Risk analysis of bridge falsework structures

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Abstract: Bridge falsework systems are traditionally used to support the formwork during the construction of concrete bridges. These structures have a significant impact on the cost, construction rate and safety of the supported structures. In recent years a high number of accidents involving bridge falsework systems have been reported particularly in the developing world. In order to increase the safety and the efficiency of these systems a risk-informed structural design methodology was developed and applied to selected proprietary system using the results of experimental tests of different types of joints, the results of advanced numerical analyses and newly developed structural robustness and structural fragility indices. Illustrative examples are given detailing the steps and calculations needed, including consideration of model and statistical uncertainties, and the results obtained are discussed. Furthermore, based on the findings obtained through the analysis of different strategies for decreasing risks two scenarios are studied in detail: a reference (baseline) scenario and one selected improved (alternative) scenario. For the cases analysed, it is concluded that if the cost of the permanent structure considerably exceeds the cost of the temporary structure, which is often the case, the extent of improvements in terms of structural and economical risks may justify the extra costs incurred by implementing simple Quality Management procedures.

Keywords: Temporary works, Bridge falsework, Risk, Robustness, Fragility

1 Introduction

The present paper concerns bridge falsework systems, steel temporary structures traditionally used to support the formwork during the construction of concrete bridges in particular proprietary modular 3-D frame systems of metallic elements connected by special couplers, such as the Cuplok® system. Figure 1(left) illustrates an example of a bridge falsework Cuplok® system and Figure 1(right) provides the identification of the various types of elements and joints that are part of the system.

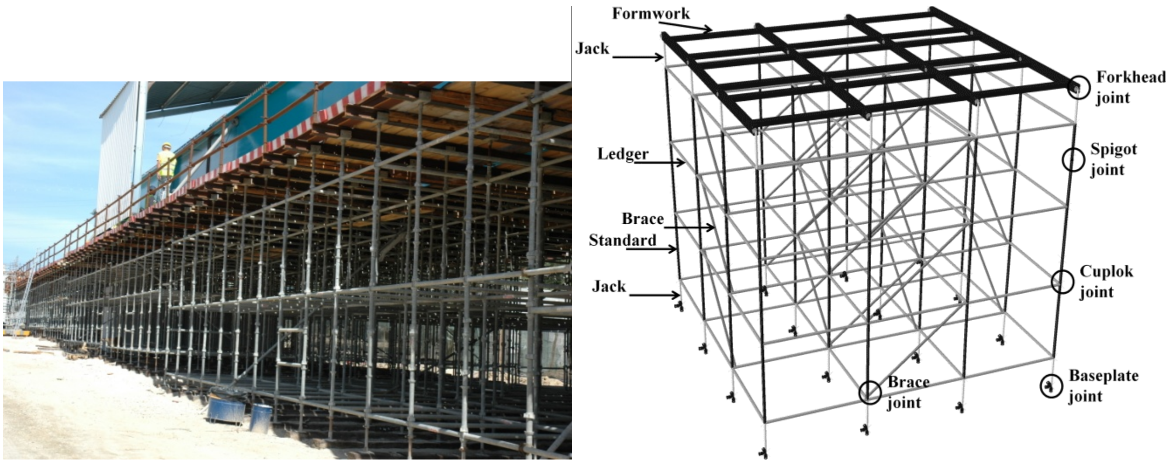

Fig. 1: Example of a bridge falsework system (left) and identification of type of elements and joints (right)

Traditionally, bridge falsework structures are usually designed using safe load tables developed by the producers of the proprietary. Often these tables do not provide information regarding (i) quality requirements (e.g. the specification of design tolerances), or (ii) risk assessment for specific applications (e.g. special loading conditions). Other factors that have a decisive influence on the behaviour, resistance and performance of falsework are also not usually directly accounted for in the design. They are expected to be covered by the safety margins adopted by the falsework system producers, but these may be insufficient to withstand the global coupled effect of the various hazards, see André et al. (2012).

In addition, the design rules applied to bridge falsework structures are not uniform due to insufficient research and code developments. The design of temporary structures based on existing codes calibrated for common building structures also raises important safety issues as clearly shown by Sexsmith (1998).

The insufficiencies highlighted above stem from a biased view from the principal stakeholders involved with falsework structures: they are “temporary” and, therefore, their role is considered to be minor compared to that of the permanent structures. This also translates to the way temporary works is used and managed on site.

The framework outlined above contributes strongly to the high number of incidents and accidents involving the use of bridge falsework systems, which frequently cause human casualties and severe injuries, work inefficiency and partial (or total) structural damage of the infrastructure, see André et al. (2012).

The severe consequences of all the accidents involving bridge falsework clearly justify the research needs for a developing a risk management framework for bridge falsework systems, see André, Beale, and Baptista (2013) for a thorough discussion.

In this paper, a risk informed structural design methodology based on new structural robustness and fragility indices is presented. An extended discussion of the topic is presented in André et al (2015a). This paper starts with an overview of the new indices and of the risk informed structural design methodology. Afterwards, an example of a structural solution of a bridge falsework using the Cuplok® system is used to illustrate the application of the methodology, and the results presented and discussed. The information given in this paper can be used to develop more rational and reliable bridge falsework structures thus safer and more efficiently designed.

2 Risk Informed Structural Design Methodology

2.1 Structural Robustness

Structural robustness is here defined as a measure of the predisposition of a structural system to loss of global equilibrium and global stability, as a result of a failure scenario, e.g. a failure of one or more elements of the structure, for a given load case. It is applicable to all design situations and not only those unforeseen, accidental, or concerning local failures (difficult to define and select). It is a very important structural property but existing design codes still do not specify a theoretically correct, consistent and practical framework for the assessment of structural robustness (André et al. 2015a).

In this paper, a robustness index, $\mathrm{I_R}$, is presented and given by (André et al. 2015a):

$$\mathrm{I_R}\left(\mathrm{A_L} \mid H\right) = \frac{D_{uc} - D_{\text{1st failure}}}{D_c - D_{\text{1st failure}}} \text{ with } \begin{cases} 0 \leq \mathrm{I_R} \leq 1 \\ D_c - D_{\text{1st failure}} = 0 \Rightarrow \mathrm{I_R} = 1 \end{cases} \tag{1}$$

where:

$\mathrm{A_L}$ represents the leading action;

$H = \{h_1, h_2, ..., h, ..., h_n\}$ is a set of hazard scenarios: {base conditions} + {impact on column 1, impact on column 2, ..., impact on column n}, a set of different actions or a combination of different actions, for example;

$D_{\text{1st failure}}$ represents the damage energy of the structure when the "first failure" state takes place for the hazard scenario considered;

D_{uc} represents the damage energy corresponding to the state where collapse is unavoidable for the hazard scenario considered;

D_c represents the damage energy corresponding to the collapse state for the hazard scenario considered.

A value of the structural robustness index equal to one means that the structure is completely optimised in terms of structural robustness, for the hazard scenario considered. In the contrary, a value of the structural robustness index equal to zero may indicate that the structure completely lacks optimisation in terms of structural robustness, for the hazard scenario considered.

2.2 Structural Fragility

The structural fragility of a system is an expression of the system's structural performance, typically in terms of damage extension, for a given hazard event.

Structural robustness is a measure of the predisposition of a structural system to progressive and disproportionate collapse. Therefore, it is not the best parameter to evaluate when the objective is to assess the system's resistance against the applied actions. A structural fragility index, $\mathrm{F_R}$, which is capable of addressing adequately these objectives, is given by (André et al. 2015a):

$$\mathrm{F_R}\left(\mathrm{A_R}, \mathrm{A_L} \mid H\right) = \frac{D_p - D_{\text{1st failure}}}{D_c - D_{\text{1st failure}}} \text{ with } \begin{cases} 0 \le \mathrm{F_R} \le 1 \\ D_c - D_{\text{1st failure}} = 0 \Rightarrow \mathrm{F_R} = 1 \\ A_\mathrm{L} \ge A_{\mathrm{L},uc} \Rightarrow \mathrm{F_R} = 1 \end{cases} \tag{2}$$

where:

$\mathrm{A_R}$ represents the reference action;

$\mathrm{A_L}$ represents the leading action, which can be different from the reference action. $A_{\mathrm{L},uc}$ represents the value associated with D_{uc};

$H = \{h_1, h_2, ..., h, ..., h_\mathrm{n}\}$ is a set of hazard scenarios: {base conditions} + {impact on column 1, impact on column 2, ..., impact on column n}, a set of different actions or a combination of different actions, for example;

D_p represents the value of the damage energy of the structure when the new static equilibrium state is reached for value p of the reference action within the considered hazard scenario. See also André (2014) and André et al. (2015a);

$D_{\text{1st failure}}$ represents the damage energy of the structure when the “first failure” state takes place for the hazard scenario considered;

D_{uc} represents the damage energy corresponding to the state where collapse is unavoidable for the hazard scenario considered;

D_c represents the damage energy corresponding to the collapse state for the hazard scenario considered.

A value of the structural fragility index equal to zero means that the structure is not damaged, for the values of the actions applied and hazard scenario considered. In the contrary, a value of the structural fragility index equal to one indicates that the structure is completely damaged (i.e. collapsed or in a state where the collapse is unavoidable).

2.3 Vulnerability and Risk Measures

Vulnerability expressed in terms of costs of consequences is related with structural fragility by a cost function. The latter includes, but is not limited to, the costs of consequences due to structural damages. An example of a possible cost function for bridge falsework is presented in André (2014).

In the suggested framework, if actions and resistance variables are simulated by their real probability distributions functions, structural fragility becomes an expression of the structural damage extension (D) of the system under the considered hazard scenario and vulnerability becomes a measure of risk that can be used in a Cost-Benefit analysis (CBA). With this approach it is possible to analyse how risk changes with structural robustness or with other risk control measures thus contributing to a better decision-making process.

Using the robustness and fragility indices, it is possible to perform progressive and disproportionate collapse analysis and also evaluate the sensitivity of damage accumulation to action values, which may be important when performing risk analysis.

In general, traditional structural risk analyses focus on probability of failure. These analyses are quite limited since they do not account for the various damage states that might occur (damage is a continuous function) but that do not directly imply the global collapse of the structure. Therefore, valuable information is lost that could be used during the risk informed decision-making process potentially leading to inefficient solutions. For instance, two structural systems A and B can have the same probability of failure but the damage evolution in A can be quite different than in B.

The newly developed structural robustness index can be used as a design option to reduce the structural risk and the newly developed structural fragility index is an analysis tool that should be used to assess the structural risk.

3 Illustrative Examples

In this section, illustrative examples of bridge falsework Cuplok® systems will be used to demonstrate the application of the newly proposed structural design methodology and the results discussed. The falsework system A2 (hereon labelled as Model A2) tested in the University of Sydney, see (Chandrangsu and Rasmussen 2011) for the complete details, will be considered in the illustrative examples. Figure 2 illustrates the numerical representation of Model A2.

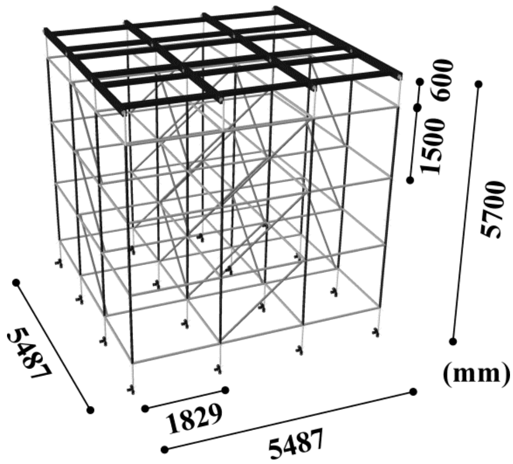

Fig. 2: Overview of Model A2 numerical model

3.1 Choice of Stochastic Variables

Early in any risk analysis it is critical to carry out a stochastic sensitivity analysis in order to understand the influence of the variability of the input values in the variability of the output results and to determine which input random variables are the most important to explain the stochastic structural behaviour.

There are many procedures available to perform stochastic analyses (Benjamin and Cornell 1970; Melchers 1999; André 2014). Here, Design of Experiments (DoE) followed by surrogate modelling and Monte Carlo analyses are used. The procedure will be explained in detail in the following section.

In the DoE, several random variables were considered, representing mechanical properties of the various types of joints present in Cuplok® systems but also the geometrical and material properties of the system. In total, 34 random variables corresponding to structural properties were selected, see Table 1 for details of a selection of input variables (the complete set is available from (André 2014)).

Table 1: Random variables considered in the DoE analysis

Parameters	Random variables	Minimum value	Maximum value
Initial geometrical imperfections	Local bow imperfection factor (l_{impf})	50	2000
	Global sway imperfection factor (g_{impf})	50	2000
Material properties	Yield stress (f_y), MPa	300	500
	Tensile resistance (f_u), MPa	450	650
	Maximum strain (ε_u)	0.1	0.3
Cuplok joint, strong bending axis	Looseness (θ_{1c}), rad	0	0.04
	Initial stiffness (k_{1c}), kN.m/rad	1	20
	Stiffness after looseness, 2 ledgers (k_{22Lc}), kN.m/rad	30	100
	Maximum bending moment (M_{uc}), kN.m	1.5	5
	Deformation capacity factor (d_{fc})*	0.5	3
Spigot joint	Looseness (θ_{1s}), rad	0	0.04
	Initial stiffness (k_{1s}), kN.m/rad	1	20
	Stiffness after looseness, ratio 1 (k_{21s}), kN.m/rad	150	300
	N/*M* Ratio 12 (r_{12s})**	30	70
Forkhead joint	Looseness (θ_{1f}), rad	0	0.04
	Initial stiffness (k_{1f}), kN.m/rad	1	20
	Stiffness after looseness (k_{2f}), kN.m/rad	20	100
	Maximum bending moment (M_{uf}), kN.m	1	4

*Deformation capacity factor (d_f) represents the ratio between the maximum joint deformation and the joint deformation at maximum force (André et al. 2015b).

***N*/*M* ratio is the axial force to bending moment ratio at the spigot joint and is used to define different values for the spigot joint constitutive model (André et al. 2015b).

The DoE assumed uniform distributions for each random variable. Table 1 also presents the minimum and maximum values considered for each random variable.

More than 700 numerical analyses were performed using Latin Hypercube (LHS) sampling of each variable. The only action considered was a uniform pressure applied on top of the formwork plate as to simulate the concrete casting action.

Afterwards, a predictive model was determined for the resistance, robustness and fragility from the DoE results and a sensitivity analysis was performed to identify the most relevant random variables which were then selected for the case study presented in a following section. Subsequently, for each predictive model, a Monte Carlo analysis was carried out with at least one million LHS samples taken from the assumed probabilistic distributions and respective parameters of each random variable, see André (2014). Existing correlations between variables were accounted for and the uncertainty in estimating the distribution parameters was also accounted for by assigning a truncated Normal probabilistic distribution for each distribution parameter, see André (2014).

All the values used were based on results presented in André et al. (2013b) and on relevant bibliographic references (Voelkel 1990; JCSS 2001; Simões da Silva et al. 2009; Chandrangsu 2010).

From the results obtained for the target output variables, the most important random variables could be identified from the total 34. These involved the initial rotation (looseness, θ_1) of the joints, the stiffness after looseness (slope of the second linear segment of the joints analytical constitutive model, k_2), the deformation capacity factor (d_f) and the initial geometrical imperfections. The joints that influence the most the variability of the falsework behaviour are the cuplok and the spigot joints, which also have been found to be the most decisive joints for the value of the falsework's resistance (André et al. 2015b).

In the end, 20 random variables were selected from the total 34, see André (2014).

3.2 Case Study Analyses

3.2.1 Introduction

Several case studies were selected and studied in detail. The results presented in this section concern only one alternative solution, labelled CS2 case study. The structural layout of the CS2 case study is depicted in Figure 3. In this layout, the top and bottom jacks are braced by a continuous brace element placed in every bay, alternating its direction in consecutive bays, along two orthogonal directions.

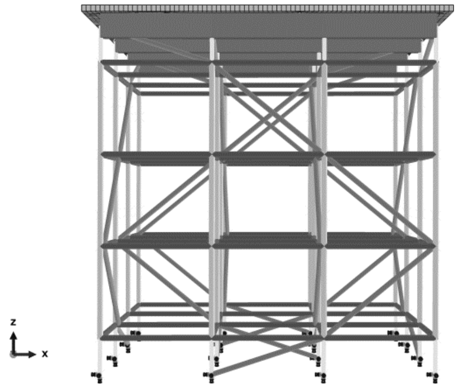

Fig. 3: Case studies structural layout

In the CS2 case study additionally to the vertical pressure applied on top of the formwork, the wind pressure corresponding to the working wind velocity and a localised differential ground settlement were also considered. The working wind velocity was considered equal to a wind pressure equal to 200 N/m^2 (BSI 2010). The differential ground settlement was applied under a central column with a value equal to 100 mm.

The pressure applied to the formwork was selected as the leading action and was increased until structural collapse occurred.

3.2.2 Predictive Models and Uncertainties

As mentioned in the previous section, 20 resistance related variables were modelled as random variables. The fragility and consequently the structural risk were analysed by means of predictive models. The procedure to validate, verify and select the predictive models is provided in André (2014). In all cases the boosted trees family provided the best predictive models, either by the Stochastic Gradient Boosting (SGB) or by the Cubist model.

Using a surrogate model to foresee the actual behaviour under unknown and uncertain conditions introduces a component to the model uncertainty, besides the uncertainty of the numerical results. Nevertheless, this uncertainty is relatively small for the SGB models and the case study considered, achieving a coefficient of determination (R^2) always higher than 95% for the test data set and relative differences not exceeding 20% at maximum, see Figure 4 for example. The abscissa represents the ratio between the observed data and the predicted data.

Regarding the numerical uncertainty, from the results presented in André et al. (2015b) it is possible to estimate it and the results are illustrated in Figure 5.

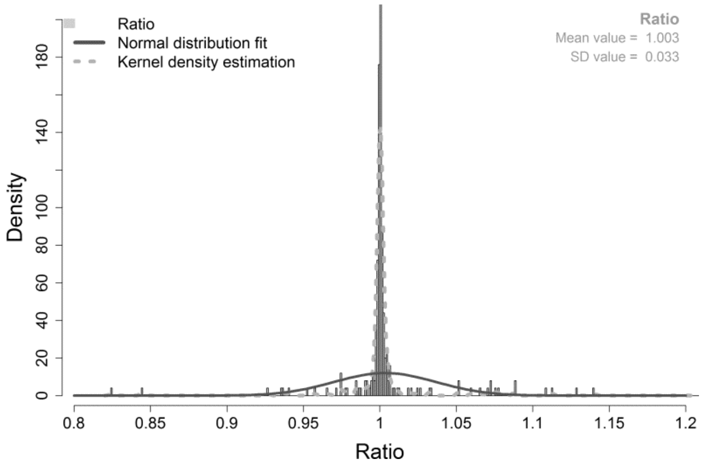

Fig. 4: Accuracy of SGB models for maximum resistance

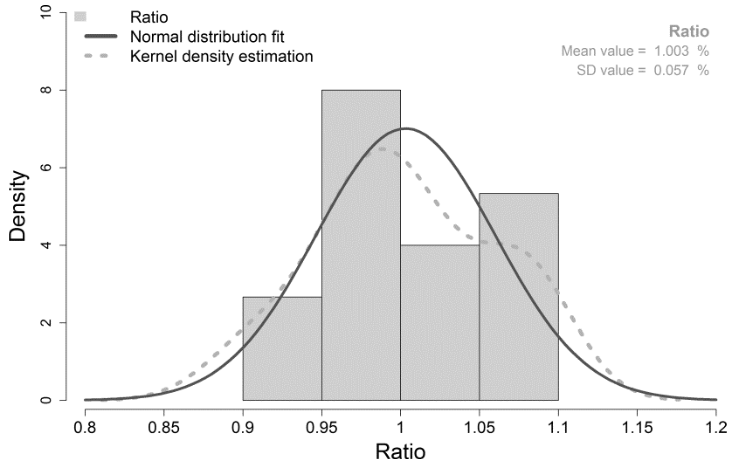

Fig. 5: Accuracy of numerical models used

3.2.3 Stochastic Analyses

Twenty variables associated with the system's resistance, while the rest of the variables were considered deterministic with values equal to the mean values of the parent probabilistic distribution, with the exception of the stiffness (k_1) associated with joint looseness which was considered equal to 1 kNm.rad.

Regarding actions, only the value of the pressure load applied on top of the formwork surface was considered random. For the reliability analyses, the performance of the system was compared against a vertical pressure action modelled by a Normal distribution with mean value equal to 24.0 kN/m^2 and a COV equal to 0.075. On the contrary, the action distribution was considered uniform with a minimum value equal to 20 kN/m^2 and a maximum value equal to 26 kN/m^2 for the structural robustness and structural fragility analyses.

The results for case study CS2 are presented in Figures 6-8. Observing the results in terms of reliability, robustness index and fragility index, it is possible to conclude that the value of the probability of failure is unacceptably high and the results for fragility demonstrate that it is very likely that significant damages will occur. The mean value of robustness index is not extremely low but it may not be enough to avoid collapse without warning. The effect of propagating uncertainty is also shown and it is seen that the variability value is considerable (between 10%-15% of variation of fragility values), see Figures 6 and 8.

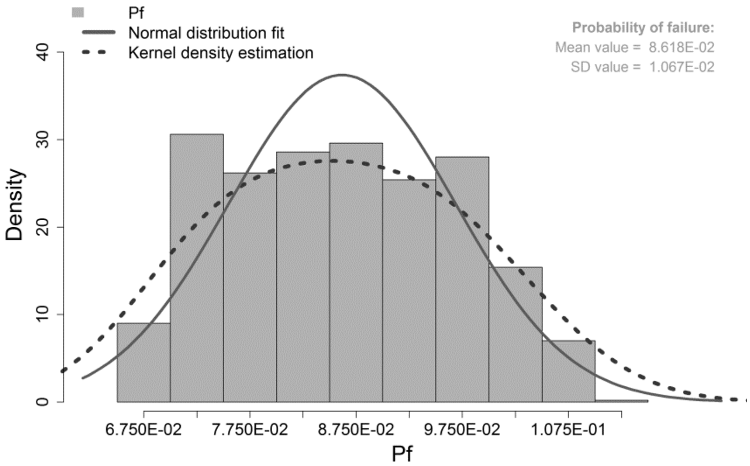

Fig. 6: Histogram of the probability of failure, P_f, CS2 model

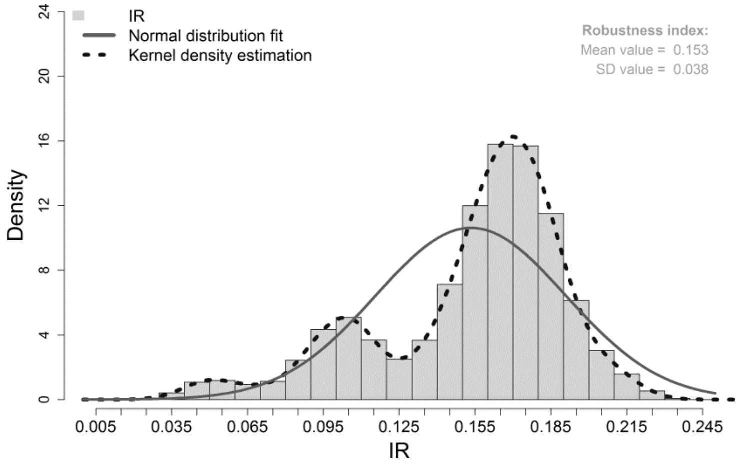

Fig. 7: Histogram of the robustness index, I_R, CS2 model

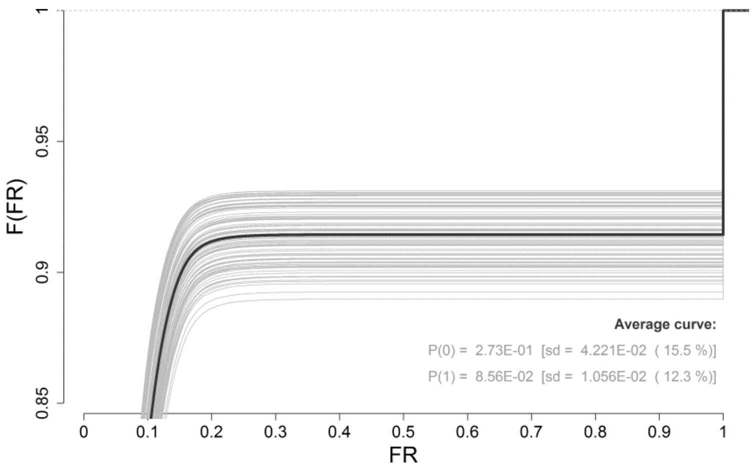

Fig. 8: Empirical cdf of the fragility index, F_R, with the dispersion due to uncertainty propagation and highlighting the average curve, CS2 model

3.2.4 Additional Stochastic Analyses

Based on the results presented in the previous section it is necessary to discuss alternative strategies to increase the structural robustness and decrease the structural fragility of the bridge falsework Cuplok® system under analysis.

As detailed in André (2014), there are several possible strategies to increase the robustness of a structure. Nevertheless, some of these will be more or less efficient depending on the type of structure. For bridge falsework systems the following strategies seem more appropriate: (i) increase resistance, (ii) increase structural integrity and (iii) increase ductility.

Applying the above concepts and guidance to the case study at hand, the values of the following random variables were modified: (i) decreasing initial geometrical imperfections, (ii) decreasing looseness rotation (θ_l), (iii) increasing the k_2 stiffness of the cuplok joints and (iv) increasing the deformation capacity of the joints (d_f). The changes are given in Table 6 and form an alternative (improved) scenario to the reference case study (CS2) discussed previously. In practice, these changes reflect simple controls related to better quality checks, inspection and maintenance plans (*i.e.* quality management).

Table 2: Improved random variables values (changes highlighted in bold)

Parameters	Random variables	Minimum value	Maximum value
Initial geometrical imperfections	Local bow imperfection factor (l_{impf})	**1000**	2000
	Global sway imperfection factor (g_{impf})	**1000**	2000
Cuplok joint, strong bending axis	Looseness (θ_{1c}), rad	0	**0.01**
	Stiffness after looseness, 2 ledgers (k_{22Lc}), kN.m/rad	**60**	90
	Stiffness after looseness, 3 ledgers (k_{23Lc}), kN.m/rad	**60**	120
	Stiffness after looseness, 4 ledgers (k_{24Lc}), kN.m/rad	**60**	140
	Deformation capacity factor (d_{fc})	**1**	2

Spigot joint	Looseness (θ_{1s}), rad	0	**0.01**
	Deformation capacity factor (d_{fs})	**1**	2
Forkhead joint	Looseness (θ_{1f}), rad	0	**0.01**
	Deformation capacity factor (d_{ff})	**1**	2
Brace joint	Deformation capacity factor (d_{fb})	**1**	2

The results of these changes are given in the Figures 9-11 for case study CS2 improved model, CS2a. Results are expressive. The mean value of resistance and structural robustness increased also over 10% and the variability of resistance, structural robustness and structural fragility also decreased considerably. In fact, the mean value of the failure probability decreased four orders of magnitude (from 80000×10^{-6} to 7×10^{-6}) when compared with CS2 model results.

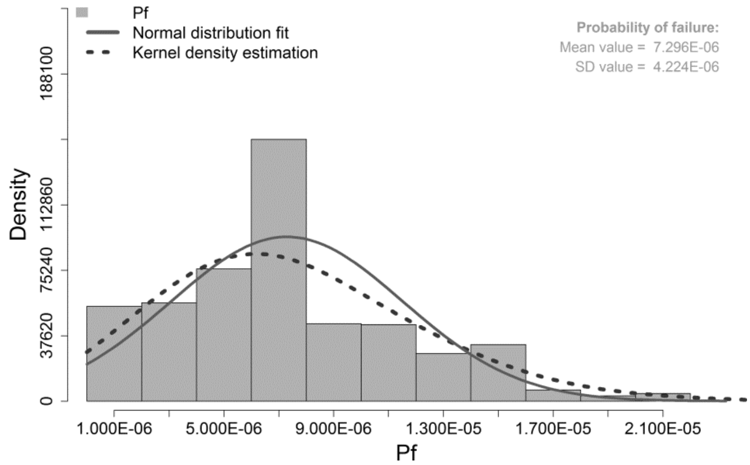

Fig. 9: Histogram of the probability of failure, P_f, CS2a model

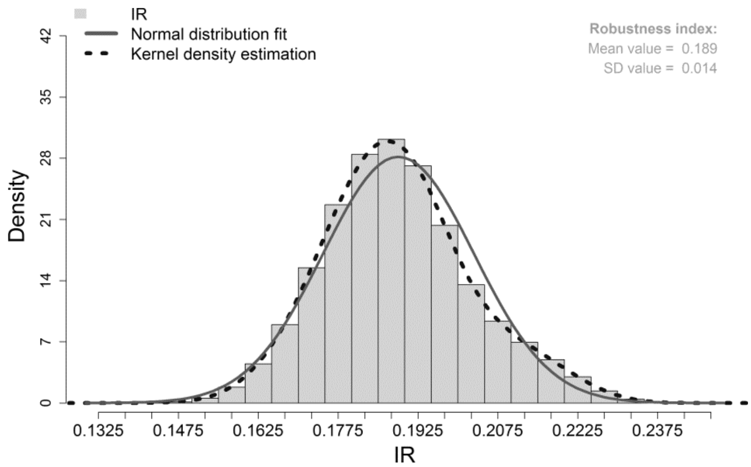

Fig. 10: Histogram of the robustness index, I_R, CS2a model

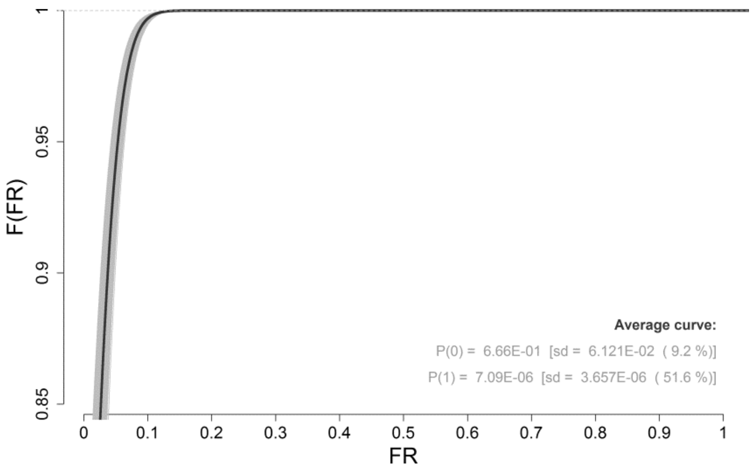

Fig. 11: Empirical cdf of the fragility index, F_R, with the dispersion due to uncertainty propagation and highlighting the average curve, CS2a model

3.3 Risk Evaluation

Assuming an acceptable probability of failure equal to 1×10^{-5}, and comparing it with the results presented in previous sections it is possible to conclude that in the original (reference) scenario the risk is exceedingly high and cannot be accepted or tolerated. Therefore, corrective measures need to be implemented to lower the risk level to acceptable, or tolerable, levels. By applying better quality management, it can be seen that it was possible to decrease considerably the failure probability to a level within the range of acceptability.

3.4 Risk Control and Risk Informed Decision-Making

In this section an economical justification for adopting the improved (alternative) scenario, instead of the reference (baseline) scenario, will be analysed. In order to perform this analysis, a cost function must be derived, such as the one suggested in André (2014).

It was assumed that the sum of the cost of the structure supported by the bridge falsework with the cost of the bridge falsework, C_{max}, was equal to £200,000.00. Additionally, the function between fragility index and damage costs was considered to be linear, see Figure 12.

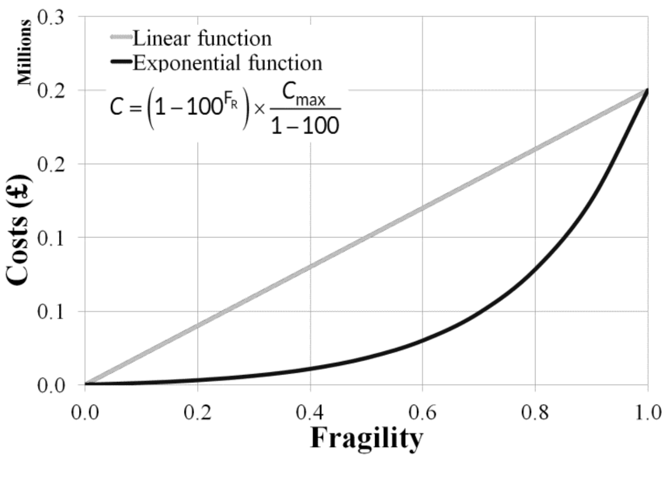

Fig. 12: Functions between Costs and Fragility

It was assumed that implementing the improved quality management represents a fraction, *e.g.* 20%, of the total cost of a new bridge falsework system, per use. Fixing this latter value at £20,000.00 (based on material's cost and labour cost), the extra costs associated with the alternative scenario are estimated to be equal to £4,000.00 (2014 prices), per use.

The benefits are calculated using the Value of Preventing a Fatality (VPF) concept, which is fixed annually in the UK by the Department for Transport (DfT). The 2014 number is equal to £1,700,000.00. Benefits are calculated by the improvements relative to the worst case scenario: the collapse of the structure, *i.e.* when structural fragility equals one. As a simplification it was considered that benefits decrease linearly with the fragility index, see Figure 13. The maximum benefits value ($B_{\max}$) was considered equal to 50% of the VPF. This value was estimated taking into account the possible differences between the probabilities of various injury levels when fragility is equal to zero (due to falls from height or being struck by an object during assembly of the falsework for example) and equal to one (due to structural collapse of the falsework).

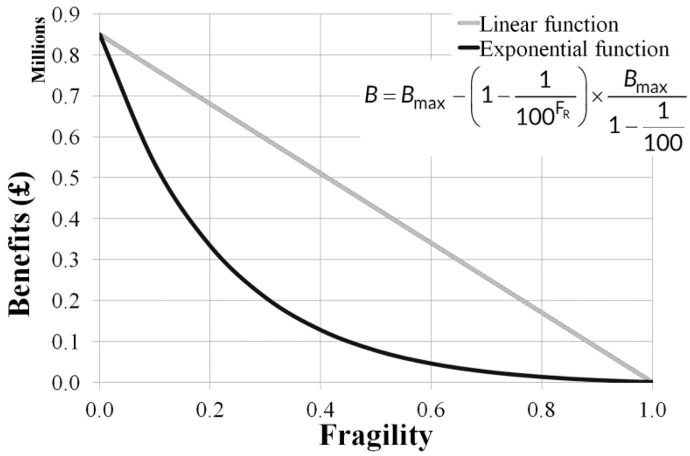

Fig. 13: Functions between Benefits and Fragility

Considering CS2 and CS2a models as two independent bridge falsework structures subject to uncertain actions, a single use per year of each structure and that only one person is at risk per use, the cdf of the relative Net Value (equal to the Net Value of the CS2a model minus the Net Value of the CS2 model) between choosing the improved scenario (CS2a model) and the reference scenario (CS2 model) is presented in Figure 14 (light grey curve). In this analysis, structural fragility was calculated considering a vertical pressure action modelled by a Normal distribution with mean value equal to 24.0 kN/m^2 and a COV equal to 0.075.

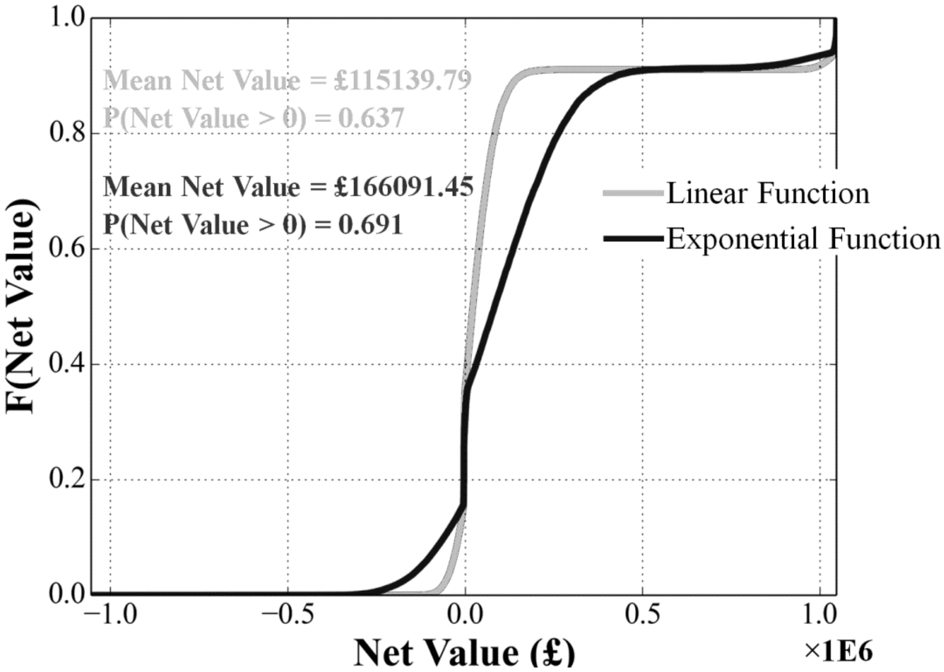

Fig. 14: Cdf of relative Net Value (for CS2 type models), linear and exponential functions

It can be observed that there is approximately 64% probability that a positive relative Net Value is obtained, with a mean relative Net Value of more than £100,000.00. It can be concluded that the choice of selecting the improved scenario, CS2a model, over the reference scenario, CS2 model, is justified since the additional costs incurred by adopting better quality management are outweighed by the dramatic reduction in individual and structural risks.

It is of interest to study how the relative Net Value varies for instance with the function between costs and structural fragility, and between benefits and structural fragility. Choosing an exponential law instead of a linear law leads to the dark curve shown also in Figure 14. It can be observed that with this modification the cdf of the relative Net Value is considerably shifted with the mean relative Net Value increasing significantly and the probability that a positive relative Net Value is obtained also increasing. This occurs because of the different configuration of the fragility curves of the reference and improved scenarios.

It can be concluded that if the cost of the permanent structure significantly exceeds (about one order higher) the cost of the temporary structure, the extent of improvements in terms of structural and economical risks completely justified the small extra costs incurred by adopting better quality management in the modified scenario.

4 Conclusions

This paper focused on risk analysis of bridge falsework structures, in particular those where the Cuplok® system is used. The paper started by giving an overview of new structural robustness and structural fragility indices, which formed the basis of the risk analysis framework. This new methodology is applicable, in principle, to all structures.

After, the data collected from the experimental campaign of tests of typical joints seen in Cuplok® bridge falsework structures was used together with other appropriate data to build probabilistic models for the most important stochastic variables. A total of 20 random variables were selected based on the results of a sensitivity analysis of the stochastic response of a chosen reference bridge falsework system.

Finally, strategies to enhance structural robustness and minimise structural risk were discussed and one possible solution was used as an application example. The approach used was to modify the most sensitive input random variables. In practice, this translated on more rigorous inspection plans and tighter quality controls (quality management activities).

Risks were estimated for the reference (baseline) scenario and the improved (alternative) scenario and later evaluated against proper risk criteria. It could be concluded that if the cost of the permanent structure significantly exceeds the cost of the temporary structure the extent of improvements in terms of structural and economical risks completely justified the small extra costs incurred by adopting better quality management in the improved scenario.

The information presented in this paper can be used to reduce the risk associated with the plan, design, assembly and operation of bridge falsework systems.

Acknowledgments

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