

* Pinned base connections were assumed for the gravity load cases in these frames.

The serviceability limit states considered in this study covered vertical deflections under gravity loads (i.e., dead (G) and imposed (Q) loads) and lateral drifts due to wind (W) loads. These load cases, including the wind load pattern considered, are illustrated in Fig. 2 and correspond to those adopted in the ultimate limit state reliability calibrations carried out in [19,20]. Since previous research works [7–13,19,20] have shown that it is not possible to achieve uniform levels of reliability for different imposed-to-dead load ratios, nor for different wind-to-dead load ratios, this study considered several nominal imposed-to-dead load ratios $\alpha = Q_n/G_n$ for vertical deflection reliability studies, while different values of design (nominal) dead loads G_n were investigated for lateral drift reliability calculations. Although the imposed-to-dead load ratio α typically ranges from 0.5 to 4.0 [25], the common load ratio for hollow section steel structures is $\alpha = 2.0$ [11,19], and thus imposed-to-dead load ratios close to 2.0, between 1.0 and 2.5, were considered. For combined gravity plus wind load cases, the effect of wind-to-dead load ratios was investigated through three values of design (nominal) dead loads for each frame, namely GW1, GW2 and GW3 load scenarios, representing relatively light, medium and heavy wind loads. The nominal values of the dead loads considered in this study can be found in [20].

2.2. Development and validation of finite element models

System strengths and deflections of stainless steel frames were estimated from advanced numerical simulations developed using the finite element (FE) software ABAQUS [26]. Frames were modelled using S4R shell elements, which have been widely used in the simulation of cold-formed stainless steel members [27,28] and frames [19,20,29], and a mesh size of approximately 25 mm × 25 mm was chosen from a mesh convergence study, striking a suitable compromise between accuracy and computational cost. The FE models included all features relevant to the response of cold-formed stainless steel portal frames, including initial geometric imperfections, connection behaviour, suitable material properties and residual stresses. Global initial geometric imperfections were introduced into the FE models by directly modifying the position of the nodes according to the relevant out-of-plumb angle, while local and member imperfections were introduced from prior linear buckling analyses, to which appropriate amplitudes (extensively discussed in [19,20]) were assigned for nominal frames and structural analysis models.

Connections at the bases were modelled using *HINGE* type connector elements available in the ABAQUS library, while for the apex and eaves connections a system of rigid *BEAM* and *UJOINT* connector elements were adopted and user-defined bi-linear moment–rotation curves were assigned, following the configuration described in [19]. The characteristic nonlinear stress–strain properties exhibited by stainless steel alloys and their considerable strain-hardening are commonly described using two-stage Ramberg-Osgood material models [30,31], which are based on basic material properties such as the Young's modulus E, the yield stress f_y, the ultimate tensile strength f_u and the corresponding ultimate strain ε_u, as well as on the strain-hardening exponents n and m. Values (or predictive expressions) for these material parameters are provided in the different structural [5,6,22,23,32] or material [33] standards. Material properties were introduced into the FE models through user-defined nonlinear true stress *vs* true plastic strain curves obtained from appropriate material parameters and the two-stage Ramberg-

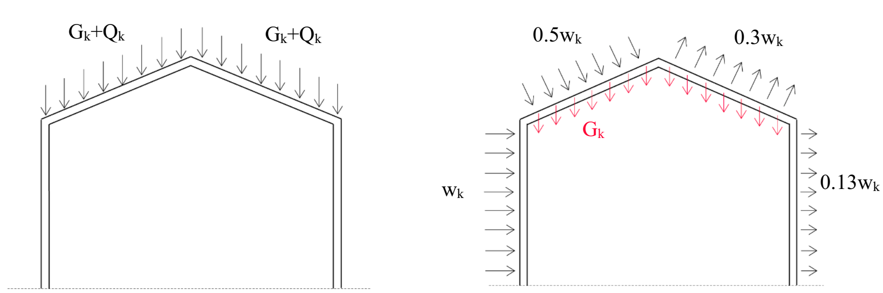

Fig. 2. Gravity and combined dead plus wind load patterns adopted in the analysis (from [19,20]).

Osgood model, while residual stresses corresponding to the model proposed in [34] for cold-formed rectangular hollow sections were input using the **INITIAL CONDITIONS, TYPE-STRESS* option. Further details of the material properties and residual stress patterns can be found in [19,20].

In the models reproducing stainless steel frames under dead and imposed loads, gravity loads were applied at the upper faces of the rafters and the geometrically and materially nonlinear analyses were performed using the Static Riks method [26] until the frames collapsed. Alternatively, the behaviour of the frames under combined dead and wind loads was modelled using static pushover analyses, in which dead loads were first fully applied at the upper faces of the rafters followed by the wind loads introduced incrementally at the rafters and columns. In this case, the analyses corresponding to the dead loading increments were performed using the Static General method, while wind loading increments were solved through the Static Riks method [26].

The finite element models were validated against the experimental results on austenitic stainless steel frames reported in [29,35]. These frames featured similar joint and base connections to those considered in this study and were subjected to vertical and horizontal loading conditions. It should be noted that small modifications were made to the developed ABAQUS scripts to adapt them to the particular characteristics of the experimental set-up, including the introduction of the loading and boundary conditions, although the main considerations discussed in this Section were still valid. Full details of the FE model validation can be found in [19], which indicated that the models were capable of accurately replicating the behaviour of the stainless steel frames under vertical and horizontal loading, as per the comparison between experimental and numerical ultimate loads, vertical load–deflection curves, horizontal load–displacement paths and failure modes.

3. Target reliability indices and serviceability loads

This Section introduces the target reliability indices required for serviceability in the three design frameworks investigated and describes the suitable load combinations and design loads to be adopted in serviceability limit state checks. The Section also presents the stochastic models available in the literature for the different loading types and a variety of reference periods.

3.1. Target reliability indices for serviceability

Safety factors and resistance factors specified in different structural standards were calibrated in order to meet specific target reliability indices in ultimate limit state (ULS) design, which are set out in the standards along with other aspects related to the reliability framework adopted in the development of the different design specifications. The minimum target reliability index recommended in prEN 1990 [36] for Class CC2 structures and a reference period of 50 years in ultimate limit state is β_0=3.8, which is commonly adopted in reliability analyses carried out in the Eurocode framework, while the values historically assumed for steel structures in the US and Australian frameworks range between β_0=2.5 and β_0=3.0 [37,38], corresponding to Risk Categories I and II structures in the ASCE 7 [39] Specification for ductile failure modes. In contrast, and since safety is not generally an issue in serviceability limit states (SLS), serviceability checks do not require the use of safety factors or resistance factors and the reliability criteria for SLS generally results in lower prescribed target indices β_0 than for ULS.

In prEN 1990 [36], the target reliability index for Class CC2 structures for irreversible serviceability limit states is β_0=1.50 for a reference period of 50 years, which corresponds to an approximate annual target index of β_0=2.90 (or an annual probability of exceedance of 0.2%). It should be noted that these two β_0 values correspond to the same reliability level but to different reference periods (50 and 1 years, respectively), which may or may not coincide with the design service lifetime of the structure. On the other hand, the serviceability load combinations included in the Serviceability Appendix in ASCE 7 [39] are based on annual exceedance probabilities of around 5–10%, which correspond to target reliability indices of about β_0=1.28–1.64. Likewise, the AS/NZS 1170.0 [40] Specification defines an annual probability of exceedance for serviceability limit states equal to 1/25 for wind and snow load cases, which is equivalent to a target reliability index of β_0=1.75.

These β_0 values for serviceability limit states (i.e., β_0=2.90, β_0=1.64 and β_0=1.75 for the Eurocode, US and Australian frameworks, respectively) will be used in Section 5 to evaluate the reliability of stainless steel frames under gravity and combined gravity plus wind load combinations. It is worth emphasizing that although the target reliability index considered for the Eurocode design framework is significantly higher than those specified in the US or Australian frameworks, the Eurocode index corresponds to irreversible serviceability considerations while the others refer to short-term effects. These target values need to be compared with reliability indices calculated using the corresponding serviceability load combinations prescribed in the different standards for serviceability, which are discussed in the following Section.

3.2. Load combinations and design loads for serviceability

International standards specify different load combinations to be adopted when checking ultimate or serviceability limit states [36,39,40], and are usually based on factored nominal loads. Nominal loads defined in international loading standards are traditionally based on a 50-year return period [39,41–43], although some wind loading standards have recently adopted considerably higher return periods (e.g., the return periods adopted in the current ASCE 7 [39] and AS/NZS 1170.2 [44] specifications are T=700 years and T=500 years, respectively). When factored according to the ultimate limit state load combinations [36,39,40], design loads have significantly small probabilities of being exceeded in 50 years (which is the average service lifetime of typical steel structures), and therefore it does not seem reasonable to base serviceability criteria on such severe requirements [45]. Conversely, suitable loads for checking deflections can be based on the assumption that the deflection limits should not be exceeded, on average, more than once during a tenancy period [45] and may therefore be only a fraction of the factored nominal loads. Thus, serviceability load combinations are defined using loads corresponding to lower periods of reference or feature short-term and long-term combination factors.

prEN 1990 [36] defines three different load combinations for serviceability, namely the characteristic, frequent and quasi-permanent combinations, each of which is affected by different combination factors. Since the target reliability index given in prEN 1990 [36] corresponds to irreversible serviceability limit states, the characteristic load combinations given by $\sum F_d = \sum G_{k,i}+Q_{k,1}+\sum \psi_{0,j}Q_{k,j}$ was adopted in this study, where $\sum F_d$ is the total service design load, $G_{k,i}$ are the characteristic permanent actions, $Q_{k,1}$ is the characteristic leading variable action, $Q_{k,j}$ represent the accompanying characteristic variable actions and $\psi_{0,j}$ are the combination factors. Since load cases investigated in this paper correspond to a permanent (dead) action G_k acting in combination with one single variable action (imposed load Q_k or wind load W_k), characteristic load combinations for serviceability can be simplified to $\sum F_d = G_k+Q_k$ and $\sum F_d = G_k+W_k$. On the contrary, target reliabilities for serviceability in ASCE 7 [39] refer to short-term effects, which for the load cases investigated in this paper result in $\sum F_d = D_n+L_n$ and $\sum F_d = D_n+W_a$, where D_n and L_n are the nominal dead load and imposed loads, respectively, and W_a refers to the wind load corresponding to the serviceability wind speed (10-year mean recurrence interval for typical buildings). Since according to [10,11,46] 10-year wind loads can be obtained by multiplying the 50-year wind loads by a conversion factor of 0.7, the wind design load $W_a = 0.7W_{n,\ 50}$ was adopted in this study for the US design framework. Finally, load combinations prescribed in the AS/NZS 1170.0 [40] Specification for serviceability limit states are based on the nominal permanent loads G_n, the short-term variable loads $\psi_s Q_n$ and the serviceability wind load W_s, based on a wind speed corresponding to the annual probability of exceedance for serviceability. The short-term factor for imposed actions in AS/NZS 1170.0 [40] is $\psi_s = 0.70$ for roofs and most common floor uses, and thus the load combinations adopted in this study for the Australian framework were $\sum F_d = G_n + \psi_s Q_n$ and $\sum F_d = G_n + W_s$, where $\psi_s = 0.70$.

3.3. Stochastic models for serviceability loads

When serviceability limit states are evaluated from a probabilistic point of view, the definition of stochastic models accounting for the variability of the structural loads involved and the selection of suitable periods of reference is fundamental. Since the target reliability indices described in Section 3.1 are based on annual probabilities of exceedance, it might seem reasonable to adopt a reference period of 1 year for reliability calculations. However, the load models reported in the literature for imposed and wind loads for reference periods shorter than the usual 50-year period do not always include annual reference periods. Load models available in the literature for short reference periods generally correspond to the mean duration of typical tenancies, which range between 8 years and 10 years for buildings [45,47,48], and thus these periods have been adopted in this paper: 8 years for vertical deflection serviceability limit states under gravity loads and 10 years for lateral drift serviceability under wind loads, in line with the availability of stochastic models for loads and previous studies [8,11,45,49]. Nevertheless, the reliabilities corresponding to a reference period of 1 year have been estimated in all cases to allow a direct comparison between load cases and with the annual target reliability indices prescribed in the different design frameworks.

Probabilistic models for the loads corresponding to several reference periods (8 years, 10 years and 50 years), based on the nominal loads for different return periods (T=50 years, T=500 years and T=700 years) are reported in Table 4 for the dead loads (G or D), imposed loads (Q or L) and wind loads (W). Note that the wind loads assumed in this paper correspond to ordinary winds, limited to the regions not affected by tropical cyclones. Mean values (as fractions of the code-specified nominal values), coefficients of variation (COV) and distribution types are provided for the Eurocode [19,20], US [25,51,52] and Australian [12,19,53] frameworks. These values were obtained from the literature, as indicated, or estimated from the models available when no information was directly available, as described below. From the load model proposed in [52] for imposed loads in the US framework and an 8-year reference period, the equivalent models for the Eurocode and Australian frameworks were estimated assuming that the factors by which the mean imposed loads are reduced from a 50-year reference period to a 8-year reference period were the same for the three design frameworks. Therefore, using the statistics for reference periods of 50 years reported in Table 4, mean values of the maximum imposed loads for an 8-year reference period equal to $0.52 \cdot Q_k$ and $0.65 \cdot Q_n$ were adopted for the Eurocode and Australian frameworks, respectively, with the same COVs.

Regarding the wind loads, stochastic models corresponding to a reference period of 10 years were assumed for the three design frameworks, in line with recent serviceability reliability studies on steel frames designed by direct analysis [8,11]. These studies assumed an average factor of 1.45 to relate the maximum wind loads corresponding to reference periods of 50 years and 10 years, and adopted a coefficient of variation of 0.50 for the 10-year reference period wind loads. Using the wind load model reported in [25] for a reference period of 50 years and the 1.45 factor, a mean maximum wind load of $0.51 \cdot W_{n,\,50}$ can be obtained for the US framework and a reference period of 10 years, as reported in Table 4. Since no stochastic models are available in the literature for the Eurocode and the Australian frameworks for reference periods equivalent to the mean duration of tenancies, a similar approach to that adopted for the US framework was assumed. Accepting that the factor by which the mean maximum wind loads are reduced from a 50-year reference period to a 10-year period are the same for the three frameworks, and using the wind load statistics proposed in [20,53] for reference periods of 50 years (see Table 4), stochastic wind load models with mean values of $0.49 \cdot W_k$ and $0.47 \cdot W_{n,50}$ can be adopted for the 10-year reference period wind loads in the Eurocode and Australian framework, respectively, with a COV of 0.50.

Note that while the characteristic wind loads W_k prescribed in prEN 1991-1-4 [42] correspond to a return period of 50 years, the nominal loads specified in the latest versions of the ASCE 7 [39] and AS/NZS 1170.2 [44] specifications correspond to return periods of T=700 years (denoted by $W_{n,700}$) and T=500 years (denoted by $W_{n,500}$), respectively. Since wind load models reported in the literature are traditionally referred to the nominal loads corresponding to return periods of T=50 years, they need to be updated to adhere to the revised nominal wind loads. This can be done using the approximate relationship between the wind loads for a T-year return period W_T and the wind load for a 50-year return period W_{50} given in Eq. 2 [25], which is valid for most of the non-hurricane-prone regions of the US [39]. Eq. 2 results in a $W_{n,700}/W_{n,50} = 1.60$ ratio for the US framework, which leads to a mean maximum wind load of $0.33 \cdot W_{n,700}$ for a wind load corresponding to a 10-year reference period, as reported in Table 4. In the absence of a specific equation for

Table 4
Statistics of loads.

Framework	Load type	Reference period	Mean	COV	Statistical distribution	Reference
Eurocode framework	Dead load G	–	$1.00 \cdot G_k$	0.10	Normal	[50]
	Imposed load Q	50 years	$0.80 \cdot Q_k$	0.25	Extreme Type I	[19]
	Imposed load Q	8 years	$0.52 \cdot Q_k$	0.32	Extreme Type I	–
	Wind load W	50 years	$0.72 \cdot W_k$*	0.36	Extreme Type I	[20]
	Wind load W	10 years	$0.49 \cdot W_k$*	0.50	Extreme Type I	–
US framework	Dead load D	–	$1.05 \cdot D_n$	0.10	Normal	[51]
	Imposed load L	50 years	$1.00 \cdot L_n$	0.25	Extreme Type I	[51]
	Imposed load L	8 years	$0.65 \cdot L_n$	0.32	Extreme Type I	[52]
	Wind load W	50 years	$0.75 \cdot W_{n,50}$*	0.35	Extreme Type I	[25]
	Wind load W	10 years	$0.51 \cdot W_{n,50}$*	0.50	Extreme Type I	–
	Wind load W	10 years	$0.33 \cdot W_{n,700}$†	0.50	Extreme Type I	–
Australian framework	Dead load G	–	$1.05 \cdot G_n$	0.10	Normal	[12]
	Imposed load Q	50 years	$1.00 \cdot Q_n$	0.25	Extreme Type I	[19]
	Imposed load Q	8 years	$0.65 \cdot Q_n$	0.32	Extreme Type I	–
	Wind load W	50 years	$0.68 \cdot W_{n,50}$*	0.39	Extreme Type I	[53]
	Wind load W	10 years	$0.47 \cdot W_{n,50}$*	0.50	Extreme Type I	–
	Wind load W	10 years	$0.32 \cdot W_{n,500}$‡	0.50	Extreme Type I	–

* Nominal wind load is based on a return period of T = 50 years.
† Nominal wind load is based on a return period of T = 700 years.
‡ Nominal wind load is based on a return period of T = 500 years.

Australian non-hurricane regions, Eq. 2 has been adopted in this study to estimate the $W_{n,500}/W_{n,50}$ ratio for the Australian framework, which results in a mean maximum wind load of $0.32 \cdot W_{n,500}$ for a reference period of 10 years.

$$W_T = W_{50}[0.36 + 0.10 ln(12T)]^2 \quad (2)$$

It should be emphasized that, in addition to the stochastic models for wind loads, it is also necessary to update the design wind loads discussed in Section 3.2 for the US and Australian design frameworks to convert these loads to the nominal wind load based on 700-year and 500-year return periods. Considering that the serviceability wind load is $0.7 \cdot W_{n,50}$, and using Eq. 2, the service loads result in $0.44 \cdot W_{n,700}$ for the US framework and in $0.46 \cdot W_{n,500}$ for the Australian framework.

In the reliability analyses carried out in this paper, the statistics corresponding to 8-year reference periods reported in Table 4 were used for imposed loads in conjunction with the load combinations described in Section 3.2 for the analysis of vertical deflections under gravity loads. For the reliability assessment of lateral drifts under wind loading, the load models corresponding to 10-year reference periods and design loads referred to the W_k, $W_{n,700}$ and $W_{n,500}$ nominal wind loads were adopted for the Eurocode, US and Australian frameworks, respectively.

4. Reliability analysis for serviceability limit state

This Section presents the input information necessary to perform the serviceability reliability analyses of stainless steel frames under different loading configurations. The statistics of the random variables affecting the response of stainless steel structures are described first, followed by the methodology adopted in the reliability study. Subsequently, the Section provides an overview of the serviceability limit state criteria available in the literature for portal frames and describes the procedure adopted in the derivation of suitable stochastic models for frame flexibility.

4.1. Uncertain variables affecting the behaviour of frames

Numerical models representing the nominal frames investigated in this paper were built following the material properties, residual stresses, initial geometric imperfections, connection behaviour and loading patterns prescribed in the prEN 1993-1-14 [6], AISC 370 [5] and AS/NZS 4100 [2] specifications for the Eurocode, US and Australian frameworks, respectively. However, the determination of suitable stochastic distributions for system strength and stiffness of actual stainless steel frames under different loading configurations is fundamental to the calculation of reliability indices in ultimate and serviceability limit states. For this, extensive numerical simulations accounting for the variability of all the variables affecting the behaviour of stainless steel frames were carried out in [19,20], based on the finite element model described in Section 2.2.

The statistical distributions adopted in the study for cross-section geometric properties, material properties, imperfections (at frame, member and local levels), residual stresses and connection behaviour were based on the probabilistic models proposed in [18] for stainless steel structural members, as well as in previous reliability studies on cold-formed steel portal frames [12] when available measurements were insufficient to derive specific stochastic models for stainless steel structures. Details of these statistical distributions, in addition to the correlations assumed for the different random variables, can be found in [18–20]. In addition to these variables, system strength and stiffness values included the effect of model uncertainly accounting for the assumptions and approximations made when using advanced FE simulations. Since it was not possible to derive a specific probabilistic characterization of the model uncertainty from the comparison between the numerical and experimental results of stainless steel frames due to the limited number of specimens available, an unbiased log-normal distribution with a COV of 0.05 was adopted in this study, as recommended in the literature [48,50].

4.2. Serviceability reliability analysis method

With the aim of assessing the reliability of stainless steel frames designed using the DDM provisions proposed in [19,20] in serviceability limit states, the probability of failure P_f (i.e., the probability of actual deflections in the structures exceeding specified allowable deflection limitations) of the six baseline frames under different loading scenarios was estimated in this study. The probability of failure can be evaluated using direct Monte Carlo simulations, which was the approach followed in previous reliability studies in the context of the DDM [8,11], but such methodology requires that approximately 10,000 simulations be carried out in order to accurately estimate P_f for serviceability limit states. Since the computational effort to perform this number of simulations is prohibitive when these are based on nonlinear shell-finite element analyses, more efficient reliability analysis methods need to be adopted.

The study presented in this paper was based on the methodology proposed in [45], in which reliability indices β were calculated based on First-Order Reliability Methods (FORM). According to [45], the limit state function for serviceability considerations given in Eq. 1 can be re-written as $g = \delta_a - K \cdot F$, where δ_a is the allowable deflection, K is the stiffness of the structure and F is the load considered, from which the probability of failure can be calculated as $P_f = P(g \leq 0)$, since $g \leq 0$ defines the failure domain. In the case of stainless steel frames subjected to gravity loads only, K corresponds to the vertical stiffness of the frame at the apex joint K_v and F represents the total gravity load, including dead G (or permanent) and imposed Q loads. Conversely, when the reliability of frames under combined gravity plus wind loads is investigated, K is the lateral stiffness of the frame K_w and F is the wind load W. Values of allowable deflections δ_a are discussed in Section 4.3 and depend on the load case: while δ_a corresponds to the vertical apex deflection $\delta_{a,v}$ when gravity load scenarios are considered, it refers to the lateral drift $\delta_{a,w}$ when dead plus wind load cases are investigated. The procedure to compute the probability of failure of each frame has been extensively reported in the literature [7–14,19,20,45], and thus only a summary of the main steps is provided in this Section.

(1) Determine the probabilistic models (distribution type and parameters) for the system stiffnesses K_v and K_w under gravity and wind loads, respectively, based on the advanced finite element models accounting for all relevant uncertainties carried out in [19,20]. Through the adoption of Latin-Hypercube sampling (LHS) techniques, the number of required simulations was reduced to 200 cases per frame and load case. This is further discussed in Section 4.4.
(2) At their limit state, nominal frames must satisfy $\delta_{a,v} = K_{v,n}(\gamma_G G_n + \gamma_Q Q_n)$ or $\delta_{a,w} = K_{w,n}(\gamma_W W_n)$ for the vertical deflection and lateral drift checks, respectively, using the nominal stiffnesses ($K_{v,n}$ and $K_{w,n}$), appropriate load combinations (i.e., γ_G, γ_Q and γ_W factors for dead, imposed and wind loads prescribed in prEN 1990 [36], ASCE 7 [39] and AS/NZS 1170.0 [40]) and design loads for the different design frameworks, as discussed in Section 3.2. Note that the lateral drifts caused by the dead loads were neglected in combined gravity plus wind load cases since the FE simulations from Step (1) showed that they represented less than 5% of the total lateral drift values obtained for the service loads. Note also that serviceability limit state checks do not require the use of safety factors or resistance factors.
(3) Define the relationship between the nominal stiffnesses $K_{v,n}$ or $K_{w,n}$ and the loads through the Load Scale Method and the design equations introduced in Step (2) [13]. Using the imposed-to-dead load ratio $\alpha = Q_n/G_n$ for gravity loads, the nominal loads G_n, Q_n and W_n can be re-written in terms of the nominal stiffness values

Table 5
Allowable vertical deflection and lateral drift limits for steel structures.

SLS criteria	Limit	Structure type	Loads	Reference
Vertical deflection at apex, $\delta_{a,v}$	s/125	Not accessible resilient roof	Imposed load	[36]
	s/250	Accessible resilient roof		
	s/240	Roof members	Imposed load	[39]
	s/300	Roof members	Long-term gravity loads	[40]
	s/200	Roofs in general	Total loads	[56]
	s/250	Roofs in general	Imposed load	[56]
	s/360	Frames with pitch angle >3°	Dead load	[58]
	s/500	Frames with pitch angle <3°		
	s/240	–	Imposed load	[58]
	s/150	–	Wind load	[58]
	s/250	–	–	[59]
Lateral drift at eaves, $\delta_{a,w}$	H_1/300	–	Wind load	[8,11]
	H_1/400	Single storey buildings	–	[36]
	H_1/600-H_1/400	–	Wind load	[39]
	H_1/500	Side sway in columns	Wind load	[40]
	H_1/400	–	Wind load	[49]
	H_1/150	Portal frames without gantry crane	Total loads	[56]
	H_1/150	Portal frames without fragile elements	Total loads	[57]
	H_1/300	Single-storey buildings with horizontal roofs without fragile elements	Total loads	[57]
	H_1/150	Portal frame with metal cladding	Service wind	[58]
	H_1/240	Portal frame with masonry cladding	Service wind	[58]
	H_1/150	Portal frame	–	[59]
	H_1/100	Profiled metal sheeting	Wind, imposed loads	[60]
	H_1/200	Precast concrete units		

Note: s is the frame span and H_1 is the height at eaves.

as per in Eq. 3 and Eq. 4 for the gravity load and wind load cases, respectively.

$$G_n = \frac{\delta_{a,v}}{K_{v,n}(\gamma_G + \gamma_Q \alpha)} \text{ and } Q_n = \frac{\alpha \delta_{a,v}}{K_{v,n}(\gamma_G + \gamma_Q \alpha)} \tag{3}$$

$$W_n = \frac{\delta_{a,w}}{K_{w,n}\gamma_W} \tag{4}$$

(4) Re-define the probabilistic models for the different loads from the relationships established in Step (3) and the models available in the literature and discussed in Section 3.3 (see Table 4).
(5) Compute the reliability index β based on FORM techniques [1,45] using MATLAB [54]. The reliability index is related to the probability of failure P_f through $\beta = \Phi^{-1}(1 - P_f)$, where Φ^{-1} is the inverse standard normal distribution function.

Using the steps described above in conjunction with the load combinations (i.e., γ_i factors) and design loads specified in the Eurocode, US and Australian design frameworks, reliability indices β can be computed for each imposed-to-dead load ratio α and for each of the three levels of dead load investigated. These β-values can be then compared with the target values β_0 discussed in Section 3.1 for the six baseline frames, which is addressed in Section 5.

4.3. Serviceability limit state criteria

Serviceability limit states are generally defined in terms of (i) excessive deflections or rotations that can affect the appearance or functionality of structures or damage secondary elements, and (ii) excessive vibrations that may cause discomfort to users or affect the performance of equipment. In this study only vertical deflections of the roof elements due to gravity loads and lateral drifts caused by wind loading were considered. Based on the historical record of structures with a satisfactory service performance, it is generally assumed that the serviceability demands of structures can be met by limiting the deflections to certain commonly accepted allowable values [45]. Precise points or limits at which a structure is deemed unserviceable are difficult to determine, since they depend on the perception of the occupants and the nature of the structure under study, and thus the definition of these allowable deflection criteria δ_a has been a subject of study in recent decades [1]. The limits are usually considered to be deterministic (or constant), but since they are sometimes associated with subjective human reactions, they have also been assumed as random in some studies [55]. International design standards and other design guides for specific structural typologies recommend limits for allowable deflections for a variety of structural members and different load cases, but these limits show considerable variability. Table 5 presents a comparison of different allowable serviceability criteria for steel structures in terms of vertical deflection limits at the apex sections $\delta_{a,v}$ and lateral drift limitations at the eaves $\delta_{a,w}$ found in different international standards [36,39,40,56,57], design guides [58–60] and recent research works investigating the reliability of steel structures in serviceability limit states [8,11,49]. Different limitations exist depending on the structure type (single storey building, portal frame), type of supported material (metal cladding, masonry cladding), occupancy, type of roof (accessible, not accessible) and loading type (dead, imposed, wind or total loads), and are subject to the judgment of the designer, among others. The overview presented in Table 5 includes allowable serviceability criteria relevant to the investigated frames only.

From the vertical deflection limitations reported in Table 5 it is evident that a significant variation exists in the recommended allowable deflection limits $\delta_{a,v}$. The values specified in prEN 1990 [36] for not-accessible and accessible resilient roofing are s/125 and s/250, respectively, where s is the span of the frame or horizontal element, while the ASCE 7 [39] and AS/NZS 1170.0 [40] specifications recommend limiting values equal to s/240 and s/300 under imposed loads and long-term gravity loads, respectively. In general, limiting vertical deflection $\delta_{a,v}$ values for apex sections lie around s/250 under imposed loads only and around s/150 under total gravity loads. Regarding the lateral drifts at eaves $\delta_{a,w}$, Table 5 shows that limitations typically range between H_1/500 and H_1/150, where H_1 is the height of the frame at the eaves. These drift limits are of about the same magnitude as the erection tolerances prescribed in different international standards [61,62]. Based on these considerations and the serviceability limit state criteria adopted in previous studies, the reliability analyses presented in this paper were premised on the assumption that allowable deflection limits were deterministic, and the limits adopted corresponded to s/150 for the vertical deflection of the apex section under total gravity loads and $H_1/300$ for the lateral drift of the frame at the eaves under wind loads, which is at the upper limit of what is customarily used by most design offices [8,11]. However, it should be noted that the reliability index is independent of the allowable deflection criteria δ_a adopted in the analysis, and that β only depends on the assumed load statistics, as highlighted in [45]. This is because the adopted serviceability reliability analysis method, as described in Step (2) of Section 4.2, considers that the structure deflects precisely to δ_a under the nominal loads. Since it is necessary to assume certain values of the allowable deflection criteria δ_a to carry out the analysis, limits equal to s/150 and $H_1/300$ have been adopted in this paper for vertical deflections and lateral drifts. The results would be, however, identical if alternative allowable deflection criteria δ_a were adopted for the analysis.

4.4. Stochastic models for frame stiffness

The fundamental input information required when deriving reliability indices through First-Order Reliability Methods (FORM) are the probabilistic models for the different random variables involved in the limit state function g(·) adopted. While the models for gravity and wind loads have been already discussed in Section 3.3, and allowable deflections were considered as deterministic in this study, suitable stochastic models for frame vertical K_v and lateral stiffnesses K_w are required, as per in Step (1) of Section 4.2. Although Galambos and Ellingwood [45] demonstrated that reliability indices were essentially the same when the stiffness of structures was considered to be random or deterministic in serviceability studies, results reported in [45] also evidenced that neglecting the variability of the stiffness leads to higher (i.e., less conservative) β-values. Moreover, nonlinear material properties might cause an additional loss of stiffness in stainless steel structures that does not occur in carbon steel frames and might further affect the calculated β-values. Therefore, this study considered the vertical and lateral stiffness of the frames as random variables. With the aim of determining suitable stochastic models for the flexibility of stainless steel frames, the load–deflection (or load–drift) curves corresponding to the simulations carried out in [19,20], which represented real frames with random properties, were used.

Structures affected by geometrical and material nonlinearities result in nonlinear load–displacement curves, and thus, the stiffness of these structures depends on the load level under consideration and is generally lower than the initial stiffness (i.e., the stiffness of the structure

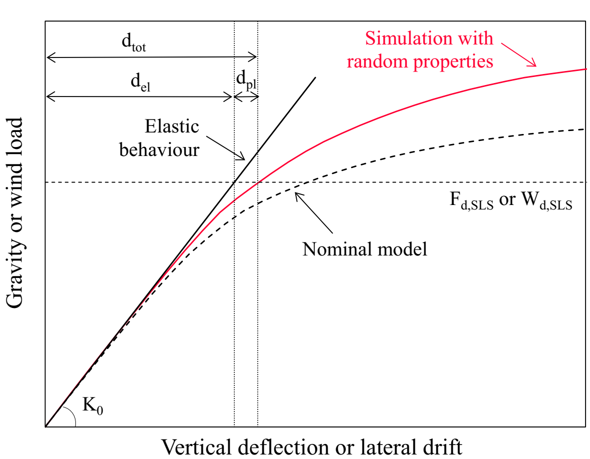

Fig. 3. Elastic and plastic components of deflections (or drifts) in stainless steel frames.

while it behaves elastically). In such cases, the total vertical deflection d_{tot} (or lateral drift) can be considered as the sum of an elastic deflection d_{el}, governed by the initial stiffness of the frame, and a plastic deflection d_{pl} that represents the additional deflection caused by the loss of stiffness due to the nonlinear geometrical or material characteristics of the structure, as shown in Fig. 3; and both components of the total deflection will typically show certain levels of variability. The variability of the elastic deflection occurs primarily as a result of the random nature of the Young's modulus E and the initial stiffness of the connections at the frame joints, while the variability in d_{pl} is due to the randomness of the yield stress f_y and the strain-hardening exponent *n*. By determining the variability of these two components, stochastic models can be derived for the vertical K_v and lateral stiffness K_w of the baseline frames investigated.

To characterize the variability of the elastic component of the deflection d_{el}, the initial stiffness of each of the simulated frames was computed from the vertical load–apex deflection curves (denoted as $K_{v,0}$) and the wind load–lateral drift curves (denoted as $K_{w,0}$). The statistics of the calculated initial stiffness values are reported in Table 6, where $\overline{K}_{v,0}$ and $\overline{K}_{w,0}$ are the mean values of the initial vertical and lateral stiffnesses, and $V_{Kv,0}$ and $V_{Kw,0}$ represent the corresponding coefficients of variation. Individual values of $K_{v,0}$ and $K_{w,0}$ were used to build histograms for each frame and load case, to which suitable probabilistic distributions were fitted. In all cases, log-normal distributions were adjusted using the Statistics and Machine Learning Toolbox in MATLAB [54], based on the maximum likelihood method. Fig. 4 shows typical examples of the histograms built from the initial stiffness results $K_{v,0}$ for frames subjected to gravity loads, while Fig. 5 and Fig. 6 show the histograms for the initial lateral stiffness $K_{w,0}$ for the GW1 and GW3 dead and wind load cases, respectively, and the fitted log-normal distributions. It should be noted that whereas for most baseline frames heavy wind load combinations (GW3) result in slightly higher mean lateral stiffness values than for light wind load combinations (GW1), an inverse trend is observed for Frames 1 and 3 in Table 6. This apparent inconsistency in mean $K_{w,0}$ values is due to the skewness of the histograms. While the mean lateral stiffness values obtained from the raw data were very similar for each baseline frame regardless the assumed wind load combination, the values reported in Table 6 correspond to the log-normal functions used to fit the histograms, which are affected by their skewness and thus result in values that are different from the mean values obtained from the raw data. Since the mean value of the fitted distribution is dependent on the skewness of the histograms, which has greater associated randomness than the mean value, it leads to the observed inconsistencies in the variation of $K_{w,0}$ shown in Table 6.

In order to evaluate the plastic component of the deflection d_{pl} due to the nonlinear stress–strain behaviour of stainless steel alloys, deviations of the actual total deflections from the purely elastic components d_{el} were evaluated for the service design loads. Since three design frameworks prescribing different nominal loads were considered in the analysis, service design loads adopted in this paper were back-calculated from the nominal FE models assuming that the frames were designed at their ultimate limit state using the DDM through the following procedure. The maximum design gravity loads and wind loads that each frame could endure were determined from the load factors specified in prEN 1990 [36], ASCE 7 [39] and AS/NZS 1170.0 [40] and the ultimate nominal frame strengths using the system safety or resistance factors recommended in [19,20] for each load case. For example, the maximum design gravity load in the Eurocode framework was estimated from the ULS-LRFD equation $R_{k,v}/\gamma_{M,s} = 1.35G_k+1.5Q_k$, where $R_{k,v}$ is the characteristic resistance of the frame under gravity loads and $\gamma_{M,s}$ is the system safety factor recommended for gravity loads, $\gamma_{M,s}$=1.15 (see Table 1). Assuming the common Q_k/G_k=2 load ratio for hollow section steel structures [11], the relationships $R_{k,v} = 2.5Q_k$ and $Q_k = R_{k,v}/2.5$ were derived. Since the gravity service design load $F_{d,SLS}$ for a Q_k/G_k ratio of 2 is $F_{d,SLS} = G_k+Q_k = 1.5Q_k$, the relationship between $F_{d,SLS}$ and

Table 6
Statistics of initial frame stiffness for stainless steel frames under gravity loads and combined gravity and wind loads.

Frame	Gravity loads		Dead and wind loads GW1		Dead and wind loads GW2		Dead and wind loads GW3	
	$\overline{K}_{v,0}$ [N/mm]	$V_{Kv,0}$	$\overline{K}_{w,0}$ [N/mm / mm]	$V_{Kw,0}$	$\overline{K}_{w,0}$ [N/mm / mm]	$V_{Kw,0}$	$\overline{K}_{w,0}$ [N/mm / mm]	$V_{Kw,0}$
Frame 1	474.4	0.066	29.87	0.088	29.63	0.081	29.67	0.082
Frame 2	730.9	0.067	44.91	0.078	45.01	0.076	45.02	0.077
Frame 3	489.6	0.056	31.60	0.077	31.50	0.077	31.43	0.077
Frame 4	499.3	0.070	38.22	0.078	38.33	0.077	38.39	0.077
Frame 5	476.6	0.079	31.18	0.090	31.26	0.089	31.31	0.089
Frame 6	602.5	0.078	41.53	0.082	41.58	0.082	41.64	0.081

$R_{k,v}$ was established as $F_{d,SLS} = R_{k,v}/1.7$. Following a similar approach, the design wind load for serviceability in the Eurocode framework was estimated from $W_{d,SLS} = R_{k,w}/(1.5\gamma_{M,s})$, which results in $W_{d,SLS} = R_{k,w}/1.8$ if the recommended $\gamma_{M,s}$ factor of 1.20 is adopted (see Table 1), where $R_{k,w}$ is the characteristic resistance of the frame under combined gravity and wind loads. Through this procedure, similar design loads were obtained for the US and Australian frameworks, which represent upper bounds to the serviceability design loads [63].

Table 7 presents the comparison between the elastic deflections $d_{v,el}$ and drifts $d_{w,el}$ with the corresponding actual total deflections $d_{v,tot}$ and drifts $d_{w,tot}$ estimated from the nonlinear load–deflection curves of the random frame simulations at the SLS design loads $F_{d,SLS}$ and $W_{d,SLS}$, including results for the different load scenarios and the six baseline frames. The mean values and coefficients of variation corresponding to the elastic-to-total deflection (or drift) ratios $d_{v,el}/d_{v,tot}$ and $d_{w,el}/d_{w,tot}$ reported in Table 7 indicate that the stainless steel frames investigated in this paper, when designed using the DDM and the $\gamma_{M,s}$ and ϕ_s factors recommended in [19,20], remain basically in the elastic range for the serviceability design loads. However, the actual total deflections were found to be consistently higher than those estimated elastically (i.e., all $d_{v,el}/d_{v,tot}$ and $d_{w,el}/d_{w,tot}$ ratios are below unity), and thus the information in Table 7 was used to modify the statistical data reported in Table 6 for the initial stiffness of the frames and to derive suitable stochastic models that also account for plastic deflections due to material nonlinearities.

Specifically, the mean values for the final stiffness distributions adopted in this study were estimated by reducing the $\overline{K}_{v,0}$ and $\overline{K}_{w,0}$ values reported in Table 6 by the corresponding average $d_{v,el}/d_{v,tot}$ and $d_{w,el}/d_{w,tot}$ ratios shown in Table 7, while the final coefficients of variation $V_{K,v}$ and $V_{K,w}$ were calculated as the sum of the respective COV values in Table 6 and Table 7 for each load case. Log-normal distributions were adopted for the final stiffness distributions, since all affecting variables (i.e., initial stiffness, yield stress and strain-hardening exponent) follow log-normal distributions [18]. The mean values ($\overline{K}_v$ and $\overline{K}_w$) and coefficients of variation ($V_{K,v}$ and $V_{K,w}$) of the final frame stiffness distributions adopted in the reliability analyses are reported in Table 8 and Table 9 for the gravity and the combined gravity plus wind load cases, respectively. The tables also include the nominal stiffness values ($K_{v,n}$ and $K_{w,n}$) corresponding to the design (nominal) FE models developed following the requirements of the Eurocode, US and Australian frameworks. Since the six nominal baseline frames exhibited a linear behaviour for the SLS design loads $F_{d,SLS}$ and $W_{d,SLS}$, the nominal stiffness values adopted in the reliability studies corresponded to the nominal initial stiffnesses (i.e., $K_{v,n} = K_{v,n,0}$ and $K_{w,n} = K_{w,n,0}$).

5. Reliability analysis results for serviceability

This Section presents the serviceability reliability results for stainless steel frames in the Eurocode, US and Australian frameworks. The calculated reliability indices correspond to vertical deflections under gravity loads and lateral drifts under wind loads. Presented separately, they are compared with the relevant target reliability indices for the different design frameworks and with the results reported in previous reliability studies.

5.1. Reliability of vertical deflections under gravity loads

Based on the K_v stiffness variability data reported in Section 4.4 and the stochastic models for dead and imposed loads reported in Table 4, this Section presents the reliability indices corresponding to the serviceability limit state of the vertical deflection. Following the procedure described in Section 4.2, reliability indices corresponding to the range of imposed-to-dead load ratios α considered (i.e., α ratios between 1.0 and 2.5) were derived for the Eurocode, US and Australian frameworks. The results showed that the reliability indices β calibrated for each of the six frames were very similar for the investigated range of imposed-to-dead load ratios for all design frameworks, with the largest observed differences in β between $\alpha = 1.0$ and $\alpha = 2.5$ being less than 5%. Consequently, and considering that the common load ratio for hollow section steel structures is $\alpha = 2.0$ [11], only the reliability indices β corresponding to an imposed-to-dead load ratio of $\alpha = 2.0$ are reported in this paper. The β indices derived for stainless steel frames under gravity loads corresponding to a reference period of 8 years β_{8yrs} are reported in Table 8 for the six baseline frames and the three design frameworks investigated. Since target reliability indices β_0 for serviceability are generally referred to annual failure probabilities, as discussed in Section 3.1, reliability indices corresponding to a reference period of 1 year, denoted by β_{1yr}, are also provided in Table 8. Annual reliability indices can be estimated from the corresponding β_{8yrs} values using Eq. 5 and Eq. 6, as indicated in [8].

$$P_{f,1yr} = 1 - \sqrt[t]{1 - P_{f,t}} \quad (5)$$

$$\beta_{1yr} = \Phi^{-1}\left(1 - P_{f,1yr}\right) \quad (6)$$

The annual reliability indices reported in Table 8 show that the highest β_{1yr} values correspond to the Eurocode framework, owing to the lowest mean-to-nominal imposed load ratios adopted by this framework (see Table 4), which means that the nominal imposed loads in prEN 1991-1-1 [41] are in general more conservative than the equivalent loads prescribed in ASCE 7 [39] and AS/NZS 1170.1 [43]. Conversely, the β_{1yr} values derived for the US and Australian frameworks are similar because they use imposed load models with the same mean-to-nominal values. The small differences observed between the US and Australian frameworks can be explained by the short-term factor ψ_s adopted by the AS/NZS 1170.1 [43] Specification in the serviceability load combination, as discussed in Section 3.2, resulting in slightly lower reliability indices since the adopted design load is less conservative. These results are in line with the β-values calibrated for the same stainless steel frames for the gravity load ultimate limit state in [19]. The average annual reliability indices β_{1yr} considering the six baseline frames are 3.19, 2.56 and 2.53 for the Eurocode, US and Australian frameworks, respectively, as reported in Table 8, which correspond to average annual probabilities of exceedance of approximately 0.1%, 0.5% and 0.6%. These β_{1yr} values should be compared with the corresponding annual target reliability indices introduced in Section 3.1, which are β_0=2.90 for the Eurocode design framework, and β_0=1.64 and β_0=1.75 for the US and the

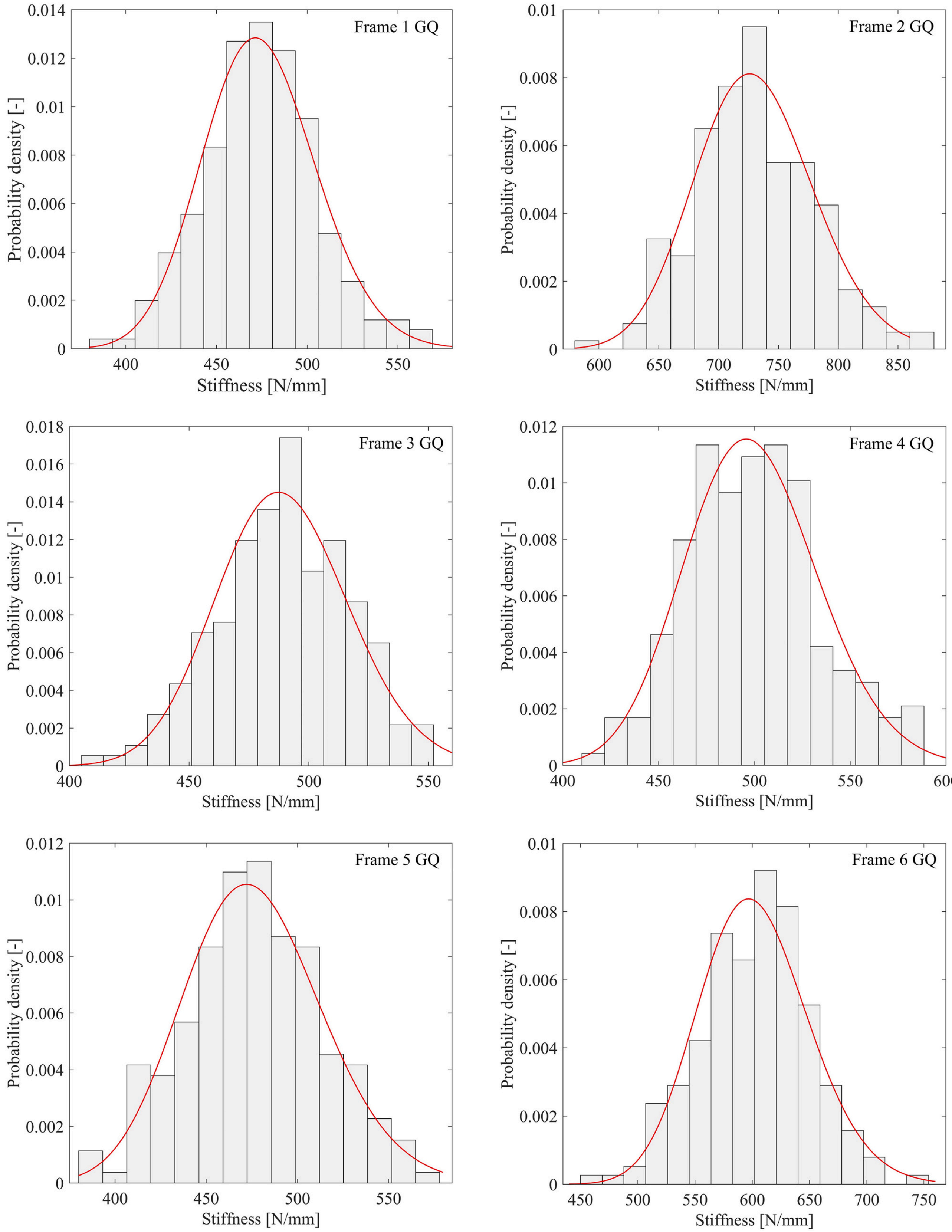

Fig. 4. Typical histograms for initial vertical frame stiffness $K_{v,0}$ of stainless steel frames under gravity loads for Frames 1 to 6.

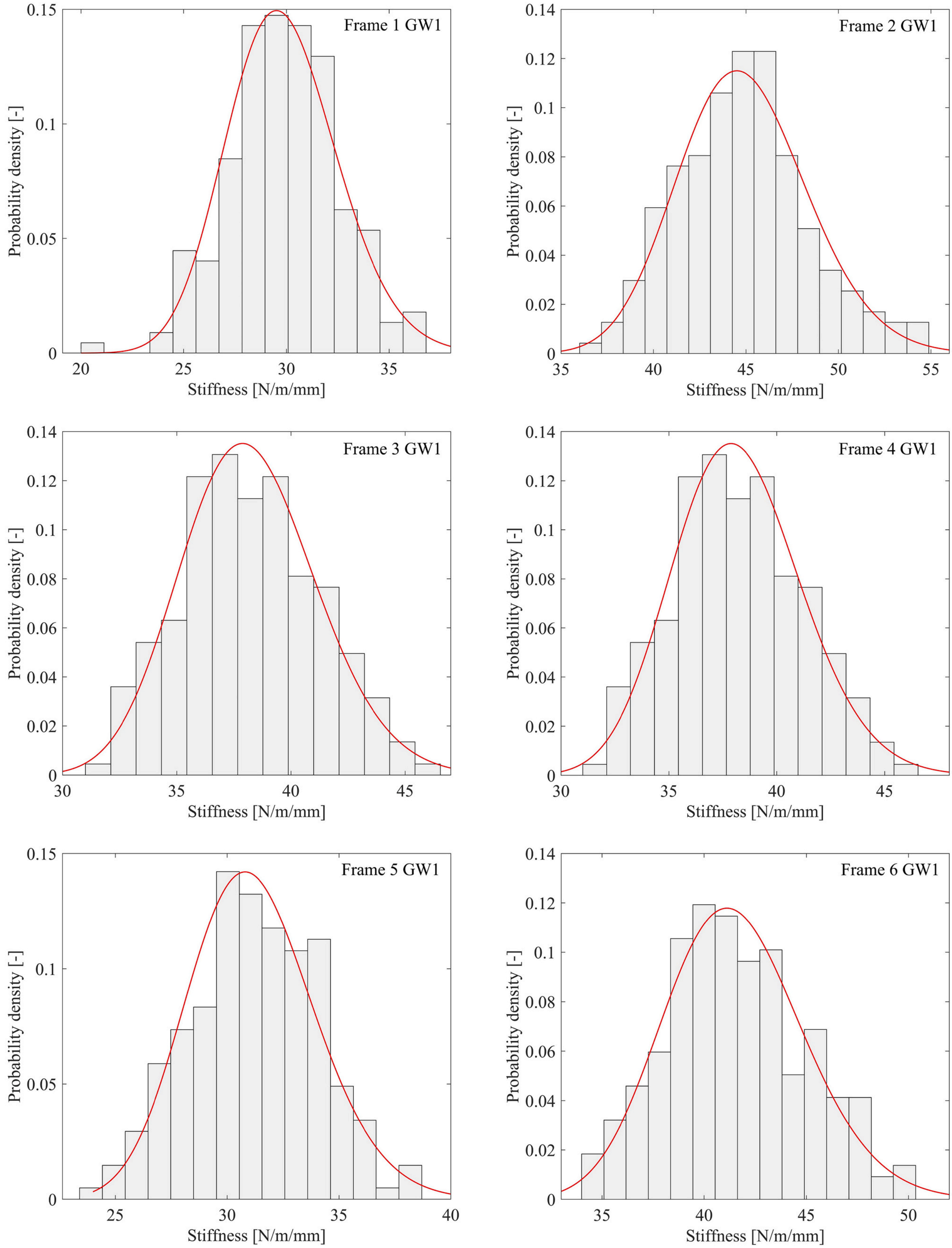

Fig. 5. Typical histograms for initial lateral frame stiffness $K_{w,0}$ of stainless steel frames under the GW1 dead and wind load case for Frames 1 to 6.

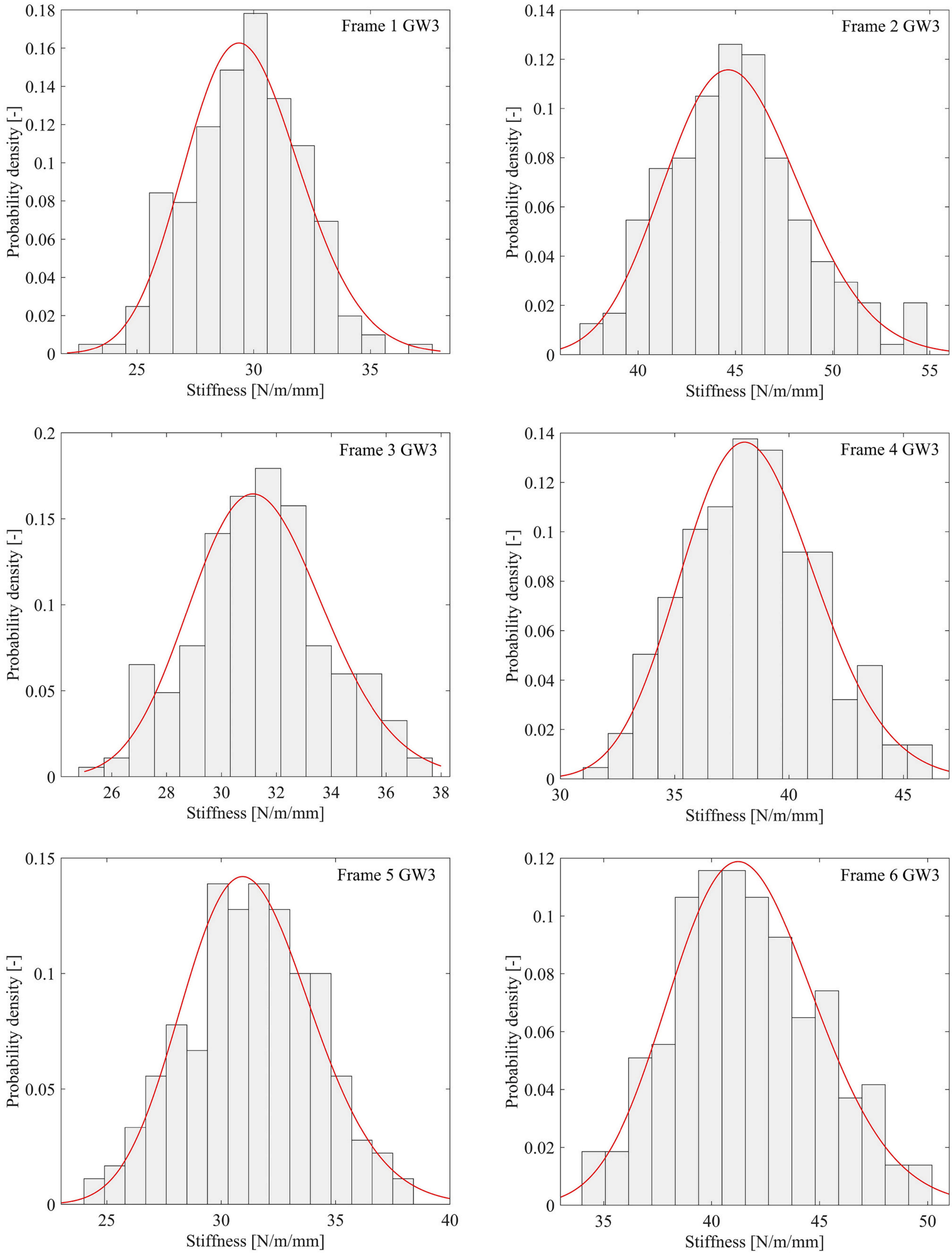

Fig. 6. Typical histograms for initial lateral frame stiffness $K_{w,0}$ of stainless steel frames under the GW3 dead and wind load case for Frames 1 to 6.

Table 7
Comparison between elastic deflections (or drifts) with the corresponding total deflections (or drifts) for design serviceability loads in different load scenarios.

Frame	Gravity loads		Combined gravity and wind loads, GW1		Combined gravity and wind loads, GW2		Combined gravity and wind loads, GW3	
	$d_{v,el}/d_{v,tot}$		$d_{w,el}/d_{w,tot}$		$d_{w,el}/d_{w,tot}$		$d_{w,el}/d_{w,tot}$	
	Mean	COV	Mean	COV	Mean	COV	Mean	COV
Frame 1	0.964	0.017	0.948	0.024	0.948	0.037	0.944	0.033
Frame 2	0.996	0.005	0.972	0.033	0.973	0.030	0.968	0.038
Frame 3	0.952	0.023	0.965	0.016	0.962	0.014	0.960	0.014
Frame 4	0.997	0.004	0.991	0.016	0.990	0.013	0.990	0.012
Frame 5	0.989	0.008	0.965	0.012	0.964	0.017	0.962	0.017
Frame 6	0.986	0.015	0.985	0.021	0.984	0.022	0.982	0.024
Average	0.981	0.022	0.971	0.026	0.971	0.027	0.968	0.029

Table 8
Statistics of vertical stiffness and reliability indices for stainless steel frames under gravity loads (for an imposed-to-dead load ratio α = 2.0).

Design framework	Frame	$\overline{K}_v$ [N/mm]	$V_{K,v}$	$K_{v,n}$ [N/mm]	β_{8yrs}	β_{1yr}
Eurocode framework	Frame 1	457.5	0.083	502.1	2.62	3.27
	Frame 2	727.6	0.073	781.6	2.52	3.18
	Frame 3	466.2	0.079	508.4	2.59	3.24
	Frame 4	498.0	0.073	526.8	2.46	3.13
	Frame 5	471.5	0.087	502.1	2.49	3.15
	Frame 6	594.1	0.093	640.6	2.54	3.20
	Average				2.54	3.19
US framework	Frame 1	457.5	0.083	487.4	1.75	2.57
	Frame 2	727.6	0.073	746.1	1.57	2.43
	Frame 3	466.2	0.079	508.4	1.85	2.65
	Frame 4	498.0	0.073	525.4	1.70	2.53
	Frame 5	471.5	0.087	501.9	1.75	2.57
	Frame 6	594.1	0.093	640.9	1.81	2.61
	Average				1.74	2.56
Australian framework	Frame 1	457.5	0.083	491.7	1.78	2.59
	Frame 2	727.6	0.073	759.6	1.62	2.47
	Frame 3	466.2	0.079	508.4	1.85	2.64
	Frame 4	498.0	0.073	525.4	1.68	2.51
	Frame 5	471.5	0.087	491.7	1.62	2.47
	Frame 6	594.1	0.093	627.5	1.69	2.52
	Average				1.70	2.53

Table 9
Statistics of lateral stiffness and reliability indices for stainless steel frames under combined gravity and wind loads (for dead load level GW1).

Design framework	Frame	$\overline{K}_w$ [N/mm / mm]	$V_{K,w}$	$K_{w,n}$ [N/mm / mm]	β_{10yrs}	β_{1yr}
Eurocode framework	Frame 1	28.33	0.112	29.28	1.83	2.71
	Frame 2	43.67	0.111	47.33	1.92	2.77
	Frame 3	30.49	0.094	30.26	1.78	2.67
	Frame 4	37.86	0.093	38.86	1.81	2.69
	Frame 5	30.08	0.102	30.50	1.78	2.67
	Frame 6	40.89	0.103	42.63	1.84	2.71
	Average				1.83	2.70
US framework	Frame 1	28.33	0.112	28.10	0.76	1.96
	Frame 2	43.67	0.111	46.49	0.91	2.06
	Frame 3	30.49	0.094	30.59	0.78	1.97
	Frame 4	37.86	0.093	39.10	0.85	2.02
	Frame 5	30.08	0.102	30.62	0.81	1.99
	Frame 6	40.89	0.103	43.05	0.89	2.04
	Average				0.83	2.01
Australian framework	Frame 1	28.33	0.112	28.76	0.98	2.11
	Frame 2	43.67	0.111	46.80	1.10	2.19
	Frame 3	30.49	0.094	30.59	0.96	2.09
	Frame 4	37.86	0.093	39.10	1.03	2.13
	Frame 5	30.08	0.102	29.29	0.90	2.05
	Frame 6	40.89	0.103	42.26	1.02	2.13
	Average				1.00	2.12

Australian frameworks, respectively. According to these values, the calculated reliability indices are above the target values for the vertical deflection serviceability designs for the three design frameworks, particularly for the US and Australian frameworks.

The reliability indices presented in Table 8 are comparable to the β-values reported in previous studies investigating the reliability of steel structures in serviceability limit states under gravity loads: Galambos and Ellingwood [45] reported average values of reliability indices β_{8yr}=1.86 and β_{1yr}=3.08 for vertical deflections for periods of reference equal to 8 years and 1 year, respectively, while an average annual reliability index of β_{1yr}=3.10 was obtained by Ellingwood [49] for beams loaded with gravity loads. The values in Table 8 also show that the reliability indices for serviceability on an annual basis are comparable to the β-values on a 50-year basis for the ultimate limit state reported in [19], as previously highlighted in [49]. It is also worth mentioning that the differences observed between the annual serviceability reliability indices and the 50-year ultimate limit state reliability indices are more significant for stainless steel structures than for carbon steel structures, because the ultimate limit state β-values for stainless steel frames are affected by the considerably higher strength reserve (i.e., mean-to-nominal frame resistance) existing in these frames due to the higher mean-to-nominal yield stress ratios, a reserve that does not affect serviceability limit states and corresponding reliability indices [19].

5.2. Reliability of lateral drifts under combined gravity and wind loads

Following a similar approach to that described for vertical deflections under gravity loads, this Section presents the serviceability reliability results for stainless steel frames under combined gravity and wind loads. Reliability indices corresponding to periods of reference equal to 10 years and 1 year (denoted as β_{10yrs} and β_{1yr}, respectively) were calculated for the Eurocode, US and Australian frameworks using the methodology described in Section 4.2, the wind load models reported in Table 4 and the lateral stiffness K_w variability data derived in Section 4.4. Although reliability studies were carried out separately for the three levels of dead load considered in the analysis (load cases GW1, GW2 and GW3), the results demonstrated that the wind-to-dead load ratio had a negligible effect on the serviceability reliability evaluation of stainless steel frames, with differences in the calculated β-values being less than 2% for all design frameworks. Consequently, and following the approach adopted for the assessment vertical deflections, only reliability indices β corresponding to the dead load case GW1, showing the lowest β-values, are reported in this paper.

Reliability indices derived for the serviceability limit state of stainless steel frames under combined dead and wind loads (load case GW1) corresponding to a reference period of 10 years β_{10yrs} for the six baseline frames are reported in Table 9. Results corresponding to a reference period of 1 year β_{1yr}, estimated using the expressions given in Eq. 5 and Eq. 6, are also included for a direct comparison with the target reliability indices β_0 for serviceability. It should be noticed that the annual β-values calculated for the roof drift limit state are generally lower than those reported in Table 8 for the vertical deflection limit state, which is in line with the reliability indices obtained for the same frames in [19,20] when assessing ultimate limit states. The annual reliability indices reported in Table 9 indicate that, consistent with the results exposed in the previous Section for serviceability considerations under gravity loads, the highest β_{1yr} values for lateral drift serviceability correspond to the Eurocode framework, while the reliability indices calculated for the US and Australian frameworks are very similar. This difference can be explained by the different load combinations or service wind loads adopted in the three design frameworks, as discussed in Section 3.2, since the load combination specified in prEN 1990 [36] adopts a service load equal to the characteristic wind load defined in prEN 1991-1-4 [42], corresponding to a return period of 50 years, while the service loads used in the US and Australian frameworks correspond to lower return periods. Average annual reliability indices β_{1yr} for the six baseline frames and the dead load case GW1 are, as reported in Table 9, 2.70, 2.01 and 2.12 for the Eurocode, US and Australian frameworks, respectively, which correspond to the approximate average annual probabilities of exceedance of 0.3%, 2.2% and 1.7%. Comparing these β_{1yr} values with the target values introduced in Section 3.1, it can be seen that the reliability of wind drift appears to be adequate for the US and Australian design frameworks, since the calculated average β_{1yr} values lie above the corresponding target values β_0, and that the average value obtained for the Eurocode framework is also very close to the target reliability index of 2.90.

The reliability associated with excessive lateral drifts under wind loads has been previously investigated in different studies of steel frames, in which the average annual reliability index of β_{1yr}=3.29 was reported by Galambos and Ellingwood [45] for lateral drift limit states, while a similar study by Ellingwood [49] reported an average value of β_{1yr}=3.40. These two studies conservatively assumed a serviceability wind load corresponding to a 50-year return period, which is equivalent to the characteristic load combination prescribed in prEN 1990 [36] for irreversible serviceability checks, and thus the annual reliability indices obtained in [45,49] are comparable to the values reported in Table 9 for the Eurocode framework. It should be noted, however, that the β_{1yr} values reported in [45,49] are slightly higher than those obtained in this study because this paper conservatively includes the effect of the variability in frame stiffness, resulting in lower reliability indices. In contrast, recent studies developed in the frame of the Direct Design Method that reported reliability results for serviceability limit states under wind loads [8,11] found that the average values of the annual reliability indices were β_{1yr}=1.86 and β_{1yr}=1.88 for hot-rolled and cold-formed steel frames, respectively. These values are similar to the reliability indices reported for lateral drifts in Table 9 for the US and Australian frameworks. The small differences observed in the annual reliability indices are due to the more recent and less conservative wind load model adopted in this study for the US framework [25] compared to the load models assumed in [8,11]. Finally, and as for the vertical deflection limit states, it can be noted that the annual reliability indices β_{1yr} reported in Table 9 are roughly comparable to those calculated in [20] for the ultimate limit state and a reference period of 50 years.

6. Conclusions

Most relevant international standards for steel structures [2,3,5,6] have recently taken major steps towards changing the paradigm of structural design by incorporating provisions of *analysis-by-design* approaches. These provisions will allow not only the design of more integrated, lighter and more efficient structures, but will also contribute to the simplification and acceleration of the design process. However, since at present no specific system reliability requirements or acceptable target reliability indices are provided in these standards for system-based design approaches, recent research efforts have focussed on the calibration of suitable system safety factors $\gamma_{M,s}$ and system resistance factors ϕ_s in the form of the Direct Design Method (DDM) for hot-rolled [7–10] and cold-formed [11,12] steel frames, racks [13], scaffolding structures [14] and stainless steel portal frames [19,20]. Nevertheless, since advanced design approaches potentially result in lighter structures, serviceability issues may become more important than for the traditional *two-step* design approach, especially when materials characterized by a nonlinear stress–strain behaviour are considered. Thus, this paper sets out an explicit analysis framework for assessing serviceability reliability at system level and presents the serviceability limit state reliability assessment of stainless steel frames designed using the DDM.

Two different serviceability limit states were assessed in this study, *viz.* excessive deflections under gravity loads and excessive lateral drifts under wind loads. The serviceability reliability analyses were based on six stainless steel portal frames subjected to different load cases, and included the three most common stainless steel grades (i.e., austenitic

EN 1.4301/ASTM 304, duplex EN 1.4462/ASTM 2205 and ferritic EN 1.4003/ASTM UNS S40977 grades). The variability of the vertical and lateral system stiffnesses was determined from extensive finite element simulations that accounted for the randomness of relevant variables (geometric properties, initial imperfections, material properties, connection behaviour and model uncertainty). In conjunction with the serviceability load combinations specified in prEN 1990 [36], ASCE 7 [39] and AS/NZS 1170.0 [40], and the stochastic models characterizing the variability of the loads, reliability indices corresponding to vertical deflections and lateral drifts were calculated and compared with the relevant target reliability indices for different design frameworks. Average annual reliability indices β_{1yr} obtained for the vertical deflection serviceability were 3.19, 2.56 and 2.53 for the Eurocode, US and Australian frameworks, respectively, while the β_{1yr} values calibrated for the lateral drift serviceability were 2.70, 2.01 and 2.12, respectively. The results indicated that the reliabilities for roof drift were generally lower than those calculated for vertical deflections, and that when the system safety $\gamma_{M,s}$ and system resistance ϕ_s factors recommended in [19,20] for stainless steel frames are adopted, the reliability indices calibrated for vertical deflections and lateral drifts lie above or very close to the target values prescribed in the different frameworks.

CRediT authorship contribution statement

Itsaso Arrayago: Conceptualization, Software, Data curation, Methodology, Formal analysis, Writing – original draft. **Kim J.R. Rasmussen:** Methodology, Supervision, Writing – review & editing.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

Data will be made available on request.

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