SMARTRAIL WP2

User Guidelines

Development of a reliability based rail transport infrastructure safety framework

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Contents

1 Introduction

These guidelines provide a description of the work necessary to deliver the benefits as described in the SMARTRAIL Work Package 2. The project website www.fehrl.smartrail.org contains full details of the results of the individual Work Packages in addition to further information concerning the project.

1.1 Purpose

This guideline document describes the application of the general rail transport infrastructure safety framework with a focus on reliability based classification. The goal is to reduce replacement costs and to provide environmentally friendly maintenance solutions for ageing infrastructure networks. Reliability analysis should be performed when the structure has failed a deterministic analysis. The flow chart presented in Figure 1.1 illustrates a brief overview of the reliability based classification procedure.

This guideline is a synthesised version of Smartrail Deliverable 2.2 – Statistical Analysis Technique, which discusses in detail the use of reliability analysis and demonstrates the application of a reliability approach through case studies.

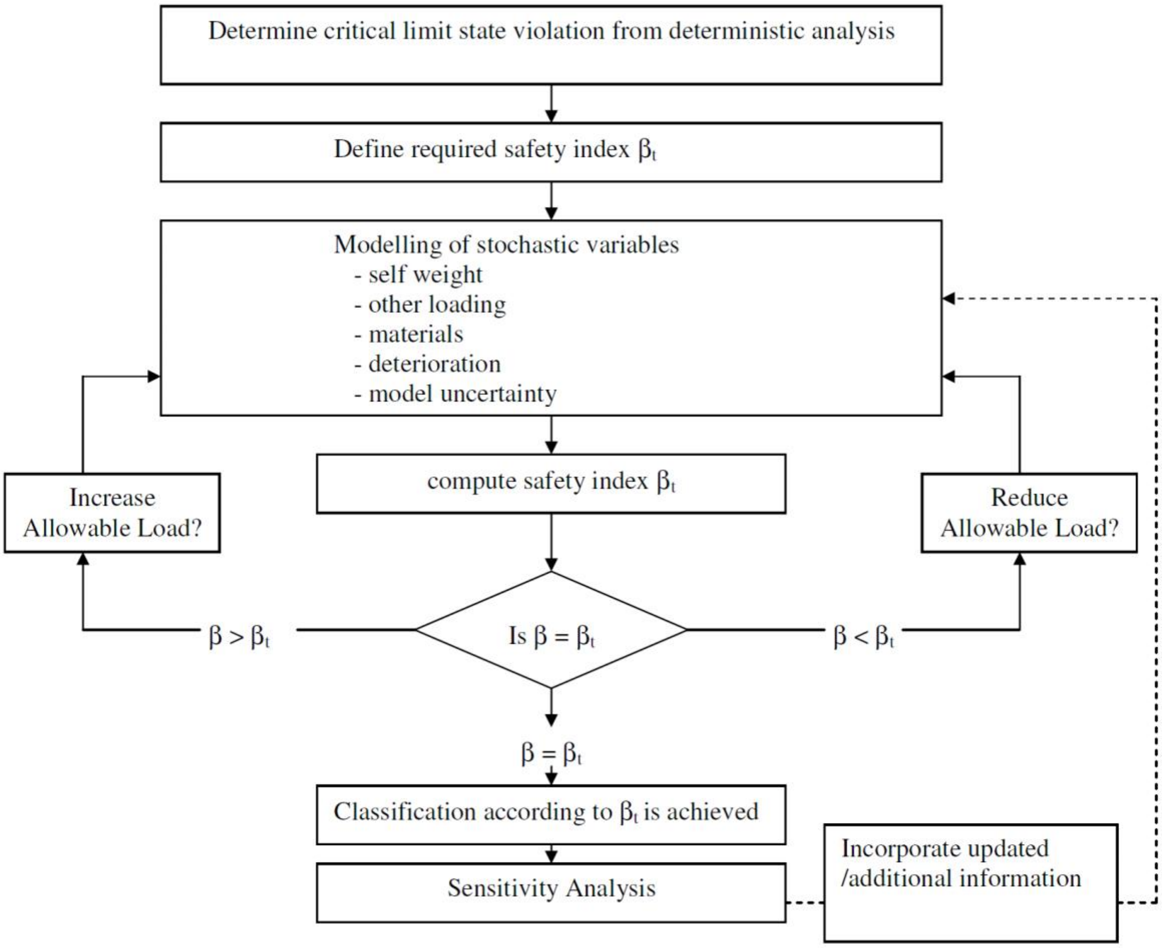

Figure 1.1: Outline of reliability based assessment

2 Assessment Procedure

This chapter discusses the concepts and procedures which are used to employ the safety and serviceability assessment of existing railway infrastructure. The concept is summarised in Figure 2.1.

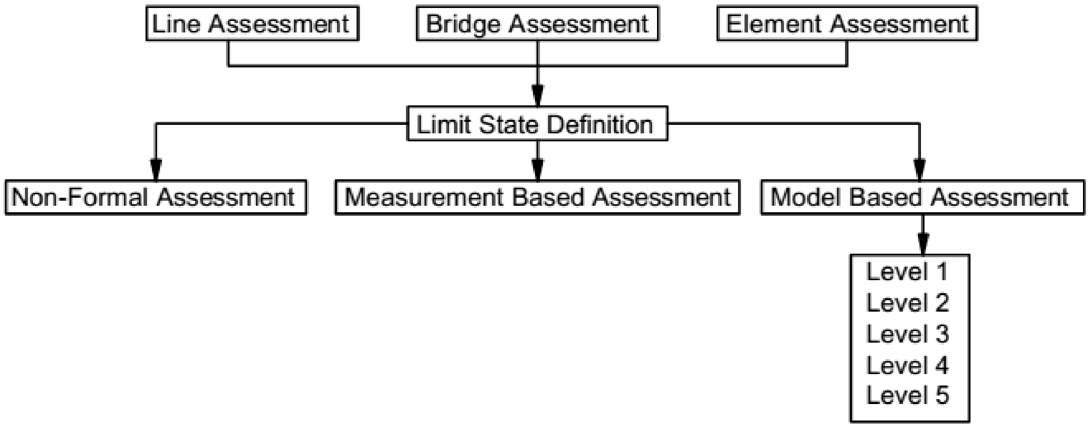

Figure 2.1: Assessment Concept

2.1 Types of assessment

If an upgrade of a line is required, this will entail a capacity assessment of the existing infrastructure along the line (e.g. a number of railway bridges). A line assessment typically initiates a primary sorting in order to identify the potentially critical bridges. For the identified 'critical bridges', where the classification load (maximum load permitted to cross bridge) is more unfavourable compared to the original design load, the assessment is then carried out at an individual bridge level.

Typical load, capacity and resistance assessment is carried out at bridge level. There are two types of analysis. Either the bridge is analysed for the critical elements and the ultimate capacity is considered to be equal to the lowest capacity of the bridge elements (i.e. the critical element), or the bridge is analysed accounting for redundancy by treating the bridge as a "system".

Element assessment can either be part of a bridge assessment or be a standalone investigation. The latter can be relevant if, for example, an element is damaged or deteriorated.

2.2 Criteria for assessment

The assessment process must always begin with the clear specification of the assessment objective. This first step is crucial to identify the most important limit states (which are discussed in detail in Section 3.1). Railway infrastructure is typically assessed taking into account the following limit state criteria:

- ultimate limit states;
- serviceability limit states;
- fatigue limit states;
- durability limit states.

2.3 Classification of assessment

In general, assessment procedures can be classified into three groups (SAMCO, 2006): Non-Formal Assessment, Measurement Based Assessment and Model Based Assessment. This document will focus primarily on Model Based Assessment, which includes all assessments where the load effects are determined by model based structural analysis. Assessment levels are ranked from level 1 to level 5, with the level of complexity and detail increasing with increasing levels.

2.4 Assessment levels (for Model Based Assessment)

Safety assessments are performed to check the capacity to safely carry or resist a specific loading level and to identify those elements of the infrastructure which are inadequate to carry or resist a required loading, e.g. infrastructures which have an unacceptable probability of failure. The consequences of finding any structure to be inadequate can be costly to both owners and users. Therefore, it is advisable that when an element of railway infrastructural fails an initial assessment, that the cost and time implications should be considered when advancing to more rigorous levels. The likelihood of changing the result should also be carefully considered. In some cases, the end result becomes self-evident at an early stage and the decision to terminate or continue the assessment can be taken at that stage.
The five levels of assessment recommended, in this document, vary in complexity from simple but conservative to complex but more accurate. These levels of assessment, numbered 1 to 5 with Level 1 being the simplest and Level 5 the most sophisticated, are well explained in the literature, including the COST 345 report (2004).

3 Reliability Requirements

3.1 Introduction

The objective of the assessment of railway infrastructure and assessments in general, is to determine whether the requirements to functionality, service life and safety, are fulfilled or not. The general requirements for a Probability Based Assessment are specified by limit states.
The ultimate limit state concerns the safety of the railway infrastructure and its contents as well as the safety of its users. Ultimate limit states which may require consideration include loss of equilibrium of the structure or any part of it, failure by excessive deformation, transformation of the structure or any part of it into a mechanism or slope failure.
Serviceability limit states relate to conditions beyond which the specified service requirements are no longer met. Serviceability limit states which may require consideration include deformations and displacements which affect appearance or effective use, vibrations which cause discomfort to people or induce damage or cracking which is likely to affect appearance, durability or function.
Fatigue is a local material deterioration caused by repeated variations of stresses or strains. The fatigue limit state includes failure caused by fatigue or other time-dependant effects and observable damage caused by fatigue and other time-dependent effects.
The durability limit state is concerned with the degradation mechanisms induced by the environment that may affect the service life. In the case of corrosion of concrete structures, the durability limit state includes the corrosion *initiation* and *propagation* periods.

3.2 Reliability class

For the purpose of reliability differentiation, consequences classes (CC) have been established by considering the consequences of failure or malfunction of the railway infrastructure. These consequences classes are presented in EN 1990 (2002).
The three consequence classes can be associated with reliability classes (RC). The reliability classes are defined by the reliability index, β, which is defined as:

$$\beta = -\Phi(P_f) \qquad \textbf{3.1}$$

where Φ is the distribution function of the standardised normal distribution and P_f is the probability of the limit state under consideration being exceeded.
Three reliability classes RC1, RC2 and RC3 exist to correspond with the three consequences classes CC1, CC2 and CC3. Table 3.2 gives the recommended minimum values for the reliability index associated with the three reliability classes at the Ultimate Limit State.

Table 3.2 Recommended minimum values for reliability index β (ultimate limit states)

Reliability Class	**Minimum values for β**	
	1 year reference period	50 years reference period
RC3	5.2	4.3
RC2	4.7	3.8
RC1	4.2	3.3

3.3 Target reliability levels

The target reliability level is the level of reliability prescribed by the railway infrastructure owner/manager to ensure acceptable safety and serviceability of the infrastructural element/network analysed. The choice of the target level of reliability should take into account the possible consequences of failure in terms of risk to life or injury, potential economic losses and the degree of societal inconvenience. The amount of expense and effort required to reduce the risk of failure should also be taken into account.
Recommended values from EN 1990 (2002) for the reliability index β for various limit states (for reference periods of 1 year and 50 years) are indicated in Table 3.4. The values of β in Table 3.4 correspond to levels of safety for reliability class RC2 structural members.

Table 3.4 EN 1990:2002 – Target reliability index for Class RC2 structural members

Limit state	**Target reliability index**	
	1 year	50 years
Ultimate	4.7	3.8
Fatigue		1.5 to 3.8*
Serviceability (irreversible)	2.9	1.5

* Depends on degree of inspectability, reparability and damage tolerance

4 Assessment with Uncertainty

4.1 Introduction

The design parameters that are involved in the definition of a structural limit state, i.e. loading, strength or geometry, have uncertainties associated with them and are thus described through the introduction of random variables. These uncertainties are therefore modelled using appropriate probability distribution functions for each basic variable. The main sources of uncertainty include:

1. **Physical uncertainty:** inherent variation in a physical parameter (e.g. strength/load intensity)
2. **Statistical uncertainty:** lack of sufficiently large samples of data to obtain a stable probability distribution function for the data.
3. **Model uncertainty:** Caused by simplifications introduced in describing model behaviour.

4.2 Methods of Reliability Analysis

Details of the different reliability-based analysis methods are widely available in the literature, including Melchers (1999). Some of the methods reviewed include FORM (First Order Reliability Method), SORM (Second Order Reliability Method) and Monte Carlo Simulation.

4.3 Model uncertainty

Model uncertainty is concerned with the differences between results predicted by mathematical models and the actual condition. Model uncertainties, denoted θ, are often modelled as normal or lognormal distributed variables. For permanent loads, the model uncertainty is normally distributed, it has a mean value about zero and is commonly introduced into the calculation model as follows:

$$\boldsymbol{Y = \theta + f(X_1 \dots\dots X_n)} \qquad \textbf{4.1}$$

For capacity and variable loads, the model uncertainty may be normally or lognormally distributed, it has a mean value about 1.0 and is introduced into the calculation models as follows:

$$\boldsymbol{Y = \theta\ x\ f(X_1 \dots\dots X_n)} \qquad \textbf{4.2}$$

Where Y is the overall response of the parameter and $f(X_1 \dots\dots X_n)$ is the model with the inherent basic variables that describes the capacity or load effect. The Danish Road Directorate Report (DRD, 2004), which borrows heavily from earlier NKB reports, provides detailed guidelines on the distributions and parameters for application of model uncertainty for load and resistance variables.

5 Load Modelling

5.1 Permanent gravity loads

In line with DRD (2004), the following statistical distributions are suggested:

- The dead load G is assumed to be normally distributed with a variation coefficient of 5%;
- The superimposed dead load GW is assumed to be normally distributed with a variation coefficient of 10%.

Note that permanent loads from different sources are assumed to be stochastically independent and it is possible to reduce uncertainties by measurement. For railway infrastructure, the self-weight generally encompasses the dead load and the superimposed dead load may refer to ballast, track, soil pressure, Water pressure etc. Further information can be found in EN 1991-2:2003 section 6.7.3.

5.2. Vertical Train loads

5.2.1 Weigh in Motion data available

Statistical distributions suitable for modelling train loads are generated using measurements of real train loads. Such measurements can be carried out using WIM techniques.

Individual train load models

The load and location of each axle of passing trains can be obtained from WIM measurements. If the measurements of the train loads are made during a sufficiently long time period and it can be assumed that the train loads are time invariant, i.e. there are not systematic changes in traffic type and train intensities, then it is possible to fit the measured data to a common statistical distribution.

The General Extreme Value (GEV) distribution (WAFO, 2000 & Coles, 2001) is used. This is a family of distributions which describes the Gumbel, Fréchet and Weibull distributions. The GEV cumulative distribution function given by:

$$F_X(x) = \exp\left(-\left(1-k\frac{x-b}{a}\right)^{\frac{1}{k}}\right) \qquad \text{if } k \neq 0 \qquad 5.1$$

$$F_X(x) = \exp\left(-\exp\left(-\frac{x-b}{a}\right)\right) \qquad \text{if } k = 0$$

where a , b and k are the scale, location and shape parameter respectively. Equation 5.1 is valid for $k(x - b) < a$, $a > 0$ and k, b arbitrary. The shape parameter k is often called the Extreme Value Index (EVI), because, if $k > 0$ the GEV is a Weibull distribution, if $k = 0$ the GEV is a Gumbel distribution and finally if $k < 0$ the GEV is a Fréchet distribution. Distributions are often fit to maximum daily values and extrapolated to represent maximum yearly data. The parameters of the train load distribution can then be input into the analysis

5.2.2 WIM data unavailable

Where WIM data is not available, alternative load models must be used in the probabilistic assessment. This is not ideal and is rather a quasi-probabilistic rather than a probabilistic analysis. In such cases, deterministic load model parameters may be taken from EN 1991-2:2003, for example, and assigned appropriate coefficients of variation.

5.3 Other variable loads

Various additional variable loads are discussed in EN 1991-2:2003. Some of the actions which may need to be considered include thermal actions, equivalent vertical loading for earthworks and earth pressure effects, centrifugal forces, nosing force, actions due to traction and braking, track bridge interaction, horizontal mass action, snow load, wind load etc. Accidental actions and dynamic effect should also be considered in accordance with EN1991-2:2003.

5.4 Fatigue loads

A fatigue damage assessment should be carried out for all structural elements which are subjected to fluctuations of stress (EN1991-2:2003). Details of the service trains and traffic mixes considered and the dynamic enhancement to be applied are given in Annex D of EN 1991-2:2003.

5.5 Application of traffic loads on railway bridges

EN1991-2:2003 recommends that a structure should be designed for the required number and position(s) of the tracks in accordance with the track positions and tolerances specified. However, each structure should also be designed for the greatest number of tracks geometrically and structurally possible in the least favourable position, irrespective of the position of the intended tracks. The simultaneous action of the vertical, horizontal and derailment loading may be taken into account by considering the groups of loads defined in Table 6.11 of EN 1991-2:2003.

6 Modelling of Resistance Variables

6.1 Introduction

Stochastic resistance models require information on material properties and dimensions. This chapter considers different material properties and identifies suitable probabilistic distributions to model these properties.

6.2 Reinforced concrete

6.2.1 Concrete

Material models for concrete include the compressive strength, f_c', the modulus of elasticity, E_c, the compressive strain and information on shrinkage and creep.

Compressive strength

In DRD (2004) the values of variation coefficient range from 0.12 for the higher strength concretes, i.e. 40MPa to 50MPa, to 0.22 for 5MPa concrete. PIARC (2000) suggests a COV of 0.2 to reflect the uncertainties associated with the material properties and the condition at the time of assessment. The other material properties of concrete (i.e. tensile strength, modulus of elasticity and ultimate strain) can be determined from the compressive strength.

Shrinkage and creep

The mean values of both shrinkage and creep can be determined using the approach in Section 2.1.6.4 of the CEB-FIP Model Code (1991). Adopting that approach, the shrinkage strain can be taken as normally distributed with a COV of 0.35 and creep strain can also be taken as normally distributed with a COV of 0.2 (DRD, 2004).

6.2.2 Reinforcing steel

The uncertainties in the estimation of the strength of steel reinforcement are due to the variation in the strength of material, variation in the cross-section, effect of rate of loading, effect of bar diameter on properties of the bar and effect of strain at which yield is defined (Mirza, 1979).

- **Tensile yield stress (f_y):** DRD (2004) suggests that f_y can be assumed to be lognormally distributed with a constant standard deviation of 25 MPa independent of the grade.
- **Modulus of elasticity:** The modulus of elasticity and the ultimate strain of the reinforcement can usually be modelled deterministically. Such an assumption will not significantly affect the safety calculation (DRD, 2004).
- **Compressive stress:** Can be determined from the tensile yield stress if no other information is available. If the reinforcement is not cold-formed then both values can be assumed to be equal. For cold-formed reinforcement the compressive yield stress is reasonably taken as 0.8 times the tensile yield stress (DRD, 2004).

Model uncertainty should be included in the analyses to account for the uncertainty in the determination of the parameters. Where tests have been carried out on the structure, the model uncertainty can be reduced. In addition geometric properties of reinforcing steel (area, effective depth, depth of conver) should be modelled according to PIARC (1999).

6.3 Prestressed concrete

In a probabilistic safety assessment involving a prestressed concrete bridge, the strength of the prestressing steel can modelled as a lognormally distributed variable (O'Connor & Enevoldsen, 2008). A low COV, circa 0.04 is generally sufficient for prestress steel. The ultimate strain and the modulus of elasticity for the prestressing steel can be modelled deterministically.

6.4 Structural Steel

For the probabilistic model for the yield stress, f_y, of structural steel, a lognormal distribution is recommended (JCSS III 2000; DRD 2004). The mean value is dependent on the steel grade and the thickness, t, and is greater than the characteristic value. A standard deviation of 25 MPa can be used for yield stress (DRD 2004). A lognormal distribution is also recommended for the ultimate tensile stress of structural steel with a standard deviation of 25 MPa (DRD 2004). The modulus of elasticity, shear modulus and Possion's ratio can either be taken as deterministic or a lognormal distribution with small COV of 0.03.

6.5 Masonry

The basic properties of masonry structures that should be included in an assessment are: elastic modulus, compressive and tensile strengths, bond strengths and shear strengths.

6.6 Soil

The relevant strength parameters for soil should be based on a geotechnical investigations and tests in the specific locality of the structure. The uncertainty in the parameters can be determined on the basis of the guidelines in NKB-rapport nr. 35 (1978) and/or JCSS (2006) (Phoon 2005).

7 Updating of Variables and Distributions

7.1 Introduction

Statistical uncertainty represents uncertainty resulting from the lack of sufficiently large samples of data to obtain a reliable probability distribution function for the data. This uncertainty can be reduced by adopting more accurate models and updating existing models. Material properties, including damage and deterioration, as well as loads can benefit from updating. This chapter outlines how testing and inspection results can be incorporated to update initial estimations or distributions.

7.2 Testing and Monitoring

7.2.1 Types of tests

Geometry and Integrity: The railway infrastructure geometry and integrity can be obtained in the first instance from visual inspection and some simple superficial measurements. This may not always provide sufficient or accurate information and in such cases measurements and other tests, preferably non-destructive, must be performed.

Material Properties: Mechanical material properties can be determined by tests performed on the structure or on specimens taken from the structure. The most reliable way is by performing destructive tests on samples taken from the structure (SB-ICA, 2007). Durability properties can also be determined using both destructive and non-destructive tests.

Load Tests: Load tests can also be performed. The objective of these tests is to apply a controlled load to the structure and monitor the response.

Real Loads: Permanent load can be obtained using the results of a geometrical survey and the expected or measured material densities. Or, the permanent load can be directly obtained by e.g. weighing the bridge deck using hydraulic jacks. The variable loads, such as traffic loading, can be determined and applied using WIM (as mentioned in Section 5.2).

7.2.2 Monitoring

Monitoring provides information on how the infrastructure and the loads acting on it are changing. Monitoring provides information on time dependent parameters and also provides more data. Therefore, the quantification of the parameters for the models is more reliable (SB-LRA, 2007).

7.3 Updating individual structural properties or reliability levels

After obtaining supplemental information from tests or monitoring, the results can be used to update the distribution parameters of a particular variable using observations obtained on that variable (e.g. concrete compressive strength), or to directly update the failure probability.

7.3.1 Individual parameters

For a particular variable (material or load), the distribution parameters such as mean and standard deviation can be estimated and updated on the basis of, for example, Bayesian statistics.

Bayesian statistics assume '*a priori'* knowledge of the distribution parameters. The distribution parameters can be the mean or the standard deviation or both (or any other parameters that describe the distribution). The *a priori* parameters can come from subjective knowledge based on expert opinion. The *a priori* distribution function can then be updated to the '*a posteriori*' distribution function using measurement or evidence data or any supplemental data available. The measurement data is used to create a likelihood function, and, using Bayes Theorem the likelihood function is combined with the *a priori* information to create the *a posteriori* distribution. The method is well documented in the literature (Raiffa and Schlaifer, 1961; JCSS, 2001; SB-LRA, 2007).

7.3.2 Direct updating of the probability of failure

Bayesian statistics can also be used to directly update the reliability of a structure based on a given event or considering a measured property. The event could be a test loading or the observation of a crack in a structure or a geometrical measurement. The deflection of a bridge at midspan, for example, can be determined with certain accuracy. The probability of failure can then be directly updated taking the measurement or event into account:

$$P_f = P(F|I) = \frac{P(F \cap I)}{P(I)} \qquad 7.2$$

Where:

F = local or global structural failure

I = information obtained from investigation or measurements

∩ = intersection of two events

| = conditional upon

8 Conclusions

The current guideline gives a summary of Smartrail deliverable 2.2 – Statistical Analysis Technique. The assessment procedure for railway infrastructure is outlined and requirements for reliability are discussed. Guidelines are given on stochastic modelling of variables for model based assessment. This includes modelling uncertainty, load modelling and modelling of resistance variables. Guidelines are also given on updating the assessment with information derived from testing / monitoring. As part of Smartrail Work Package 2, a state of the art review of non-destructive test methods was also performed. All of the above is discussed in more detail in the Smartrail Deliverable 2.2 report. In addition the Smartrail deliverable 2.2 report gives guidelines on reliability analysis of railway embankments and probabilistic assessment considering climate change. Practical examples are also provided. A simplified approach to the assessment framework is also discussed in the full report, along with a sample application. A brief outline of the procedure is given here.
Load and Resistance Factor Design (LRFD) is a simple approach for everyday use by engineers at a local level. Partial safety factors in this case are calibrated for the entire bridge stock. Using the framework discussed herein, partial safety factors can be calibrated more accurately and less conservatively for a smaller range of similar bridges. Application of these optimised partial safety factors can potentially result in cost savings for bridge owners by showing a bridge to have sufficient reliability, even after failing a traditional deterministic assessment.

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