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Towards a practicable GMNIA procedure in the footsteps of the Direct Design Method

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Abstract

Eurocode 3 permits to utilize nonlinear finite element analysis (NFEA) in design process by carrying out Geometrically and Materially Nonlinear Analysis with Imperfections (GMNIA). Design by GMNIA can consider material plasticity and redistribution of forces in a system level such that it can result in an economical design regarding the material consumption. Unfortunately, GMNIA is scarcely adopted among structural engineers, most likely due to the complexity of the method. The determination of the safety factor for GMNIA requires numerical or experimental test results which may not be available for the designed structural system. Recently, a more holistic approach, Direct Design Method (DDM), has been introduced. In DDM, structural systems are prequalified by determining the system safety factor in advance based on system level reliability studies, thus simplifying the design process for a structural engineer. This study develops a Eurocode-compliant DDM, i.e. practicable GMNIA, for Warren roof trusses. The paper presents the prequalification process based on three trusses made of cold-formed hollow sections of the steel grade S700. Additionally, modelling techniques are presented for cold-formed hollow sections such that an accurate nominal NFEA model is achieved.

Keywords

Advanced design method, GMNIA, Direct Design Method, Reliability study, Warren truss, cold-formed, hollow section, high-strength steel

1 Introduction

In Advanced Design Method (ADM), the entire structural system is designed by verifying condition [1]:

$$R_n/\gamma_{ADM} \geq \sum \gamma_i Q_i \quad (1)$$

Where R_n is the nominal resistance of the system obtained from Nonlinear Finite Element Analysis (NFEA), γ_{ADM} is the system safety factor, and Q_i and γ_i are characteristic values and partial factors for actions according to relevant design standards, respectively. In NFEA, loads are scaled up incrementally until the failure of the system occurs at the maximum load proportionality factor α_u. α_u is thus the highest load factor in the load-displacement curve of the system. Alternatively, if the system does not have a descending branch or a clear maximum peak in the curve, α_u can be determined at the point at which the stiffness of the structure has been reduced e.g. to 5% from the initial stiffness determined from the load-displacement curve [1]. α_u represents the ratio of nominal resistance to applied loads, hence Eq. (1) can be rewritten as $\alpha_u \geq \gamma_{ADM}$.

ADM is permitted in many design standards such as in AISC 360-16 [2], AS/NZS 4600 [3] and Eurocode 3 [4], [5]. It has been shown that design by ADM can reduce material consumption even 14% when compared to conventional member-based design methods in which the resistance of structural members are verified by using e.g. buckling curves or individual interaction equations [1]. Additionally, ADM leads to more uniform system reliability than conventional methods [6] also revealing the failure mode of the system, thus allowing the designer to avoid the most fatal modes. Consequently, the safety of the design is enhanced.

Eurocode 3 uses the term GMNIA for ADM, which stands for Geometrically and Materially Nonlinear Analysis with Imperfections [7]. Unfortunately, despite the advantages, design by GMNIA has been scarcely adopted for the practical design of entire structural systems. One probable reason for this disfavour has been insufficient guidelines in performing the design by GMNIA. The second generation of Eurocodes will provide guidance in prEN 1993-1-14, “Design assisted by finite element analysis” [8], which is a great step forward for GMNIA. However, regarding entire structural systems, the determination of the γ_{ADM} according to EN 1993-1-14 causes two challenges because the γ_{ADM} is determined by [8]:

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$$\gamma_{ADM} = \gamma_M \cdot \gamma_{FE} \quad (2)$$

Where γ_M is the partial factor for resistance and γ_{FE} the model factor that accounts for the uncertainties of the numerical model and analysis method. The first challenge is that the determination of γ_{FE} requires experimental or numerical capacity test results, which may not be available for the designed structural system because, in general, very few experimental tests for structural systems have been carried out or published. The second challenge is that the γ_M in Equation (2) is determined based on the relevant failure mode thus possibly leading to various γ_{ADM}-values and to overconservative design. Consider the example of roof truss, for which γ_{ADM} = 1.15 for buckling and γ_{ADM}=1.20 for tensile fracture have been determined due to different failure mode-dependent γ_M-values. It may happen during the NFEA, that truss brace buckles at $1.15 \leq \alpha_u < 1.20$. In this situation, the buckling capacity is verified but the tensile fracture, e.g. resistance of the bottom chord, is not. Consequently, the design engineer must strengthen the model such that $\alpha_u \geq 1.20$ is obtained thus leading to an overconservative design for compressed members. Therefore, the current GMNIA procedure seems suitable for such isolated structural products which can be designed individually against each failure mode and for which experimental capacity tests can be carried out to obtain γ_{FE}. For a general structural system, however, test results do not usually exist, and governing failure mode may not be evident thus leading to an unfeasible procedure.

In recent years, a more holistic approach called Direct Design Method (DDM) has been developed for hot-rolled products [9], storage rack frames [10], and cold-formed stainless [11] and carbon steel frames [1]. DDM overcomes the challenges of the current GMNIA procedure by substituting the experimental tests required for γ_{FE} with system-level reliability studies. In these reliability studies, the γ_{ADM} is determined based on system-level Monte Carlo simulations (MCS) in which preselected structural systems are studied thus prequalifying such families of systems. The development phase of DDM is laborious because various structural systems with different cross-sections and materials must be studied. Once the families of systems are prequalified, however, the utilization of the DDM is straightforward with a readily determined system safety factor γ_{ADM}. Because γ_{ADM} is determined for the entire system considering various failure modes in a single value of γ_{ADM}, issues with overconservativity are also solved. Nevertheless, the main advantage of DDM is that γ_{ADM} is based on system-level reliability studies considering all the relevant uncertainties in a single system safety factor thus leading to a simple approach. Consequently, due to simplicity, the outcome of the design by DDM will become harmonized and safe among practitioners.

This study presents the prequalification process and usage of the Eurocode-compliant DDM, i.e. “practicable GMNIA” procedure. The system safety factor γ_{ADM} is determined for the ultimate limit state design of three Warren roof trusses shown in Figure 1. The trusses are made of cold-formed rectangular hollow sections (CFRHS) and of the steel grade S700. Additionally, the paper presents techniques to accurately build the nominal NFEA model for CFRHS structures. Finally, a short discussion underlines the major challenges that are yet to be solved such that the practicable GMNIA procedure becomes widely adopted among design engineers.

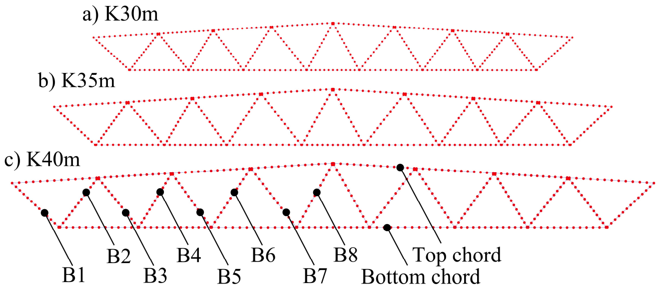

Figure 1 Geometries and FEM meshes of studied trusses [12].

2 Cold-formed rectangular hollow sections

The present study considers CFRHS fabricated by the continuous forming method [13]. In these sections, corner regions have higher yield and ultimate stresses than the flat region of the cross-section [14]. Additionally, a significant magnitude of bending residual stresses exists in these sections for longitudinal and transversal directions [13]. The effects of hardened corner region material and residual stresses have been incorporated into the NFEA by using the so-called Effective Material Model (EMM) [14]. In EMM, the stress-strain curve is determined as a weighted average from flat and corner region materials and by reducing the stiffness of the stress-strain curve due to residual stresses.

Table 1 presents the properties of CFRHS which are considered as random variables in reliability studies with varying probabilistic distributions. It should be noted that only a brief description is given in this paper for the presented procedure and more detailed descriptions can be found for the residual stress model in [13], for the EMM in [14], and for the rest of the features in [12].

Table 1 List of random properties considered for CFRHS [12].

Yield strength, Ultimate strength, 0.05% proof stress, Strain at ultimate strength, Elastic modulus, Corner strength enhancement factor, Width, Height, Thickness, Bow imperfection, Longitudinal bending residual stress, Transversal bending residual stress, Modelling uncertainty

3 Warren trusses

Figure 1 and Table 2 present layouts and configurations of three Warren, i.e. K-jointed trusses, for which the practical GMNIA procedure is developed. These trusses are denoted as K30m, K35m and K40m, having the span lengths L_s (see Figure 2) of 30m, 35m and 40m, respectively. Heights at the mid-span are selected as 1/10 of the span lengths as recommended in [15] and the roof slope ratio is 1:16 which is common in industrial buildings.

Trusses are designed for the self-weight of CFRHS members and the line load q_{Ed}=22kN/m at the top chord as shown in Figure 2. The line load consists of permanent load (0.5kN/m^2) and snow load (2kN/m^2) on the roof with a truss spacing of 6m and by applying the partial factors γ_G=1.35 for the permanent load and γ_Q=1.5 for the variable snow load [16]. The top chord is assumed to be restrained in the Y-direction (see Figure 2) with roofing such

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that buckling of the top chord can occur only in XZ-plane.

Cross-sections presented in Table 2 were originally selected in [12] to fulfil the GMNIA-design $\gamma_{ADM}=\alpha_u=1.21$: The value $\gamma_{ADM}=1.21$ was determined based on reliability studies of 15 various trusses, which were designed according to EN 1993-1-1 [4] forcing various failure modes to occur by alternating the target utilization ratios for the truss members [12]. In this study, reliability studies are conducted for the presented three trusses which were designed by using GMNIA instead of the conventional method. Because these two design methodologies result in slightly different cross-sections, they may also lead to differing system safety factors. Therefore, the γ_{ADM} is calculated for the presented three trusses and compared with the value of 1.21. A more detailed description of the trusses is given in [12].

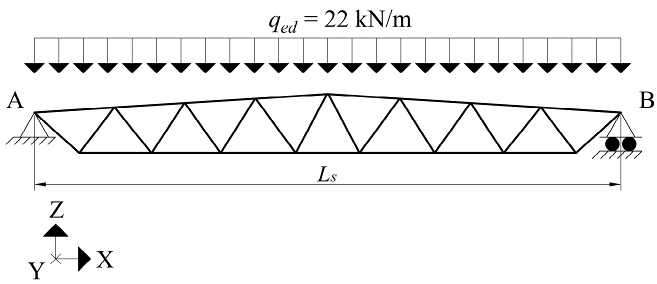

Figure 2 Loads and boundary conditions of trusses.

Table 2 Truss configurations [12].

Member	K30m	K35m	K40m
Top chord	150x150x6	180x120x7.1	200x120x8
Bot. chord	50x100x5	60x120x5	50x120x6
Brace B1	60x60x3	70x70x3	70x70x4
Brace B2	100x100x5	110x110x5	120x120x6
Brace B3	30x40x3	40x40x3	40x40x4
Brace B4	90x90x4	100x100x4	110x110x5
Brace B5	25x25x3	25x25x3	25x25x3
Brace B6	80x80x4	90x90x4	100x100x4
Brace B7	50x50x3	60x60x3	80x80x4
Brace B8	25x25x3	25x25x3	25x25x3

4 Finite element model

Finite element analyses have been carried out by the Abaqus [17] software. Abaqus beam element B31 has been used with "ARBITRARY" cross-section definition which allows for the modelling of rounded corners in CFRHS. Mesh sizes were determined based on a mesh convergence study and 12 elements for each member between connections was deemed sufficient, see Figure 1. All translations have been fixed in support A (see Figure 2) whereas only translations in Z- and Y-directions have been restrained in support B. Additionally, translations in Y-direction have been restrained in the top chord. Connection eccentricities have been considered between braces and chords by modelling brace ends to their actual locations at chord surfaces with "Beam"-type multipoint constraints. Rotational degrees of freedom about both bending axes (not torsional) have been released from brace ends thus assuming pinned connections for braces. Chords are assumed as continuous members over the mid-span. Analyses are carried out by the arc-length method.

5 Reliability studies

Reliability analysis is a powerful tool to determine the probability of failure of a system and consequently the required system safety factor [18]. The probability of failure can be determined by P_f=P{g(X)≤0}, in which g(X) is the so-called limit state function and X represents the vector of random variables. EN 1990 [16] determines the required reliability of the structure by target reliability index β_T, which relates to the failure probability by $P_f=\Phi(-\beta_T)$, in which Φ() is the cumulative distribution function of standard Normal distribution. In this study, γ_{ADM} is determined for a reference period of 50 years and for reliability class 2 structure, for which EN 1990 [16] requires $\beta_T=3.8$.

Eurocode does not yet consider target reliability for structural systems. However, prEN 1990 [19] states that "*if the partial factor design format that is being calibrated can be related to an existing partial factor design format for which the safety level is considered satisfactory, the corresponding average reliability level should be used as a target value for calibration*." Consequently, the target reliability index β_T may be lowered from prescribed 3.8 when considering "series" or "weakest link systems" such as isostatic trusses, because conventional member-based design methods correspondingly result in a lower reliability level in such structures [18]. Accordingly, in addition to $\beta_T=3.8$, the system safety factor is determined in this study for varying reliability levels such that the correct γ_{ADM}-value can be chosen once a proper reliability level is known for truss structures.

5.1 Reliability procedure

This study adopts a two-step procedure for determining the γ_{ADM}-values [1]: (1) First, statistical distributions of resistance are determined by Monte Carlo simulations (MCS) in which loads are assumed as deterministic. (2) Subsequently, the required γ_{ADM}-values are calculated by considering the variability of resistance and loads in the First-Order Reliability Method (FORM) [18]. Latin hypercube sampling has been employed to reduce the required number of samples in MCS [18]. A sample size of 500 was estimated sufficient by preliminary tests in which distributions for sample sizes of 60, 125, 250, 500 and 1000 were determined, and 250 samples converged already closely with the distribution of 1000 samples [12].

This study considers Eurocode 0 load combination 6.10 for a single variable load:

$$\frac{R_n}{\gamma_{ADM}} \geq \gamma_G G_k + \gamma_Q Q_k = 1.35 G_k + 1.5 Q_k \quad (3)$$

in which G_k is the characteristic value of the permanent load and Q_k characteristic value of the variable load. This equation yields the limit state function of:

$$g(\boldsymbol{X}) = R - G - Q \leq 0 \quad (4)$$

in which R, G and Q are random values for the resistance,

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permanent load, and variable load. The system safety factor is determined based on the assumption that structures are designed economically at their limits, thus characteristic values for actions are obtained by [11]:

$$G_k = \frac{R_n/\gamma_{ADM}}{\gamma_G + \gamma_Q \alpha_Q} \quad (5)$$

$$Q_k = \frac{\alpha_Q R_n/\gamma_{ADM}}{\gamma_G + \gamma_Q \alpha_Q} \quad (6)$$

in which $\alpha_Q = Q_k/G_k$. For the $\beta_T = 3.8$, the γ_{ADM} is determined by minimizing the weighted least-squares of error $S = \sum_{i=1}^{5}(\beta_T - \beta_i)^2 \cdot w_i$ [18], where β_i is the nominal reliability index of case i and w_i is the weight factor. This study considers five cases i, for which the selected α_Q-(w_i) value-pairs are 1.0 (6%), 1.5 (17%), 2.0 (22%), 3.0 (33%) and 5.0 (22%) [1], [20]. Table 3 presents statistical distributions for loads and resistance. Although trusses were designed originally against snow load, in FORM, imposed and wind loads are additionally considered to obtain comparable system safety factors with other studies. However, only a single variable load Q is considered in FORM one at a time.

5.2 Nominal models

Nominal model is the model which is used by the design engineer. Nominal resistance R_n is the lowest resistance among multiple nominal models which differ from each other in imperfection directions. In this study, a total of 20 nominal models with randomly generated imperfection directions, and additionally assuming all directions positive or negative, were studied in each XZ-, YZ- and 3D directions. In these 66 nominal analyses, bow imperfection in the shape of a sinusoidal half wave was modelled for every free span between connections. [12]

Parameters for the nominal model must be chosen such that a sufficiently low variation in reliability level is obtained between varying failure modes to find a single meaningful system safety factor that is proper for all the modes. Article [12] presents the chosen nominal parameters which yield rather uniform reliability levels among tensile and buckling failure modes. It is worth mentioning that with more conservative nominal parameters the derived system safety factor is also more beneficial, and vice versa. Hence, selecting nominal parameters does not directly affect the reliability level of the structure but their selection is important in finding the balance and uniform reliability level among various failure modes. The exact nominal resistances were 1.209 for K30m, 1.226 for K35m, and 1.215 for K40m. [12]

5.3 Monte Carlo-simulations

Table 4 presents the resistances obtained from MCS in terms of mean-to-nominal ratios (R_{MtoN}) and coefficient of variation (CoV). Two sets for each truss, denoted as NC and FC, were studied in MCS. In NC, no correlation was assumed between truss members. In FC, however, the full correlation was assumed between members such that the same randomly sampled values were assigned for each member in a truss. This attempts to simulate a situation in which all truss members are of the same batch. Fully correlated trusses have higher CoVs, but also higher mean-to-nominal ratios as shown in Table 4.

Resistances of steel structures are usually assumed as Log-normally distributed [1]. However, in the case of weakest link systems, Weibull distributions can also be observed [21]. In studied trusses, a somewhat hybrid distribution consisting of both Weibull and Log-normal distributions was observed before the application of Log-normally distributed modelling uncertainty. However, after considering the modelling uncertainty, Log-normal gave the best fit for resistances and was therefore chosen.

5.4 System safety factor γ_{ADM}

Table 4 presents the derived system safety factors by FORM for $\beta_T = 3.8$. In addition to individual loads, the table presents weighted averages 33-33-33 and 50-40-10, the former having even weights for all variable loads and the latter 50% for imposed, 40% for wind and 10% for snow as suggested in [22]. Fully correlated trusses give lower system safety factors than trusses without correlation, hence it is conservative to assume uncorrelated members.

System safety factors for 15 previously studied trusses without correlation had an average $\gamma_{ADM} = 1.21$ for weighting 50-40-10 [12], whereas the corresponding γ_{ADM} is 1.22 for the three currently studied trusses as shown in Table 4. A close correspondence between these two values indicates that there is no remarkable difference in whether the truss configurations for MCS are determined based on conventional methods or GMNIA.

Figure 3 presents γ_{ADM}-β curves for the case K35mNC, which required the highest γ_{ADM}-values among the studied trusses as shown in Table 4. The uppermost curves in the figure correspond to $\alpha_Q = 1$, the next highest to $\alpha_Q = 1.5$, and the lowest curves to $\alpha_Q = 5$. Once the target reliability index is known for corresponding weakest link truss systems, the required γ_{ADM} can be determined based on the figure.

Table 3 Probabilistic models of the resistance and loads.

Property	Description	Distribution	Mean and CoV	Characteristic value	Ref.
R	Resistance	Log-normal	$\mu = R_{MtoN} \cdot R_n$, CoV of R_{MtoN}, see Table 4	R_n	MCS
G	Permanent load	Normal	$\mu = G_k$, CoV=0.1	G_k	[23]
$Q_{Imp,50}$	Imposed load, 50 years	Gumbel	$\mu = 0.6Q_{k,Imp,50}$, CoV=0.35	$Q_{k,Imp,50}$	[23]
$Q_{wind,50}$	Wind load, 50 years	Gumbel	$\mu = 0.7Q_{k,wind,50}$, CoV=0.35	$Q_{k,wind,50}$	[23]
$Q_{snow,50}$	Snow load, 50 years	Gumbel	$\mu = 1.0Q_{k,snow,50}$, CoV=0.22	$Q_{k,snow,50}$	[24]

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Table 4 Statistics for the resistance and γ_{ADM}-values for β_T=3.8.

Truss	Statistics for resistance		γ_{ADM} for β_T=3.8				
	R_{MtoN}	CoV (%)	$G+Q_{Imp,50}$	$G+Q_{Wind,50}$	$G+Q_{Snow,50}$	Weighted 33-33-33[1]	Weighted 50-40-10[2]
K30mNC	1.07	5.29	1.14	1.29	1.36	1.26	1.22
K30mFC	1.10	5.91	1.12	1.28	1.34	1.25	1.21
K35mNC	1.07	5.71	1.14	1.30	1.36	1.27	1.23
K35mFC	1.10	5.93	1.12	1.27	1.33	1.24	1.20
K40mNC	1.09	5.32	1.13	1.28	1.34	1.25	1.21
K40mFC	1.11	6.05	1.11	1.26	1.33	1.23	1.19
Average NC	-	-	1.14	1.29	1.35	1.26	1.22
Average FC	-	-	1.12	1.27	1.33	1.24	1.20
Average All	-	-	1.13	1.28	1.34	1.25	1.21

[1] Average, i.e. even weights for imposed, wind and snow load

[2] Average by giving 50%, 40% and 10% weights for imposed, wind and snow load, respectively.

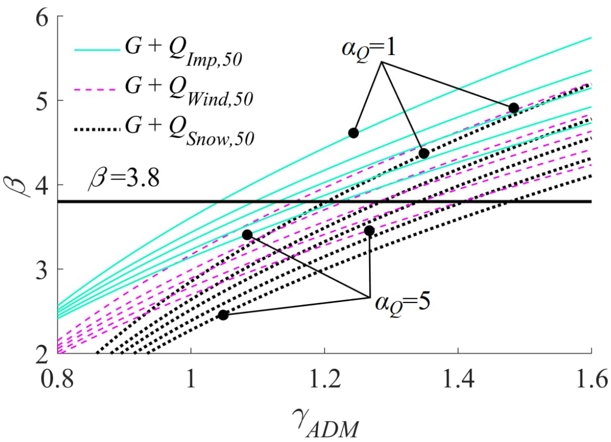

Figure 3 γ_{ADM}-β curves for the case K35mNC

6 Further research

It has been estimated that γ_{ADM}=1.21 yields an uneconomical design outcome when compared to the conventional method and $\gamma_{ADM}\approx$1.15 could be more suitable for truss structures [12]. Although design by advanced methods can result in a more uniform system reliability than conventional methods enhancing the safety of the design [6], potential cost savings are however the best incentives for structural engineers and their clients to adopt the presented GMNIA procedure. The following three issues have a huge impact on the system safety factor and effectiveness of the method and need further research and discussions. Firstly, Table 4 and Figure 3 show how wind and snow loads require much greater γ_{ADM}-values than the imposed load for the same reliability index because climatic loads have more onerous statistical distributions, see Table 3. The snow load would require γ_{ADM} =1.35 on average without correlation to fulfil β_T=3.8, which would lead to an extremely uneconomical design when compared to conventional methods [12]. On the other hand, γ_{ADM} =1.21 by the weighting 50-40-10 leads to a much lower reliability index than 3.8 for climatic loads, see Figure 3. In some countries and regions, snow load can be the primary load for roof trusses, hence it is questionable whether it is justified to assume only a 10% weight for snow in such situations. Therefore, well-justified and commonly accepted loading models and weight factors are needed for the presented method. Document "Reliability background of Eurocodes" [22] will deliver this information once it is officially published, and system safety factors may need recalculations for the updated load models and weight factors.

The second issue affecting the system safety factor and effectiveness of the method is the target reliability index β_T. As cited in Section 5, the target reliability level may be taken the same as the reliability level of a structural system designed by the conventional method. For weakest link structural systems such as truss structures, the system reliability level is lower than the reliability level of a single member [12], [18], thus justifying a lower β_T to be used. Therefore, additional reliability studies are needed to determine the actual reliability level of structural systems which are designed based on the conventional method, thus to obtain β_T to be used in deriving γ_{ADM}.

The third issue affecting the effectiveness of the method is the assumption of pinned connections for truss braces [12]. The conventional method can use reduced buckling lengths for truss members to consider the connection stiffness [4]. Unfortunately, no modelling method has yet been developed for welded hollow section connections in GMNIA. However, Generalised Component Method [25] may solve this issue once proper connection models are developed for CFRHS.

7 Concluding remarks

This study presented the prequalification process and derivation of the system safety factor for the practicable GMNIA procedure. The target structures were three Warren roof trusses made of cold-formed rectangular hollow sections of the steel grade S700. These trusses were originally designed by using GMNIA with a system safety factor of 1.21. The required value of 1.21 was determined based on reliability studies conducted for 15 trusses [12] designed by the conventional Eurocode 3 method [4]. The corresponding system safety factor for the three Warren trusses presented in this study was 1.22. Consequently, the close correspondence between these safety factors indicates that the reliability procedure is not sensitive to the methodology used in selecting the cross-sections for trusses.

Reliability studies for the three trusses were conducted by assuming a full correlation of random properties for each

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member in a truss, and additionally by assuming no correlation such that each member in a truss had independent random properties. Full correlation yields 1-2% lower system safety factors than without correlation, hence it is conservative to assume no correlation for such truss structures.

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