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Probability-based assessment of highway bridges according to the new Danish guideline

A. O'CONNOR* and I. ENEVOLDSEN

Ramboll, Bredevej 2, DK-2830 Virum, Denmark

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This paper describes the newly developed Danish guideline for probability-based assessment of highway bridges. The guideline, the first of its kind in the world, describes how probability-based assessment of bridges can be performed in accordance with the requirements for the safety level prescribed by the Danish Roads Directorate (DRD). The guideline specifies principles for modelling uncertainties including treatment of model uncertainties. The requirement in the ultimate limit state for the structural safety is specified with reference to failure types and failure consequences. The guideline, in conjunction with codes of practice, provides the DRD with the legal justification necessary for application of probability-based approaches in Denmark. While the purpose of the guideline is to serve as a basis for the probability-based assessment of Danish bridges, it can also easily be applied in other countries. This paper presents, through a practical example, the application of the guideline to actual bridge structures.

Keywords: Bridge; Probabilistic assessment; Traffic modelling; Stochastic

1. Introduction

In past years, large efforts have been put into the assessment of the load carrying capacity of the bridges managed by the Danish Road Directorate (DRD). The assessments mostly deal with the passing of heavy vehicles, where a heavy vehicle is defined as a vehicle with a weight larger than 48 t that needs special permission from the police.

A classification system has been developed for the administration of heavy vehicles. The system is based on the idea that both the bridges and the heavy vehicles should be classified in a way such that the classes are comparable. In this way, the assessment of bridges regarding heavy vehicles should only be carried out once. Once the bridges are classified, it is easy for the police and road/bridge administrators to decide if a specific heavy vehicle with a certain vehicle class can pass the bridge with a certain bridge class. The classification system (Vejdirektoratet 1996) is based on a set of standard vehicles representing vehicles

with a total weight ranging from 20 to 200 t. In a regular deterministic assessment of the bridge class, a relatively conservative traffic load combination is applied comprising the standard vehicle of 50 t with a second standard heavy vehicle (weight 50 to 200 t). The resulting bridge class is equal to the weight in tonnes of the heaviest standard vehicle in the load combination. Furthermore, bridge classes are determined for passages of heavy vehicles with imposed restrictions such as speed limits and exclusion of other traffic.

- *Normal passage* is defined as passage involving both vehicles, with no restrictions on speed or position. Vehicles may be placed at the most severe location in respect of the load effect under consideration with speed of passage inducing the maximum dynamic amplification.
- *Conditional passage type I* is defined as passage involving both vehicles with no restrictions on speed, but position is restricted to lanes within the defined carriageway.

*Corresponding author. Email: alo@ramboll.dk

- *Conditional passage type II* is defined as passage involving both vehicles with restricted speed (10 km h^{-1}) and position. Vehicles may be placed, within lanes, at the most severe location in respect of the load effect under consideration with speed of passage reduced to limit dynamic amplification.
- *Conditional passage type III* is defined as passage involving only the heaviest vehicle with restricted speed (10 m h^{-1}) and position. The vehicle is placed in the least severe location in respect of the load effect under consideration with speed of passage reduced to limit dynamic amplification. This case simulates temporary bridge closure and prescribed passage route for the heavy transport.

In line with this classification system, a relatively detailed so-called *Blue Road Network* has been established comprising roads with no bridges having a class less than 100. As the blue road network includes all motorways (see figure 1) and many other major roads it ties together transport routes throughout all of Denmark. The police and road/bridge administrators use a map of the blue road network when preparing special weight permits, while haulage contractors use it when planning transport and selecting routes.

It has been the aim of the DRD for some time that all state roads, and as many other main roads as possible,

should be included in the blue road network. As about 98% of all heavy vehicles are classified below 100 t, and hence need no special investigation when passing on blue roads, an easy and efficient administration for the police and bridge administrators has been hereby established together with the provision of satisfactory service to the industry.

Inclusion on the blue road network requires initially that bridges have been assessed by applying general deterministic methods with elastic or plastic limit state analysis according to the Danish guideline for classification of bridges (Vejdirektoratet 1996). Where the obtained bridge class is insufficient ($< \text{class } 100$), traditionally the alternatives of expensive rehabilitation or replacement have been considered. In recent years, the DRD have consistently considered a third alternative, probability-based approach, before an expensive strengthening or rehabilitation project is implemented. The probability-based approach is often combined with advanced response models. The results of the probability-based assessment have often been found to raise considerably the bridge class achieved in deterministic assessment (Enevoldsen and Jensen 2000, Enevoldsen 2001, Enevoldsen *et al.* 2002, Jensen *et al.* 2002, O'Connor *et al.* 2004, Sloth *et al.* 2004). In many cases, these analyses have resulted in a satisfactory bridge class (i.e. > 100), thereby minimizing or avoiding strengthening projects. It is important to stress that at no stage is the safety of the structure compromised, rather the bridge specific safety is calculated. Therefore, these methods have proven to be very beneficial for bridge managers, resulting in large cost savings. Internationally, the result of the application of this methodology has also proven very beneficial, as evidenced by a number of publications (Nowak and Yamani 1995, Neves and Frangopol 2004, Czarnecki and Nowak 2006, Imam *et al.* 2006).

It is obvious that it is a fundamental requirement for a bridge owner such as the DRD, in order to be able to use probability-based management, that the legal justification for the methods is present. Some codes simply state that it is legal to use alternative assessment methods if it can be shown that the safety level is maintained. However, this statement is not operative for the code users or for the DRD who are going to approve the assessment results. To provide this legal justification, the DRD decided to commission the preparation of a guideline to be used in conjunction with the relevant codes of practice for probability-based assessment (Vejdirektoratet 2004). Uniquely, this guideline, believed to be the first of its kind in the world, is developed by a bridge owner and, as such, specifically specifies the requirements for probability-based assessment of bridges. It is a practical document providing comprehensive information on how to perform probability-based modelling and assessment of bridges in Denmark.

The guideline itself consists of seven chapters. Chapter 1 is an introduction, which proposes a definition of where,



Figure 1. Blue motorway network.

and on which bridges, the guideline can be applied. Chapter 2 describes how the probability-based approach for classification of road bridges works. This chapter includes a useful diagram, which describes the process in the assessment, as shown here in figure 2. Chapter 3 defines the DRD's requirement for the structural safety in both the ultimate and serviceability limit state. In Chapter 4, specifications on methods for probability-based safety analysis are given with guidelines on how model uncertainties are introduced into the modelling. Chapter 5 concerns load modelling, including requirements and specifications on how uncertainty modelling is to be performed. This chapter describes modelling of traffic loads by

applying Poisson models. Further, models on special heavy transports and ordinary trucks are given, together with information on modelling of transverse and longitudinal load location in lanes and specifications for modelling of dynamic impact factors. In Chapter 6 information for modelling of material strengths is given. For the three generations of the deterministic codes of practice in Denmark from 1949, 1973 and 1984 (Danish Standards 1949, 1973, 1984) it is specified how concrete compressions strength, yield stress of various reinforcement types and yield stress of structural steel must be modelled as stochastic variables. In addition, the chapter includes information regarding model uncertainties in modelling

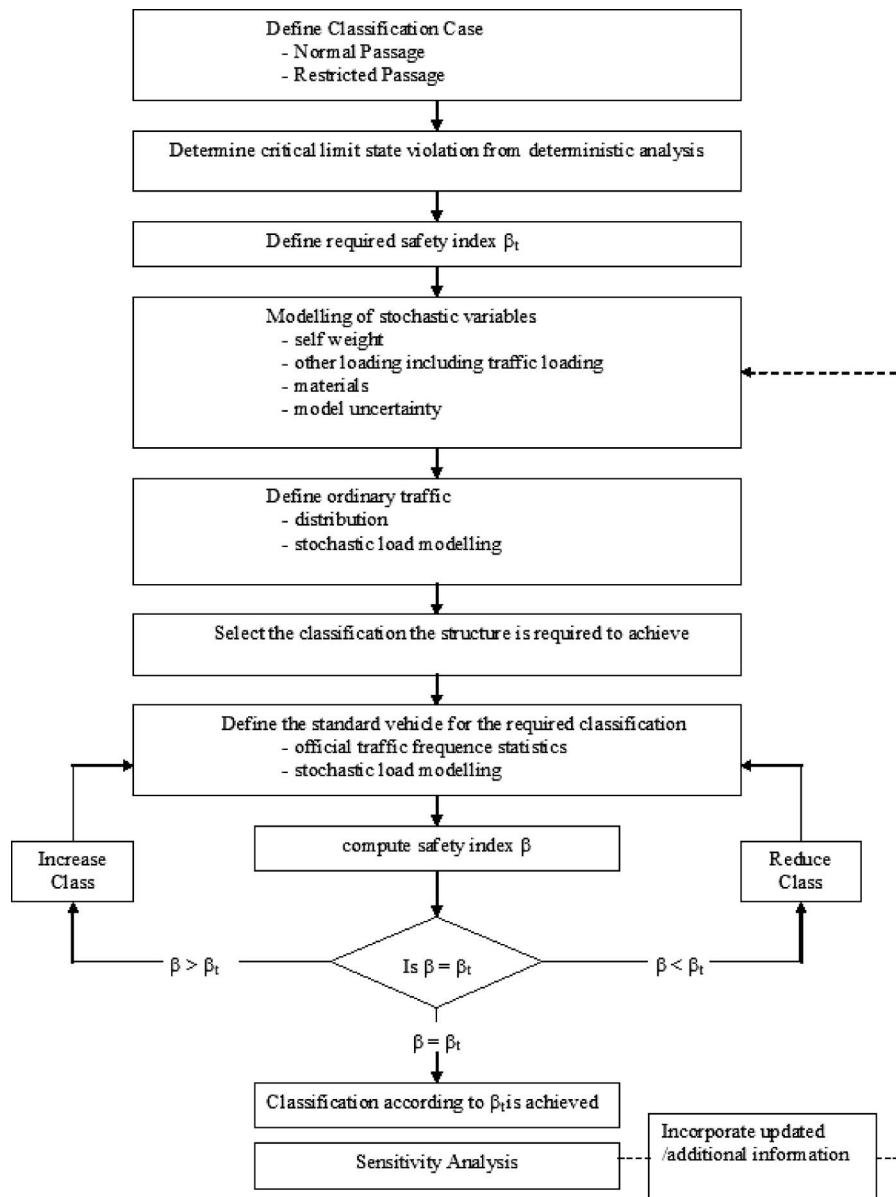


Figure 2. Flowchart of probability-based classification according to the new DRD guideline.

material strengths. Chapter 7 describes how new information from tests can be included in the probability-based assessment. Standard techniques for parameter estimating and reliability updating are described. The guideline does not provide information for modelling time dependent deterioration of structures; this topic is covered in an additional guideline currently in preparation.

This paper details selected topics from the guideline. A selected case study demonstrates the practical application of the guideline. A summary of savings achieved through the adoption of these approaches is presented.

2. Probability-based classification of bridges

The procedure for the probability-based safety assessment of an existing bridge according to the new DRD guideline is illustrated graphically in figure 2.

2.1 Formal requirements for safety assessment

The first stage in the process illustrated in figure 2 is to define the classification case for which the structure is to be considered (i.e. normal passage or conditional passage) and, by implication, to set the formal requirements for a specific safety assessment. The guideline specifies that for a bridge to receive classification N, the following condition must be fulfilled:

$\beta^{common} \geq \beta_t$ for common passage, i.e. vehicles of gross weight < 50 t, where β_t is the minimum required safety index,

$\beta_i^{normal} \geq \beta_t$ $i = 50, 60, \dots, N$ for normal passage,

$\beta_i^{restricted} \geq \beta_t$ $i = 50, 60, \dots, N$ for restricted passage,

where β is the safety index. Where the above condition is fulfilled, the safety rating of the structure is assigned according to:

$$\begin{aligned} \beta^{normal} &= \min \{ \beta_{50}^{normal}, \beta_{60}^{normal}, \dots, \beta_N^{normal} \} \\ &\text{for normal passage,} \\ \beta^{restricted} &= \min \{ \beta_{50}^{restricted}, \beta_{60}^{restricted}, \dots, \beta_N^{restricted} \} \\ &\text{for restricted passage.} \end{aligned}$$

2.2 Determine critical limit state violation from deterministic analysis

The procedure for the safety assessment of an existing bridge is first to identify specific problems using the general approach, i.e. traditional deterministic code. This is cost-efficient because the subsequent probabilistic modelling of the problems with related limit states requiring more

careful consideration, in a so-called individual approach, are narrowed down to a minimum. The condition of bridges in Denmark is in general very good, so the majority of bridges can still be assessed using a deterministic approach.

2.3 Define the required safety index β_t

The requirements at the ultimate limit state for the structural safety are specified with reference to failure types and failure consequences, i.e. safety class with requirements for the formal annual probability of failure p_f . The definition of the safety index, β , in this regard is taken from the existing Nordic recommendation for loading and safety regulations for structural design (NKB 1978). The safety index, β , is formally defined in terms of the probability of failure:

$$\beta = -\Phi^{-1}(p_f), \quad (1)$$

for which $\Phi^{-1}(\cdot)$ is the inverse function of the standardised normal distribution. Table 1 outlines the requirements of the guideline (Vejdirektoratet 2004) in this regard.

At the serviceability limit state, the guideline distinguishes between reversible and irreversible conditions. The ISO guideline on general principles on reliability for structures (ISO 1998) defines a reversible limit state as a limit state that will not be exceeded when the actions that caused the excess are removed, while an irreversible limit state is defined as a limit state that will remain permanently exceeded when the actions that caused the excess are removed. In selection of the appropriate safety index, β_t it is suggested that consideration be given to the cost of loss of serviceability and repair and of the cost of reducing the risk of attaining such a limit state. Reference is made to suggested values in other codes/guidelines indicated in table 2.

The guideline also requires that a sensitivity analysis be performed to determine the sensitivity of the computed β to variations in the parameters describing the stochastic variables modelled in the analysis. An advantage of the sensitivity analysis lies in its identification of the most

Table 1. Vejdirektoratet guideline requirements (2004).

Failure Consequences (Safety class)	Failure type I: Ductile failure <i>with</i> remaining capacity	Failure type II: Ductile failure <i>without</i> remaining capacity	Failure type III: Brittle failure
Very serious: High safety class	$p_f \leq 10^{-5}$ $\beta_t \geq 4.26$	$p_f \leq 10^{-6}$ $\beta_t \geq 4.75$	$p_f \leq 10^{-7}$ $\beta_t \geq 5.20$

Table 2. β , the serviceability limit state.

Code	Limit state	β_t
NKB (1978)	Reversible and irreversible	1.0–2.0
ISO (1998)	Irreversible	2.9
	Reversible	2.2
ENV1991-1 (EC1 1995)	Irreversible	1.5

critical parameters with respect to β . If, for example, it is determined that β is highly sensitive to the concrete strength, tests may be performed on the structure to update assumed models of material behaviour and the analysis re-performed with greater confidence. Another important sensitivity check is the control of the design values. It should always be checked that the outcome corresponding to the design value for all the stochastic variables is realistic.

2.4 Modelling of stochastic variables

In probability-based assessment to determine the safety index β , both the loading on, and resistance of, the structure are modelled as random variables (Madsen *et al.* 1986, Ditlevsen and Madsen 1996). The guideline is quite clear in its recommendations for treatment of these stochastic variables.

2.4.1 Self-weight. The self-weight is treated in terms of its permanent/dead and quasi-permanent/superimposed components, G and G_w respectively. The guideline recommends that self-weight be modelled as a normally distributed random variable, in the standardised form of the normal distribution, the mean of both the dead and superimposed-dead loads is modelled as 1.0, while their coefficients of variation are taken as 5% and 10% respectively. The inclusion of model uncertainty is treated later in this section.

2.4.2 Traffic load. Of the loads to be modelled on a highway bridge, by far the most variable are those termed *traffic loads*. This variability results not only from the stochastic variables describing the individual vehicles themselves, i.e. weight, axle spacing, speed, impact, etc. but also from the probability of multiple presence, both longitudinally within an individual lane, or transversely in multiple lanes.

Typically, the critical loading events for bridges with an influence length up to approximately 50 m in the extreme occur due to: (1) meeting events between ordinary trucks, and (2) meeting events involving heavy transport with ordinary trucks. In both cases, the extreme distribution function of the load effects can be obtained from the so-called thinned Poisson process (Ditlevsen 1994, Ditlevsen and Madsen 1994), i.e. only arrival and meeting events including the heaviest groups of trucks in the various traffic situations are considered. Figure 3 indicates a typical meeting event.

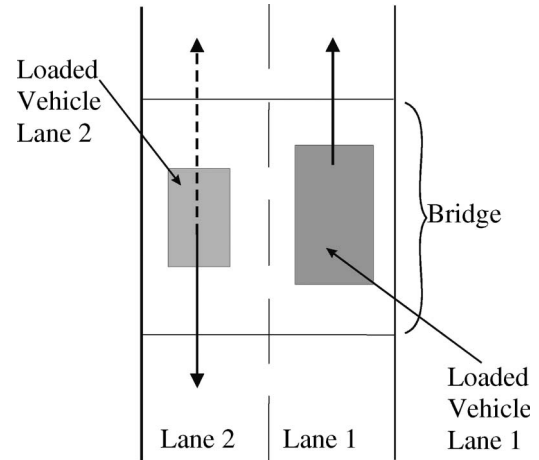


Figure 3. Typical meeting event: normal passage situation, heavy transport and standard vehicle.

For a two-lane bridge, the extreme distribution $F_{\max}$ of the considered load effect q can be obtained from:

$$F_{\max}(q) = \exp[-(v_1 - v_{12})T(1 - F_1(q))] \exp[-(v_2 - v_{12})T(1 - F_2(q))] \exp[-v_{12}T(1 - F_{12}(q))] \quad (2)$$

where v_1 and v_2 are the intensities of the considered traffic in lanes 1 and 2 respectively; v_{12} is the intensity of the meeting, i.e. of the considered traffic in both lanes at the same time; and T is the considered reference period for the extreme distribution (one traffic year). Furthermore, the distribution for the load effect in lane 1, $F_1(q)$, lane 2, $F_2(q)$, and the distribution of load effects due to simultaneous traffic load in both lanes, $F_{12}(q)$, must be determined. These three distributions do, in general, include modelling of:

The number, configuration and weight of trucks

Table 3 indicates the statistics provided by the guideline for the expected annual frequency of special heavy vehicles of various classes (> 100 t) for various route types. The configuration of the various vehicle types is provided in the guideline. For ordinary trucks, the six axle vehicles with the axle configuration illustrated in figure 4 should be employed. This axle configuration was derived following a nationwide survey of vehicles. For each truck class, the mean gross vehicle weight is specified by the guideline, while the standard deviation on the weight is taken as 5 t for all classes.

The longitudinal and transverse appearance in bridge lanes

In normal passage, the guideline recommends that the transverse location of the vehicles be modelled as a normally distributed random variable with the mean

Table 3. Annual frequency of standard heavy transports, N_i .

Route/class	100	125	150	175	200
Motorway	100	50	50	50	50
Primary	50	20	20	20	20
Secondary	20	10	10	10	10

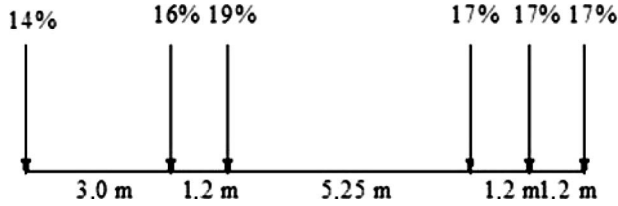


Figure 4. Configuration of ordinary transport vehicle.

position taken as the centre of the driving lane and a standard deviation of 0.24 m. Longitudinally, the vehicles are generally located at the critical influence ordinate.

The dynamic amplification of the static truck load

The guideline recognises the conservatism of ignoring the inverse relationship between vehicle weight and dynamic amplification factor. The dynamic increment is modelled as:

$$K_s = 1 + \varepsilon, \quad (3)$$

where ε is the dynamic increment, which for vehicles in normal passage is modelled as two independent normally distributed stochastic variables $N(41.5/W; 41.5/W)$ for an influence length $l > 2.5$ m, and as $\mu = (83/W) - (16.6/W) \cdot l$; $\sigma = \mu$ for $l < 2.5$ m, where W is the total vehicle weight in kN. Modelling the dynamic amplification factor implicitly assumes: (1) an inverse proportionality between the dynamic amplification and vehicle weight, and (2) a reduction in the coefficient of variation with increasing weight, which compare well with the literature (Hwang and Nowak 1991, Nassif and Nowak 1996, Kirkegaard *et al.* 1997).

The expected value for model uncertainty in the traffic load is defined in terms of a judgement factor, I_f , which is assumed to be normally distributed with a mean value equal to 1.0 and a coefficient of variation V_{I_f} taken as 10%, 15% or 20% for a level of uncertainty considering the loading assumed as small, medium and large respectively (Vejdirektoratet 2004).

2.5 Materials

The guideline provides significant details for establishing the statistical properties of materials based upon characteristic values assumed in design, or indicated on drawings. For concrete, the guideline recommends that a log-normal distribution be used to model compressive strength. Tables

provide the required mean values and coefficients of variation of the distribution based upon characteristic strength values, f_{ck} , or values for cylinder strength, or mix proportions. In addition, recommendations regarding modification of the values to account for the age of the concrete are provided.

The guideline also recommends that the strength distribution for steel be modelled as log-normally distributed. Tables provide the mean values based upon the specified steel type indicated on drawings. The guideline suggests a conservative standard deviation of 25 MPa be assumed for all steel types.

2.6 Model uncertainty

The model uncertainty takes account of: (1) the accuracy of the calculation model, (2) possible deviations from the strength of material properties in the structure involved as compared with that derived from control specimens, and (3) material identity. The model uncertainty is taken into account by introducing judgement factors I_m related to the material properties. The judgement factor I_m , which is assumed to be log-normally distributed with mean value equal to 1.0 and coefficient of variation V_{I_m} , is introduced by multiplying the basic material variables by $I_m \cdot V_{I_m}$, is calculated as (NKB 1978, 1987):

$$V_{I_m} = \sqrt{V_{I_1}^2 + V_{I_2}^2 + V_{I_3}^2 + 2(\rho_1 V_{I_1} + \rho_2 V_{I_2} + \rho_3 V_{I_3}) \cdot V_M}, \quad (4)$$

where the variation and correlation coefficients, V_{I_i} and ρ_i respectively, are determined from table 4 which is reproduced from the guideline of Vejdirektoratet (2004), and V_M is the coefficient of variation of the basic material variable.

2.7 Inclusion of tests and inspections

The guideline permits the inclusion of the results from tests and inspections in the calculation of the safety index. The guideline suggests the use of methods such as Maximum Likelihood or Bayesian Statistics in updating.

3. Probability-based classification of a concrete slab bridge

The DRD now employs probability-based assessment as a matter of course for any structure that fails an initial deterministic assessment. The newly developed guideline, which follows on from other Nordic documents (NKB 1978), is central to this process, as it ensures that at no stage in the process is the safety of the structure compromised. The result of this policy has been the ability to avoid unnecessary

rehabilitation or replacement of existing structures with considerable cost savings. A recent example of the assessment of a concrete slab bridge from 1942 is presented here.

The structural form consists of two continuous spans, carrying the motorway over a footpath and canal. In 1960, repairs to the structure were performed. The bridge was widened on the south side to incorporate an additional lane. The structure is illustrated in figures 5 and 6. The structure is in good condition as evidenced by the results of periodic inspections.

The bridge spans are 3.56 m and 5.56 m between support centrelines. The structure is skewed at an angle of 55° with the direction of travel so the load bearing spans are 4.35 m and 6.79 m respectively. Following widening of the structure in 1960, the structural width is 28.74 m. The structural thickness of the slab varies from 0.53 m at the middle to 0.37 m at the edge beam. The surfacing depth is 0.11 m.

The principal reinforcement is positioned parallel to the direction of travel. The transverse reinforcement is positioned along the bridge's long side and, as such, is

skewed at an angle of 55° to the principal reinforcement. On the bottom layer the main reinforcement is $\varnothing 20$ (20 mm bar diameter), while the transverse reinforcement is $\varnothing 14$. The intensity of the reinforcement varies with the dense bottom slab reinforcement in the centre of the span transferring to the top of the slab over the support. Transverse reinforcement is $\varnothing 14/250$ (14 mm bar diameter at 250 mm centres) and $\varnothing 14/500$ in the bottom and top of the slab respectively in the larger span and $\varnothing 10/250$ and $\varnothing 10/500$ in the bottom and top of the slab respectively in the smaller span.

A deterministic load-carrying capacity analysis of the structure was performed according to the guidelines set out by the DRD and national standards (Vejdirektoratet 1996, 2002, DS411 1999). The deterministic assessment investigates the capacity of the structure in terms of: (1) bending capacity assessment of the slab at the Ultimate Limit State (ULS), together with assessment of the abutment walls and foundation forces, (2) check of the shear capacity of the slab at ULS, and (3) bending capacity assessment at the Serviceability Limit State (SLS).

Table 4. Model uncertainty factors.

Accuracy calculation model				Material property deviations				Material identity			
	Good	Normal	Poor		Small	Medium	Large		Good	Normal	Poor
V_{I_1}	0.04	0.06	0.09	V_{I_2}	0.04	0.06	0.09	V_{I_3}	0.04	0.06	0.09
ρ_1	−0.30	0.00	0.30	ρ_2	−0.30	0.00	0.30	ρ_3	−0.30	0.00	0.30



Figure 5. Reinforced concrete slab structure.

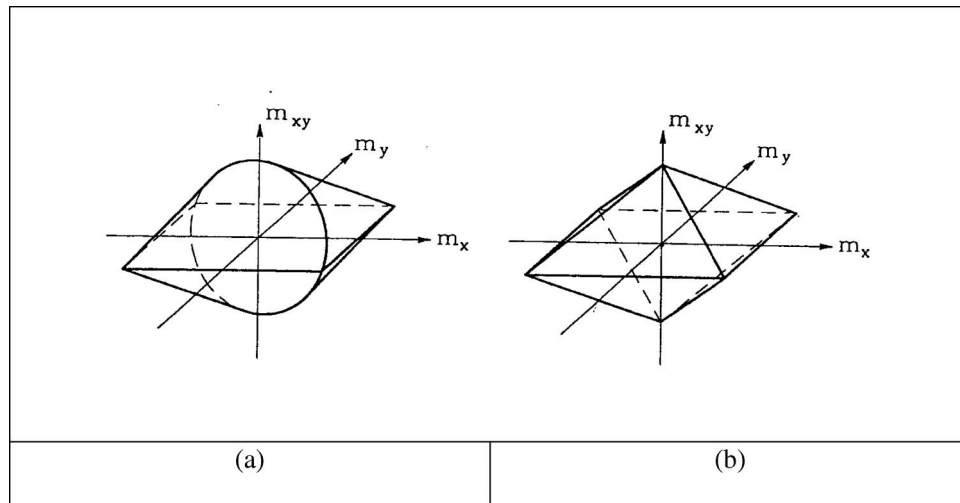


Figure 7. Yield criteria (m_x , m_y , m_{xy} = moment/unit width about associated axes): (a) idealised form, and (b) linearised form.

figure 7(b). A plastic hinge is formed when the applied moment reaches any of the eight surfaces. The load factor for variable load is changed in an iterative procedure until a mechanism is formed. The load factor at the formation of the mechanism is reported. The program checks for flexural capacity only.

3.3 Requirements for safety level

The requirement in the ultimate limit state for the structural safety are specified with reference to failure types and failure consequences; for the slab bridge deck of the structure, the critical limit state is a ductile failure mode. It is considered appropriate to select *Failure Type II – Ductile failure*. The implication is that the safety requirement for the structure at the ultimate limit state is $\beta \geq 4.75$.

3.4 Reliability-based analysis using PROBAN

Reliability analysis is carried out using the software package PROBAN (DNV 2006). Determination of the probability of failure p_f is performed using the FORM and/or SORM techniques.

3.5 Traffic load modelling

The normal passage case is considered.

3.6 Modelling of stochastic loading parameters

In the traffic load model, values are entered for the following: traffic loads of the vehicles, flow rates and proportions, vehicle transverse locations, lane importance factors, vehicle impact factors, and model uncertainty for the load.

Traffic load of heavy transport vehicles

The traffic loading for the heavy transport vehicles is modelled with an axle configuration equivalent to the class 100 vehicle illustrated in figure 8. The total weight is modelled by a normally distributed stochastic variable where the expected value and the standard deviation are 1072 kN and 49.1 kN respectively (Vejdirektoratet 2004).

Traffic load of ordinary transport vehicles

The ordinary transport vehicle, illustrated in figure 4, has a total weight of 520.9 kN and a prescribed load distribution to internal axles. The standard deviation is 49.1 kN.

Vehicle speed

The special transport is put in a normal passage for class 100 driving with a speed of 60 km h⁻¹. The speed for the ordinary trucks is conservatively modelled as 80 km h⁻¹.

Vehicle length

The vehicle length is modelled according to the length of the standard vehicles in equivalent to 19.0 m for the class 100 and 11.9 m for the ordinary transport vehicles respectively (Vejdirektoratet 2004).

Bridge influence length

The bridge influence length is modelled as 11.16 m. This value is equivalent to the total bridge length. The influence length is defined as the length of the structure which, when loaded, contributes to the magnitude of the sectional force at a specified location (where relieving zones are excluded from this length).

CLASS	AXLE CONFIGURATION AXLE WIDTH IN TONNES: AXLE SPACING IN m	AXLE WIDTH (m)
50		2.6
80		2.6
100		2.6
200		2.8

Figure 8. Representative vehicle silhouettes.

Vehicle annual frequency

The annual frequency of the standard transports vehicles is taken as 100 in accordance with table 3. The daily frequency of ordinary vehicles is found to be 1900 from the Vejdirektoratet records for the site of the bridge.

Vehicle annual hours/day

The duration of the day over which trucks are assumed to use the road network is assumed to be 15 h.

Modelling uncertainty in the traffic load

The expected value for model uncertainty in the traffic load is 1.0 and the standard deviation/coefficient of variation is set in normal passage to 0.15 (Vejdirektoratet 2004). However, as in this case, the influence of the standard transport was shown to be minor, the value of $V_{I_f} = 0.10$.

Dynamic factor

The dynamic factors are modelled as two independent normally distributed stochastic variables $N(41.5/W; 41.5/W)$ as discussed previously.

Vehicle transverse location in the lanes

Transverse location is modelled as a normally distributed random variable with an expected value equivalent to the location of the centre of the lane and a standard deviation of 0.24 m (Vejdirektoratet 2004).

Vehicle longitudinal location in the lanes

The vehicles are placed in the most severe longitudinal locations, l_1 and l_2 , in the two lanes where these locations are taken from the deterministic analysis.

Self-weight of the structure

The sectional forces induced by the self-weight of the structure are a superposition of the effects of the beams, slab and pavement. The magnitude of the sectional force induced is modelled as stochastic. For the dead load, the mean value is taken as 1.0 with a coefficient of variation including model uncertainty of 7.071%. For the surfacing, the mean value is taken as 1.0 with a coefficient of variation including model uncertainty of 11.18% (Vejdirektoratet 2004).

4. Modelling of stochastic variables – capacity

Two strength parameters that are modelled as stochastic in the PROBAN analysis are:

- the concrete compressive strength, f_c ,
- the strength of the ordinary reinforcing steel, f_y .

Concrete compressive strength f_c

The concrete compressive strength is determined from construction drawings where in it is specified that the concrete consists of 300 kg C m^{-3} , which is taken to imply a mix ratio cement/sand/stone of 1:2:3. From the

guideline it may be deduced that $E[f_c] = 29$ MPa with a corresponding coefficient of variation (CoV) of 0.35. The guideline specifies that f_{ck} at 28 d may be increased by 50% for concretes over 50 y so that $1.5 E[f_c] = 43.5$ MPa.

The model uncertainty is modelled as a logarithmic normal distributed stochastic variable with an expected value of 1.0, which is multiplied by the strength variable. The accuracy of the calculation model is taken as normal, the uncertainty of the material properties in construction is classified as medium, while the material identity is classified as normal. Thus, the coefficient of variation including model uncertainty is taken as 36.01%. The resulting standard deviation including model uncertainty is 15.67 MPa.

Yield strength of the reinforcing steel, f_y

The classification of the reinforcing steel is taken as St 52. Tests performed on the steel in the old portion of the deck indicated a characteristic yield strength of 348 MPa and a mean strength of 362.33 MPa, with a standard deviation of 5.54 MPa (CoV 1.5%). These tests were performed on bars of $\phi \leq 16$ mm whose expected characteristic strength is $f_{yk} = 355$ MPa with a corresponding mean of 426 MPa and standard deviation of 25 MPa.

Test results were not available for the bars with $\phi \geq 16$ mm, and so, according to the guideline, their yield strength was taken to be $f_{yk} = 345$ MPa. The corresponding mean value specified by the guideline is 416 MPa and the standard deviation is 25 MPa. For the original reinforcing, the presence of test evidence was taken to reflect a reduction in the uncertainty associated with the reinforcement properties and as such for $\phi \leq 16$ mm: (1) the accuracy of the calculation model was taken as normal, (2) the uncertainty for the material resistance in the construction was assumed as medium, and finally (3) the material identity was taken as good due to the presence of test results. For $\phi \geq 16$ the uncertainties were taken as: (1) normal, (2) medium, and (3) normal.

Variation coefficients, including model uncertainty of 10.91% and 12.00%, were derived for the old reinforcement with $\phi \leq 16$ mm and $\phi \geq 16$ mm respectively. These values are equivalent to deviations of 39.53 and 49.94 MPa.

5. Modelling of deterministic variables

The elastic modulus of the materials, parameters describing the cross section of the slab under investigation, the steel areas and covers to the top and bottom reinforcement layers are modelled deterministically.

6. Classification for safety analysis

Safety assessments of the carrying capacity of the bridge are performed based upon the probabilistic modelling

outlined previously. The analysis returned a safety index $\beta = 4.88 > 4.75$. It was therefore concluded that the bridge can receive the classification Class 100 for normal passage. The rating of Class 50 for normal passage, resulting from the deterministic assessment, is revised.

6.1 Sensitivity analysis

An important aspect of any safety assessment is to perform a sensitivity analysis of the results to identify which of the modelled parameters has the greatest influence on the safety. This analysis provides a means of checking the rationality of the values of the random variables at the design point.

Figure 9 illustrates the relative influence of the modelled parameters. It is apparent that the steel strength, denoted

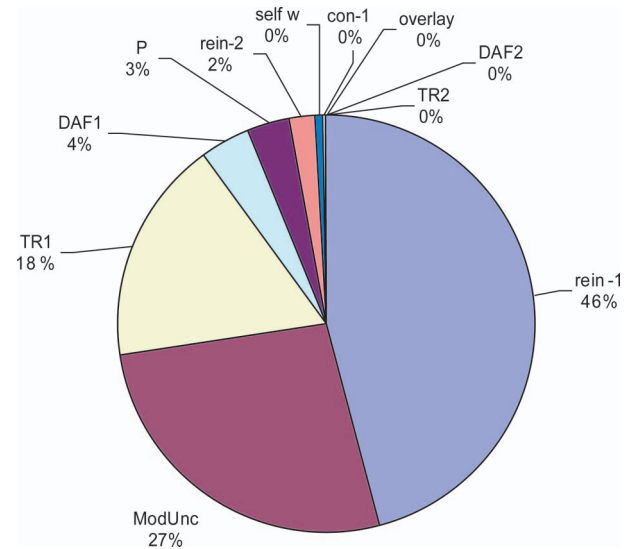


Figure 9. Relative importance of modelled parameters.

Table 6. DRD savings from probability-based assessments.

Bridge	Result of deterministic analysis	Probability-based assessment	Cost saving \$ ($\times 10^6$)
Vilsund	Max $W = 40$ t	Max $W = 100$ t	4
Skovdiget	Lifetime ~ 0 y	Lifetime > 15 y	15
Storstroem	Lifetime ~ 0 y	Lifetime > 10 y	20
Klovtofte	Max $W = 50$ t	Max $W = 100$ t	2
407-0028	Max $W = 60$ t	Max $W = 150$ t	1.5
30-0124	Max $W = 45$ t	Max $W = 100$ t	0.5
Norreso	Max $W = 50$ t	Max $W = 100$ t	0.5
Rødbyhavn	Max $W = 70$ t	Max $W = 100$ t	0.5
Åkalve Bro	Max $W = 80$ t	Max $W = 100$ t	1.5
Nystedvej Bro	Max $W = 80$ t	Max $W = 100$ t	2
Avdebo Bro	Max $W = 80$ t	Max $W = 100$ t	3
Total			50.5

rein-1; the model uncertainty on loading, ModUnc; and the transverse position of the vehicle in the outer lane, TR1, are controlling. The other variables (DAF1(2)=dynamic amplification in lane 1(2), P=live load exceedance probability, rein-2=steel strength widened portion of slab, self w=self weight, con-1=concrete strength in original slab, overlay=surfacing, TR2=transverse location in lane 2) are of minor influence only.

7. Conclusion

The newly developed Danish guideline for probability-based assessment of bridges is believed to be the first in the world of its kind and describes, in practical detail, how a probabilistic-based assessment can be performed in accordance with the requirements for the safety level prescribed by the Danish Roads Directorate (DRD). The guideline, in conjunction with codes of practice, provides the DRD with the legal justification necessary for the application of probabilistic-based approaches in Denmark. While the purpose of the guideline is to serve as a basis for the probability-based assessment of Danish bridges, it can also easily be applied in other countries.

The DRD now pursues reliability-based assessment as a matter of course for all structures that have failed a deterministic assessment. The results of this policy have provided significant savings in both the direct and indirect costs associated with bridge rehabilitation or replacement. Table 6 lists the direct monetary benefits, in excess of a total of fifty million US dollars, accrued in some recent cases where probability-based assessments have been employed.

The Guideline for Reliability Based Classification of the Load Carrying Capacity of Existing Bridges may be downloaded (in English) from the website of the Danish Roads Directorate at <http://www.vd.dk>.

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