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Interaction between Planning, Execution and Evaluation of Tests

Peter TANNER
Research Engineer
Inst. of Const. Science (IET-CSIC)
Madrid, Spain

A graduate of ETH Zurich, in 1989 Peter Tanner joined ICOM (Steel Structures) at EPFL where he worked in the fields of fatigue and fracture mechanics. Since 1992 he has worked with consultants in Madrid. Presently, at IET-CSIC, he is performing research into durability of concrete marine structures and reliability of existing structures

Summary

The evaluation of a riveted wrought iron bridge from the last century is described. The case study concentrates on the planning of the test and inspection programme for the collection of site data and the introduction of this data in the structural calculations. The site data is used to calibrate updated deterministic models of action effects and resistance, applying reliability methods to a simple structural model. Based on the updated action effects and resistance, the required strengthening of structural members can be established through a deterministic analysis of a refined structural model.

1. Introduction

When assessing the structural safety of an existing structure, the information is different from that available during design because many characteristics may be measured from the structure under consideration which, at the time of its design, were just anticipated quantities. The level of accuracy for the load and resistance models, which are needed for the assessment, can be increased for example by visual inspection, material testing or field testing. It is always possible to improve these models by collecting more data about the assessed structure. However, the updating of information by collecting site data may result expensive, time consuming or even ineffective if the choice of the test programme is not made to suit the characteristics of the structure under investigation and if the updated information can not easily be introduced in the calculation models used for the assessment. Tests should therefore carefully be planned, executed and evaluated.

This paper deals with the evaluation of the structural safety of a 100 year old wrought iron truss-girder bridge. The relationship between planning, execution and evaluation of tests is emphasized.

2. Description of the bridge

The bridge investigated crosses the Duero river in Zamora, Spain, and was built around 1895. It is a continuous riveted wrought iron truss-girder bridge over five spans (43.2, 54, 54, 54, 43.2 m) with a total length of 248.4 m. The two main girders beams consist of parallel horizontal U-section members and crossed diagonals (Figure 1).The platform is composed of a wrought iron framework which supports the deck, consisting of a wrought iron sheeting, a sand fill and an asphalt layer. At present, the main girder bottom U-section members are affected by severe corrosion due to poor detailing and reduced maintenance in the past. For this reason, a bridge evaluation is to be initiated.

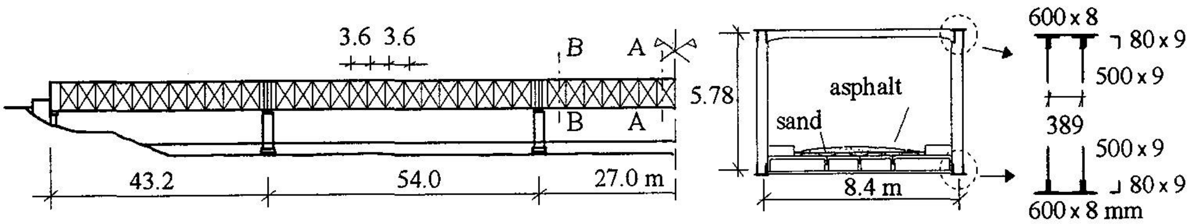

Fig.1 View and cross-section of the investigated truss bridge

3. Evaluation procedure

The assessment of the structural safety is carried out applying a staged procedure. Figure 2 shows the concept of the staged evaluation procedure and its relation to the collection of site data by inspection, material- and field testing.

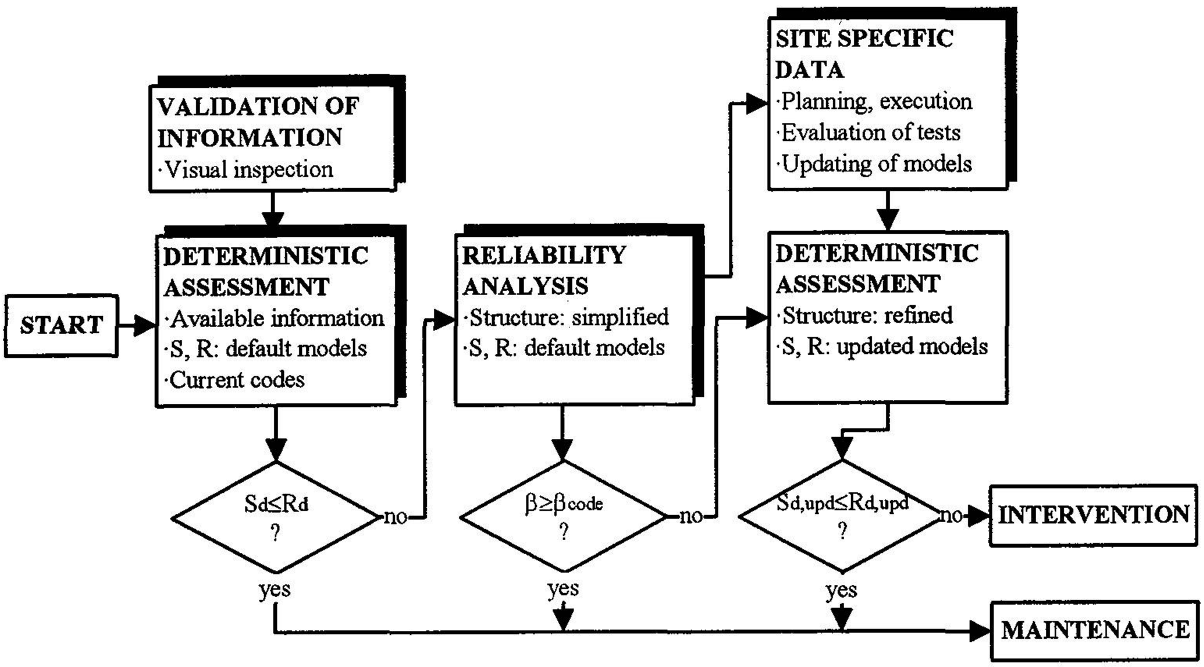

Fig.2 Staged evaluation procedure and its relation to the collection of site data

In a first step, a preliminary deterministic assessment is carried out, using the verification criteria defined in the current Spanish bridge design code [1]. For this, the calculation models are based on the available information about the structure, validated by visual inspection. No further evaluation is necessary for the members for which structural safety is verified in this first step.

For the most critical area, identified in the first step, a simplified structural model can be established that permits a reliability analysis using default probabilistic models of action effects and resistance. If the structural safety of this area is not verified, further evaluation is possible based on improved load and resistance models. The improvement of these models is possible through the collection of site data. The aforementioned reliability analysis aids the planning of site data collection: From the results it can be deduced which parameters can be most effectively updated.

The site data can be used to calibrate updated deterministic models of action effects and resistance. For the calibration, reliability methods are applied to the simplified structural model mentioned above. The updated deterministic models of action effects and resistance are then used for a detailed deterministic assessment using a more refined structural model.

For the structural members for which safety is not verified by deterministic assessment with updated models, a reliability analysis could be used for a more accurate assessment of structural safety. However, due to the large number of different structural elements, nodes and riveted connections, a full reliability analysis is not considered viable for the investigated bridge. An intervention must be planned for the members for which safety is not verified by any of the aforementioned assessment methods.

4. Collection of site specific data

4.1 Critical areas

4.1.1 Validation of information

The available information about the structure is validated by a first visual inspection before carrying out the preliminary deterministic assessment. The most important findings can be summarised as follows:

- Important eccentricities exist at main girder nodes, not visible from the geometry of the original plans (Figure 3).
- Advanced global corrosion of the truss girder bottom U-section member can be observed, facilitated by its channel like geometry. A large number of holes with dimensions of the order of 20·40 cm exist (Figure 3).
- Buckling is found of the slender "web plates" (with a height to thickness ratio of 55 and a free edge as can be seen from Figure 1) of top and bottom U-section compression members.
- Fatigue cracks in truss top lateral sway frames are observed, spreading out from rivet holes (this finding is important with a view to the evaluation of fatigue safety and the planning of maintenance and inspection strategies [2, 3]; however, fatigue and brittle fracture are not further considered in the present paper).
- The foundations are in a very good state.

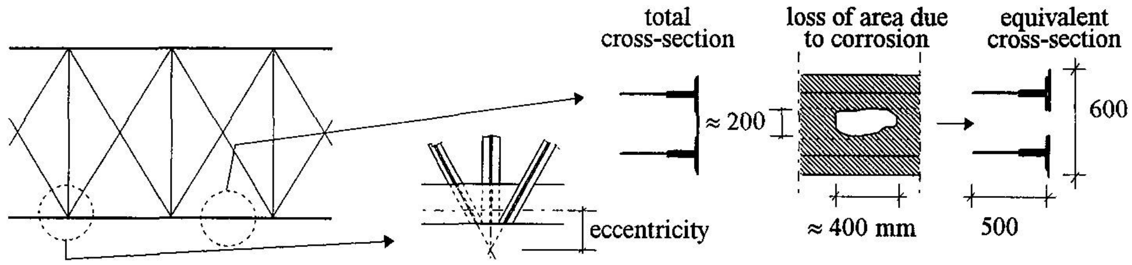

Fig.3 Validation of information

4.1.2 Preliminary deterministic assessment

Structural safety is evaluated by applying the verification criteria defined in the relevant design standards. The action effect, S, is calculated by using actions and load factors according to [4] and by introducing in the structural model the aforementioned eccentricities at main girder nodes. The corrosion of the truss girder bottom U-section members is taken into account by adopting an equivalent cross-section according to Figure 3 for the calculation of the resistance, R. Information about the material properties of wrought iron is available from literature [2], and resistance factors are adopted from [1]. The structural safety can be expressed by a rating factor, r:

$$r = \frac{R / \gamma_R}{S_d} \tag{1}$$

R	resistance
S_d	design load effect

γ_R resistance factor (=1.10)

If r is greater than or equal to 1.0, the investigated member or connection reaches the required structural safety level according to the Spanish codes. If the rating factor is less than 1.0, then structural safety is not verified and there is a need to perform a more accurate evaluation. The preliminary deterministic assessment reveals that the governing elements regarding load carrying capacity of the bridge are the top and bottom U-section compression members at midspan and over the piers, respectively (sections A-A and B-B, respectively, in Figure 1). Quite a number of these elements do not reach the required safety level. The minimum value for the rating factor r, equal to 0.57, is obtained for the main girder top U-section member at midspan (Figure 1, section A-A).

4.2 Importance of different variables for safety

4.2.1 Simplified structural model

Once the compression members at midspan and over the piers are identified as the critical areas, it can be assumed that the structural behaviour is brittle and that there is no significant system redundancy. Therefore, the failure of the most critical member leads to the failure of the system. Consequently, the failure probability for the bridge is governed by the failure probability of the most critical member [5].

Due to the aforementioned eccentricities at main girder nodes, the most critical member is subject to combined bending and axial compression. Although the "web plates" of the U-section are slender (Figure 1), the governing combination of bending moment, M, and axial compression, N, which defines the Ultimate Limit State of the critical member, leads to a loss of stiffness due to plate buckling of the order of only 18.5%. Therefore, the ends of the member are not free to rotate in the plane of buckling (plane of the girder, Figure 4). According to [1], the buckling length, l_p, of a truss girder top compression member corresponds to the length of a "pin ended" member which has the same buckling resistance. In the present case it can be assumed that $l_p = 0.9 \cdot l$. The reliability analysis can now be carried out for the simplified structural model, consisting of a "pin ended" member with a length of $0.9 \cdot l$ which is subject to combined bending and axial compression according to Figure 4.

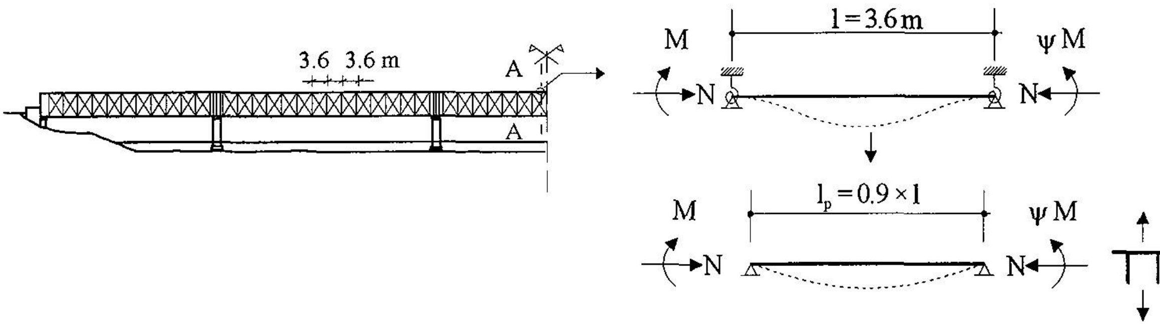

Fig.4 Simplified structural model for reliability analysis

4.2.2 Reliability analysis

Basic variables which are considered for the assessment of structural safety are associated with uncertainty. The safety of a structure can therefore be measured in terms of, for example, its reliability which takes account of uncertainty and is represented by a probability of failure.

The safety of a structure is expressed in terms of the basic variables by the Limit State Function (LSF). The most simple LSF defines safety as the requirement that resistance, R, is greater than or equal to the total action effect, S:

$$R - S \geq 0 \tag{2}$$

The probability of failure, p_f, is thus equal to the probability that S is greater than R.

Different numerical or analytical reliability methods exist for the analysis of structural safety. The First Order Second Moment (FOSM) method [6] introduces for example a reliability index, β, for which a direct link to the failure probability exists. Even though the FOSM reliability method only produces an estimate of failure probability, the resulting errors are small if it is used to compare the failure probabilities for a given LSF and varying basic variables. This is what the FOSM method is used for in the present study: Going out from the *axiom* that a correct application of the current codes results in a safe structure, the verification of structural safety of an existing structure consists of three steps [7]:

- Dimensioning of the existing structure according to a consistent set of codes,
- Calculation of the reliability index, β_{code}, related to the dimensions obtained in the first step, considering the parameters (mean value, standard deviation, probability distribution) of the variables assumed to lie behind the rules of codes,
- Calculation of the reliability index, β, related to the actual structure using default probabilistic models of action effects and resistance.

The structure may be considered safe if

$$\beta \geq \beta_{code} \tag{3}$$

In the case of the investigated truss bridge, in the first step the main girder top U-section member at midspan (Figure 4) is to be dimensioned according to the current codes [1, 4]. The analysis reveals that a rolled profile HEB 300 is required with a specified nominal yield strength of f_y = 235 N/mm^2. Such a main girder top member at midspan may be considered safe according to the aforementioned *axiom*.

In the second step the reliability index, β_{code}, of the above safe member is to be calculated. The LSF which is used in this reliability analysis is derived from the Spanish code [1] for the verification of structural safety of members subject to combined bending and axial compression:

$$f_y - \frac{\left(N_a + N_s + N_p + N_q\right)}{\chi\, A_{eff}} - \frac{k\left(M_a + M_s + M_p + M_q\right) + e\left(N_a + N_s + N_p + N_q\right)}{W_{eff}} = 0 \tag{4}$$

f_y	elastic limit of structural steel (or wrought iron)
N_a	axial compression due to the self weight of the steel
N_s	axial compression due to the sand fill
N_p	axial compression due to the asphalt layer
N_q	axial compression due to the traffic actions
M_a, M_s, M_p, M_q	moments due to the different aforementioned actions
A_{eff}	effective area of the cross-section when subject to uniform compression
W_{eff}	effective section modulus of the cross-section when subject only to moment about the relevant axis
χ	reduction factor for the relevant buckling mode
e	shift of the relevant centroidal axis when the cross-section is subject to uniform compression
k	factor which takes into account the distribution of the moments and the characteristics of the cross-section

The parameters of the variables involved in the LSF that are assumed to lie behind the rules of the codes are taken from the literature [5]. This LSF and the parameters of the variables (mean value, standard deviation, probability distribution) may now be introduced in a computer program [8] which handles the variables in accordance with the method from [6] and calculates the FOSM reliability index β_{code}. In the present case we obtain β_{code} = 4.06.

The third step of the verification consists of the calculation of the reliability index, β, of the actual member. *A priori* values for the parameters of the variables (Table 1), are either taken directly or interpreted from [2, 5, 9, 10] and introduced in the LSF (4). The FOSM reliability index is calculated to be β = 1.12.

Obviously, according to the inequality (3), the member under consideration is not safe. Site data should therefore be collected in order to improve the load and resistance models for the continuation of the evaluation (Figure 2).

Variable	Type	bias μ_X/X_{nom}	cov σ_X/μ_X	Nominal value X_{nom}	Mean μ_X	Standard deviation σ_X	Influence coefficient α_X^*	Design value X^*
f_y	LN	1.195	0.115	220 N/mm^2	263	30.3	0.826	234.8
N_a	N	1.01	0.03	234 kN	236.3	7.1	-0.025	236.5
N_s	N	1.20	0.25	273 kN	327.6	81.9	-0.29	354.3
N_p	N	1.20	0.25	82 kN	98.4	24.6	-0.087	100.8
N_q	Gumbel	0.88	0.125	1070 kN	941.6	117.7	-0.45	980.9
M_a	N	1.01	0.03	5.9 kNm	6.0	0.18	-0.004	6.0
M_s	N	1.20	0.25	9.5 kNm	11.4	2.85	-0.061	11.6
M_p	N	1.20	0.25	2.8 kNm	3.4	0.85	-0.018	3.42
M_q	Gumbel	0.88	0.125	43 kNm	37.8	4.7	-0.094	37.48
A_{eff}	N	1.02	0.01	14061.6 mm^2	14342	143.4	0.034	14340
W_{eff}	N	1.02	0.01	$1.68 \cdot 10^6$ mm^3	$1.71 \cdot 10^6$	$1.71 \cdot 10^4$	0.038	$1.71 \cdot 10^6$
χ	N	1.05	0.024	1.0	1.05	0.025	0.081	1.048
e	N	1.02	0.01	82.5 mm	84.2	0.84	-0.025	84.22
k	N	1.04	0.02	1.15	1.20	0.024	-0.025	1.2

***Table 1** Assumed values of the parameters of the variables for the estimation of β and results of FOSM analysis*

4.2.3 Conclusion

In addition to the reliability index, β, the method according to [6] provides the design values, X^*, and the importance factors, α_X^*, corresponding to the variables involved in the LSF (Table 1). The design values, X^*, correspond to the most probable set of values of the variables at failure. The importance factor is a function of the relative importance of a given basic variable within a given LSF. The greater the absolute value of α_X^* (the importance factor is negative for variables which have an unfavourable effect on safety), the bigger the influence of the variation of the corresponding variable on the reliability index. In the above example the yield strength of wrought iron, f_y, and axial compression due to traffic actions, N_q, are most critical. For these variables, updating efforts would be most effective.

4.3 Collection of site data - Planning and execution

4.3.1 Overview

The definition of a test program includes the choice of the parameters which should be updated, the definition of the method of observation and recording, the selection of test specimens, test conditions and arrangements, the number of tests and the method of evaluation. The execution of tests should be in accordance with the planning, and the measurement techniques in accordance with the required tolerances. For the evaluation of the test results, methods should be used which enable an easy introduction of the updated information in the calculation models. In the present case, according to 4.2.3 updating is carried out for the wrought iron yield strength and the traffic actions. For two reasons it is also decided to carry out measurements of the actual dimensions of wrought iron member cross-sections: the influence of corrosion is to be assessed and the assumed dimensional variation in the reliability analysis (4.2.2) corresponds to modern welded steel elements, for which fabrication tolerances are very small, and not to wrought iron members.

In the following, some information about the planning and execution of site data collection is given. Section 4.4 contains some thoughts on test evaluation, and the obtained site specific data is summarised in Table 2.

4.3.2 Material properties

Material properties are determined from miniaturised specimens, which can be drilled from structural members without reducing their resistance [11]. In the present case for example, the dimensions of the cylindrical specimens for tensile tests are: 40 mm of total length and 3 mm of diameter. Chosen test temperatures are room temperature (20°C) and -20°C corresponding to the lowest service temperature expected to occur within the intended remaining life of the structure.

Test samples should be representative and a sufficient number should be taken in order to determine variability with adequate certainty. In normal daily practice, however, only a limited number of tests can be carried out for economical reasons. In the present case for example, the number of tensile tests is eight. In section 4.4, the influence of the number of tests on the characteristic value of the wrought iron yield strength is discussed.

4.3.3 Cross-section area

The influence of the severe corrosion of the truss girder bottom U-section members is directly taken into account in the corresponding resistance model by introducing an equivalent cross-section (Figure 3). The influence of the dimensional variation due to corrosion and fabrication tolerances on the structural resistance of the other members is to be assessed. This is done by extensive measurement of the actual dimensions of wrought iron cross-sections.

4.3.4 Traffic actions

For economical reasons, neither vehicle surveys nor measurements of the effects of vehicle actions on the bridge with a view to obtaining data describing traffic actions are possible in the present case. Only traffic counting can be carried out: a daily traffic volume of 10059 vehicles, of which 12.5% are Heavy Goods Vehicles, is physically measured. This means that an average of 1257 HGV per day cross this urban bridge. Furthermore, frequent traffic jams are observed due to the traffic lights situated at both ends of the bridge. It is also known that the percentage of overloaded HGV in Spain is around 25% [12]. The effects of traffic actions on road bridges is described by a certain frequency distribution which determines the extreme action effects to be considered during the assessment of structural safety [5]. These effects may be obtained based on numerical simulations by generating random traffic actions for the considered traffic type [5, 9].

4.4 Evaluation of tests

If only a limited number of tests on material samples are available, as normal in daily practice, the evaluation of test results according to standard statistical methods may lead to unrealistic low characteristic or design values [13]. This drawback can be avoided, if the evaluation of test samples with a limited number of tests is carried out according to statistical models which permit the introduction of prior knowledge. Based on knowledge about the distribution of the investigated variable, a posterior distribution is derived in combination with the obtained test results. Such an approach is applied in the present study. In the case of the wrought iron strength, for example, a mean value of the yield strength m_{fy} = 225 N/mm^2 and a standard deviation of s_{fy} = 17.1 N/mm^2 are obtained from the sample of eight tensile tests. The corresponding characteristic value, which is based on a 5% fractile with a confidence level of 75%, evaluated with standard statistical methods [13], is f_{yk} = 187.5 N/mm^2. It is known from previous experience that for the yield strength of wrought iron a lognormal distribution can be expected. Furthermore, the sample standard deviation, s_{fy}, underestimates the standard deviation of the whole population, σ_{fy}, depending on the sample size. Taking into account this prior information, the estimate for the characteristic value of the yield strength is f_{yk} = 196.8 N/mm^2.

5. Introduction of test results in the calculation models

5.1 Overview

As mentioned in chapter 3, a full reliability analysis is not considered viable for the investigated bridge. A simplified deterministic method should therefore be used. The aim of a deterministic assessment of structural safety is to verify that the inequality (2) is satisfied by using nominal values of basic variables and partial safety factors in order to obtain the values that they would have at the design point in a reliability analysis [5]. The link between reliability concepts and deterministic methods is the design point which is the most probable failure point on a limit state surface [5]. The relation between the design point, partial safety factor and nominal value is given by

$$X^* = \gamma_X \cdot X_{nom} \tag{5}$$

X^* value of the basic variable at the design point
γ_X partial safety factor
X_{nom} nominal value of the basic variable

The Limit State Function is the same for both methods (reliability and deterministic), only the representation of the variables is different. Partial safety factors, which are introduced in a deterministic analysis, are therefore attributed individually to the variables in the LSF and vary according to the degree of uncertainty and the importance of the variable within the LSF. The aim of the collection of site specific data is the reduction of the uncertainty associated with the variables. The influence of this change can not be considered explicitly in a deterministic assessment (only changes in the mean value of a variable can be accounted for). As mentioned in chapter 3, the site specific data is therefore used to calibrate updated deterministic models of action effects and resistance, by applying reliability methods to the simplified structural model according to 4.2.1.

5.2 Calibration procedure

According to the *axiom* mentioned in 4.2.2, the calibration procedure consists of the following five steps:

- Dimensioning of the existing structure according to a consistent set of codes,
- Calculation of the reliability index, β_{code}, for this structure,
- Calculation of the reliability index, β_{upd}, for the actual structure using the updated parameters of the variables. β_{upd} may be greater or smaller than β_{code}, depending mainly on the state of the structure (corrosion) and the aggressivity of the actual traffic.
- Find the required actual resistance, $R_{upd.req}$, by multiplying the actual resistance, R_{upd}, by a factor, κ_R, in a way that results $\beta_{upd} = \beta_{code}$ for the actual effect of actions, S_{upd} (Figure 5).
- Derive partial safety factors, in analogy with equation (5), which can be applied to the nominal values of basic variables (S_{nom} for action effects and R_{nom} for resistance) in a deterministic assessment:

$$\gamma_{S,upd} = \frac{S^*_{upd}}{S_{nom}} \tag{6}$$

$\gamma_{S,upd}$ updated partial safety factor for action effects
S^*_{upd} updated action effect at the design point
S_{nom} nominal value of the action effect

$$\gamma_{R,upd} = \frac{\kappa_R \cdot R_{nom}}{R^*_{upd.req}} \tag{7}$$

$\gamma_{R,upd}$ updated partial safety factor for resistance
$R^*_{upd.req}$ updated required resistance at the design point
R_{nom} nominal value of the resistance
κ_R factor for the calculation of the required actual resistance

The updated partial safety factors, which take into account the influence of a change in uncertainty associated with the variables and are attributed individually to the basic variables in a LSF, can now be used in a deterministic assessment (using a more refined structural model) of structural safety, together with the nominal values of action effects and resistance. The requirement for structural safety can therefore be derived from the inequality (2) and is expressed by the following condition:

$$\gamma_{S,upd} \cdot S_{nom} \leq \frac{R_{nom}}{\gamma_{R,upd}} \tag{8}$$

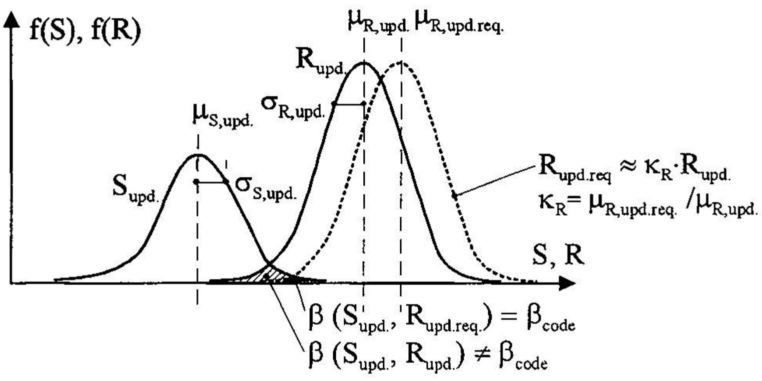

***Fig. 5** Calibration of updated load and resistance models*

5.3 Case study

The first two steps of the calibration procedure correspond to the first two steps of the reliability analysis from 4.2.2. Therefore, the reliability index according to the current codes is: $\beta_{code} = 4.06$. The collection and evaluation of site data according to 4.3 and 4.4 results in updated parameters of the corresponding variables, listed in Table 2.

Variable	Type	bias $\frac{\mu_{X,upd}}{X_{nom}}$	cov $\frac{\sigma_{X,upd}}{\mu_{X,upd}}$	Mean $\mu_{X,upd}$	Standard deviation $\sigma_{X,upd}$
f_y	LN	1.023	0.079	225 N/mm^2	17.7
N_q	Gumbel	0.80	0.125	856 kN	107
M_q	Gumbel	0.80	0.125	34.4 kNm	4.3
A_{eff}	LN	1.013	0.023	14249.4 mm^2	336.6
W_{eff}	LN	1.013	0.023	$1.7 \cdot 10^6$ mm^3	$3.9 \cdot 10^4$

***Table 2** Updated parameters of the variables*

Action effects				Resist.
Iron $\gamma_{Ga,upd}$	Sand $\gamma_{Gs,upd}$	Asph. $\gamma_{Gp,upd}$	Traff. $\gamma_{Q,upd}$	$\gamma_{R,upd}$
1.01	1.45	1.3	1.4	1.06

***Table 3** Updated partial safety factors*

For the other variables of the LSF (4), the parameters from Table 1 are adopted. The calculation of the FOSM reliability index for the actual structure gives $\beta_{upd} = 0.493$. This value is even lower than the one calculated in 4.2.2 using default probabilistic models of action effects and resistance. This is mainly due to the fact that in the bridge under investigation the elastic limit of the wrought iron is lower than usual values for this type of material. For the aforementioned factor, κ_R, a value of $\kappa_R = 1.484$ is found. The values of the basic variables of the LSF (4) at the design point, $X^*_{upd.(req)}$, result from the FOSM analysis, carried out for S_{upd} and $R_{upd.req}$. These values are then used to derive updated partial safety factors according to the equations (6) and (7). The obtained results are listed in Table 3. In a detailed deterministic assessment with updated models of action effects and resistance (according to (8)) it is now possible to determine the structural elements, nodes and riveted connections which need to be strengthened (Figure 2). The proposed solution for the strengthening is presented in [12].

6. Conclusions

A proper assessment of an existing bridge based on incomplete or defective information may be completely wrong. Therefore, correct updating of data is probably the most important step in a bridge evaluation. For the choice of the test and inspection programme some guidelines should be observed:

- The expected structural behaviour, loading and environmental conditions should be investigated by a qualitative analysis.

- Based on the results of the preliminary analysis, the objectives of the tests can be formulated and correct choices for the test programme are possible.
- The tests should be undertaken following the established plan.
- The evaluation of test samples with a limited number of tests should be carried out taking into account prior knowledge in order to avoid unrealistic low design values.

There is a need for simplified load and resistance models for the assessment of existing bridges. Furthermore, methods should be developed which enable an easy introduction of the collected site data in these simplified models.

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