Risk-informed requirements for design and assessment of structures under temporary use

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ABSTRACT

The relatively high failure rates, with important consequences in many cases, suggest that the implicitly acceptable risk levels corresponding to temporary civil engineering structures and activities might exceed the bounds of normally acceptable levels associated with different societal activities. Among other reasons, this may be attributed to the lack of a rational approach for the assessment of risks associated with the different technologies supporting these activities in general, and for structures in particular. There is a need for establishing appropriate target reliability levels for structures under temporary use taking into account specific circumstances such as reduced risk exposure times. This issue is being addressed in the present paper. Acceptance criteria for building structure related-risks to persons obtained in prior studies are adapted to the special circumstances of non-permanent risk exposure. Thereby, the general principle followed is to maintain the same risk levels per time unit as for permanently occupied buildings. The adaptation is based on the statistical annual fatality rate, a life safety risk metric which allows for a consistent comparison of

risks across different societal activities and technologies. It is shown that the target reliability indices taking account of the temporary use of buildings might be significantly higher than the values suggested for permanently used structures.

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The inference of time dependent target reliabilities for structures under temporary use is addressed. As a general principle, the same risk levels per time unit as for load bearing systems of permanently occupied buildings are maintained.

KEYWORDS

Structural reliability; collapse; risks to persons; acceptance criteria; temporary use.

1. INTRODUCTION

Current life safety risk regulations are subject to a number of inconsistencies originating from different sources, such as, for example, the lack of a unified definition of risk metrics across different societal activities and their corresponding supporting technologies (Faber, Sørensen, & Vrouwenvelder, 2015). One of the major consequences arising from these inconsistencies is a large scatter in the implicitly acceptable risk levels associated with the mentioned activities. This is schematically shown in Fig. 1, where the time-dependant exposure of an individual person to varying acceptable risk levels (per time-unit) associated with different societal activities A_i is represented (Tanner, Hingorani, Bellod, & Sanz, 2018). As shown in the Figure, the acceptable risk-levels corresponding to most of the societal activities (and the corresponding applied

technologies) a person may perform during the major part of time fall within the bounds that are considered normal for a broad range of matters, such as residence, transport or professional tasks. However, due to the mentioned lack of consistency in current life safety risk regulations not all activities necessarily follow the general norm. Certain activities might involve implicitly accepted risk levels that temporarily exceed (see activity A_3) or fall below (see A_{m+1}) the bounds of normally acceptable levels associated with the mentioned everyday activities. The former case (A_3) would call for an improvement of best practices of the applied technologies supporting the activity and hence for an investment in risk-reducing measures, whereas the latter (A_{m+1}) would be indicative of a non-efficient allocation of available resources for such measures. It is one of the principle purposes of life safety risk regulation to ensure that persons are safeguarded to the highest affordable level and not exposed to excessive levels of life safety and health related risks (Faber et al., 2015).

Fig. 1. Schematic representation of acceptable risks per time unit associated with different societal activities, A_i , according to current regulations and best practices (Tanner et al., 2018)

A field of application where the variability in the acceptable risk levels might be inappropriate are temporary civil engineering structures or activities associated with such structures. Structural

collapses during temporary use of civil engineering facilities are reported frequently from all over the world, often related to construction processes (André, Beale, & Baptista, 2012b; Andresen, 2012; Eldukair & Ayyub, 1991; Hingorani & Tanner, 2019; Tanner & Hingorani, 2013; Tanner et al., 2018; Terwel, Boot, & Nelisse, 2014) but also to leisure activities (Crosti, Bontempi, & Arangio, 2016). The observable relatively large failure rate of temporary structures, with important consequences in many cases, suggests that the implicitly admissible risks associated with such structures or activities might be too high in relation to generally accepted risk levels corresponding to permanent structures under persistent situations, referring to conditions of normal use within a time period of the order of the design working life (EN 1990, 2002). André *et al.* (André, Beale, & Baptista, 2012a) support this statement by affirming that due to a lack of a consistent approach in current design codes, the reliability levels associated with temporary structures exhibit a great variation and are often smaller compared to permanent structures. Indeed, for the time being the current codes and standards do not provide a coherent framework for design or assessment of structures under temporary use and there is a need for guidelines with respect to the choice of appropriate target reliability levels (Caspeele, Steenbergen, & Taerwe, 2013). It has been recently suggested that the fundamental basis for choosing the levels of safety for temporary structures or structures under temporary use shall not be different from those applied to permanent structures and should be fixed taking account of both, possible failure consequences and relative costs for risk-reduction measures (JCSS, 2015). Moreover, in view of the important consequences the failure of structures under temporary use might entail, there is no meaningful reason to choose a priori lower safety levels for such structures just because of their temporary use conditions (JCSS, 2015). Taking into account these considerations, the present contribution addresses the development of risk-based target reliability levels for structural members in buildings under temporary use

conditions. The developments are based on the results of prior studies (Tanner & Hingorani, 2010, 2015) where structural safety requirements were inferred from implicitly acceptable risks to persons associated with building structures in compliance with present best structural engineering practice. Using a life safety risk metric proposed by Faber *et al.* (Faber *et al.*, 2015) which relates risk exposure due to different activities and applied technologies (Fig. 1), the general approach for the definition of the time-dependent requirements is to maintain the acceptable life safety risk level per time unit during the temporary use activity of a building as for structures under permanent use conditions.

Sections 2 and 3 of this contribution provide a brief summary of the prior studies by the authors and the results obtained, including recent updates, posterior to the developments described in (Tanner & Hingorani, 2010, 2015). In particular, these updates consist in an improvement of the models for estimation of structural collapse-induced fatalities (section 2.4) and of the conditional probabilities of death of building user(s) given collapse (sections 3.2, 3.3). Moreover, the influence of risk aversion is mitigated with respect to the assumptions adopted in (Tanner & Hingorani, 2015) (section 3.3). Subsequently, based on these updated results, section 4 describes the derivation of acceptance criteria under temporary use conditions of buildings. The application of these criteria is illustrated in section 5 for two buildings under different temporary use conditions and with different potential failure consequences. The first example refers to the extension of a grandstand of an existing stadium, where people congregate periodically during a very short period of time and where potential failure consequences may be large. The second example corresponds to a multifunctional building designed for the purpose of international sailing competitions taking place a few times only within the considered design working life of the structure and where, due to the

structural layout and the building use characteristics, the consequences for human life are moderate in case of failure of one of the constitutive structural members.

2. ACCEPTABLE STRUCTURE-RELATED RISKS TO PERSONS

2.1 Approach

Although according to the international standard about general principles on reliability of structures (ISO 2394: 2015, 2015) there is no need to define acceptable levels of risk to persons, in the opinion of social scientists it is unlikely that the public would accept higher failure rates than those associated with current best practice, even if they are based on rational acceptance criteria such as the Marginal Life Saving Cost (MLSC) principle. Indeed, the aforementioned standard states that “an activity which is found acceptable should be assessed in regard to the corresponding absolute level of life safety risk”, and that the practical implementation of the MLSC principle by using the Life Quality Index (LQI) might require the specification of “absolute values of the acceptable life safety risks”. For these reasons, building structure-related risks to persons in compliance with current best practice were derived in prior studies and acceptance criteria deduced from the findings (Tanner & Hingorani, 2010, 2015). The developments were based on the following guidelines.

In explicit analysis, risks are quantified and compared to established safety requirements. In the present study, acceptable risk is defined as the risk associated with structures that are strictly compliant with the safety requirements set out in existing structural standards, which reflect current best practice and are regarded to be acceptable by definition. Risk acceptability therefore depends on the degree of reliability implicitly required by such standards, which in turn depends on the state of uncertainty associated with the standardized rules.

A procedure was established to determine the risks implicitly accepted by standards such as the Eurocodes (EN 1990, 2002) and applied to a representative set of building structures (Fig. 2). Structure-related notional risks (R) for a given structure were obtained from equation (1) on the grounds of the probability of occurrence ($p_{f,j}$) of each of the n_{sc} failure scenarios (j) associated with the structure and its consequences in case of occurrence in terms of the expected number of fatalities (N_j) (Tanner & Hingorani, 2010, 2015).

$$R = \sum_{j=1}^{n_{sc}} (p_{f,j} \cdot N_j) \quad (1)$$

A short summary of the underlying assumptions and the results obtained is given in sections 2.2 to 2.5 below. The deduction of acceptance criteria based on these results is subsequently addressed in section 3.

Fig. 2. Procedure for derivation of risk-based acceptance criteria for structural safety

2.2 Representative set of structures and associated failure scenarios

A series of hypothetical but realistic building structures was obtained by varying the parameters with the greatest effect on design within reasonable ranges to cover the vast majority of the cases encountered in practice: building material and geometry, use category, loads, material strength and

the like. Sets of representative structures made from different constituent materials such as structural steel, reinforced concrete, composite steel and concrete and glued laminated timber were established and analysed. In all, the analysis covered 10872 buildings classified under consequence category CC2, where the number of persons at risk per unity of net-room building area affected in a structural failure scenario is moderate (e.g. residential and office buildings), and 8136 buildings corresponding to category CC3, where this number would be large (e.g. buildings with areas where large numbers of people may congregate (see section 2.4 concerning the definition of consequence classes). The series included buildings with total net room areas ranging from 200 to 990000 m², with up to 10 storeys (excluding the ground storey and roof) in category CC3 and up to 30 in category CC2. The number of main members (floor beams, roof beams and columns) per structure ranged from 6 to 40931 in both categories. Since the hypothesis adopted is conservative with regard to the reliability level and in light of their wide use in conventional building structures, only statically determinate members were considered in the study. The most representative failure scenarios (j), characterised by a specific load combination and failure mechanism for a specific structural member, were then identified to formulate the respective limit state functions, based on the Eurocodes for basis of design (EN 1990, 2002), actions (EN 1991-1-1, 2002) and resistance (EN 1992-1-1, 2004; EN 1993-1-1, 2005; EN 1994-1-1, 2004; EN 1995-1-1, 2004). The members were subsequently designed strictly to the code specifications ($E_d=R_d$), i.e. the provided design value for structural resistance R_d complied exactly with the design value for the considered action effects E_d .

2.3 Implicitly acceptable failure probabilities

The implicitly acceptable failure probabilities $p_{f,j}$ for each representative failure scenario (j) were determined for all the strictly designed members using the First Order Reliability Method (FORM)

(Hasofer & Lind, 1974). For this purpose, previously developed probabilistic models (fib, 2016; Tanner, Lara, & Hingorani, 2007) for the structural design variables were employed that represent the state of uncertainty associated with the rules laid down in the Eurocodes. The probabilistic models proposed are consistent with those laid down in the Probabilistic Model Code (JCSS, 2000). For sake of example, Fig. 3 shows the obtained implicitly acceptable failure probabilities $p_{f,j}$ for structural steel members (Reference period: $T_{ref} = 50$ years), represented as a function of ratio v between the variable loads and the total loads, formulated in terms of characteristic values. The left Figure distinguishes the results corresponding to the bending and shear failure mode of 144 different roof beams whereas on the right the findings for the compression failure mode of 12168 columns are shown. It can be observed that in many cases, especially for the beams, the implicitly admissible failure probabilities of the strictly designed members exceed the current Eurocode requirement ($p_{f,EN1990} = 7.2 \cdot 10^{-5}$, corresponding to a reliability index of $\beta_{EN1990} = 3.8$, $T_{ref} = 50$ years). In addition, it is shown that the implicitly acceptable probabilities are generally subject to a wide scatter. Given the many simplifications and assumptions adopted in the calibration of the code rules for which the implicit level of reliability was determined in the present study, the dispersion observed is not surprising however (Tanner & Hingorani, 2010; Tanner et al., 2007).

Fig. 3. Implicitly acceptable failure probabilities for structural steel members (left: Roof beams, right: Columns); reference period: $T_{\text{ref}} = 50$ years.

2.4 Failure consequences

In conformity with their designation (not their definition) in the current Eurocodes (EN 1990, 2002), three consequence classes, CC1 to CC3, are established as a function of the number of persons at risk (O_{col}) per unit of building area affected by the collapse of a particular structural member (A_{col}). A threshold occupancy rate ($O_{\text{col}}/A_{\text{col}}$) of 1/10 persons/m² is defined for distinction between CC2 and CC3. This value represents an approximate upper bound to typical occupancy rates corresponding to building use categories “Office”, “Residential”, “Hospital”, “Industrial” and “Storage”. Moreover, it constitutes a lower bound to building use categories such as “Congregation” and “Educational”. On the contrary, the suggested limit occupancy rate $O_{\text{col}}/A_{\text{col}}$ between CC1 and CC2, of 1/100 persons/m², provides generally a lower bound to all the mentioned use categories. Moreover, a set of representative values for the occupancy rates associated with the three CC has been defined. The value established for CC2, $(O_{\text{col}}/A_{\text{col}})_{\text{rep}} = 1/30$ persons/m², corresponds approximately to the average occupancy rate of residential buildings in Spain (INE, 2001), which is probably the most representative use category for moderate failure consequences. For sake of simplicity, the representative values for CC1 and CC3, have been chosen one order of magnitude smaller and higher, respectively, 1/300 and 1/3 persons/m².

Consequence models associated with the defined CC were derived, based on a statistical evaluation of data corresponding to more than 150 total or partial building collapses in mainly Western countries. Equation (2), deduced with multiple linear regression analysis, predicts the consequences of a structural member collapse scenario j in storey buildings, expressed in terms of the number of fatalities expected, N_j , relative to the area affected by the collapse, $A_{\text{col},j}$ and its occupancy $O_{\text{col},j}$.

$$N_j = 0.26 \cdot A_{col,j}^{0.19} \cdot Ocu_{col,j}^{0.7} \quad (2)$$

Factoring the established representative occupancy rates $(Ocu_{col}/A_{col})_{rep}$ into (2) delivers consequence model (3), where CC_j is a consequence class-dependent model constant, equal to 300, 30 and 3, for CC1, CC2 and CC3, respectively.

$$N_j = 0.26 \cdot \frac{A_{col,j}^{0.89}}{CC_j^{0.7}} \quad (3)$$

2.5 Implicitly acceptable risks

Fig. 4 shows the notional risks to persons (R) versus total net room area (A) for the totality of the buildings analysed: each point represents the implicitly acceptable risk associated with a specific building structure considering a reference period of $T_{ref} = 50$ years. A statistical analysis of the findings yields the functions for the mean, R_m , and the 5 and 95-% fractiles, $R_{k,5}$ and $R_{k,95}$, respectively, shown in the Figure. The expected number of fatalities was observed to be higher in buildings in consequence category CC3 than in category CC2. In addition to this reasonable result, the derived notional risks were found to vary widely due, among others, to the scale effect. Risks associated with structures designed in strict accordance with the existing legislation rise with the number of failure mechanisms, structural members and combinations of actions that may lead to structural failure (Tanner & Hingorani, 2010, 2015) and, as Fig. 4 shows, with the size of the net room area of the building. As shown in (Tanner & Hingorani, 2010, 2015), the scale effect can be eliminated by normalising the results, e.g. for the total net room area (A) or for the total number of failure mechanisms (number of failure scenarios, n_{sc}).

Fig. 4. Notional risks (R) versus total net room area (A) [m²] for all the buildings analysed; left: CC2; right: CC3; reference period: 50 years

3. INFERENCE OF ACCEPTANCE CRITERIA

3.1 Generalities

The risks and failure probabilities obtained in the present study are primarily notional. They are nonetheless related to the real values, inasmuch as the probabilistic models used to determine the implicitly acceptable failure probabilities (section 2.3) represent the physical properties of the structural design variables modelled and since the consequence models were derived from actual instances of building failure (section 2.4). The order of magnitude obtained for implicitly acceptable risks is consequently suitable for present best practice and the results may be used as a basis for the definition of risk acceptance criteria.

3.2 Fatality rate

The fatality rate or the probability that a given building user will lose his or her life due to structural collapse in a specific reference period (T_{ref}) can be estimated by relating risks normalised for the building net room area (R/A) to the building occupancy rate, i.e., the number of building users per unit of net room area (Ocu/A), which depends on building use. For the purposes of the present study, the representative occupancy rates defined in section 2.4, $(Ocu/A)_{rep,CC2} = 1/30 \text{ persons/m}^2$

and $(Ocu/A)_{rep,CC3} = 1/3$ persons/ m², are adopted for consequence categories CC2 and CC3, respectively. Factoring these values to the normalised risks $(R/A)_{1y}$ ($\approx (R/A)_{50y} / 50$), associated with a reference period $T_{ref} = 1$ year, yields the implicitly accepted, annual fatality rate due to structural failure ($r_{LR,adm}$) in CC2 and CC3 buildings, according to equation (4). Results based on the mean value of variable $(R/A)_{1y}$, $(R/A)_{1y,m}$, and the 5 and 95-% fractile values, $(R/A)_{1y,k,5}$ and $(R/A)_{1y,k,95}$, respectively, are distinguished (Table I). Interestingly, the $r_{LR,adm}$ values for both consequence classes, CC2 and CC3, are of the order of $10^{-6} y^{-1}$ if $(R/A)_{1y,m}$ is considered, whereas $r_{LR,adm}$ tends toward values on the order of $10^{-5} y^{-1}$ when the evaluation is based on $(R/A)_{1y,95}$. In general, the findings are consistent with the criteria laid down in standards, guidelines and technical papers on the safety requirements for built systems in general and structures in particular (SIA 269, 2011; Steenbergen, Sykora, Holicky, Diamantidis, & Vrouwenvelder, 2015; Steenbergen & Vrouwenvelder, 2010; Sykora, Diamantidis, Holicky, & Jung, 2017; Vrijling, Van Gelder, & Ouwerkerk, 2005; T. Vrouwenvelder & Scholten, 2010).

$$r_{LR,adm} \cong (R/A)_{1y} \cdot (A/Ocu) \quad (4)$$

Life safety risk-based target failure probabilities ($p_{ft,LR}$) are deduced from the $r_{LR,adm}$ values which may serve, for instance, as a basis for the consistent calibration of partial factors for routine structural design and assessment. This is achieved by factoring in the conditional probability of the death of a person present in the building at the time of collapse ($p_{d|f}$) according to (5) below:

$$p_{ft,LR} \cong \frac{r_{LR,adm}}{p_{d|f}} \quad (5)$$

The statistical analysis of the data on collapsed building structures used to derive the consequence model presented in section 2.4 delivers an estimate for the mean value of probability $p_{d|f}$ of about 0.3-0.35. Moreover, the analysis suggests that $p_{d|f}$ is independent of the consequence class (CC), a

finding that, although in contrast to the commonly adopted assumption of higher $p_{d/f}$ for higher CC's (Steenbergen et al., 2015; Steenbergen & Vrouwenvelder, 2010; Sykora et al., 2017), seems plausible. Factoring $p_{d/f} = 0.3$ into equation (5) yields the fatality rate-based target failure probabilities ($p_{ft,LR}$) and the corresponding target reliability indices ($\beta_{t,LR}$), shown in Table I. Again, findings based on the mean, and the 5 and 95-% fractile values of variable $(R/A)_{1y}$ are distinguished. It should be observed that due to the similar fatality rates $r_{LR,adm}$ for, respectively, CC2 and CC3, and the common probability $p_{d/f}$, the difference between the CC2 and CC3 safety requirements is almost negligible.

Table I. Mean and 5 and 95-% fractile values for admissible risks normalised for building net room area in 1-year period $((R/A)_{1y})$; annual fatality rate $(r_{LR,adm})$; respective target failure probability $(p_{ft,LR})$ and 1-year target reliability index $(\beta_{t,LR})$

Consequence class	CC2				CC3			
Value	$(R/A)_{1y}$	$r_{LR,adm}$	$p_{ft,LR}$	$\beta_{t,LR}$	$(R/A)_{1y}$	$r_{LR,adm}$	$p_{ft,LR}$	$\beta_{t,LR}$
	$d/(m^2 \cdot y)$	$1/y$	-	-	$d/(m^2 \cdot y)$	$1/y$	-	-
5 % fract.	$5 \cdot 10^{-9}$	$1 \cdot 10^{-7}$	$5 \cdot 10^{-7}$	4.9	$2 \cdot 10^{-8}$	$6 \cdot 10^{-8}$	$2 \cdot 10^{-7}$	5.1
Mean	$1 \cdot 10^{-7}$	$3 \cdot 10^{-6}$	$1 \cdot 10^{-5}$	4.3	$6 \cdot 10^{-7}$	$2 \cdot 10^{-6}$	$6 \cdot 10^{-6}$	4.4
95 % fract.	$4 \cdot 10^{-7}$	$1 \cdot 10^{-5}$	$4 \cdot 10^{-5}$	4.0	$2 \cdot 10^{-6}$	$7 \cdot 10^{-6}$	$2 \cdot 10^{-5}$	4.1

3.3 Reference system

As decision making is often geared to prevent accidents with a large number of fatalities, events with more casualties tend to be regarded as more important than those with fewer, even where the annual fatality rate is the same. To account for this attitude, frequency of collapse under the current best practice was consequently analysed for several reference systems and admissible failure probabilities were deduced from the findings (Tanner & Hingorani, 2015), as summarised below.

Frequency–consequence curves, also termed F–n curves, may be used to relate risks to the size of the reference system considered (e.g. all structures designed and built on a national level further to a specific code or a particular structure or structural member compliant with that code) for a given activity or technology (Vrijling, van Gelder, Goossens, Voortman, & Pandey, 2004). They plot the relationship between the annual probabilities of the occurrence of events and $n \geq N$ fatalities (Diamantidis & Vrouwenvelder, 2012). The mathematical definition of the F-n curve is given by equation (6), where $F(1)$ and α are constants:

$$F(n) = p(N \geq n) \leq F(1) \cdot n^{-\alpha} \quad (6)$$

The constant $F(1)$ represents the frequency of occurrence of an event with $N \geq 1$ fatalities. As a general rule, its value should be consistent with the reference system to which the F-n criterion is applied. Furthermore, in order to avoid irrational decision-making, the definition of an F-n acceptance criterion should also be consistent with the expected number of fatalities $E(N)$ associated with the system (Trbojevic, 2009), i.e. the area under the F-n acceptance line, and the reference period considered. Further to Vrouwenvelder *et al.* (A. Vrouwenvelder, Lovegrove, Holicky, Tanner, & Canisius, 2001) an approximate solution for this area is given by (7) assuming a risk-aversion exponent $\alpha = 1$, as well as lower and upper integration limits of, respectively, $N_{\min}=1$ and $N_{\max}$.

$$E(N) \approx F(1) \cdot (\ln(N_{\max}) + 1) \quad (7)$$

A value $\alpha = 1$ reflects a risk neutral approach, characterised by a proportional decrease of the target exceedance frequencies with rising consequences of undesired collapse events, what according to Sykora et al. implies a fair decision making process (Sykora et al., 2017). Data on F-N curves associated with different technological systems compiled by (Proske, 2008) supports the adoption

of a risk-neutral approach. The available data shows that the slopes of the implicitly acceptable frequencies-fatalities relationships are often not significantly different from unity, at least for an order of magnitude of failure consequences relevant in the present context, of the order of up to $N = 100$ fatalities with a few exceptions, for instance the World Trade Center collapse (FEMA 403, 2002). The assumption of $\alpha = 1$ is moreover advantageous since it enables a comparison of the findings with safety criteria based on the Life Quality Index (LQI) according to (ISO 2394: 2015, 2015), to be addressed in future studies. A comparison between the F-N curve based human safety acceptance criterion suggested in (fib, 2016) and the LQI-approach, conducted by (Sykora, Holicky, Jung, & Diamantidis, 2018), revealed similar target reliability indices if $\alpha = 1$ is adopted for the former.

Based on equations (6) and (7), F-n acceptance criteria for structural safety decision-making can be derived. In view of practical applications of these criteria, the reference system is reduced from whole buildings to individual members (Tanner & Hingorani, 2015), i.e. individual failure scenarios represented by specific failure modes in given structural elements. This is achieved by establishing F-n criteria consistent with the expected number of fatalities associated with a building structure (R) normalised for the total number of failure scenarios (n_{sc}), associated with all the members comprising the structure in question (Tanner & Hingorani, 2015). The expected number of fatalities associated with a system ($E(N)$), in turn, is given by the area under the respective F-n curve, calculated approximately with equation (7). Hence, when $E(N)$ in (7) is substituted by the normalised risks $(R/n_{sc})_{1y}$, referring to a period of $T_{ref} = 1$ year, the annual frequency of occurrence for structural failure scenarios of specific elements in given building structures with $N \geq 1$ fatalities, $F(1)$, can be deduced as a function of N_{max} . It should be observed that due to the logarithmic function in equation (7), $E(N)$ is not very sensitive to variations of N_{max} within realistic ranges. For

purpose of the present study, $N_{\max}$ has been replaced by the number of persons at risk in a specific collapse scenario (Ocu_{col}), given by the product of the representative occupancy rate for a specific consequence class, $(Ocu_{\text{col}}/A_{\text{col}})_{\text{rep}}$ (section 2.4), and the area affected by the collapse (A_{col}).

Given $F(1)$ and α , the (normalised) F-n acceptance criteria are defined by equation (6). Finally, the $F(n)$ frequencies are converted to admissible structural failure probabilities ($p_{\text{ft},\text{LR}}$) with equation (8), where $p_{N|f}$ describes the conditional probability of $N \geq n$ fatalities in the event of failure (Tanner & Hingorani, 2015). This probability can be estimated by means of (9), yielded from statistical analysis of the collapse data compiled to establish a model for prediction of the number of collapse-induced fatalities (section 2.4). In equation (9), CC is a consequence class-related model constant equal to -2.57, -2.31 and 0, for CC1, CC2 and CC3, respectively. Moreover, (9) is a function of the so-called damage class (DC), established in dependency of the area affected by the collapse A_{col} according to Table II.

$$p_{\text{ft},\text{LR}} \cong \frac{F(n)}{p_{N|f}} \quad (8)$$

$$p_{N|f} = 13.83 \cdot N^{-1.04} \cdot e^{(CC+DC)} \quad (9)$$

Table II. Model constant DC corresponding to different building damage classes

Damage class	Description	A_{col} (m ²)	DC
DC1	Small damage	≤ 100	-1.88
DC2	Medium damage	$>100; \leq 500$	-0.63
DC3	Large damage	> 500	0

The target reliability indices ($\beta_{\text{t},\text{LR}}$) corresponding to the admissible failure probabilities $p_{\text{ft},\text{LR}}$ according to (8) are reflected by the step-functions shown in Fig. 5 (left CC2, right CC3), under distinction of the findings based on, respectively, the mean (dashed lines), and the 95-% fractile

value (continuous lines) of the implicitly acceptable risks $(R/n_{sc})_{1y}$. The $\beta_{t,LR}$ values are represented as a function of the area affected by the collapse (A_{col}) of the structural member considered, converted from the expected number of fatalities n , applying the consequence model (3) used to derive the implicitly accepted risks (section 2.4). The $\beta_{t,LR}$ - A_{col} step functions following from (8) are intercepted by horizontal lines representing the target reliability index based on the implicitly acceptable fatality rates, corresponding to (5), summarized in Table I. In case of CC2, this intercept corresponds to an area $A_{col} \approx 30 \text{ m}^2$ (Fig. 5, left), whereas for CC3, criteria (5) and (8) intercept at $A_{col} \approx 6 \text{ m}^2$.

In order to obtain operational acceptance criteria suitable for practical applications, a least squares fitting procedure is applied to the obtained step functions (including the fatality rate based result) within the limits $1 \leq A_{col} \leq 10000 \text{ m}^2$, resulting in the continuous functions shown in Figure 5. In comparison to the criteria suggested in the prior study conducted by the authors (Tanner & Hingorani, 2015), these functions are characterized by slightly higher target reliabilities in the range of small A_{col} , what can be mainly attributed to the more refined assumptions concerning the conditional probability of death for a building user given collapse, $p_{d|f}$, (section 3.2). On the contrary, the updated criteria suggest lower reliability requirements for large A_{col} , due to the mitigation of risk aversion ($\alpha = 1$ instead of 2) and the improved modelling assumptions for probability $p_{N|f}$ of $N \geq n$ fatalities in the event of failure (eq. (9)), which in (Tanner & Hingorani, 2015) was approached with a constant value for want of a more detailed model.

Fig. 5. Target reliability index (β_{TLR}) versus the area affected by collapse (A_{col}) for members in CC2 (left) and CC3 (right) structures; $T_{ref} = 1$ year.

The updated target reliability indices shown in Fig. 5 may be used for structural design or assessment of building structures. In particular, the findings based on the mean value are recommended for design of new structures, whereas the 95% fractile value is suggested to be employed in the context of the assessment of existing structures, following the considerations in (Tanner & Hingorani, 2015). For sake of comparison, the target reliability index demanded in the Eurocodes for structural design (EN 1990, 2002) for CC2 and CC3 structures are likewise plotted in Fig. 5. It can be observed that up to a certain area A_{col} , the Eurocode requirement is stricter than the herein proposed criteria based on the mean value of the intrinsically accepted life safety risks, suggested for design of new structures. In particular, for CC2 and CC3 structures this is the case if $A_{col} < 200 \text{ m}^2$, and $A_{col} < 1200 \text{ m}^2$, respectively. For higher A_{col} , the herein deduced mean target reliability indices are higher than the current Eurocode design requirements. On the contrary, the proposed criteria based on the 95% fractile value of the implicitly accepted life safety risks, suggested for structural assessment of existing facilities, are generally less demanding than the current Eurocode requirements. Only for very large A_{col} (CC2: $A_{col} > 3500 \text{ m}^2$; CC3: $A_{col} \gg 10000 \text{ m}^2$), extremely rare in practice, the current EN 1990 target values would be exceeded.

4. TEMPORARY USE OF STRUCTURES

4.1 Life-safety risk metric

The risk-informed acceptance criteria presented in Fig. 5 may be used for the design or assessment of structures for buildings under permanent use. In order to adapt the derived target reliability indices to the circumstances of temporary building use, the statistical fatality rate γ_{LR} (10) is used, a life safety risk metric which enables a consistent comparison of risks associated with different activities or applied technologies which potentially cause loss of lives (Faber et al., 2015), i.e. in the present context a comparison between a permanent and temporary use of building structures. The parameter γ_{LR} is defined as the ratio between the expected number of lost lives per time unit caused by a given type of activity or applied technology, n_F , and the expected value of the total number of life years per time unit exposed to the considered activity or applied technology, N_L (Faber et al., 2015):

$$\gamma_{LR} = \frac{n_F}{N_L} \quad (10)$$

4.2 Risk-based time-dependent failure probabilities

The condition to maintain the acceptable life safety risk level per time-unit of risk exposure as under permanent use conditions (section 1) can be formulated in terms of equation (11), by using γ_{LR} (10). Here, $\gamma_{LR,adm,P}$ stands for the admissible statistical fatality rate per time-unit associated with structures under permanent use conditions and $\gamma_{LR,adm,T}$ represents the corresponding rate for buildings under temporary use.

$$\gamma_{LR,adm,P} = \gamma_{LR,adm,T} \quad (11)$$

In case of permanent risk exposure, the statistical fatality rate $\gamma_{LR,adm,P}$ is given by (12), where the total number of life years exposed to the risk, N_L in the notation of (10), corresponds to the number of persons present on a specific building net room area A during the reference period considered. The expected number of fatalities, n_F , in the notation of (10), is replaced by the admissible value for the notional risks $R_{adm,P}$ (associated with the specific building net room area A and reference period T_{ref}) to persons associated with buildings under permanent use conditions. As explained in section 3.3, the notional risks R associated with a specific building are represented by the area under the corresponding $F(n)$ curve, $E(N)$, approximated by equation (7). Hence, $R_{adm,P}$ can be expressed as a function of the admissible failure frequency $F(1)_{adm,P}$ and the considered maximum consequence level N_{max} . Finally, based on (6) and (8), frequency $F(1)_{adm,P}$ can be substituted by the product of the target failure probability $p_{ft,LR,P}$, taking account of life safety risk to persons under permanent building use conditions, and the conditional probability of $n \geq N$ fatalities at the time of collapse, $p_{N|f}$ (9):

$$\gamma_{LR,adm,P} = \frac{R_{adm,P}}{N_L} = \frac{F(1)_{adm,P} \cdot (\ln(N_{max}) + 1)}{N_L} = \frac{p_{ft,LR,P} \cdot p_{N|f} \cdot (\ln(N_{max}) + 1) \cdot n^\alpha}{N_L} \quad (12)$$

In order to factor in the time-dependent use of a structure, (Faber et al., 2015) introduce parameter μ which, in the present context, defines the part of the considered reference period T_{ref} during which persons are temporarily exposed to risk. Making use of μ , the admissible statistical fatality rate under temporary use conditions $\gamma_{LR,adm,T}$ is defined by (13), in terms of the target failure probability for members in building structures under temporary use conditions, $p_{ft,LR,T}$, and the conditional probability, $p_{N|f}$.

$$\gamma_{LR,adm,T} = \frac{R_{adm,T}}{\mu \cdot N_L} = \frac{F(1)_{adm,T} \cdot (\ln(N_{max}) + 1)}{\mu \cdot N_L} = \frac{p_{ft,LR,T} \cdot p_{N|f} \cdot (\ln(N_{max}) + 1) \cdot n^\alpha}{\mu \cdot N_L} \quad (13)$$

Finally, substituting equations (12) and (13) into (11) yields the admissible member failure probability $p_{ft,LR,T}$ taking account of the temporary use conditions of the structure as a function of the failure probability $p_{ft,LR,P}$ associated with permanent risk-exposure:

$$P_{ft,LR,T} = P_{ft,LR,P} \cdot \mu \quad (14)$$

Equation (14) provides a rational basis for deriving risk-based reliability requirements for structures taking account of temporary risk exposure. According to this relation, the shorter the risk exposure, i.e., the smaller μ , the stricter these requirements will be. This is generally a reasonable finding taking account of the fact that human safety requirements are usually based on the assumption of a constant annual failure probability (Steenbergen et al., 2015).

In the present context, $p_{ft,LR,P}$, in the notation of (14), corresponds to the annual ($T_{ref} = 1$ year) failure probability $p_{ft,LR}$ defined in section 3 by means of (8). Hence, $p_{ft,LR,T}$ can be readily obtained as a function of μ and subsequently converted to the corresponding target reliability indices, $\beta_{t,LR,T}$. These indices are shown in Figs. 6 and 7. The former contains the results based on the *mean value* of the implicitly accepted, normalised risks, the latter refers to those corresponding to the *95% fractile value*. The target reliability indices for CC2 (left) and CC3 (right) structures are distinguished. The obtained reliability indices, which are associated with a reference period $T_{ref} = 1$ year, are represented as a function of the area affected by member collapse, A_{col} , and different assumptions for the share μ of T_{ref} during which persons are temporarily exposed to risk. The bottom curve in each of the Figures corresponds to permanent risk exposure of building users ($\mu = 1.0$), adopted from Fig. 5. When the risk exposure time diminishes (i.e. μ reduces), the target reliability indices increase. A value of $\mu = 0.1$ represents an effective reference period equal to 10%

of the total reference period T_{ref} , i.e. about 37 days per year. The curves for $\mu = 0.001$ correspond to an effective reference period of about 9 hours per year.

Fig. 6. Target reliability index ($\beta_{L,R,T}$) versus the area affected by collapse (A_{col}) for members in CC2 (left) and CC3 (right) structures, based on mean value of implicitly accepted risks

Fig. 7. Target reliability index ($\beta_{L,R,T}$) versus the area affected by collapse (A_{col}) for members in CC2 (left) and CC3 (right) structures, based on 95% -fractile value of implicitly accepted risks

4.3 Final remarks

In Figs. 6 and 7 it is shown that the target reliability indices taking account of temporary use may be significantly higher than the values suggested for structures under permanent use conditions. However, it is important to stress that when calculating the reliability indices for structural

members, that are to be compared with the deduced target ceilings, the parameters of the probabilistic models should be adjusted for reference periods in accordance with the duration of the temporary use. For this purpose, it must be distinguished between structures which are used only once during a short period and those which are reused several times, at the same or at different locations (JCSS, 2015). In these cases, duration dependent models should be used, particularly for the relevant variable actions, what may lead to higher reliability levels. Special attention is required for erection processes of structures where different temporary stages are to be considered (JCSS, 2015; Tanner et al., 2018).

5. PRACTICAL APPLICATION EXAMPLES

5.1 Introduction

In the present section the application of the preceding developments is illustrated by means of two buildings under temporary use: The grandstand of the Chapin track and field stadium in Jerez (Spain), built in 1988 and refurbished and extended on the occasion of the horse-racing world championship 2002 – as an example for a CC3 structure where people may congregate – and the Alinghi Base building in Valencia (Spain), designed and built for the America's Cup held between 2004 and 2007 – as an example for a CC2 structure. For purpose of example, in both cases the target reliability index for a relevant structural member is established. The first case (Jerez stadium) concerns the assessment and extension of an existing building, the latter (Alinghi base) applies to the design of a new structure.

5.2 Chapin stadium

The Chapin stadium in Jerez was to be refurbished and extended by adding a new roof (Fig. 8) as well as a new gallery to the existing grandstand, implying an important change of the structural system and leading to completely different internal forces and moments in the existing reinforced

concrete frames (Tanner & Bellod, 2003). The accesses for the public were also reorganised. Furthermore, a new sports centre as well as a hotel were integrated into the stadium (Fig. 8). The extended stadium has a capacity for about 22000 seated spectators.

Fig. 8. Grandstand of the Chapin stadium before (left) and after (right) refurbishment and extension

(courtesy of Cesma Ingenieros and Cruz y Ortiz Arquitectos, © Duccio Malagamba)

The grandstands are constituted by in situ concrete frames of about 24 m length at 6 m centres and by precast concrete elements as platforms for the public. Hence, the possible failure of one main member of an existing frame for which the safety requirements are to be established, may affect an area of approximately $A_{col} \approx 24 \cdot 6 \cdot 2 \cdot 2 \approx 580 \text{ m}^2$, considering situations in which persons are present on and below the frame in question.

For the establishment of share μ of the reference period T_{ref} the stadium is effectively used, it is assumed that the local football team plays a match in the own stadium every two weeks. Consequently, the frequency of use of the stadium is about 25 times per year. It is further assumed that each supporter spends about 2 hours in the stadium per match. Hence, a value of $\mu = (25 \cdot 2) / (365 \cdot 24) = 0.0057$ is being obtained.

In order to determine the target reliability level, the *95% fractile values* may be used for the admissible failure probabilities since the *assessment* of an existing structure is under concern (section 3.3). Thus, the safety requirements shown in the right part of Fig. 7 apply, where the particular circumstances of the herein analysed example case ($A_{col} = 580 \text{ m}^2$, $\mu = 0.0057$) are reflected. A target reliability index $\beta_{t,Chapin} \approx 5.8$ ($T_{ref} = 1 \text{ year}$) is obtained for the members of the existing frames of the grandstand which is significantly higher than the requirement suggested for members whose failure would affect a similar A_{col} in permanently occupied CC3 buildings ($\mu = 1.0$), such as railway stations, hospitals and others, of about $\beta_{t,LR,CC3}(A_{col} = 580 \text{ m}^2, \mu = 1.0) \approx 4.9$ (Fig. 7, right). Likewise, it considerably exceeds the current Eurocode (EN 1990, 2002) requirement for CC3 structures, $\beta_{EN1990} = 5.2$ (Fig. 5). On the other hand, it should be recalled that the need for adjusting the models for the variable actions to the effective, temporary use period (section 4.3) will entail higher structural reliability however. In the present case study, this period is roughly estimated as $0.0057 \cdot 365 \approx 2$ days within the considered reference period of 1 year. Hence, a return period for climatic actions of 2 years may be assumed according to Table 3.1 in (EN 1991-1-6, 2005). In the case of wind loads, for instance, assuming a return period of 2 years leads to a reduction of around 40% in comparison to a permanent use of the structure (EN 1991-1-4, 2005).

Finally, it should be mentioned that structural collapse does not necessarily has to occur during the effective use period, i.e. it might as well occur when the building is not used, when no persons are at risk. For these design situations, the target reliability should be obtained based on economical criteria, taking into account all relevant costs during the future service period.

5.3 Alinghi Base building

The Alinghi base is a prism-shaped building, 68 m long, 39.15 m wide and 13.5 m high (Fig. 9). Its total usable area, nearly 7000 m² counting the four storeys (including ground floor and the accessible roof) into which the building is divided, is occupied by offices, meeting rooms, a gymnasium, a restaurant and a shop, in addition to the boatshed and a VIP lounge. The roof is also fitted with a bar.

The conceptual design of the building is fairly conventional, with composite columns, beams and slabs (Tanner & Bellod, 2008). The design called for continuous columns, from the foundations to the roof. Longitudinally, they were spaced at from 5 to 18 m, whereas transversally the standard spacing was 5 m. The main beams were to run parallel to the longitudinal facades of the building. The distance between these beams was to be spanned by a composite slab with shaped steel sheeting to avoid the need for any transverse beams and thereby to minimize the number of elements and on-site connections (Tanner & Bellod, 2008).

Fig. 9. Alinghi Base structure (Tanner & Bellod, 2008). a) Plan view at first storey level; b) longitudinal section; c) cross-section

In order to illustrate the application of the derived structural safety requirements taking account of the temporary use conditions of the building, one of its columns situated in the first floor of the office sector will be considered, highlighted in Fig. 9. In this sector, columns are spaced at 5 m in both, longitudinal and transverse direction of the building. Hence, it is reasonable to assume that a potential collapse of the considered member affects an area in the first and second floor, as well as on the accessible roof, of about $A_{col} \approx 10 \cdot 10 \cdot 3 \approx 300 \text{ m}^2$ (considering that the slab of the first floor and the ground floor columns sustaining this slab will resist the impact forces due to falling debris).

The Alinghi base was used for sailing competitions between May and June 2006 and between April and July 2007, when actions due to invited guests were simultaneous to actions in consequence of the normal building use. In the former period (May to June 2006), the building was effectively used during approximately 48 days and in the latter (April to July 2007) during about 24 days. Adopting for sake of simplicity the mean value of these two use periods (36 days) for establishment of the reliability requirements, the share μ of the reference period T_{ref} (1 year) during which the office rooms of the Alinghi building are effectively used is roughly estimated to $\mu \approx 36/365 \approx 0.1$. It is assumed that within its total service life, the building would be used on a yearly basis during similar time periods, i.e. a similar μ would be expected for each of these periods.

As stated previously, the target reliability indices inferred from the *mean values* of the implicitly acceptable risks are recommended to be used for *design* of new building members. These are given in the left part of Fig. 6 where the target reliability index corresponding to $\mu = 0.10$ is being shown as a function of A_{col} . Intercepting the obtained curve at $A_{col} \approx 300 \text{ m}^2$ yields $\beta_{t,Alinghi} \approx 5.2$ ($T_{ref} = 1$ year) for the design of the composite column in the office sector of the building. This value is

higher than the requirement suggested for members whose failure would affect a similar A_{col} in (quasi-) permanently occupied CC2 buildings ($\mu = 1.0$), such as residential buildings, of about $\beta_{t,LR,CC2}(A_{col} = 300 \text{ m}^2, \mu = 1.0) \approx 4.7$ (Fig. 6, left). It is also higher than the current Eurocode requirement for CC2 structures, $\beta_{EN1990} = 4.7$ (Fig. 5, left).

Again it should be recalled that structural reliability may also be higher due to the required adjustment of the relevant variable action models (imposed loads) taking account of the temporary use conditions (section 4.3). For the column under concern, this applies to imposed loads on the floors and the accessible roof. Wind actions are not relevant here since the building is fitted with bracings (Tanner & Bellod, 2008).

6. CONCLUSIONS

The present paper addresses the challenging issue of establishing acceptable risks and associated target reliability levels, taking account of temporary use conditions of structures. The developments are based on the condition to maintain the same acceptable risk levels per time-unit as for permanently occupied building structures that are strictly compliant with the safety requirements set out in the current Eurocodes (EN 1990, 2002) and which, further to international standard (ISO 2394: 2015, 2015) constitute present best structural engineering practice associated with risk acceptance criteria for human safety. Using a life safety risk metric, which relates risk exposure due to different activities and applied technologies, it is shown that the target reliability index for structural members significantly increases when short risk-exposure times are considered. Depending on the expected failure consequences and the share of the reference period T_{ref} the building is effectively used, the required risk-based reliability levels may exceed the general target values demanded by current codes and standards. The derived criteria may be applied within the framework of an explicit reliability analysis or constitute the basis for consistent calibration of the

partial factor models used in everyday practice. Regardless of the approach adopted, they should be consistently used in connection with a time-dependent adjustment of the probabilistic models for the relevant variable actions.

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