Application of Probabilistic Robustness Framework: Risk Assessment of Multi-Storey Buildings under Extreme Loading

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Abstract

Risk assessment is a requirement for robustness design of high consequence class structures, yet very little guidance is offered in practice for performing this type of assessment. This paper demonstrates the application of the probabilistic risk assessment framework arising from COST Action TU0601 to multi-storey buildings subject to extreme loading. A brief outline of the probabilistic framework is first provided, including the main requirements of describing uncertainty in the hazards and the associated local damage as well as the consequences of global failure. From a practical application perspective, it is emphasised that there is a need for (a) computationally efficient deterministic models of global failure for specific local damage scenarios, and (b) effective probabilistic simulation methods that can establish the conditional probability of global failure on local damage. In this respect, this work utilises a recently developed multi-level deterministic assessment framework for multi-storey buildings subject to sudden column loss, which is coupled with a response surface approach utilising first-order reliability methods to establish the conditional probability of failure. The application of the proposed approach is illustrated to a multi-storey steel-composite building, where it is demonstrated that probabilistic risk assessment is a practical prospect. The paper concludes with a critical appraisal of probabilistic risk assessment, highlighting areas of future improvement.

Keywords: risk assessment; robustness; multi-storey buildings; extreme loading; progressive collapse.

Introduction

The assessment of structural robustness using a risk-based approach is widely considered to be the most rational, and increasingly the most effective, treatment.[1] Such an approach offers the ultimate criterion for evaluating the risks of failure for existing structures subject to different hazards and for the meaningful comparison of candidate designs for new structures. Indeed, despite the fact that recent design codes have maintained prescriptive guidelines as a practical option for the design of low to medium consequence class structures, systematic risk assessment is still demanded by these codes for the design of high consequence class structures,[2,3] yet virtually no guidance is offered on how such an assessment may be undertaken.

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Within this context, COST TU0601 was initiated to establish an objective risk-based assessment for the robustness qualities of a structure.[4] Such an approach is meaningful only if adequate methods for quantifying failure probabilities and corresponding risk are available. This requires adequate sets of data with respect to exposure conditions, structural response and consequences on the one hand, but also operational calculation procedures on the other. The present paper intends to demonstrate that such calculations can indeed be effectively made for realistic structures.

Towards this end, the paper considers the risk assessment of multi-storey buildings under extreme loading, where, without loss of generality, focus is placed on local damage scenarios consisting of sudden column loss. The adopted probabilistic framework for risk assessment is first described, highlighting its treatment of uncertainty in the hazard, the associated local damage and the ensuing structural failure, as well as the consideration of consequences. Two important components are identified for establishing the structural failure probability associated with a specific local damage scenario, namely an efficient and realistic deterministic model, and an effective probabilistic simulation approach accounting for uncertainty in the structural variables. In this respect, a recently developed approach for deterministic assessment of multi-storey buildings is described, which considers sudden column loss scenarios within a practical multi-level framework.[5] Furthermore, the application of this deterministic approach in probabilistic failure assessment is illustrated in a case study of a multi-storey steel-composite building subject to sudden column loss in alternative locations, where consideration is given to uncertainty in gravity loading, material strength and component ductility parameters. It is finally shown through this case study that probabilistic simulation can be undertaken effectively, thus rendering risk-based robustness assessment of real structures a practical prospect.

Probabilistic Framework for Risk Assessment

Robustness is typically concerned with the consequences of local damage, where the local damage itself may be caused by normal overloading, extreme loads such as fire and explosions, or human errors in design, construction or use.[6] The basic equation for the corresponding risk calculation may be formulated as:

$$\text{Risk} = \sum P(H)\, P(D|H) \times P(S|D)\, C(S) \quad (1)$$

where H represents the hazard, D the direct local damage, S a subsequent failure scenario and C the cost of the final consequences. The summation is over all relevant hazards, local damage and failure scenarios, and the risk is evaluated over a period which is typically a single year or the lifetime of the structure. As the central event, this paper considers hazards leading to the sudden removal of a column, a local damage scenario which is often considered in codes and is also performed in practical design.

Depending on the sophistication of the assessment, H, D and S can be expressed in terms of discrete and/or continuous variables. Examples of discrete variables are event type (fire, explosion,…), event location (ground floor, fifth floor,…), extent of local damage (one or two columns,…), etc. On the other hand, examples of continuous variables are event intensity, amount of local deformations, material strength, gravity loading, etc. Such variables are typically chosen in view of anticipated uncertainty, and as such relevant data on variability is required for a rational probabilistic assessment.

For the purpose of illustration, this paper focuses on types of hazard that could lead to the removal of a single column in multi-storey buildings (*Fig. 1*), namely fire, explosion and human error. The assumed probabilities $P(H)$ for the occurrence of such events, regardless of intensity, over a 50 year period at any location within the building are as given in *Table 1*. It is worth noting at this point that extreme events arising from planned human action, such as vandalism and terrorism, are excluded, as the associated probabilities cannot be rationally established from statistical data. However, even in such cases conditional scenario-based robustness assessment utilising $P(S|D)$ could still prove very useful, not least with regard to comparing design alternatives.

With the hazard expressed at a low resolution (i.e. neglecting intensity, duration, etc.), the correspondence to local damage scenarios of complete column loss is expressed by means of relatively low conditional probabilities $P(D|H)$, which are again assumed as given in *Table 1*. Of course, for higher resolution hazards, a combination of event and structural modelling could be used to establish $P(D|H)$, considering also variables affecting event propagation and its impact on the structure. This would then allow damage to be in turn described at a higher level of resolution, including for example the amount of deformation, the extent of damage, etc. Although variable levels of damage are not explicitly considered here, recent work has shown that sudden column loss offers an upper bound on the ensuing structural response in comparison with intermediate levels of column damage caused by blast loading,[7] thus justifying the adoption of this simplified column loss scenario for practical robustness assessment.

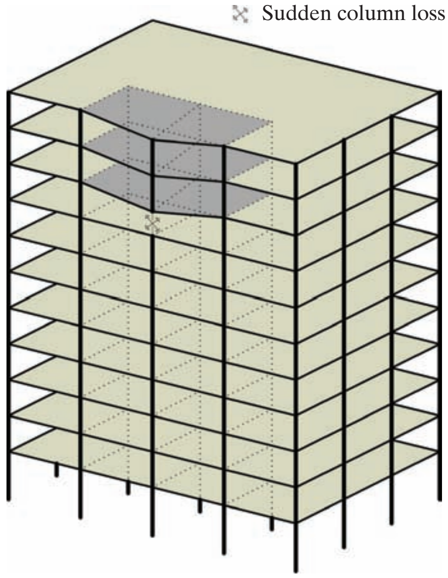


Fig. 1: Multi-storey building subject to column loss

	$P(H)$ (50 years)	**$P(D|H)$**
Explosion	2×10^{-3}	0,1
Fire	20×10^{-3}	0,1
Human error	2×10^{-3}	0,1

Table 1: Estimated probabilities for the column removal case (somewhere in the building)

Subsequent failure scenarios S, as influenced by local damage D, are denoted for simplicity in terms of a binary outcome of no structural failure $\bar{F}$, associated only with direct consequence, and complete structural failure F, associated in addition with severe indirect consequences. Both types of consequences C can be classified into human, economic and environmental categories, and are highly dependent on the specific system under consideration.[8]

The main focus of this paper is the evaluation of the conditional structural failure probability $P(F|D)$ for a given initial damage condition, which requires that a proper failure function is introduced. For the case of multi-storey buildings under sudden column loss, failure is expressed in terms of the vertical loading exceeding the pseudo-static capacity,[5] where dynamic effects are readily accounted for. Each of these terms is function of more basic parameters, such as dead and live load levels, for the applied load, and component strength and ductility, for the pseudo-static capacity. As these parameters are associated with significant variability, they need in general to be represented as random variables. A more detailed exposition of the parameters affecting the evaluation of $P(F|D)$ is provided in the case study presented in a subsequent section.

Simplified Deterministic Framework for Sudden Column Loss

While the concept of "notional member removal" for structural robustness assessment has for long been considered in design guidelines,[2,9] the importance of such factors as geometric/material nonlinearity, ductility and dynamic effects has only recently been recognised.[10] Towards this end, a simplified deterministic framework was proposed[5] for the assessment of multi-storey buildings subject to sudden column loss scenarios, which is particularly suited for application in probabilistic simulation due to computational efficiency. This framework utilises three stages (a) nonlinear static push-down analysis, (b) simplified dynamic assessment and (c) ductility assessment.

The first stage focuses on the system nonlinear static response under vertical loading, and this can be determined using simplified analytical models or detailed numerical models.[5] The transformation of the nonlinear static response to a maximum dynamic response is performed using a novel approach based on energy balance (*Fig. 2*), where the resulting load–deflection response is termed *pseudo-static response*. In this approach, and with reference to *Fig. 2a and b*, the maximum dynamic displacement ($u_{d,n}$) arises from equating the two hatched areas under the constant gravity load and the nonlinear static resistance,

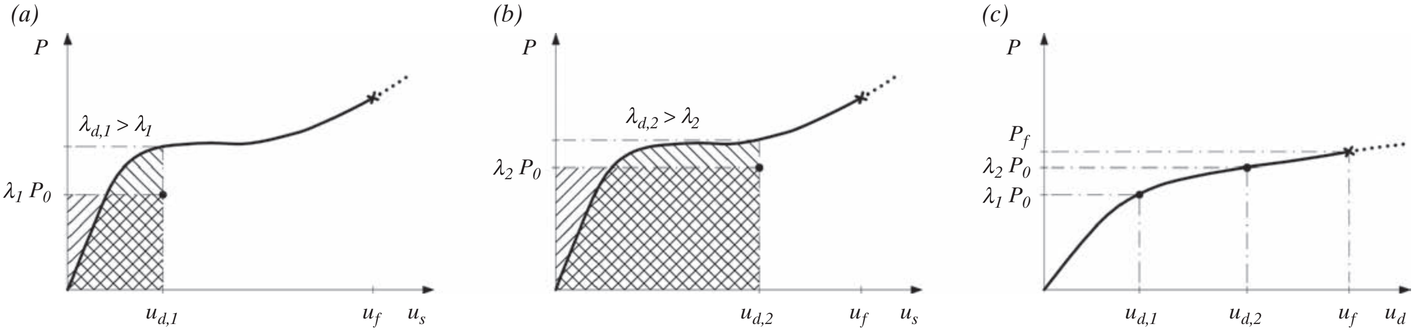


Fig. 2: Simplified dynamic assessment and definition of pseudo-static response.[5] (a) Dynamic response ($P = \lambda_1 P_0$), (b) dynamic response ($P = \lambda_2 P_0$) and (c) pseudo-static response

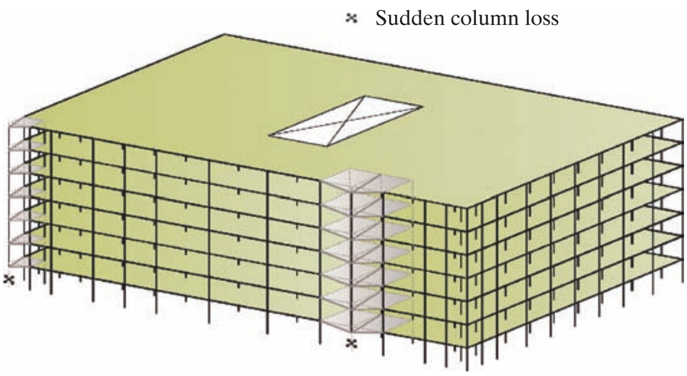


Fig. 3: Layout of the seven-storey steel-framed composite building[11]

respectively, leading to a pseudo-static resistance ($\lambda_n P_0$) at a specific displacement which is equal to the average nonlinear static resistance up to the same displacement. Finally, structural failure is considered to occur when the maximum dynamic response exceeds the ductility limit, which is typically defined by the deformation capacity of connections. Overall, this deterministic framework accounts for the most important factors affecting the resistance of building structures to sudden column loss, including redundancy, ductility and energy absorption capacity.

A key benefit of the simplified deterministic framework is its multi-level characteristic, where assessment may be carried out at different levels of structural idealisation.[5] Depending on structural regularity and the feasibility of model reduction, great computational savings can be achieved by limiting the assessment to relatively low levels of idealisation and assembling the response at the desired level from the individual member response at lower levels,[5] as illustrated in the next section.

Case Study

The illustration of the probabilistic robustness framework is carried out using a typical seven-storey steel-framed composite building designed for office use (*Fig. 3*), as mentioned in Reference [11]:

- the building, located in the UK, is designed in accordance with rules for simple construction,[12]
- the superstructure consists of a 9 × 6 m^2 steel primary structural grid acting compositely with a reinforced concrete slab,
- lateral restraint is provided by a braced core situated at the central atrium of the building in order to improve sway stability and resistance to wind loads, and
- joints are designed as simple noncomposite connections, with detailing that satisfies the UK design guidelines for steel construction.

In order to determine the sensitivity of structural robustness to different structural solutions, two alternative slab reinforcement ratios are studied, both complying with code prescribed tying force requirements: (a) EC4[13] minimum reinforcement ratio of 0,84%, and (b) 2% reinforcement ratio.

As noted in the previous section, only three types of hazard are considered for the application example: gas explosions, fire and human error, with $P(H)$ as given in *Table 1*. From the possible local damage scenarios that can be induced to the structure, only the sudden loss of a single column is taken into consideration, where again, as noted before, corresponding $P(D|H)$ is provided in *Table 1*. For illustrative purposes, it is assumed here that $P(H)$ and $P(D|H)$ are associated with hazards affecting columns on the exterior of the building only.

Failure Assessment

Structural failure assessment for sudden column loss is undertaken using the simplified deterministic framework[5] described in the previous section. In view of the vertical regularity of the structure (*Fig. 3*), assessment is applied at the lowest level of idealisation consisting of a single floor system within the bay affected by column loss. Furthermore, planar regularity is assumed, which means that all corner column loss scenarios and all peripheral column loss scenarios become identical in outcome, respectively. By considering the loss of external columns only, this regularity reduces the number of column loss scenarios to be investigated to two: (a) peripheral column loss and (b) corner column loss, as illustrated in *Fig. 4*. Further information on the floor system structural characteristics and connection details can be found elsewhere.[11]

Structural Models

Following the simplified assessment framework,[5] the pseudo-static response of the individual floor systems (*Fig. 4*) is obtained from a grillage approximation as the assembly of individual beam contributions. Therefore, the nonlinear static response of each of the composite beams is first determined, and then transformed into a pseudo-static response before assembling into the floor response.

In order to represent the steel beam and the concrete "flange", cubic elastoplastic elements[14] are used, which are linked by rigid-plastic elements representing full shear connectors designed

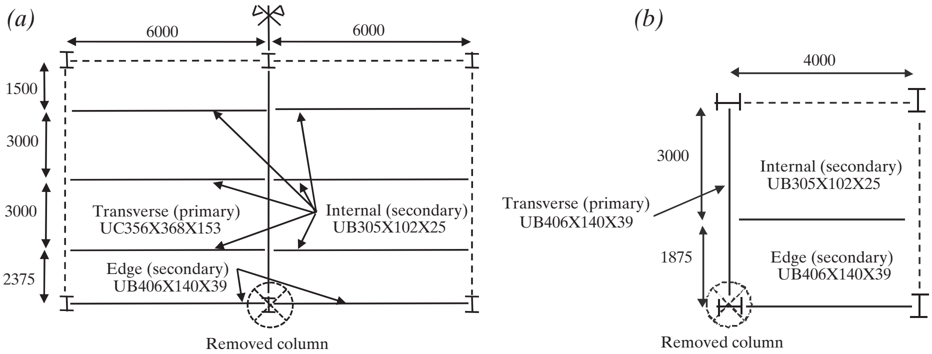


Fig. 4: Representative floors affected by sudden column loss. (a) Peripheral column loss and (b) corner column loss

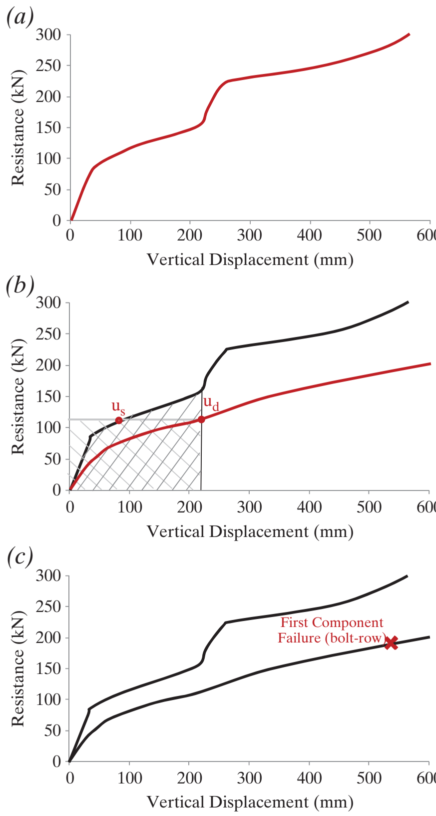


Fig. 5: Stages of simplified assessment for edge beam of peripheral column loss (ρ = 0,84%). (a) Nonlinear static response, (b) pseudo-static (maximum dynamic) response and (c) ductility assessment

according to EC4.[13] The concrete "flange", with an effective width also given by EC4, uses a compressive trilinear material model for C30 concrete and a bilinear elasto-plastic material model for 460B reinforcing steel. As for the steel beam, the bilinear material model is used for S355 structural steel. All material properties are provided elsewhere.[13]

An explicit mechanical joint model based on the EC3[15] component-based approach is utilised in the individual beam models. For this purpose, piecewise linear spring elements are used to represent the various joint components,[11] including (a) reinforcement bars in hogging region, (b) extreme fibre joint components to model the gap between the steel beam and column web, and (c) internal joint components to model bolt-rows and the panel zone component for the major axis connections.

As structural failure is based, in this study, on first component failure, the simulation of the effects of component failure on the ensuing structural response is not required. Accordingly, for simplicity, the nonlinear structural response may be obtained initially assuming components with unlimited ductility, with the influence of component ductility considered afterwards in a post-processing stage.

The three stages of the deterministic assessment approach are illustrated in *Fig. 5* for the edge beam of the floor system affected by the peripheral column loss scenario (*Fig. 4a*), considering a reinforcement ratio of 0,84%. Similar responses are obtained for the internal secondary and transverse beams.

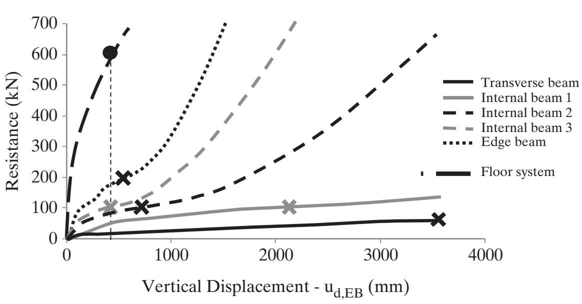
The overall floor system pseudo-static capacity is assembled from individual member contributions, utilising a simplified floor grillage idealisation,[5,11] as exemplified in *Fig. 6*. The work-related α and compatibility $\bar{\beta}$ factors adjust the individual member contributions to account for load distribution and the assumed collapse mode, respectively.[5,11]

Uncertainty in Failure Assessment

Structural failure is defined in terms of the demand exceeding capacity at the adopted level of structural idealisation. For the current study considering sudden column loss, the demand is the gravity loading applied to the floor system in a typical affected bay, whereas the capacity is the pseudo-static resistance accounting for strength and ductility. Accordingly, uncertainty in failure assessment is directly related to the uncertainty in the parameters affecting the applied gravity loading and the floor system pseudo-static capacity.

It is noted that spatial variability is ignored for this illustrative study, without loss of generality, so as to reduce the computational effort of the probabilistic simulation. For example, joint component ductility is considered using a single variable, thus identical variation in ductility is assumed for all components of the affected floor system. Similar assumptions are made with regard to material strengths and connection component strengths, respectively.

Both structural capacity and demand are expressed in terms of equivalent work-conjugate load and resistance values with respect to the chosen displacement parameter in a single degree of freedom (SDOF) idealisation of the deformed configuration.[5] Other simplifications arising from the adopted floor model, such as the use of a grillage approximation which neglects floor slab membrane action,



α_{EB}	α_{IB1}	α_{IB2}	α_{IB3}	α_{TB}	α
0,5	0,5	0,5	0,5	1,0	0,287

$\bar{\beta}_{EB}$	$\bar{\beta}_{IB1}$	$\bar{\beta}_{IB2}$	$\bar{\beta}_{IB3}$	$\bar{\beta}_{TB}$
1,00	0,152	0,456	0,759	1,00

$$P_{floor} = \frac{1}{\alpha}\sum_i \alpha_i \bar{\beta}_i P_i = 593{,}70\ kN$$

Fig. 6: Assembly of pseudo-static capacity for floor system from individual beam contributions for peripheral column loss (ρ = 0,84%)

could be addressed by the adoption of more sophisticated failure assessment models, or through the incorporation of model uncertainty in probabilistic assessment.

Structural Demand

As noted before, structural demand consists of the aggregate effects of gravity loading, considering both dead and live loads. The mean values for coupled floor and façade dead loads are determined from the specific weight of the materials and its mean volume, which correspond for the current example to 4,2 kN/m^2 and 8,3 kN/m, respectively. The coefficient of variation (CoV) for the Gaussian distribution associated with the dead load is taken as 0,10.[16] For the live load, a mean value of 0,70 kN/m^2 is considered, and a CoV of 1,0 is assumed for the corresponding lognormal distribution.[16]

Structural Capacity

The structural pseudo-static capacity is determined by the structural configuration, material response, as well as connection strength and ductility. Neglecting variability in the structural dimensions, and noting that the response of composite floor systems with partial strength connections, as considered here, is largely determined by the connection response, variability in the structural capacity becomes dominated by uncertainty in the strength and ductility of the connection components. Such uncertainty is therefore studied here in some detail, as very little can be found in the research literature, particularly in respect of the ductility of connection components.

In modelling connections, each bolt-row component is represented by a T-stub model, with a geometric configuration as given by EC3.[15] Due to the lack of realistic analytical approaches to determine the T-stub response under large deformations, finite element (FE) numerical simulations were performed accounting for nonlinearity. For model calibration, preliminary numerical tests were also successfully validated against existing experimental data[17] in terms of mode of failure and collapse load/displacement prediction. In order to determine the probabilistic variation of the bolt-row capacity and ductility, a sample of 25 runs of the T-stub model was considered, assuming five different fractiles of the lognormal distribution curve for end-plate and bolt strengths, where the assumptions and results are summarised in *Table 2*. Interestingly, the results show that the T-stub deformation capacity is subject to the greatest uncertainty with a CoV of 0,15.

Unlike the bolt-row component, the component representing the reinforced concrete slab in tension (RFT) can be described by an analytical model. Its yield/ultimate resistance is calculated by considering the reinforcing steel area and the yield/ultimate rebar strength as given by EC4,[13] where the ultimate deformation capacity is obtained from the average strain of the concrete slab over a defined tension bar length.[19] By considering the variability in the rebar and concrete strength properties, the CoVs are derived by statistical evaluation of the resistance and deformation capacity, as summarised in *Table 3*. Again, it is interesting to note that the deformation capacity of the RFT component is subject to the greatest uncertainty with a CoV of 0,26.

Probabilistic Assessment

Considering a specific local damage scenario D, namely the loss of corner or peripheral column (*Figs. 3 and 4*), failure F of the floor system, and therefore the whole structure using the regularity argument, is established from the deterministic assessment framework in terms of the applied load exceeding the pseudo-static capacity. In this respect, F depends on variables defining the dead/live loads as well as the strength and ductility of connection components, as discussed earlier, and hence the evaluation of $P(F|D)$ requires a probabilistic simulation which considers uncertainty in these variables.

Because a large number of outcomes may be required for the application of probabilistic assessment tools, such as first-order reliability methods (FORM) or Monte Carlo simulations, the resulting computational burden can become significant, particularly for systems with a large number of variables. While the adopted deterministic assessment framework is already characterised by computational efficiency, further computational savings can be achieved by the calculation of the so-called response surfaces. This implies that both demand and capacity terms in the failure function are expressed directly by approximate expressions which involve the capacity and demand variables.

For illustrative purposes, four variables are used in this case study: (a) X_1 and X_2 capacity variables for the joint component resistance and ductility, respectively, and (b) X_3 and X_4 demand variables for the dead and live loads, respectively. It is noted that X_1 and X_2 correspond to the resistance and deformation capacities of the T-stub components, because bolt row failure was found to be more critical in this case study than the failure of the RFT components. Moreover, these variables are used for the T-stub components in all parts of the floor system and, by extension, all parts of the structure. Of course, a more sophisticated application should consider variables for T-stub and RFT components and at different locations within the structure,

Material/component	Type	Mean	CoV
Tensile strength of steel (mild steel)[16]	Lognormal	$f_{y,mean} \times 1,4$	0,04
Tensile strength of bolts[18]	Lognormal	1,15 × Nominal	0,03
T-Stub resistance	Lognormal	—	0,05
T-Stub ultimate deformation capacity	Lognormal	—	0,15

Table 2: Probabilistic parameters for bolt-row components

Material/component	Type	Mean	CoV
Area of rebar[16]	Lognormal	—	0,02
Yield strength of rebar[16]	Lognormal	$f_{y,nom} + 2\sigma$*	$\sigma/f_{y,mean}$*
Tensile strength of rebar[16]	Lognormal	$1,2 \times f_{y,mean}$	0,04
Ultimate strain of rebar[19]	Lognormal	—	0,1
Compression strength of concrete[16]	Lognormal	—	0,06
RFT resistance	Lognormal	—	0,05
RFT ultimate deformation capacity	Lognormal	—	0,26

*σ = 30 N/mm^2.

Table 3: Probabilistic parameters for RFT components

thus requiring a larger number of variables for probabilistic simulation.

In constructing the response surface approximation, a second-order polynomial is employed for the structural pseudo-static capacity in terms of X_1 and X_2. A total of nine combinations of (X_1,X_2) are considered, each taking three alternative values $(\mu, \mu \pm \sigma)$, to establish a complete quadratic approximation of the pseudo-static capacity. *Figures 5 and 6* exemplify the determination of the pseudo-static capacity[5] of the edge beam and floor system, respectively, for the peripheral column loss scenario with a reinforcement ratio of 0,84%, considering $(X_1 = \mu - \sigma, X_2 = \mu + \sigma)$. It is worth noting that because ductility assessment is undertaken in a post-processing stage, the number of nonlinear analyses is reduced to 3 as the number of X_1 instances, thus realising further computational benefits. For illustrative purposes, the capacity term of the response surface is obtained for the peripheral column loss scenario with 0,84% reinforcement ratio, and with (x_1,x_2) representing (X_1,X_2) normalised relative to μ, as:

$$
\begin{aligned}
R(x_1,x_2) &\\
&= 21348{,}245 - 41861{,}234x_1\\
&\quad - 3252{,}92x_2 + 20696{,}104x_1^2\\
&\quad + 4466{,}64x_1x_2 - 16822{,}539x_2^2\\
&\quad + 35827{,}504x_1x_2^2 - 685{,}63x_1^2x_2\\
&\quad - 19150{,}230x_1^2x_2^2 \text{ (kN)} \qquad (2)
\end{aligned}
$$

With regard to the structural demand term in the response surface, a first-order polynomial in terms of X_3 and X_4 is sufficient, as the dead and live loads are simply additive in the aggregate gravity loading. Again, taking (x_3,x_4) as (X_3,X_4) normalised relative to μ, the following demand term is obtained:

$$L(x_3,x_4) = 593{,}865x_3 + 82{,}95x_4 \text{ (kN)} \qquad (3)$$

Accordingly, failure is defined in terms of the capacity and demand variables as:

$$
\begin{aligned}
F(x_i) &\equiv (g(x_i)\\
&= R(x_1,x_2) - L(x_3,x_4) \leq 0) \qquad (4)
\end{aligned}
$$

The analytical form of the failure function implies that FORM[20] is well suited for the evaluation of failure probabilities. The basic principle behind this approach is illustrated in *Fig. 7*; the curved failure surface is approximated by a hyperplane (i.e. a straight line for the two-dimensional case) at the critical point in the region which gives the highest contribution to the failure probability. A specific numerical search algorithm, such as the Rackwitz–Fiessler algorithm, is generally required in order to find this point which is typically referred to as the "design point". Subject to normalisation of the original distributions of the different variables, the distance of the design point from the origin β, can be used to approximate the conditional probability of failure $P(F|D)$ as $\Phi(\pm\beta)$, where Φ is the cumulative standard Gaussian distribution.[20]

The application of FORM with the response surfaces determined for each of the two column loss scenarios and the two floor reinforcement ratios leads to the conditional failure probabilities $P(F|D)$ given in *Table 4*. Because of the inherent approximation of the capacity response surface, refinement may be desirable around the design point obtained in the first FORM iteration, especially in the case of very low or very high failure probability in a highly nonlinear system, though such refinement is not considered here.

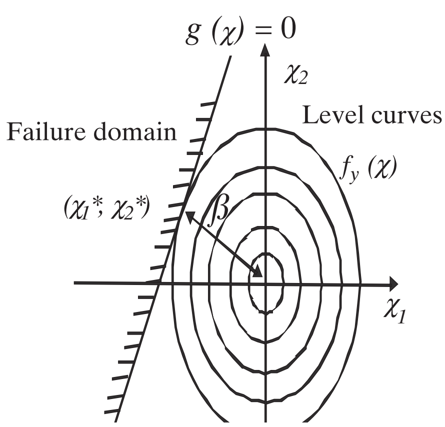


Fig. 7: Schematic illustration of FORM[20]

Following the determination of the conditional failure probabilities $P(F|D)$, these can be combined with the probability of occurrence of these scenarios. By adjusting the event probabilities in *Table 1* for the relative number of corner and peripheral columns, and noting that $P(D|H)$ is the same for all considered events and that $P(\bar{F}|\mathrm{D}) = 1 - P(F|\mathrm{D})$, the overall risk can be determined. The results for the two considered reinforcement ratios are provided in *Table 4*, where risk is expressed in terms of the consequences of failure F and non-failure $\bar{F}$; in practical application, these consequences depend on the use, importance and environment of the building structure, and are therefore left uninstantiated in this study. However, assuming that $C(F)$ is much greater than $C(\bar{F})$, it is clear from this study that the structure with the reinforcement ratio of 2% is associated with a much reduced risk compared with the structure with the minimal 0,84% reinforcement ratio. This highlights the benefits of undertaking risk assessment, not least in respect of comparing alternative design solutions.

Conclusions

Risk assessment is widely recognised as the most rational approach—and is indeed required by design codes—for assessing the robustness of high consequence class structures, yet virtually no guidance is provided on how such an assessment may be undertaken. This paper considers the requirements of probabilistic risk assessment for multi-storey buildings subject to extreme loading and demonstrates its practical application to a steel-composite building structure.

The adopted approach hinges on the availability of the probability of hazard occurrence, which may be obtained from statistical data. Hazards which cannot be associated with a meaningful probability (e.g. malicious planned actions) may be dealt with in a conditional scenario-based manner. With the additional requirement of conditional

Scenario	***P(F|D)***	***P(H)***	***P(D|H)***	**Risk**
EC4 minimum slab reinforcement				
Peripheral column loss	0,868	$(38/42) \times 24 \times 10^{-3}$	0,1	$1{,}88 \times 10^{-3}\,C(F_{\mathrm{Per}}) + 2{,}87 \times 10^{-4}\,C(\bar{F}_{\mathrm{Per}})$
Corner column loss	$5{,}77 \times 10^{-5}$	$(4/42) \times 24 \times 10^{-3}$	0,1	$+\,1{,}32 \times 10^{-8}\,C(F_{\mathrm{Cor}}) + 2{,}29 \times 10^{-4}\,C(\bar{F}_{\mathrm{Cor}})$
2% slab reinforcement				
Peripheral column loss	0,217	$(38/42) \times 24 \times 10^{-3}$	0,1	$4{,}71 \times 10^{-4}\,C(F_{\mathrm{Per}}) + 1{,}70 \times 10^{-3}\,C(\bar{F}_{\mathrm{Per}})$
Corner column loss	$1{,}58 \times 10^{-6}$	$(4/42) \times 24 \times 10^{-3}$	0,1	$+\,3{,}61 \times 10^{-10}\,C(F_{\mathrm{Cor}}) + 2{,}29 \times 10^{-4}\,C(\bar{F}_{\mathrm{Cor}})$

Table 4: Conditional failure probabilities and overall risk

probability of local damage on hazard occurrence, which may also be evaluated statistically or with the aid of modelling tools, the evaluation of risk reduces to the probabilistic assessment of failure given local damage and to consideration of direct/indirect consequences. This paper focuses on the process of evaluating the conditional probability of failure for specific local damage scenarios consisting of single column loss in multi-storey buildings, and its incorporation within the probabilistic risk assessment framework.

As the probabilistic failure assessment can present a computational bottleneck, practical application in design practice requires efficient deterministic assessment models coupled with efficient probabilistic simulation. In a case study dealing with a multi-storey steel-composite building, it is shown that risk assessment may be practically performed using (a) an efficient multi-level deterministic framework for sudden column loss, (b) a response surface approach utilising a relatively small number of sampling points, and (c) the FORM assessment approach.

Further improvement of the risk assessment may also be achieved through an increase in sophistication, though there is clearly a balance to be struck between sophistication and practicality. Such improvements might include (a) better event resolution including intensity/duration which requires continuous variables, (b) event/local damage models, (c) enhanced multi-degree of freedom (MDOF) structural failure models, (d) treatment of model uncertainty, and (e) more sophisticated probabilistic simulation, for example, using a Monte Carlo method. In all of this, the availability of statistical data at all levels, including hazard, component strength/ductility, structural failure, etc., is of paramount importance, and would serve to improve predictability and reduce model uncertainty. In this paper, probabilistic risk assessment is shown to be a practical prospect for structures subject to extreme loading, yet there is clearly significant scope for further research and development.

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