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Safety Concept and Partial Factors for Military Assessment of Existing Concrete Bridges

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This work is a result of my involvement at the University of Bundeswehr in Munich where I was employed as scientific assistant at the Structural Engineering Institute from July 2010 until December 2013. Large portion of the research presented in this work can be contributed to my participation in the project for development of new and improved methods of bridge classification. The idea behind this work was to develop a concept that would be applicable within the frame work of military vehicles but at the same time in line with current provisions and related to the general problem of existing bridge assessment. The path was not easy as some my ideas would sometimes prove to be a dead-end, not applicable or too difficult to implement, so an iteration and reinvention was necessary. I wanted a concept that is straightforward, easy to understand and traceable. Hopefully this all was accomplished.

My first large thank you is directly aimed at my supervisor Prof. Manfred Keuser. It was actually his idea, after discussing some options upon my return from the USA, that I will become part of his Concrete Structures team. The topic of my thesis was also selected by him. It was short proven to me, that not only is Prof. Keuser a great supervisor, but also an unspeakably generous person. Working together with him allowed me to gain additional knowledge, improve my skills and broaden my horizons. I shall be forever grateful for his supervision, support and motivation.

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Abstract

Military traffic often utilizes bridges designed and maintained by civilian authorities. These bridges are located not only in home and allied countries, but also in foreign theatres of operation. The use of civilian bridges by military vehicles is in the NATO countries regulated and governed by STANAG 2021. This standard, however, does not fully deliver the essential aspects such as defined safety concept or specified values for partial factors that are important for a safe and reliable crossing of military vehicles over existing bridges. This work is aimed at investigating the military loads and developing a suitable safety concept that can be utilized during the military assessment of concrete bridges. A number of factors must be taken into account, including target reliability index and probabilistic models related to dynamic amplification factor, static load due to military vehicle and applied load effect model. The calculated partial factors for semi-probabilistic safety concept are significantly dependent on the selected crossing condition. The results show that the military partial factors for assessment of existing bridges can be generally considered lower than those factors listed in modern structural codes for bridge engineering.

Table of Contents

1 Introduction

1.1 Motivation

It is a recognized phenomenon that the assessment of existing bridges is becoming increasingly important throughout the world. Due to the aging infrastructure, increased civilian traffic loads and load intensity, many structures are yielded as obsolete. When the performance of existing bridges calculated according to the most current design codes is considered, then many of those bridges show an inadequate performance to the design loading. Many researchers have recognized this fact and there is a significant effort beyond establishing and improving possible methods for bridge structural assessment, assessment of remaining service life and proposing improved repair and maintenance techniques.

Present and most widely used methods of bridge assessment are evolved around a semi-probabilistic safety concept with partial factors applied to both the load and the resistance side of the limit state equation. As this safety concept is mainly developed and calibrated for the design of new structures it fails to recognize some of the fundamental characteristics of already existing structures and is therefore in many cases overly conservative [86], [122]. Existing structures could be in most cases described with a larger degree of certainty, especially when the data resulting from inspection, testing or measurements are available. Such data are simply not available during the design and therefore must be a priori accounted for in the development of design partial factors. Recent advancements and studies have presented new methods for the evaluation of existing bridges [29], [90], [132]. This is accomplished by the use of modified partial safety factors values or even abandoning the semi-probabilistic safety concept and changing to, for example, the full probabilistic assessment.

There are additionally differences in the treatment of existing bridges either under the generic common traffic or under the well-defined traffic loading [81]. The well-defined loading could be for example defined as permit legal vehicles, or any other form of vehicles not conforming to the standards, such as construction equipment. The most important fact is that the well-defined traffic can be better described in terms of the expected load effect and the associated uncertainty. It is therefore inconsistent to use design methods intended for common traffic when assessing bridges under the well-defined loading.

The topic of improving assessment methods and updating partial factors to reflect the changing traffic and bridge conditions is not restricted to civilian engineering professionals and scientists, but is also interesting for the military community considering the fact, that the military frequently utilize bridges built and maintained by civilian authorities. Besides development of concepts for assessment of existing bridges, there has been an increased effort in establishing a proper safety format for the assessment of bridges under military loading. Emphasis of this work is the investigation of the well-defined loading as represented by military vehicles on bridges.

The current standard for assessment of existing civilian and military bridges is STANAG 2021 [93]. This regulation is a NATO Standardization Agreement and provides guidance on the assessment of military vehicles, bridges, rafts and ferries. It is a general document that primarily aims at establishing common grounds and language among military engineers within NATO. In the respect of national interests, it does not set nor requires any specific procedures or safety concepts for the bridge classification itself, although there are minimum outlined criteria. No provisions regarding the safety format and more importantly no partial factors are provided. It is assumed from the language there that military engineers would utilize current civilian structural standards. Most European countries within NATO are using Eurocodes designated for bridge design and detailing (EN 1990 [45], EN 1991-2 [46], EN 1992-2 [48], etc.). Therefore, Eurocodes and the listed provisions are generally accepted for military bridge assessment as well. Specifically, the partial factor for variable loading γ_Q would be used for establishment of design load effects of military vehicles. It is however problematic to use the factor originally developed for civilian traffic when dealing with military loads [80],[82]. Eurocodes and their National Annexes have never been calibrated for the assessment of existing bridges under military loading and fundamental differences exist between the two traffic models:

1. The civilian loading is described by loading models developed to represent the complete actual and future predicted traffic. The military loading is distributed in defined time invariant classes.
2. Dynamic effects are included in traffic models in current bridge codes. There are no specified dynamic allowances listed in STANAG 2021 [93].
3. The characteristic value of civilian traffic load corresponds a 1000-year return period [61] while a nominal (mean) value is considered for military vehicles; considerable reliability margin is thus included already in the characteristic value of civilian traffic load.

It is in this respect inconsistent to use partial factors intended for civilian traffic when assessing bridges under military loading.

1.2 Military Traffic on Bridges

Within the NATO, the military traffic has been studied under the Land Capability Group 7 (LCG 7) on Battlefield Mobility and Military Engineer Support, and especially with regards to bridge assessment by the Team of Experts on Military Bridge Assessment [124]. These groups were created in order to further investigate military load and military bridge assessment as a supplement to the STANAG 2021 [93], which provides only limited guidance. The main accomplishment and advantage of STANAG is the introduction of standardized Military Load Classes (MLC) for vehicles, which is useful for a number of reasons:

- loading the vehicles on trains or airplanes as required by operational deployment,
- loading the ferries or rafts for transportation over water surfaces,
- and most importantly, loading of the bridges.

The military utilizes number of civilian bridges for many of its operations and often in many foreign countries. It is a common practice that most of these bridges are assessed for its maximal load carrying capacity in terms of MLC in order to properly plan for routing of military convoys. It is important for both peace and wartime strategic and operational capabilities. There is a variety of reasons for crossing of military vehicles. In peace time, naturally, the bridges serve in terms of transport and mobility. They may be used during the response to natural threats (flooding, earthquake, fire or snow [140]) or response to danger in home countries presented by terrorist threats [137]. It can also be the case, that civilian network is used to reach to and operate on during the foreign deployment (KFOR [138], operations in Bosnia [139]). It is then for a number of reasons necessary to properly assess the bridges in order to allow for the safe and reliable transport of military vehicles over existing civilian bridges as dictated by the mission.

1.3 Goals

The goal of this work is to properly reflect the military traffic on bridges as suggested by LENNER [80] and to develop a safety concept and calibrated partial factors for the bridge assessment in the ultimate limit state. There are substantial differences in treatment of bridges under civilian and military traffic. A systematic identification of these differences is necessary to include the investigation of the military traffic definition, its characteristic values, load model, uncertainty, dynamic load effects and crossing conditions. When the bridge assessment is considered according to the STANAG 2021 [93], it states that “a safety factor appropriate to the bridge type and mission must be included in the consideration when determining a bridge rating. The safety factor should reflect a high degree of confidence for the bridge under specific loading levels”. This work is aimed at developing partial factors which could be used for the military variable loading in the semi-probabilistic safety format based of the current EN 1990 [45]. Simple principles of structural statics, traffic modeling and structural reliability theory are employed to duly account for knowledge about load models, uncertainties, dynamic load effects and crossing conditions.

The advantage of accepting current structural codes and modifying the selected factors dwells in maintaining the continuity of developed design and assessment calculations as known to the engineers, while reflecting the specifics of military loading and providing a framework for safe, reliable and efficient classification of existing civilian bridges.

2 Safety and Reliability of Structures

The core part of this work is based on the reliability theory of structures and it is therefore necessary to introduce the safety concept, basic theory, terms and analytical solutions for the relevant problems.

2.1 Safety Concept Introduction

The purpose of a safety concept is to ensure a minimal overall structural performance to prevent damage, failure or disruption of operational capacity of a structure. There is a number of developed concepts and related structural codes that aim at providing at least the minimal required performance. In order to introduce current civilian codes for bridge assessment and design, an overview of different safety concepts needs to be provided. In general, all concepts compare the numerical relation between the load and the resistance. The overview of various formats is shown in Table 1 [15]. In the current practice, there is a mixture of different approaches to the safety concept as dictated by the explicit needs and environment of each specific profession. The structural engineering community tends to accept the Level 1 approach, semi-probabilistic, as a standard. Semi-probabilistic approach allows for both resistance and load effect variables to be modified by partial factors in order to ensure desirable behavior and performance of the design element/structure. The semi-probabilistic approach recognizes differences in the characteristics of possible elements contributing for resistance or load effect and therefore treats each of them individually. In comparison the conventional deterministic concept, Level 0, recognizes only a single central safety factor and to some respect fails to recognize qualitative differences between the various elements in the limit state equation. Probabilistic approximation, Level 2 approach, utilizes the reliability index, or target reliability, as a preset value that estimates the robustness in the respect of probability of failure. In another words, it predicts the possibility of a single, or multiple events to take place and cause the structure to fail.

Table 1. Safety concept levels [15]

Safety Concept	Level	Reliability Dimension	General Equation
Conventional deterministic	0	Central safety factor γ	$\gamma = R/E \geq$ code γ (*R*=Resistance, E=Load)
Semi probabilistic	1	Partial safety factors for Resistance R (γ_R) and for the Load E (γ_E)	$R/\gamma_R \leq \gamma_S \cdot E$
Probabilistic approximation	2	Reliability index β	existing β - required $\beta \geq 0$
Probabilistic accuracy	3	Probability of failure P_f	permissible P_f - exist. $P_f \geq 0$
Economic optimum	4	Permissible probability of failure P_f, required reliability index β	Optimization reliability

The particular partial factors values used in the Level 1 approach are determined by the process of combining previous experiences with probabilistic calibration. The experiences were collected throughout the time and were intrinsically presented in previous codes. Any probabilistic calibration method would take these codes as a starting point or base, and thus utilize the previous lessons-learnt. However, the calibration process provides the option of adjustment of the necessary parameters or results according to the newest advances in models or desired performance. In the respect of the semi-probabilistic safety concept this is exactly accomplished by the calibration of partial factors.

Properly calibrated safety factors of both load and resistance side should consistently produce reliable performance of various structural elements. Furthermore, the calibration process of the partial factors must not only take in account unique properties of each individual structural element, but it also must focus on statistical properties and uncertainties associated with all variables in the limit state equation. Each variable on the resistance side, such as concrete compressive strength or steel yield strength is associated with statistical uncertainty. Same uncertainty applies to the load portion of the limit state equation. Both permanent and variable load have associated statistical parameters as well as model uncertainties. All these factors play a role in the proper calibration of the partial factors and thus are essential in the semi-probabilistic safety concept. It is therefore necessary to study the structural theory and the basic principles in order to understand the theory behind safety concepts and mathematical operations.

2.2 Structural Reliability Theory and Basic Concepts

General principles of structural reliability are described in EN1990 [45] and ISO 2394 [72]. Additional information is provided by the Joint Committee on Structural Safety – JSCC [74]. In-depth theoretical provisions are provided by the books of SPAETHE [121], SCHNEIDER [111], FABER [52] and Reliability Handbook [70]. The here provided background information on structural reliability is inspired mainly by the above mentioned documents and often refers to them.

The basic concept of structural reliability theory is to secure certain structural behaviour by essentially providing a large enough margin between the loading and structural resistance. The structures should with high probability sustain all the loading during the foreseen life time without losing the capacity or limiting the serviceability, where the probability of occurrence of uncertain events is one the main points under consideration.

2.2.1 Random Variable

FABER [52] notes that the performance on a structural system may be modelled in mathematical physical terms in conjunction with empirical relations. The basic random variables are defined as parameters of the performance evaluation. The random variables must be able to represent the uncertainties that are tied to any quantity in their physical or statistical form and at the same time to represent approximation or idealisation of such quantity in a mathematical operation. Detailed classification of uncertainties is provided in the *Reliability Handbook* [70] and by JCSS [74]. It is generally sufficient to model the associated uncertainties by using a random variable with an assigned distribution function and statistical parameters. Any real random variable can take on any value based on the probability. The probability that a random variable X is less or equal to value x is described by the cumulative distribution function:

$$F_X(x) = P(X \le x) \qquad \text{Eq. 2-1}$$

A full characterization of random variable is sufficiently accomplished by the cumulative distribution function but for some of the problems it is appropriate to use a complementary function [121]:

$$G_X(x) = 1 - F(x) \qquad \text{Eq. 2-2}$$

A derivative of cumulative distribution function is called the probability density function. It describes the probability of X falling within an interval (x_1, x_2) by integrating the surface within this interval:

$$f_X(x) = \frac{dF(x)}{dx} \qquad \text{Eq. 2-3}$$

$$F_X(x) = \int_{-\infty}^{x} f(u)du \qquad \text{Eq. 2-4}$$

$$P(x_1 < X \le x_2) = \int_{x_1}^{x_2} f(x)dx \qquad \text{Eq. 2-5}$$

The area under *f(x)* is equal to 1 and the probability that a random variable take on a specific value is zero:

$$\int_{-\infty}^{\infty} f(x)dx = 1 \qquad \text{Eq.2-6}$$

$$P(X = x_1) = 0 \qquad \text{Eq. 2-7}$$

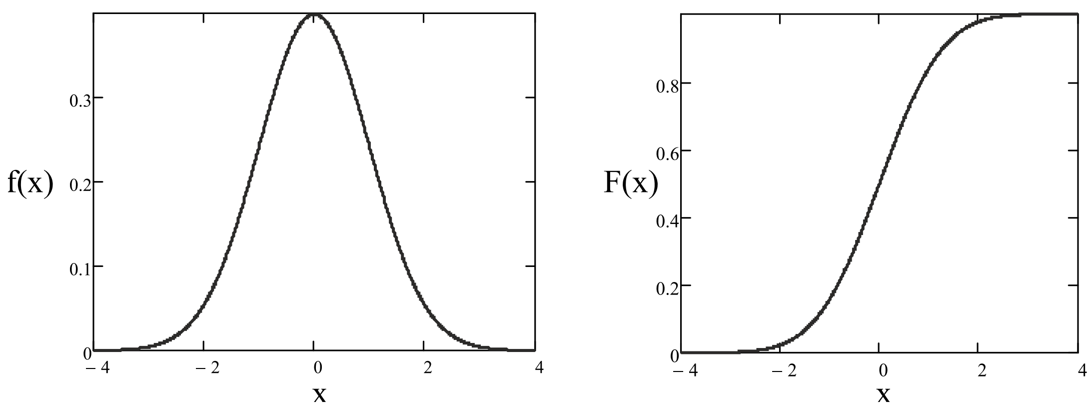

Figure 1: Illustration of probability density function *f(x)* and cumulative distribution function *F(x)*.

Random variables can be additionally described in terms of moments. It however is a simplified characteristic and it does not fully describe the variable [121]. Nevertheless, it can be very useful and the description is in most case sufficient. The *i* moment of a continuous random variable is defined with the help of probability density function as [52]:

$$m_i = \int_{-\infty}^{\infty} x^i \cdot f(x)dx \quad \text{Eq. 2-8}$$

The mean value μ_X or the expected value $E[X]$ of a random variable is defined by the first moment. It corresponds to the centroid of the probability density function. Variance σ_X^2 can be described by the second moment. Standard deviation is simply, by the definition, a square root of the variance.

$$\mu_X = E[X] = \int_{-\infty}^{\infty} x \cdot f(x)dx \quad \text{Eq. 2-9}$$

$$\sigma_X^2 = E[(X - \mu_X)^2] = \int_{-\infty}^{\infty} (x - \mu_X)^2 \cdot f(x)dx \quad \text{Eq. 2-10}$$

$$\sigma_X = \sqrt{\sigma_X^2} \quad \text{Eq. 2-11}$$

It is sometimes more convenient to use the coefficient of variation V_X since it is dimensionless parameter and the mean value does not have to be known, on contrary to standard deviation, in order to visualize the actual variation of the random variable. It is however problematic to use this measure when the mean value is close to zero as the coefficient of variation approaches infinity. For most practical applications, such as expression of the variation of loadings or measurements, it is fully sufficient. The coefficient of variation is defined as:

$$V_X = \frac{\sigma_X}{\mu_X} \quad \text{Eq. 2-12}$$

In practice, it is often necessary to distinguish between numerous distribution functions for description and modelling of uncertainties. The distribution is often dependant on properties of the population which the random variable should represent. An overview of the most common distribution functions in structural engineering is provided in for both loading and resistance parameters [15], [106] and summarized in Table 2.

Table 2: Overview of statistical distribution functions in structural engineering as provided by RACKWITZ [106]

Random variable	Distribution function	Example
Permanent loading	Normal	Self-weight of structure
Variable loading based on a wide range of single values	Extreme (Gumbel)	Extreme values of traffic loading with long measurement duration
Variable loading based on a narrow range of samples	Gamma	Snow loading, or short duration variable loading
Material strength	Normal	Compressive strength of concrete Tension strength of steel
Material strength	Lognormal	Yield strength of reinforcement
Measurements	Normal	Geometrical measurements
Fatigue working life	Weibull	Fatigue strength of steel

A numerical description of the here listed and other often used functions is provided by SCHNEIDER [111]. Some of the functions particularly important for this work are investigated in detail.

Normal distribution

Normal distribution is one of the most important distribution functions as it is frequently used to describe the most common variables such as self-weight, material strengths, geometrical properties or dimensions. It is convenient to use for symmetrically distributed random variables with a small variance defined on an unlimited interval $-\infty < x < \infty$. The normal distribution depends only on two parameters, and that is mean value μ and standard deviation σ. This is symbolically denoted as $N(\mu,\sigma)$.

The numerical definition of cumulative distribution function and the probability density function is given as follows:

$$F_X(x) = \frac{1}{\sigma_X \sqrt{2 \cdot \pi}} \int_{-\infty}^{x} \exp\left(\frac{-1}{2} \left(\frac{x - \mu_X}{\sigma_X} \right)^2 \right) dx \qquad \text{Eq. 2-13}$$

$$f_X(x) = \frac{1}{\sigma_X \sqrt{2 \cdot \pi}} \exp\left(\frac{-1}{2} \left(\frac{x - \mu_X}{\sigma_X} \right)^2 \right) \qquad \text{Eq. 2-14}$$

The standard normal distribution function should be mentioned, since it has a great practical use. It means, that for a standard normal distribution function the expected (mean) value equals to zero and standard deviation is exactly unity. The cumulative distribution function is commonly denoted as $\Phi(x)$ with corresponding density function as $\varphi(x)$.

The numerical definition is given as:

$$\Phi(x) = \frac{1}{\sqrt{2 \cdot \pi}} \int_{-\infty}^{x} \exp\left(\frac{-x^2}{2}\right) dx \qquad \text{Eq. 2-15}$$

$$\varphi(x) = \frac{1}{\sqrt{2 \cdot \pi}} \exp\left(\frac{-x^2}{2}\right) \qquad \text{Eq. 2-16}$$

The following Figure 2 shows the standardized normal distribution *N(0,1)* while the other curves are showing reduced standard deviations. It can be clearly observed that description of reduced standard deviation is narrower.

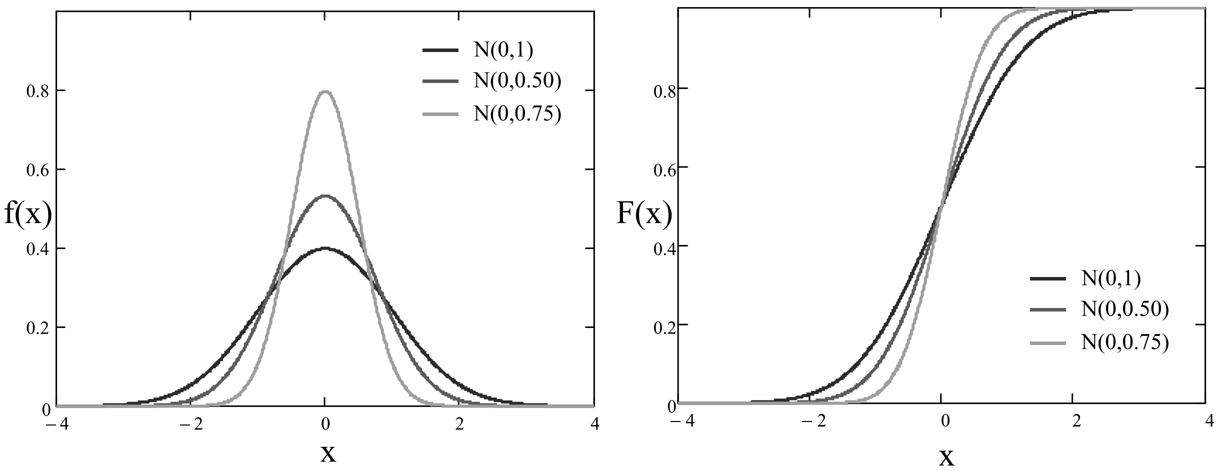

Figure 2: Probability density function *f(x)* and cumulative distribution function *F(x)* for normal distribution *N(μ,σ)*.

Lognormal distribution

Another often used distribution function is the lognormal distribution. A variable *X* has a lognormal distribution if the transformed variable *Y* is normally distributed.

$$Y = \ln X \qquad \text{Eq. 2-17}$$

The main advantage of lognormal over normal distribution is the one-sided distribution over $x_0 < x < \infty$ or $-\infty < x < x_0$ limit interval. In comparison to normal distribution, it might be practical to consider negative values in some cases. Additionally, lognormal distribution can be used to describe an unsymmetrical distribution of the population. A lognormal distribution generally depends on three parameters - mean value μ, standard deviation σ, and skewness ω. This is symbolically denoted as $LN(\mu,\sigma,\omega)$.

Without considering the skewness the cumulative density and probability density function are given as [34]:

$$F_X(x) = \Phi\left(\frac{\ln(x/\xi)}{\delta}\right) \qquad \text{Eq. 2-18}$$

$$f_X(x) = \frac{1}{x\delta\sqrt{2\cdot\pi}}\exp\left(\frac{-1}{2}\left(\frac{\ln(x/\xi)}{\delta}\right)^2\right) \qquad \text{Eq. 2-19}$$

with the following defined parameters:

$$\xi = \frac{\mu_X}{\sqrt{V_x+1}} \qquad \text{Eq. 2-20}$$

$$\delta^2 = \ln(V_X{}^2+1) \qquad \text{Eq. 2-21}$$

Two following Figure 3 quantitatively shows the difference between the normal and lognormal distributions, where the same mean values and two different standard deviations of lognormal distribution are considered. It can be clearly seen, that the lognormal distribution does not yield any negative values and is asymmetric in relation to the standard deviation.

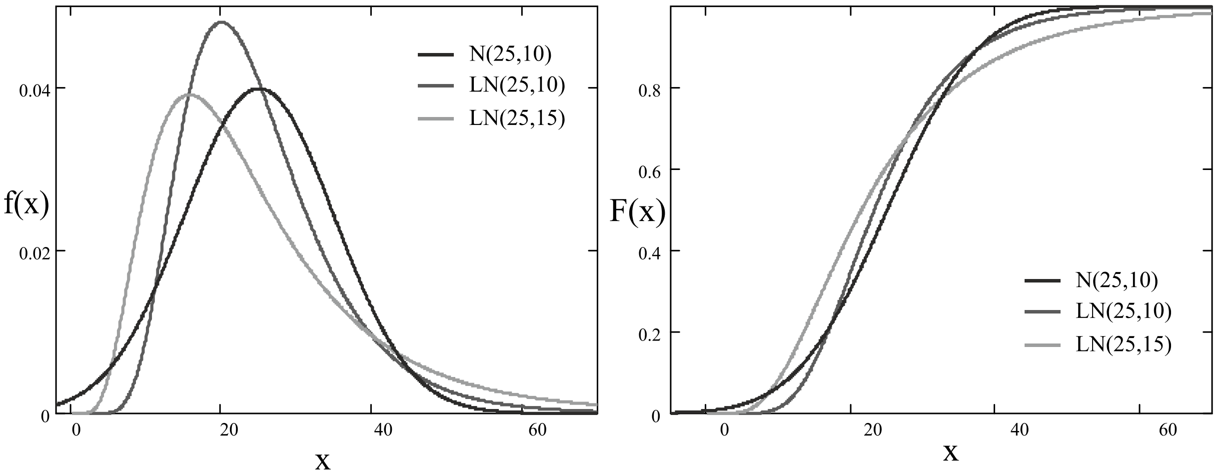

Figure 3: Probability density function *f(x)* and cumulative distribution function *F(x)* for normal *N(μ,σ)*and lognormal *LN(μ,σ)* distribution.

Consideration of skewness ω_X is accomplished by the introduction of lower or upper bound parameter x_0. The transformed variable can be expressed as:

$$Y = \ln|X - x_0| \qquad \text{Eq. 2-22}$$

The lower or upper bound parameter is expressed in terms of mean value, standards deviation and coefficient c, that is turn determined by the value of ω_X:

$$x_0 = \mu_X - \frac{\sigma_X}{c} \quad \text{Eq. 2-23}$$

$$\omega_X = c^3 + 3c^3 \quad \text{Eq. 2-24}$$

$$c = \left(\sqrt{\omega_x{}^3 + 4} + \omega_x\right)^{\frac{1}{3}} - \left(\sqrt{\omega_x{}^3 + 4} - \omega_x\right)^{\frac{1}{3}} \cdot 2^{\frac{-1}{3}} \quad \text{Eq. 2-25}$$

The effect of skewness can be observed in Figure 4. It can provide for even more asymmetric distribution.

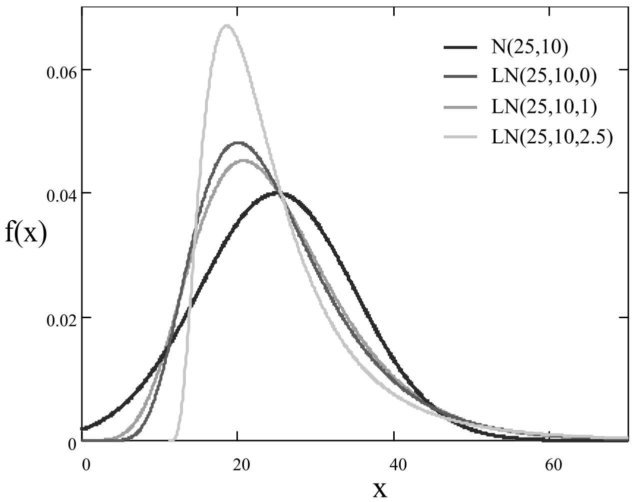

Figure 4: Probability density function *f(x)* and for normal *N(μ,σ)*and lognormal *LN(μ,σ,ω)* distribution.

Other distribution functions

Core of this work is concerned with normal and lognormal distribution. But for illustration and comparison purposes, two relevant distribution functions Gumbel extreme and Gamma are briefly shown. For a numerical definition see SCHNEIDER [111].

Gumbel extreme distribution can be often found describing the distribution of long term variable loading. In comparison to normal distribution it can encompass extreme values of the population and is therefore often used for the description of loading in a long time interval. This distribution has two alternatives for the minimal and maximal values. Since skewness and kurtosis are constant, it is sufficient to describe this distribution with the mean value and the standard distribution – *GUM (μ,σ)* for both min and max [70].

Gamma on other hand, similar in shape to lognormal is a one-sided distribution with a zero lower limit, but with a lower skewness, and can be used for description of for example snow loading. It is sometimes more convenient to use gamma over lognormal when describing geometrical properties or variable loadings do not have a large skewness [70]. It is described by the mean and standard deviation values, denoted in this work as *GAM (μ,σ)*.

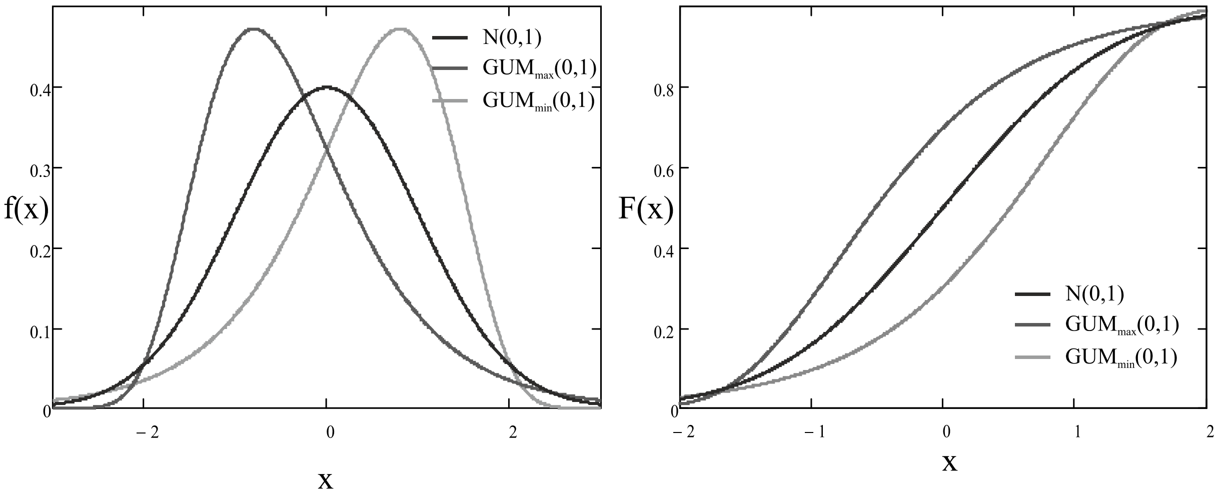

Figure 5: Probability density function *f(x)* and cumulative distribution function *F(x)* for normal *N(μ,σ)* and Gumbel *GUM (μ,σ)* distributions.

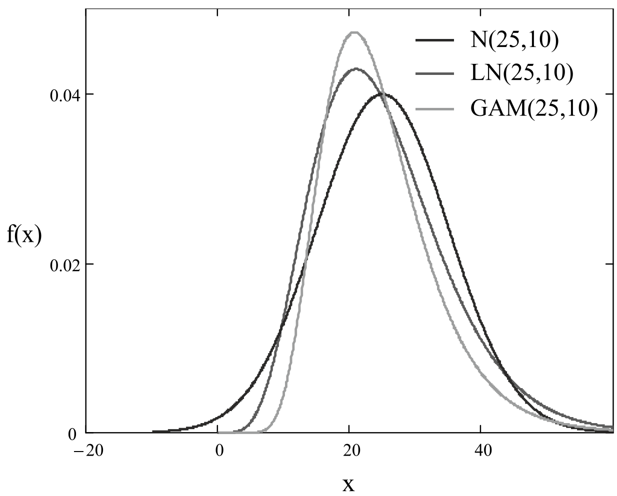

Figure 6: Probability density function *f(x)* for normal *N(μ,σ)* and lognormal *LN(μ,σ)* and Gamma *GAM(μ,σ)* distributions.

2.2.2 Reliability calculations and solutions

The evaluation of relationship between resistance R and load effect E is the fundamental task of a structural reliability calculation. The evaluation should provide an insight whether the structural component under consideration has sufficient resistance to carry the applied loading.

$$R > E \quad \text{Eq. 2-26}$$

Assuming a special case of two fundamental normally distributed variables, the structural safety evaluation of a given component may be accomplished by the so-called margin of safety M. The evaluation of resistance and loading on simple terms yields the following relationship:

$$M = R - E \quad \text{Eq. 2-27}$$

where the moment parameters of M are evaluated as:

$$\mu_M = \mu_R - \mu_E \quad \text{Eq. 2-28}$$

$$\sigma_M{}^2 = \sigma_R{}^2 + \sigma_E{}^2 + 2\rho_{RE} \cdot \sigma_R{}^2 \cdot \sigma_E{}^2 \quad \text{Eq. 2-29}$$

The term ρ_{RE} indicates the correlation between R and E. It may be often in structural engineering assumed that the variables are independent and therefore $\rho_{RE} = 0$. This may be an invalid assumption in geotechnical engineering.

It is practical to express the safety margin in terms of probability of failure. This can be expressed as a probability that the loading exceeds the resistance of a given component or the margin safety takes on a negative value:

$$P_f = P(E > R) = P(M < 0) \quad \text{Eq. 2-30}$$

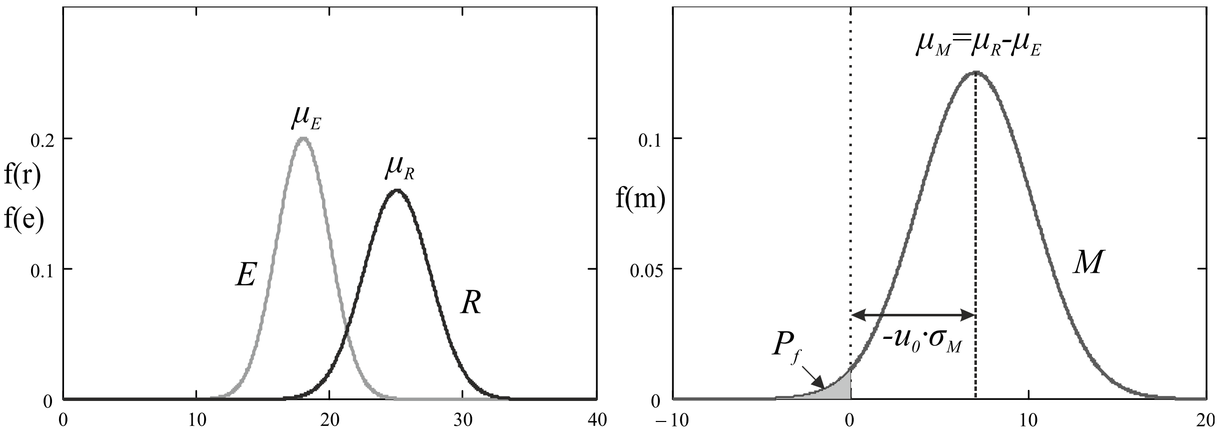

Figure 7: Probability density function f(x) for resistance R and load effect E; and distribution of margin of safety M.

The relationship of probability of failure may be reduced into a simple case of standard distribution function $\Phi(M)$ evaluation and an assessment for the realization $m = 0$. This directly leads to the probability that the safety margin M is negative and gives the probability of failure:

$$P_f = P(M < 0) = \Phi_M(0) \quad \text{Eq. 2-31}$$

The $\Phi_M(0)$ distribution function can be obtained through transformation of variable M into standardized variable U, the necessary realization of u_0 for that determines the value on a standardized distribution function is given as:

$$u_0 = \frac{0-\mu_M}{\sigma_M} = -\frac{\mu_M}{\sigma_M} \quad \text{Eq. 2-32}$$

The probability of failure P_f is then obtained accordingly as:

$$P_f = \Phi_M(u_0) \quad \text{Eq. 2-33}$$

If a normal distribution is assumed, then the term $-u_0$ is generally known as reliability index β. The probability of failure P_f is then generally defined as:

$$P_f = \Phi_M(-\beta) \quad \text{Eq. 2-34}$$

It follows equations Eq. 2-30, Eq. 2-31 and Eq. 2-34 that β can be defined as:

$$\beta = \frac{\mu_M}{\sigma_M} = \frac{\mu_R - \mu_E}{\sqrt{\sigma_R{}^2 + \sigma_E{}^2 + 2\rho_{RE} \cdot \sigma_R{}^2 \cdot \sigma_E{}^2}} \quad \text{Eq. 2-35}$$

Since the variable R and E are assumed as mutually independent, the assessment of reliability index is narrowed down to the evaluation of the distance between the mean value μ_M from the origin in terms of standard deviation σ_M, in another words, it describes how many standards deviations are necessary to reach the mean value from the null point. Figure 7 shows this in detail.

The probability of failure can be conveniently expressed through the reliability index. It must be noted that the reliability index and the corresponding probability of failure are notational terms and are primarily intended for development of consistent design rules rather than quantifying the actual structural failure frequency [106].

The reliability index can be also expressed in terms of probability of failure:

$$\beta = -\Phi^{-1}(P_f) \quad \text{Eq. 2-36}$$

Table 3 provides an overview of the relationship between P_f and β.

Table 3: Relationship between P_f and β

P_f	50	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}
β	0	1.28	2.32	3.09	3.72	4.27	4.75	5.20

However, the considered fundamental case with two independent normally distributed random variables is not always valid due to the nature of structures. More commonly, the reliability analysis of structures may be considered as a function of basic variables described by a vector *X* [121]:

$$X = [X_1, X_2, ..., X_n] \quad \text{Eq. 2-37}$$

and realization of these as *x*:

$$x = [x_1, x_2, ..., x_n] \quad \text{Eq. 2-38}$$

Where for example resistance *R* or loading *E* may be given by a function:

$$Z = f(X) \quad \text{Eq. 2-39}$$

The resulting variable *Z* is a random variable where the characteristics are derived from the set of basic variables *X*. The safety margin can be in turn expressed as:

$$M = R - E = f_1(X) - f_2(X) = g(X) \quad \text{Eq. 2-40}$$

Due to the nature of vector *X*, the margin safety is no longer a simple normally distributed variable, but rather a function. The function g(*x*) is called a limit state function. The structural behavior is described by the limit state equation, which can be essentially evaluated in two outcomes: a state of failure in Eq. 2-41 or a safe state in Eq. 2-42:

$$g(X) = g(X_1, X_2, ..., X_n) \leq 0 \quad \text{Eq. 2-41}$$

$$g(X) = g(X_1, X_2, ..., X_n) > 0 \quad \text{Eq. 2-42}$$

FABER [52] notes that setting g(*X*) = 0 defines a failure hyper surface in the basic variable space. This hyper surface divides the safe state from the failure state by separating all possible realizations of *x* of the basic random variables *X* resulting in failure from the *x* realizations resulting in a safe state. The probability of failure can then be expressed as [52]:

$$P_f = \int_{g(x) \leq 0} f_x(x)dx \quad \text{Eq. 2-43}$$

where $f_x(x)$ is the joint probability density function of the random variables *X*.

The solution of this integration is not a trivial one and is possible only in some specialized cases. Various approximation methods have been developed in past years, but one of the widely applied ones is the First-Order-Reliability-Method or for short FORM.

The core of this method, as originally developed by HASOFER & LIND [60], is the assumption of a linear or linearized limit state with mutually independent normally distributed random variables. The random variables are often not normally distributed and therefore a transformation to standard normal distribution with zero mean and unity variance must follow (Figure 8), this is generally possible for all variables, or is possible at least in some domain. For example, Rosenblatt transformation may be performed [19].

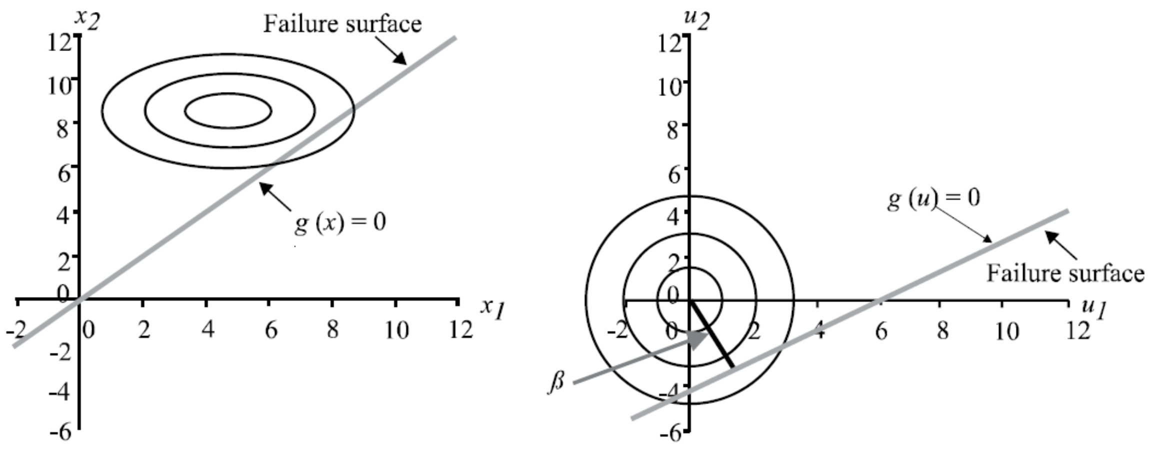

Figure 8: Illustration of the two dimensional case of a linear limit state function and standardized normal distribution of variable *U* [52].

A considered simple case of a linear failure surface g(*u*) = 0 and normally distributed variables in limit state equation leads to a relatively straightforward solution. The transformation of the *R* and *E* variables into normal space is accomplished by:

$$U_1 = \frac{R - \mu_R}{\sigma_R} \qquad \text{Eq. 2-44}$$

$$U_2 = \frac{E - \mu_E}{\sigma_E} \qquad \text{Eq. 2-45}$$

The evaluation of limit state is given by [15], [53]:

$$G = R - E = (U_1 \cdot \sigma_R + \mu_R) - (U_2 \cdot \sigma_E + \mu_E) = 0 \qquad \text{Eq. 2-46}$$

$$G = R - E = (\mu_R - \mu_E) + U_1 \cdot \sigma_R + U_2 \cdot \sigma_E = 0 \qquad \text{Eq. 2-47}$$

and Hesse normal formal for calculating distances in a higher dimension space yields the following solution:

$$-\frac{(\mu_R - \mu_E)}{\sqrt{\sigma_R{}^2 + \sigma_E{}^2}} - \frac{U_1 \cdot \sigma_R}{\sqrt{\sigma_R{}^2 + \sigma_E{}^2}} + \frac{U_2 \cdot \sigma_E}{\sqrt{\sigma_R{}^2 + \sigma_E{}^2}} = 0 \qquad \text{Eq. 2-48}$$

The design point is found as the shortest distance from the origin to the point on failure surface defined by g(u) = 0. This point is obtained by:

$$\beta = \frac{\mu_R - \mu_E}{\sqrt{\sigma_R{}^2 + \sigma_E{}^2}} \qquad \text{Eq. 2-49}$$

the additional factors in Eq. 2-47 indicate the direction of the vector pointing from the origin to the design point u^* [15]. They can be evaluated from the following equations as sensitivity factors denoting the influence of each variable on the reliability index:

$$\alpha_E = \frac{\sigma_E}{\sqrt{\sigma_R{}^2 + \sigma_E{}^2}} \qquad \text{Eq. 2-50}$$

$$\alpha_R = \frac{\sigma_R}{\sqrt{\sigma_R{}^2 + \sigma_E{}^2}} \qquad \text{Eq. 2-51}$$

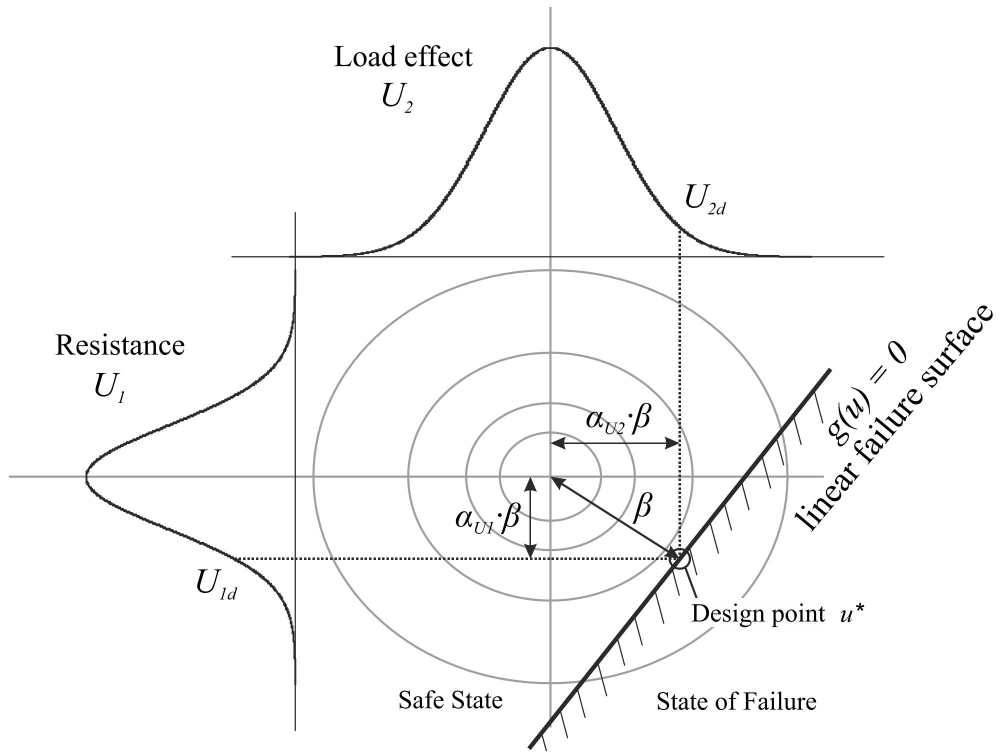

Figure 9: Illustration of the simple solution in a standard normal space.

It can be observed in Figure 9 that the load effect and resistance have specific values at design point and they can be written in a standard normal space as:

$$U_{1d} = -\alpha_{U1} \cdot \beta \qquad \text{Eq. 2-52}$$

$$U_{2d} = \alpha_{U2} \cdot \beta \qquad \text{Eq. 2-53}$$

It follows the transformation back from the standard space into the original space in order to obtain the realizations for load effect *E* and resistance *R* at which failure occurs with the highest probability:

$$r^* = -\alpha_R \cdot \beta \cdot \sigma_R + \mu_R \qquad \text{Eq. 2-54}$$

$$e^* = \alpha_E \cdot \beta \cdot \sigma_E + \mu_E \qquad \text{Eq. 2-55}$$

This is in particular important for development of partial factors, where is evaluated the difference between characteristic and design value of variables, here are design values denoted as r^* and e^*.

A general solution of either more complicated or non-linear limit states is not as trivial as just shown and commonly requires advanced approximation techniques. A linearization of a failure surface at the design point in a normalized space is a suggested solution for non-linear limit state. The linearized failure surface, as schematically shown in Figure 10, is at point u^* given by:

$$g'(u) = 0 \qquad \text{Eq. 2-56}$$

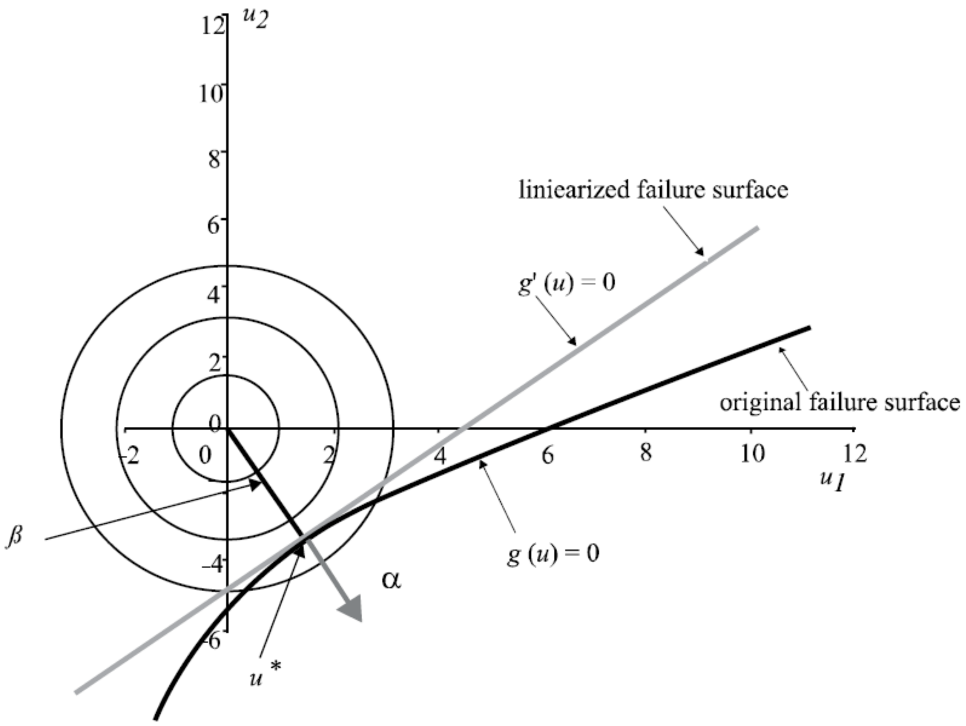

Figure 10: Illustration of the linearization in a standard normal space [52].

Finding the location of *u** is reduced to the iteration of following optimization problem [52]:

$$\beta = \min_{u \in \{g(u)=0\}} \sqrt{\sum_{i=1}^{n} {u_i}^2} \qquad \text{Eq. 2-57}$$

where β is again the point on the linearized failure surface closest to the origin point. The result is exact if the failure surface is linear, approximated in this case of non-linear failure surface. It must be noted that the optimization solution is not limited to non-linear failure surface, but also to a general limit state that may not be considered on simplified bases by Eq. 2-43 to 2-49.

The iteration will generally converge if the limit state is differentiable. It provides as a result the design point *u** as well as the reliability index β that is on the outward normal vector α pointing from the origin point to the failure surface, with its components representing the sensitivity factors. They again indicate the influence of each component from the limit state on the reliability index. In essence, it follows the results of a linear limit state. All of the mentioned assumptions for the reliability analysis of structures as a function of basic variables described by X are not always realistic, but the shown methods are proven to deliver consistently accurate results for most practical applications. For more detail regarding reliability solutions refer to the works mentioned in the introduction.

2.2.3 Definition of reliability based partial factors

An example consideration of the previously investigated fundamental problem of two mutually independent normally distributed variables yields a limit state governed by a simple expression with the reliability index calculated based on the properties of resistance and loading:

$$M = R - E \qquad \text{Eq. 2-58}$$

$$\beta = \frac{\mu_M}{\sigma_M} = \frac{\mu_R - \mu_E}{\sqrt{{\sigma_R}^2 + {\sigma_E}^2}} \qquad \text{Eq. 2-59}$$

The semi-probabilistic design concept can be easily explained in Figure 11 with schematically shown resistance R and load effect E. Refer to Figure 9 for a display of design values in standard normal space and Eq. 2-54, Eq. 2-55 for transformation into the original variable space. The reliability (or probability of failure) is given by the distance between the R and E mean values and by their second moment properties. The larger the distance between the two peeks, the larger reliability index β and respective lower probability of failure can be reasonably expected.

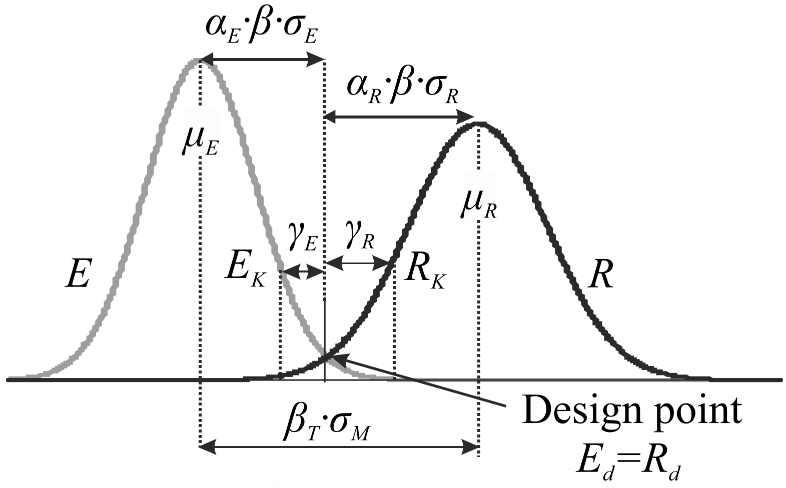

Figure 11: Semi-probabilistic safety concept.

The currently valid semi-probabilistic safety concept in EN 1990 [45] is based on the use of partial factors for resistance and load effect. The already introduced limit state equation can be rewritten as:

$$\gamma_R R_d - \left(\gamma_G G_d + \gamma_Q Q_d\right) = 0 \qquad \text{Eq. 2-60}$$

where γ_R stands for resistance partial factor, R_d design resistance, γ_G permanent action partial factor, G_d design permanent action, γ_G variable action partial factor and Q_d design variable action.

Generally, the partial factors in design codes are composed of a partial factor accounting for the model uncertainty and is reliability-based partial factor accounting for intrinsic properties of the random variables with all the related uncertainty. It is particularly important for this work to investigate the definition of reliability based partial factors. The previous section introduced some of the solutions for structural reliability problems; this section aims at introducing some of the principles regarding the partial factors inherently built into Eurocodes.

The design value of resistance and load effect is given in EN 1990 [45] as:

$$P(E > E_d) = \Phi(\alpha_E \cdot \beta_T) \qquad \text{Eq. 2-61}$$

$$P(R \leq R_d) = \Phi(-\alpha_R \cdot \beta_T) \qquad \text{Eq. 2-62}$$

The α-values are in further text defined as negative for loading and positive for resistance to clearly differentiate the variables and to maintain continuity of design value definitions. The following approximation is defined in EN 1990 [45]:

- $\alpha_E \approx$ -0.7 for the leading action
- $\alpha_R \approx$ 0.8 for the secondary action

but only as long as the following equation remains valid [45]:

$$0.16 < \frac{\sigma_E}{\sigma_R} < 7.6 \quad \text{Eq. 2-63}$$

This result in the following definition of design values:

$$P(E > E_d) = \Phi(-0.7 \cdot \beta_T) \quad \text{Eq. 2-64}$$

$$P(R \le R_d) = \Phi(-0.8 \cdot \beta_T) \quad \text{Eq. 2-65}$$

Mainly three distribution functions are considered for description of resistance and loading and that is normal, lognormal and Gumbel. The overview of respective design values corresponding to the distribution function is provided in Table 4.

Table 4: Design values according to EN 1990 [45]

Distribution	Design value
Normal	$\mu - \alpha \cdot \beta \cdot \sigma$
Lognormal	$\mu \exp(-\alpha \cdot \beta \cdot V)$
Gumbel	$u - \frac{1}{\alpha} \ln(-\ln \Phi(-\alpha \cdot \beta))$ $u = \mu - \frac{0.577}{\alpha}; \alpha = \frac{\pi}{\sigma\sqrt{6}}$

From other perspective, if a certain structural behaviour has to be secured in terms of probability of failure, the distance between the two peaks can be set by target reliability index β_T. This means, that a target reliability β_T dictates either the maximum expected loading if resistance is known, or the value of resistance to carry the design loading. It was already determined how to describe the design values for both resistance and loading, it is also schematically shown in Figure 11. Slightly modified expression in terms of the required β_T dictates then the necessary values for design (assuming normal distribution) as:

$$X_d = \mu_X - \alpha_X \cdot \beta_T \cdot \sigma_X \quad \text{Eq. 2-66}$$

Since the resistance or load effect are normally expressed as characteristic values during the design that correspond to a certain x quantile values (generally regarded 5% for resistance and 90% to 98% for loading). Statistically speaking this means, that a selected characteristic will not be higher or lower with certain probability during the whole working life or reference period.

The characteristic value can be expressed with the help of standard normal cumulative distribution function $\Phi(x)$ as:

$$R_k(x) = \Phi^{-1}(x) \cdot \sigma_R + \mu_R$$ Eq. 2-67

$$E_k(x) = -\Phi^{-1}(x) \cdot \sigma_E + \mu_E$$ Eq. 2-68

Figure 12 exemplarily shows the 5% and 95% quantile values.

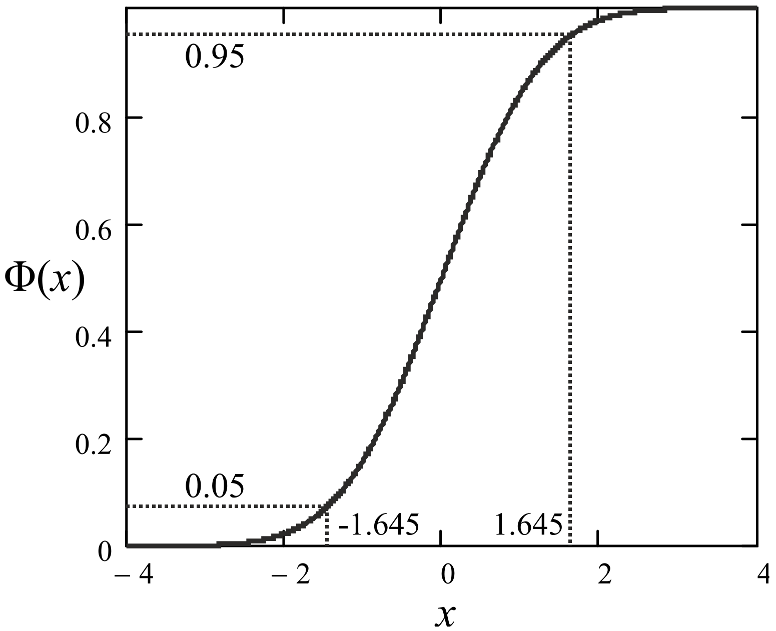

Figure 12: 5% and 95% quantile values.

The partial factors for the considered case can be expressed as the distance between the characteristic value entering the limit state and the required design value securing the desired structural performance. In essence, the partial factors reduce the characteristic value of resistance and increase the characteristic value of load effect so that both are equal at the design point. The numerical expressions are given as:

$$\gamma_R = \frac{R_k(x)}{R_d} = \frac{\Phi^{-1}(x) \cdot \sigma_R + \mu_R}{\mu_R - \alpha_R \cdot \beta_T \cdot \sigma_R}$$ Eq. 2-69

$$\gamma_E = \frac{E_d}{E_k(x)} = \frac{\mu_E - \alpha_E \cdot \beta_T \cdot \sigma_E}{-\Phi^{-1}(x) \cdot \sigma_E + \mu_E}$$ Eq. 2-70

It can be clearly observed that the particular value of partial factor is highly dependent on the selected target reliability index and stochastic properties of considered random variables for both resistance and load effect partial factors. Only normal distribution is shown, naturally, other distribution functions can be considered as well given their probability and cumulative density functions.

An assumption of characteristic values equal to mean values, as correct for example of self-weight or measured variables, yields to following simplified expressions for normally distributed variables:

$$\gamma_R = \frac{R_k}{R_d} = \frac{\mu_R}{\mu_R - \alpha_R \cdot \beta_T \cdot \sigma_R} = \frac{1}{1 - 0.8 \cdot \beta_T \cdot V_R} \qquad \text{Eq. 2-71}$$

$$\gamma_E = \frac{E_d}{E_k} = \frac{\mu_E - \alpha_E \cdot \beta_T \cdot \sigma_E}{\mu_E} = 1 + 0.7 \cdot \beta_T \cdot V_E \qquad \text{Eq. 2-72}$$

Since such definition is not only valid for normally distributed variables, similar assumption regarding the characteristic values yields for lognormally distributed variables following definitions of partial factors:

$$\gamma_R = \frac{R_d}{R_k} = \frac{\mu_R}{\mu_R \exp(-\alpha_R \cdot \beta_T \cdot V_R)} = \frac{1}{\exp(-0.8 \cdot \beta_T \cdot V_R)} \qquad \text{Eq. 2-73}$$

$$\gamma_E = \frac{E_d}{E_k} = \frac{\mu_E \exp(-\alpha_E \cdot \beta_T \cdot V_E)}{\mu_E} = \exp(0.7 \cdot \beta_T \cdot V_E) \qquad \text{Eq. 2-74}$$

3 Bridge Engineering and Structural Codes

Modern structural codes are often divided according to the structure type such as building, bridge or a tunnel and the purpose of the structure is decisive for the code development. Additionally, different construction materials (concrete, steel, timber, etc.) dictate the contents of codes. In essence, structural codes are aimed at primarily securing certain performance by providing for a sufficient resistance of structural elements to the expected applied loading and at maintaining the serviceability. The civilian community has been developing methods and standards for design of new bridges for extensive periods of time. Many concepts are well thought of and have been carefully developed over the years relying on the experiences and performance of completed structures.

A potentially different approach of structural codes has been meanwhile somewhat standardized across the structural community through cooperation and sharing of ideas, experiences and results. It is necessary to briefly study look at the current structural codes and scientific concepts in order to understand their background before any proposal for military traffic is made. Especially the bridge loading and safety, as they are regarded by the Eurocode [45] - [47], are important due to their ties to military bridge assessment in Europe. More importantly, the developments in assessment of existing bridges are particularly attractive.

3.1 Current Design Codes

When compared, the format of most modern structural design codes is quite similar in the respect of using the semi-probabilistic safety concept and even allowing the probabilistic approximation in some special cases. The particular values of current partial factors are commonly obtained through a calibration of previous codes, which were in large part based on the mentioned collected experiences of engineers and previous practice, and setting up desired performance levels in terms of reliability index. For details see Section 2.1. For calculation purposes and quantification of the performance, the characteristic load effects are increased, while the resistance side of the limit state equation is decreased by the use of partial factors. This increases the safety margin between the characteristic values in the limit state equation and fundamentally decreases the probability of failure of a given element or a structure.

The definition of safety format in EN 1990 [45] is provided in Table 5. For illustration purposes, AASHTO [1] is also included to demonstrate a different structural standard. It can observed, that both codes follow principally the same safety concept and evaluate the difference between the factored load and resistance in a limit state equation. The manner of the limit state evaluation is somewhat different for the listed two, when EN 1990 [45] uses the so-called combination coefficients for a combination of different actions and AASHTO [1] tends to rather evaluate many different limit states separately.

Table 5. Example of structural codes for bridge design

EN 1990 [45]	AASHTO [1]
Limit State function (ultimate limit state): $E_d \leq R_d$ where $E_d = \sum \gamma_g G_k + \gamma_Q Q_{k1} + \sum \gamma_{Qi} \psi_{0i} Q_{ki}$ $R_d = R\left(\frac{\alpha \cdot f_{ck}}{\gamma_c}, \frac{f_{sk}}{\gamma_s}, ...\right)$	The basic limit state equation: $\sum(\eta_i \cdot \gamma_i \cdot Q_i) \leq \phi \cdot R_n$
E_d Design force R_d Design resistance γ_G Partial factor for permanent actions G_k Permanent load γ_{Q1} Partial factor for dominant variable actions Q_{k1} Dominant variable load γ_{Qi} Partial factor for other variable actions Ψ_i Combination coefficient Q_{ki} Other variable load R Resistance function f_{ck} Concrete compressive strength f_{sk} Steel yield strength γ_c Partial factor for concrete γ_s Partial factor for reinforcing steel	η_i Limit state load modifier factor for ductility, redundancy and importance γ_i Load factor Q_i Load (dead load, live load, etc) ϕ Resistance factor R_n Nominal or ultimate resistance

Additionally, as can be observed in Table 6, the respective partial factor values defined in each of the shown codes vary from each other. Each design code has its specifics when it comes to the modeling of variables in the limit state equation to include characteristic value and uncertainty, reference period, performance level or the target reliability index. The calibration then naturally results in a set of numerically different partial factors. The differences between the specific values of partial factors are observable between almost any separately developed structural codes. It is however particular apparent when the codes from Europe and North America are compared.

Table 6. Example Partial Factors in Design Codes

Variable	Eurocode ULS		AASHTO Strength I	
Permanent / Dead Load	γ_G	1,35	γ_{DL}	1,25
Variable / Live Load	γ_{Q1}	1,35#	γ_{LL}	1,75
Concrete	γ_c	1,50*	φ_c	0,75
Reinforcement	γ_s	1,15*	φ_s	0,90

For LM1 Load Model

* Values are in denominator, see limit state function

It should be noted, that there are also differences in the National Annexes to Eurocode. The particular modifications according to each nation are however marginal and in many cases insignificant for the overall performance, as the loading models and performance level often remain constant and generally only the design approach differs. It needs to be however mentioned, that German National Annex does not allow the use of 6.10a and 6.10b equations for combinations of different leading action effects and only a single limit state is evaluated.

3.2 Assessment of Existing Bridges

The codes for design of new structures are often based on conservative assumptions regarding the resistance and expected loading. However, there are considerable differences between the design and the assessment of existing structures and a significant effort has been aimed in the last decade into the improvement of classification methods of existing structures. Identification of the differences between the approach to design and assessment may be utilized in the development of a suitable safety concept for military loads on existing concrete bridges. The points that are in particular important for bridge assessment under consideration of well-defined loading are:

- fundamental format of safety concept,
- target reliability for the assessment,
- definition of the variable load model – traffic model to include stochastic properties and uncertainty.

The safety format for the assessment dictates the approach and the required input for the numerical quantification of the loading and resistance. The assessment of existing bridges generally follows the format of design codes. The target reliability levels recommended in various national and international documents for both new and existing structures are inconsistent in terms of the recommended values and the criteria according to which appropriate values are to be selected. Specification of the target reliability levels is required for the probabilistic assessment of existing bridges. In addition, the target reliabilities can be used to modify the partial factors used in a deterministic assessment [31], [83], [117].

It is widely recognized [119], [135] that the reliability assessment of existing bridges differs from design of new structures in a number of aspects including:

- Increased safety levels usually involve more costs for existing bridges than for new bridges.
- The remaining working life of existing bridges is often different from the standard design working life of about 100 years assumed for new bridges.
- Information on actual structural conditions may be available for assessment (inspections, tests, measurements).
- Target reliability can be modified to reflect the existing nature of the bridge and associated cost of strengthening.

- Application of advanced structural analysis techniques accounting for additional reserves in load carrying capacity may be justified by excessive upgrade cost.
- Updating variable load models for site-specific situations may improve estimates of the applied loading.
- Proof load testing of existing bridges may serve to verify the resistance models.

At present, existing bridges in Europe are mostly verified using simplified procedures based on the partial factor method commonly applied in design of new bridges [86]. Such assessments may often be conservative and therefore may lead to expensive upgrades or bridge replacement. More realistic and at the same time more demanding verification of actual performance of existing bridges can be achieved by modifying the respective partial factors to recognize existing nature of the bridge, or by employing full probabilistic methods when uncertainties of basic variables are described by appropriate stochastic models. It is often a case, that a bridge assessed using traditional techniques according to design codes yields unsatisfactory performance, but is actually able to carry the loads safely without requiring strengthening or replacement when investigated in more detail using advanced techniques. The economics of the advanced approach must be recognized.

A brief overview of the current status regarding the assessment of existing bridges from the perspective of national standards and up-to-date research is provided further in this section. Currently, the Eurocode does not recognize the differentiation between the design and the assessment of bridges. But there are some advanced guidelines or research projects proposing the changes to the established codes and practice. A number of countries developed their national codes and guidelines for the assessment of existing bridges, as provided by the comprehensive overview of WISNIEWSKI [135]. This practice is demonstrated the best by the Canadian CSA-S6-06 [29] and Danish Guideline [108], [109]. On the national level in Europe, a number of research projects evolved into codes and guidelines – Austria [101] , Netherlands [94], UK [62], and Switzerland [112].

Following the lead of the few pioneers, a number of research projects have been completed in Europe to include COST 323 [36], COST 345 [37], ARCHES [10], BRIME [18] and SAMARIS [110] as noted by KOTES & VICAN [79]. WISNIEWSKI [135] hopes, that it could advocate a development of a new Eurocode for Bridge Safety Assessment. This standard should recognize the particular characteristics of bridge assessment and potentially would extend the current design practice by allowing advanced methods of assessment and reach more economic solutions.

The common idea behind establishment of these advanced codes is the optimization of target reliability index to reflect the relationship between cost and safety and utilization of better defined permanent and variable loading, as could be the form of legal trucks or measured site specific loading.

This results in two main approaches for the assessment of existing structures:

- modification of partial factors for semi-probabilistic assessment,
- full probabilistic assessment.

The advantage of modified partial factors is the possibility of introducing well known limit state concept from design codes. This simplifies the assessment by only introducing the calibrated partial factors according to the modified target reliability index and stochastic models. In comparison, the full probabilistic assessment requires an employment of structural reliability theory along with advanced methods such as FORM or Monte-Carlo methods and places higher burden on the engineer in terms of expertise and time consumption.

It is therefore proposed to introduce a multi-level procedure for the evaluation of the existing capacity of bridges. It essentially dictates the use of standard semi-probabilistic methods with partial factors in a first step. If the capacity is sufficient to safely carry the loads, the analysis is finished. If the capacity is not adequate during the initial check, additional techniques of assessment may by employed in accordance to the increased level of required expertise, data and involvement as shown in WISNIEWSKI ET AL. [135]. Such multi-level process is allowed in the newly published Guideline for Recalculation of Bridges in Germany [27]. It must be noted, that not all levels have been fully defined at this point and especially the ultimate level lacks the full description about the application of structural reliability theory for probabilistic assessment or the application of nonlinear methods. MALJAARS ET AL. [86] proposed a four level assessment of existing highway bridges with specific description of each:

- Level 1 – Partial Factors and Load Reduction Factors for Existing Infrastructure; it makes use of modified partial factors based on the reduced target reliability levels on account of economic and human safety.
- Level 2 – Current Use of the Structure; where the actual loading conditions are considered accounting for the limiting geometry or shorter reference period.
- Level 3 – Design Stress Based on Measurements; the application of WIM technology [36] along with load cells in order to quantify the loading and the response.
- Level 4 – Full Probabilistic Assessment; numerical methods such as FORM or Monte-Carlo are applied along with the guidance of JCSS Probabilistic Model Code [74].

It should be noted that besides the mentioned work for multi-level assessment of existing bridges, the approach for a case-specific modification of partial factors or advanced methods of structural resistance and risk assessment has been further pursued by a number of scientist and engineers. The focus has been to a large extent on resistance of the structures. For example, FISCHER [53] attempted in his thesis to modify the partial factors for existing concrete structures reflecting the existing nature and additionally taking in account various ratios of permanent to variable loading. VAL & STEWART [129] considered bridges and buildings in terms of resistance and capacity reduction factors. Their proposed method with Bayesian statistical approach [95], which can systematically account for information obtained prior to inspection and during the inspection, has been used for updating of characteristic resistance and the selection of partial factors. The resistance of existing bridges has been largely investigated by BRAML [15] where stochastic models for resistance were developed to represent an existing damage of bridges. A solution is offered based on the reliability evaluation and in terms of required reduction of the allowable loading. An actual structural resistance can be much better determined with a targeted scheme for monitoring of critical components as provided by KOHLBREI [78] and may lead to an economically sound decision regarding the risk management. Partial factors for existing structures are investigated by MOSER ET AL. [92] with a goal to develop a method for adjustment due to collected stochastic models of the properties of historic and new structures with additionally reduced reliability levels. On the national level of building codes and factor calibration should be mentioned the work by VROUWENVELDER & SIEMENS [133] in Netherlands and SØRENSEN [120] in Denmark. Extending over the theoretical approach to the structural assessment, a detailed methodology for service life oriented design, construction and assessment of structures is provided in *Betonkalender 2013* [4].

Loading of bridges has been certainly investigated as well. Ghosn ET AL. [56] for example investigated the procedures of AASHTO LRFR [2] specifications and developed modified load factors for load rating in accordance to regulations of NYSDOT (New York State Department of Transportation) by essentially assessing the remaining service life of bridges and by utilizing actual traffic measurements from Truck Weight-in-Motion data. Similar, but somewhat different approach is demonstrated by accepting well-defined variable loading based not on a WIM data sample, but by defining certain permit vehicles or vehicles not conforming to the design standards, as could the case of heavy construction vehicles. This practice then establishes common classes of vehicles that are used for the re-evaluation of the capacity. A set of legal vehicles that produce considerably less loading in comparison to the design load model is used for so called load rating according to the above mentioned LRFR [2]. Permit vehicle routing is also accepted for example in Denmark [98] and Spain [32].

It can be seen that a lot has been accomplished in the field of bridge assessment. A number of research projects were carried out and even structural codes have been established. However, up to this point no work has been accomplished regarding the military loading. In particular the idea about accepting well-defined loading and developing a set of partial factors is attractive for the safety concept regarding the military vehicles crossing civilian bridges. Due to their properties, the vehicles can be regarded as a well-defined variable loading, moreover, STANAG introduces a number of classes of vehicles, or one could argue, a set of legal permit vehicles. It is the aim of this work to apply the reached advances to the military traffic and to develop a suitable safety concept.

4 Safety Concept for Military Bridge Assessment

The focus of this work up to this point was aimed at the civilian community and the developments in the field of existing bridge assessment. It is actually a frequent phenomenon that bridges built and maintained by civilian authorities are utilized by military traffic. These bridges are located not only in home and allied countries, but also in foreign theatres of operation. Military bridge assessment is no different task judging from the structural engineering point of view; the term military only refers to an assessment performed for the loading represented by actual the military loading. It should be noted, that sometimes the recalculation of bridges is performed with an equivalent civilian loading representing the military traffic [63], this is however problematic due to the reasons further investigated here. This work is aimed at the bridge assessment where the variable loading is represented by the military vehicles.

As any structural engineering calculation, the military bridge capacity assessment requires properly defined safety concept that assures required minimal safety and sufficiently reliable outcomes. A safety concept for the capacity assessment of existing fixed civilian bridges under military loading has not been fully defined by any of the existing military standards or operation procedures, although there has been an increased need at the NATO international level for its establishment [124].

Within NATO countries, the current available document for the assessment of both civilian and military bridges is STANAG 2021[1] [4]. This regulation is a NATO Standardization Agreement and provides guidelines on the assessment of military vehicles, bridges, rafts and ferries. It is a largely general document that aims at establishing common rules among military engineers within NATO. However, in the respect of national interests, it does not set nor requires any specific procedures or concepts for the capacity assessment itself, although in order to ensure a consistent level of safety any used methodology should adhere to the guidelines and minimum criteria outlined in Paragraphs 8 - *General parameters for military load classification of all bridges* and Paragraph 9 - *Military Load Classification of civilian fixed bridges* regulating the general parameters for military load classification, crossing conditions, loading requirements and loading situations. The standard sets forward the specific requirements for a safety factor in Paragraph 8e:

> “A safety factor appropriate to the bridge type and mission must be included in the consideration when determining a bridge rating. The safety factor should reflect a high degree of confidence for the bridge under specific loading levels and frequencies and consider both the static and fatigue life characteristics of the bridge. Due to the fact each country has its own procedure and safety factors, no specific method will (be)[2] imposed.”

[1] Further referenced as STANAG in this text
[2] Author´s correction to referenced text

The problem that dwells within the properly defined safety format is exactly in the point, that the standard does not specifically deliver any explicit formats of safety concept or any values to be used. Since there are no specific methods imposed, it is overall generally accepted practice for engineers to utilizing their national bridge codes in the framework of bridge assessment for military vehicles.

Currently most of the bridge codes in effect are based on the semi-probabilistic format (see Section 3.2). For those NATO nations with the implemented Eurocodes, the safety concept is defined by the EN 1990 [45] (in Germany regulated by ARS22/2012 Appendix 2 [8]) and specific procedures for bridge design and assessment are provided in detail by EN 1991-2 and EN 1992-2. Disregarding the fact that the Eurocodes currently do not supply any provisions concerning the assessment of the existing bridges, the EN 1991-2 [46] and EN 1992-2 [48] are specifically used as guidance for structural calculations concerning the assessment of reinforced concrete bridges under military loading due to the lack of different provisions. It further detail, it means that in the frame work of semi-probabilistic military bridge assessment all partial factors for resistance γ_M and for permanent γ_G and variable loading γ_Q are taken from the current standards.

There are fundamental differences in the treatment of bridges under either civilian or under military traffic and identification of these differences is in the particular scope of this work. It is problematic to adapt Eurocodes and their National Annexes, since they have never been calibrated for the assessment of existing bridges under military loading. Fundamental differences between the civilian and military approach are summarized below:

1. The civilian loading (based on observations on European highways [25]) is described rather generally by time variant loading models developed to represent the complete actual and predicted traffic by extrapolating the measured data. The military loading is assigned to a defined time-invariant MLC and therefore the expected traffic can be captured more accurately.

2. Civilian codes usually assume design life ranging from 50 to 100 years. For military needs such time frame is in many cases impractical as it depends on a number of factors, such as location, strategic and tactical significance or purpose of assessment. These conditions dictate the expected time frame, which is in many cases significantly less than 50 to 100 years. For emergency or crisis situation the time reference can be even regarded in days or weeks.

3. Dynamic effects, accounting for the interaction between passing vehicles and bridge superstructure, are included in traffic models in current bridge codes [24]; no dynamic allowances are provided in STANAG, since the vehicles are measured as static loads.

4. The characteristic value of civilian traffic load corresponds a 1000-year return period (EN 1991-2 [4]) while generally a nominal (mean) value is considered for military vehicles as defined in STANAG; considerable reliability margin is thus included already in the characteristic value of civilian traffic load.

It is therefore inconsistent to use partial factors intended for civilian traffic when assessing bridges under military loading. In fact, the statement in STANAG that the safety factor should reflect high degree of confidence for the appropriate mission and bridge type under the specified loading suggests the need for an explicitly stated safety format, especially because the differences between civilian and military traffic yield EN 1991-2 [46] and EN 1992-2 [48] incompatible for the military loading.
The following sections of this work aim at investigating the methods of military load classification, identification of specific characteristics of military vehicles when compared to civilian traffic and most importantly at defining a proper safety format with for the assessment of existing bridges carrying military traffic. A careful review of existing safety concepts and investigation of the relevant parameters reveals the possibility of modifying the parameters relevant to the military bridge assessment. It is proposed in this work to adapt the semi-probabilistic concept from EN 1990 [45] and to modify the relevant partial factors for a use in the ultimate limit state in order to reflect the specifics on military traffic and the nature of existing bridges as suggested by LENNER AT AL. [84]. Such concept will deliver the continuity of structural engineering calculations according to the current codes while allowing to take the particular aspects of military in account.

4.1 Military Load Class

It is necessary to study the process of military load classification as described in STANAG in order to understand the background of the safety format proposal. Somewhat similar to the legal truck concept, STANAG operates with prescribed procedures defining the military loading. It is accomplished by the means of a vehicle classification in one of the so-called Military Load Classes[3]. The MLC vehicle classification is a standardized procedure enforced and practiced by all the NATO members. The aim of STANAG and vehicle classification is to provide “a standard method of enabling bridges, ferries, rafts (including their landing stages) and laden vehicles to be allocated a MLC number indicating the relationship between the load carrying capacity of the former and the effect produced by the latter” [93].

STANAG defines thirty-two different MLCs – sixteen classes for wheeled vehicles and sixteen for tracked vehicles. Each MLC is represented by a hypothetical vehicle, which is defined by the axle weights and axle spacing in the case of wheeled vehicles, and by the total weight and length in the case of tracked vehicles. Figure 13 shows an example of MLC 40 for both wheeled and tracked vehicles. In addition, wheeled vehicles have defined maximum single axle load. The mass in "short tons" (907 kg) of each tracked hypothetical vehicle is chosen as the numeric MLC, but the mass of the wheeled hypothetical vehicle is slightly different from its MLC number. Figure 14 shows the MLC relationship to the metric ton. The range of hypothetical vehicles varies from the lightest MLC 4 to the heaviest MLC 150. Table 7 provides an overview of all hypothetical MLCs as defined by STANAG.

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[3] Further referenced as MLC in this text

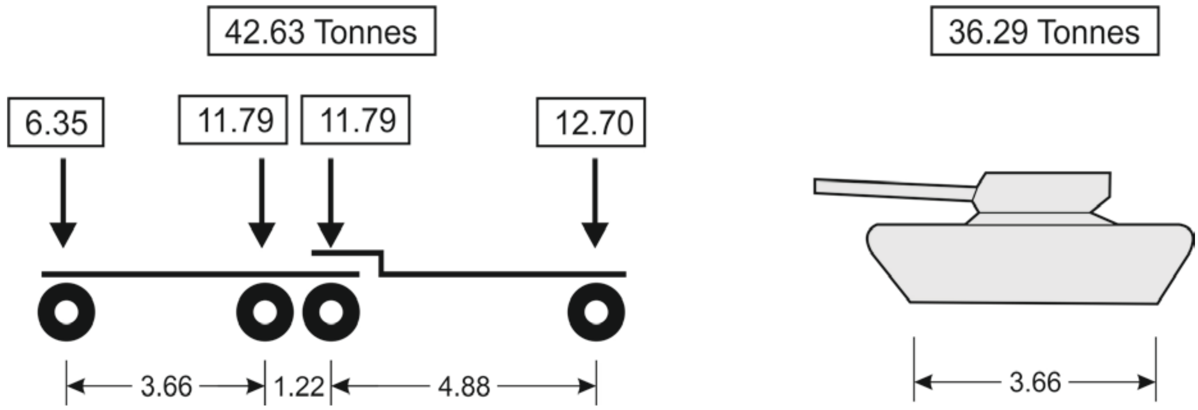


Figure 13: Example Definition - Wheeled and Tracked MLC 40 [93].

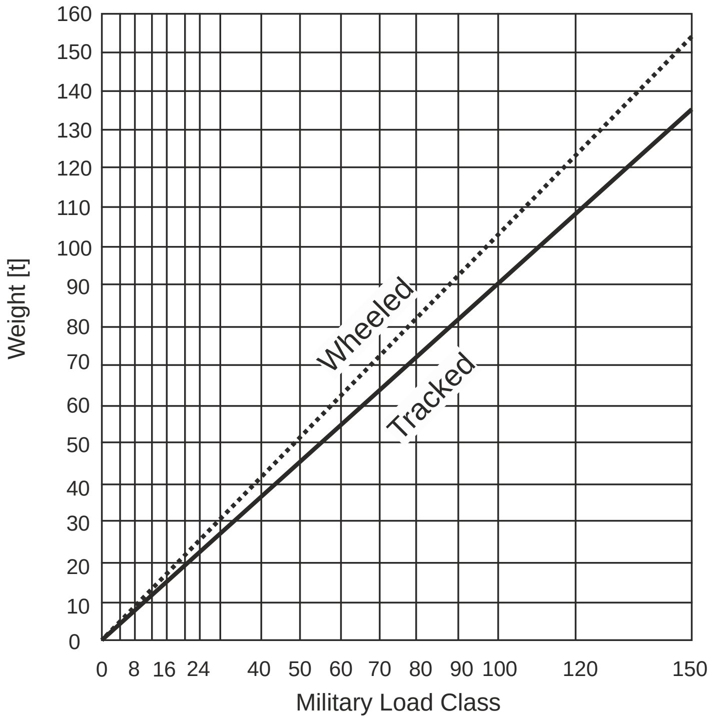


Figure 14: Comparison of total vehicle weight for tracked and wheeled vehicles [17].

STANAG additionally defines loads set up by either hypothetical vehicles or by the maximum single-axle load. This is accomplished by calculating maximum bending moments and shear forces on single span beams with lengths ranging from 1m to 100m (Figure 15 and Figure 16). The results, for simplification purposes, are plotted in form of so-called MLC curves, where the bending moments are additionally divided by the corresponding span length in order to produce unit bending moments.

All standard MLC curve calculations are performed without any dynamic allowances and with assumption of 30.5m spacing (30.5m ≈ 100ft standard convoy spacing in STANAG) between the contact points of nearest two vehicles, since it is always assumed that military operates in convoys.

The MLC curves are established to serve two primary reasons:

1. Rapid determination of internal forces for single span bridges of the referenced lengths for any given MLC class. With the calculated results, the internal forces resulting from variable action are readily at hand for any MLC and any single span length from 1m to 100m. It simplifies the bridge assessment process in the ultimate limit state. Continuous beams or any other different bridge configurations are necessary to be analysed and recalculated for the given MLC class hypothetical vehicle with the above described characteristics. Section 4.2 provides additional details.
2. Classification of real vehicles, where the internal forces resulting from passage of a real vehicles are compared to values established by hypothetical vehicles. Classification of vehicles follows the procedures of loads set up by hypothetical vehicles where a maximum load effect is calculated. In core, the classification process essentially compares the resulting internal forces of any real vehicle to those of MLC hypothetical vehicles. A subject discussed in the next section in detail.

Table 7 lists all of the tracked and wheeled vehicles with their properties such as vehicle length, axle loads and spacing between the adjacent axles.

Table 7: List of all MLC Vehicles [93]

MLC	Tracked		Wheeled								
	Weight [kN]	Length [m]	Axle Loads [kN]					Axle Spacing [m]			
4	36.30	1.83	9.10	15.90	15.90			2.44	1.22		
8	72.60	1.98	27.20	27.20	27.20			3.05	1.22		
12	108.80	2.74	27.20	45.40	45.40	18.10		3.05	1.22	3.66	
16	145.10	2.74	27.20	59.00	59.00	27.70		3.05	1.22	3.66	
20	181.40	2.74	36.30	77.10	77.10	27.20		3.05	1.22	3.66	
24	217.70	2.74	45.40	90.70	90.70	27.20		3.05	1.22	3.66	
30	272.20	3.35	54.40	99.80	99.80	54.40		3.05	1.22	3.66	
40	362.90	3.66	63.50	117.90	117.90	127.00		3.66	1.22	4.88	
50	453.60	3.96	72.60	136.10	136.10	181.40		3.66	1.22	4.88	
60	544.30	4.27	72.60	163.30	163.30	117.90	117.90	3.66	1.52	4.57	1.22
70	635.00	4.57	95.20	190.05	190.05	127.00	127.00	3.66	1.52	4.57	1.22
80	725.80	4.88	108.90	217.70	217.70	145.10	145.10	3.66	1.52	5.49	1.52
90	816.50	5.18	125.50	244.90	244.90	163.30	163.30	3.66	1.52	5.49	1.52
100	907.20	5.49	136.10	272.20	272.20	181.40	181.40	3.66	1.68	6.25	1.52
120	1088.60	6.10	163.30	326.60	326.60	217.70	217.70	3.66	1.83	6.10	1.52
150	1360.80	7.32	199.60	381.00	381.00	290.30	290.30	3.66	2.13	6.71	1.83

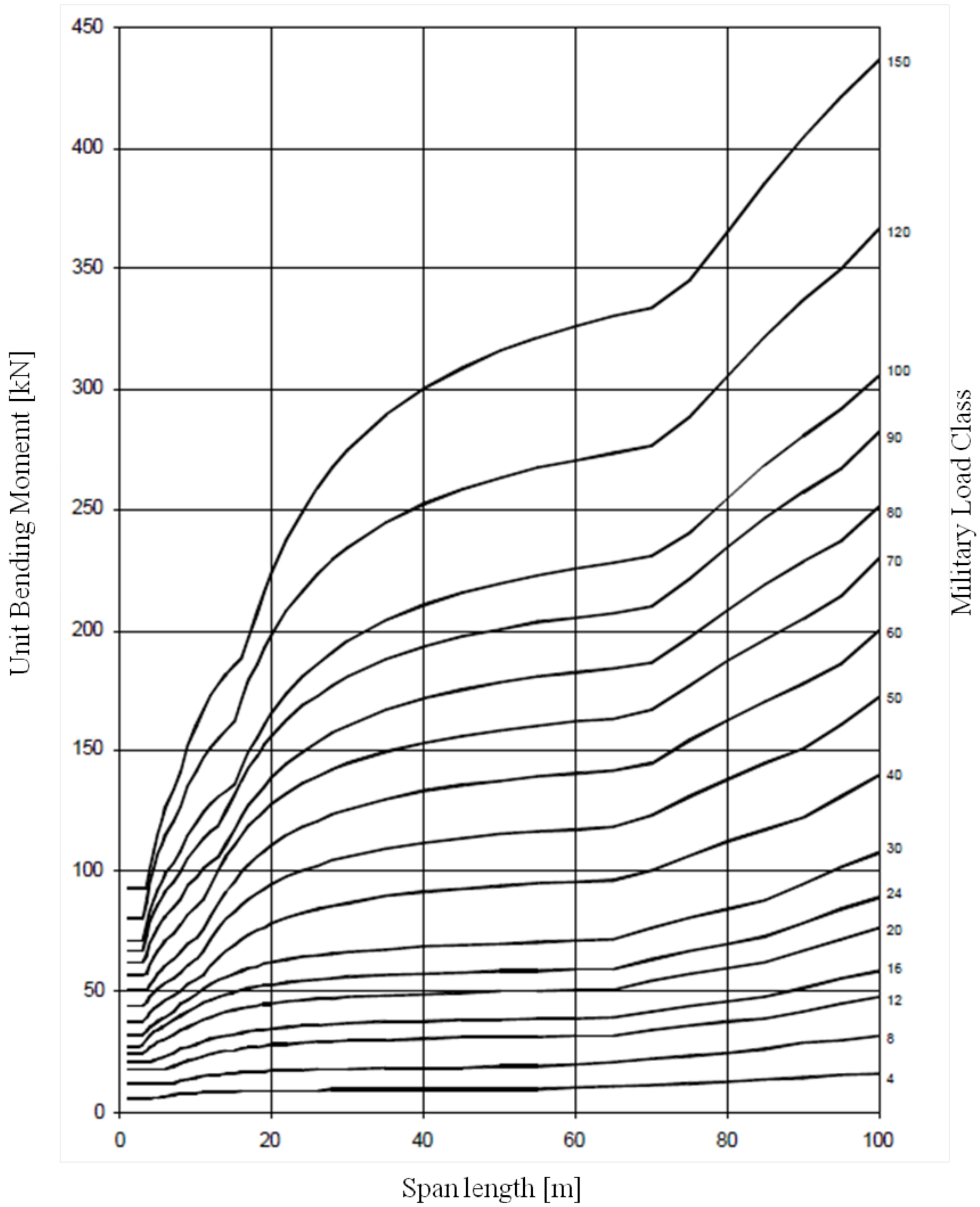


Figure 15: MLC Curve – Unit Bending Moment [93].

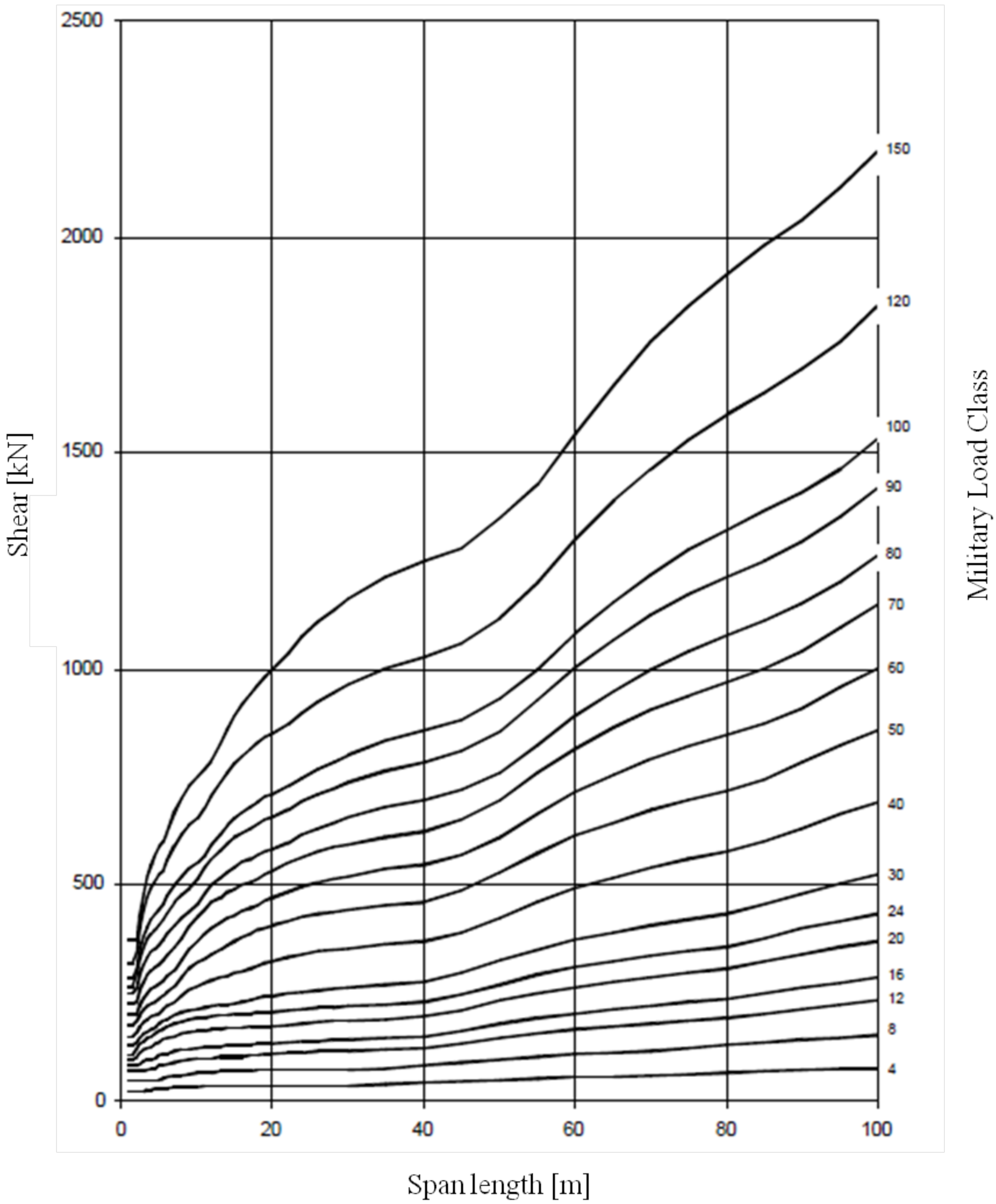


Figure 16: MLC Curve – Shear Force [93].

Every single vehicle[4] used by the military forces is assigned a certain integer MLC number. This number then serves as an indicator to which Military Load Class this vehicle belongs. Vehicle MLC number is assigned based on the resulting internal forces in a single span beam element on contrary to the total weight as sometimes done for legal civilian vehicles [55] . The internal forces of MLC are resulting from the positioning of a vehicle in the critical position on a single span in order to achieve the largest bending moment in the midspan or the maximum shear response at the support. The particular axle loads and axle configurations of wheeled vehicles are therefore fundamental properties in assignment of MLC number, since the position and magnitude of point loads (axle loads) can significantly influence the resulting unit bending moment or shear. Vehicle classification process in essence follows the procedures of calculating the internal forces set up by hypothetical vehicles.

For the classification of a real vehicle, as a first step, axle loads and axle spacing have to be measured. That means physical measurements of the distances between the respective axles and weight measurements of each axle are obtained by methods described in STANAG. As an example, Belgian Army Engineers measure axle loads five times, and axle spacing three times along with Dixon Q 95% confidence validation of the measurements for rejection of observations that greatly deviate from the rest of data [43]. However, commonly is the MLC number assigned by the manufacturer of the vehicle and presented to the military user along with the other supplemental data such as maximum loading capacity, etc. This means that the definition of loads is a quite accurate measure.

With the obtained measurements, the MLC classification procedure continues with a calculation of various bending moments and shears for 1m to 100m span lengths. These calculated internal forces are then compared to the tabulated hypothetical vehicle internal forces to find out which vehicle represents the obtained values. Due to the random configuration of real vehicles it is often the case that calculated internal forces do not match the exact values of hypothetical vehicles. An MLC integer number for real vehicles is then derived through interpolation between the two adjacent classes.

Table 8: DINGO 2 Configurations used by Belgian Army [42]

Vehicle ID	Calculated MLC	Assigned MLC
574	11.56	12
575	11.62	12
573	13.44	13
576	12.87	13
579	13.59	14
584	13.51	14

[4] Term “vehicle” should be for further reference understood as any of the following configuration - solo vehicle, assembly of vehicles, or vehicles with trailers.

Within the armed services, there are many different types of vehicles and, more importantly, most of the generic vehicles are produced with numerous modifications or versions in order to maximize the vehicle potential while maintaining reasonable costs. Each version, regardless of the extent of modifications, is rated and assigned a separate MLC number as shown in Table 8, where different versions of the same base vehicle are assigned different vehicle ID. Additionally, a vehicle receives a MLC number in accordance to the loading – for both empty and fully laden state. Figure 17 shows some of the vehicles in the military forces and their MLC ratings.

French P4 - MLC 3

Czech DINGO 2 - MLC 12

IVECO 8T - MLC 22

MiRPz Keiler - MLC 56

German Leopard A2 - MLC 66

SaZgM heavy (8x6) with 56t trailer - MLC 99

Figure 17: Exemplarily military vehicles and their MLC classification. Photos per [142] - [147].

STANAG MLCs are defined ranging from 4 to 150 (Table 7). Real vehicles take any of the values within the given range based on their configuration and load characteristics. Very light vehicles used by the military – such as reconnaissance or light troop transport, example French Peugot 4 rated MLC 3 – generally fall in the lower MLC categories, MLC 4 to MLC 10. Personnel carriers with additional protective armour (DINGO 2 rated MLC 12) tend to be heavier and therefore fall in respectively higher MLCs, MLC 12 to MLC 20. Transportation trucks, for example IVECO8T or IVECO16,5T Eurotrakker, are rated when fully loaded over MLC 20 and up to MLC 40 when equipped with a trailer. The wheeled vehicles are in STANAG defined up to hypothetical MLC 150, but actually most of the assigned MLC do not exceed MLC 40 to MLC 50 - the approximate equivalent of heavy civilian truck traffic. However, due to specialized needs, some vehicles with rating around MLC100 exist. For example, German Army possesses various heavy duty tractor units and tank transporters for its operational needs. One of them, SaZgM heavy (8x6) with trailer also known as “Franziska” is rated MLC 99 when fully laden and is able to haul up to 56.000kg, which even includes the main battle tank Leopard – MLC 66.

Tracked vehicles often fall in heavier MLC categories and tend to have higher MLC numbers due to their relatively concentrated weight when compared to large wheeled assemblies with trailers. Large combat military vehicles are often tracked, due to the weight distribution, terrain clearing capabilities and high traction on sub-quality surfaces. An example is the mentioned main battle tank Leopard A2 used by the Bundeswehr and other NATO nations with its rating MLC 66. Other illustration is offered in the form of German mine clearing vehicle MiRPz rated MLC 56. However, this described characterization of tracked vehicles does not always apply, since there are other specialized light vehicles and exceptions not always requiring combat deployment and terrain clearing (ie. construction equipment, etc.).

The generalization of vehicles and their respective classes is not always true, but it provides an overview of what vehicles are used by the military forces and what MLC range is realistic to be expected as bridge traffic. The specific way of MLC classification process is important, because it provides valuable information about each vehicle used by the military. When compared to a simple classification purely based on the weight, the MLC number is a much suitable description in terms of expected results. The separation in many different classes allows for much narrower division and more accurate description of loads, a very important aspect in terms of stochastic description of the loading (investigated in Section 5.1). More importantly, such defined loads are time-invariant – newly developed and constructed vehicles might get larger and heavier, but that would only result in assignment of a higher MLC class. This significantly simplifies the loading models, because it means that the prediction of possible future traffic loads and its characteristic loading is in end effect unnecessary.

It is also important to note, that the conversations with the military engineers produced a couple of interesting and important points. The military is strict about overloading of vehicles and it is generally sternly prohibited. The rules about classification are generally thoroughly followed in accordance to STANAG. Moreover, a MLC is often determined by the manufacturer. Assigned MLC numbers are the rule and are respected. Therefore the probability of a vehicle actually exceeding its assigned MLC, when disregarding statistics related to weight and length, is quite negligible due lack of overloading and due to classification of vehicles and its modification in its both empty and fully laden state.

4.2 Bridge Assessment

Within the military a great care is devoted to the route assessment and planning as they are essential to all ground movement activities. Each route is carefully selected in order to accomplish the assigned task. From that point of view, it is essential to properly select a route with a sufficient height and width clearances and an adequate capacity. Bridges, in particular, often present a bottle-neck and limit the maximal allowable MLC on the selected routes and therefore could hinder the outcome of the whole task or operation. It is necessary to assess each bridge on the proposed route for its maximal load carrying capacity in order to determine the route suitability. It should be noted that many countries operationally assess most of the bridges on national networks for capacity and rate them for MLC. This is accomplished either by civilian or military engineers depending on particular country. An example is offered by MANAS [87] for the route assessment in the Czech Republic.

The development of German regulations for the assessment of bridges carrying military traffic is described in a large detail by BRANDT [17]. The assessment of bridges for military loads in Germany according to STANAG2021 [93] was recognized by German Department of Transportation Guideline in 1957 [26], which was additionally supplemented in 1964 [22] and 1968 [23]. However, it was first in 1979 as mentioned by BRANDT [17] that the recalculation of bridges for the equivalent loading was required for all civilian bridges as long that they were not specifically designed for military loading. But already ARS Nr. 11/1981 [6] required that:

- New federal bridges shall be designed according to STANAG2021 to carry MLC 50 as two way traffic and MLC 100 in a single lane, designated as MLC 50/50 – 100.
- Existing federal bridges without MLC classification shall be assessed and classified.

Later introduced ARS 6/1987 [7] offered a simplified approach where the military loading could be regarded as fraction of civilian loading and a bridge class 60/30 [40] was automatically rated as MLC 50/50 – 100 [26]. The result of the bridge military rating was that a number of bridge structures in Germany were equipped with one of the MLC postings as shown in Figure 18 [17].

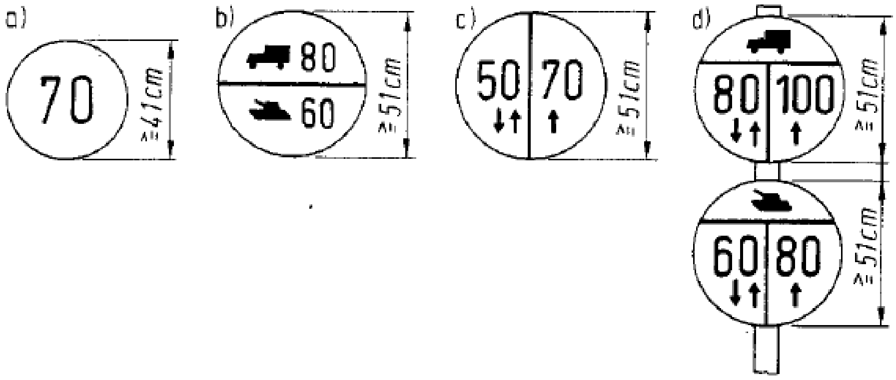


Figure 18: Permanent marking of bridges with military signs [17],

a) Normal sign for roadway widths < 5.50m (single lane traffic),

b) Special sign for roadway widths < 5.50m (single lane traffic),

c) Normal sign for roadway widths ≥ 5.50m (two lane traffic),

d) Special sign for roadway widths ≥ 5.50m (two lane traffic).

Currently, it is somewhat ambiguous, which bridges should be assessed for MLC, as the Department of Defense (Bundesministerium der Verteidigung) decided that it was no longer necessary to mark new bridges due to the increased civilian loading in newest standards [85], [141]. However, the most actual federal requirements of Department of Transportation (Bundesministerium für Verkehr) from 01.01.2013 [28] still point at ARS Nr. 11/1981 [6]. Additionally, military uses bridges not only in home countries, but also abroad and the legal definition of crossings is not always crystal clear. The national regulations are often not valid or fully applicable in foreign countries.

The actual structural standards for bridge design and assessment in many NATO countries are the Eurocodes. In Germany, according to the ARS Nr. 22/2012 [8], it is necessary to regard the STANAG military loading with the partial factors from EN 1990 [45] during the bridge evaluation. In fact, the military load should be treated as an additional variable loading where the partial factor is listed as γ_Q = 1.50.

Asides the legal requirements for bridge assessment it should be mentioned, that HOMBERG [63], [64],[65] published the most comprehensive guidelines for recalculation of bridges for military load classes in Germany. The backbone of this recalculation process, in spirit of legal requirements at the time of publication, is the numerical comparison of the military loading to the civilian loading (specifically DIN 1072 [40]) for different bridge configurations and treating the resulting load as an equivalent civilian load within the safety concept valid at the time [21]. This method has been updated by FORMAN ET AL. [54] in terms of tabulated comparison of military loads to civilian loads according to loading model from DIN FB-101 [41]. The resulting equivalent load is still treated as civilian load and the sectional forces are calculated for some typical structural static systems for three specific hypothetical vehicles - MLC 50, MLC 100 and MLC 150. These mentioned methods do not recognize some of the particular aspects of military loading, but do simplify the structural analysis for the selected MLC vehicles and familiarize civil structural engineers with the military loading.

During the bridge MLC classification according to the actual structural codes, the bridge receives an MLC number corresponding to its structural capacity. The MLC number is currently assigned on the basis of load capacity calculations resulting from the traditional semi-probabilistic concept in the ultimate limit state (Eq. 4-1), where appropriate partial factors are applied to both loads (Eq. 4-2) and resistance variables (Eq. 4-3). It is hereafter assumed that loads and resistances can be treated separately (which may not be the case e.g. for geotechnical structures).

$$E_d \leq R_d \qquad \text{Eq. 4-1}$$

where

$$E_d = \sum \gamma_G G_k + \gamma_Q Q_{k1} + \sum \gamma_{Qi} \psi_{0i} Q_{ki} \qquad \text{Eq. 4-2}$$

$$R_d = R\left(\frac{\alpha \cdot f_{ck}}{\gamma_c}, \frac{f_{sk}}{\gamma_s}, ... \right) \qquad \text{Eq. 4-3}$$

According to STANAG, the permanent action shall consist of the self-weight of all structural elements as well as equipment such as curbs, rails and utilities. Snow and mud load should also be accounted for when present according to STANAG, unfortunately with no guidance on how to account for this loading. Additional information regarding the loading is provided in the document Trilateral Design and Test Code for Military Bridging and Gap-Crossing Equipment [127], as accepted by Federal Republic of Germany, United Kingdom and United States of America, that shall override their national standards expect for STANAG and ISO publications.

The variable action for the calculations is represented by point loads or uniform loads of the hypothetical MLC vehicles, or by the defined maximum single axle load. The maximum load effect is to be obtained from the critical position on the bridge. Obviously, the exact load path and expected internal forces are dependent on the structural system and therefore the critical position of a vehicle is not always the same for all structural elements in consideration. There are no provisions for an additional distributed load supplementing the vehicular loading as in civilian bridge codes and only the MLC vehicle is to be considered in the assessment of load effects.

While EN 1991-2 [46] has provisions for the combination of secondary variable actions on a bridge, such as seismic or braking forces, according to STANAG only vehicular loading should be regarded in the military load classification of existing bridges, unless one of the secondary variable actions has a significant influence. There is no mention of what and how significant the other action shall be and it should be left to an engineering judgment as necessary. Braking forces are commonly considered and BRANDT [17] offers a numerical quantification of braking forces to be applied.

No provisions for a mixed civilian and military traffic on a bridge are available. It is generally assumed that civilian traffic is not present when military vehicles are crossing the bridge under consideration. In addition to the case of a single standard vehicle on the bridge, an indefinitely long convoy of vehicles with 30.5m spacing between contact points of the nearest two vehicles is to be accounted for. RAY & STANTON [107] list in their publication regarding load rating of bridges within US Military Installation additional details about the application of the loading. They specify the critical load path of variable loads and distribution factor for multi-girder bridges while declaring that the bridges are to be rated separately for civilian, military and pedestrian traffic.

A bridge receives a MLC number corresponding to the maximal structural capacity. Any vehicle or a convoy operating under the maximum MLC can pass the bridge without restrictions. In reality it means, that a MLC 40 bridge carries traffic composed from MLC 4 to MLC 40. There is a random mix of real vehicles with MLC numbers such as 8, 17, 24, 32 or 38 and the maximum 40 corresponding to the bridge ultimate capacity. The likely distribution of MLC vehicles is difficult to determine in general terms, since the use of the particular bridge is upfront unknown. However, no vehicle with a larger MLC then posted on the bridge is allowed to cross, unless a different crossing condition is considered and the crossing is regulated in terms of speed and vehicle position as described in Section 4.2. It should be noted, that the expected bridge traffic is more likely to be made up of wheeled vehicles, especially when operations in homeland are considered. Regulations are often preventing tracked vehicles to freely operate on paved surfaces maintained by civilian authorities. Nevertheless it is possible for tracked traffic to operate, either with rubber track protectors or during deployment operations and they should subsequently be accounted for.

STANAG additionally recognizes two possible bridge classifications procedures - permanent and temporary classification of bridges. Permanent MLC is achieved through the use of analytical methods. Expedient classification methods may only determine temporary marking and thus the bridge must be reclassified analytically as soon as practically possible. This work in all further reference is only concerned with the analytical methods of classification. Although, there is no specific ruling on what time frame the term permanent stands for and is therefore regarded as the remaining design life

The assessment and marking of bridges after their construction simplifies the situation for military route planners, since the bridge MLCs are readily available. The route, for example in emergency situations, maybe be planned very quickly in order to respond to the threat. The planning may be accomplished inclusive potential detours, should there be encountered a damaged bridge during the transport. The situation slightly changes in deployment situations, where most of the existing bridges are not rated, but are commonly used by military – as was shown recent operations in Kosovo, Bosnia, Iraq, Afghanistan or Mali. The military might stay for years and use the bridges on daily basis and therefore it is a priority for the engineers to assess the bridges as soon as possible in order to develop a suitable routing for the required operations. The quality of bridges, or the applied use of sound engineering principles during the design are in some of the less developed countries questionable, as shown by experiences, but it does not change the fact, the bridges need to be assessed and rated for MLC.

4.2.1 Military Loading

The variable action for the calculation in the ultimate limit state is represented by the STANAG hypothetical MLC vehicle, hypothetical maximum axle load, or an indefinitely long convoy of MLC vehicles spaced at the minimum 30.5m. The convoy of vehicles is generally rated according to the highest MLC present. The convoy should be regarded for analysis as homogeneous. It is assumed that the corresponding highest MLC is spaced at the minimum 30.5m and lower MLC numbers are disregarded. Figure 19 shows the convoy spacing in a two directional traffic.

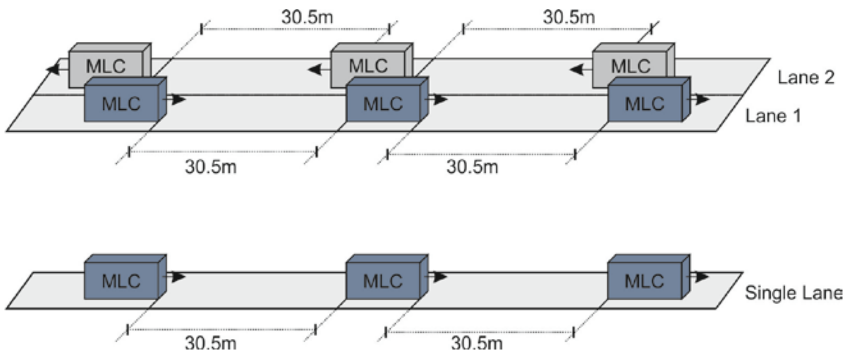


Figure 19: MLC Column on single lane or two lane bridge.

The convoy considerations are particularly important for the analysis of multi-span structures in terms of bending moment. Shear response of a beam is always governed by the maximum number of vehicles that can potentially load the same span. There are no special considerations provided for extraordinary cases of vehicles spaced possibly closer than the allowable range as dictated by the specific conditions or needs. These have to be regarded on case-specific bases. A maximum of two lanes is defined for the military traffic. A variable minimum lane width is required for different ranges of MLC classes. Moreover, roadway widths with less than 5.50m are limited to a single lane one-way traffic (Table 9).

Table 9: Minimum roadway widths [93]

MLC	One-way traffic	Two-way traffic
4-12	2.75m	5.50m
13-30	3.35m	5.50m
31-70	4.00m	7.30m
71-100	4.50m	8.20m
above 100	5.00m	not allowed

For analytical considerations of a two-way traffic effect, the vehicles in both lanes should be regarded as moving simultaneously. Their load effect is then combined. MLC calculations must account during the normal crossing for the vehicular loadings anywhere on the roadway surface. The critical position will be that which produces the maximum load effect as determined by static analysis.

4.2.2 Crossing Conditions

In addition to defining the MLC, STANAG 2021 [93] also provides regulations regarding the different modes of crossing of military vehicles over bridges. This is to maximize the allowable load by minimizing the load effects resulting from load positioning and dynamic amplification. These conditions are dictated by the tactical or emergency situations when crossing of vehicles with a higher MLC is necessary. The crossing of heavier MLC vehicles is accomplished by either more controlled crossing conditions or relaxed safety criteria.

Normal Crossing

The normal crossing condition is the main crossing mode and should be regarded as standard for the assessment if not stated otherwise. The minimum criteria for safety outlined in STANAG are valid for this condition. A normal crossing allows for an unrestricted use of bridge by military traffic and for all vehicles or convoys operating at or below the maximum allowable MLC. Only rating associated with the normal crossing may be permanently assigned to a bridge.

Caution Crossing

While maintaining the same safety level as the normal crossing, the caution crossing allows for a higher allowable MLC by limiting the maximum speed to 5km/h and restricting the use of braking, accelerating and switching gears. Vehicles must be driven along the centerline and are only allowed to cross one at a time across each structurally independent span. Rating associated with the caution crossing may be only regarded as temporary.

Risk Crossing

The risk crossing allows for a transportation of higher MLC vehicles by adapting the same conditions as the caution crossing (speed up to 5km/h, single vehicle at the centerline of an independent span, no braking, accelerating and changing gears), but additionally decreasing a minimum required safety. It increases the probability of failure, and even if the bridge does not fail, permanent damage to the bridge may occur. Crossing conditions are therefore quite important element when discussing assessment of bridges and the development of suitable partial factors.

During the year 2012 and through the discussion among military engineers in framework of Team of Experts on Military Bridge Assessment [124], a proposal for crossing conditions was created that would combine the rules from STANAG and practical aspect of bridge assessment. Current working version was adopted in the form of Technical Paper PoW [125]. The proposed assessment condition rules are summarized in Table 10.

Table 10: Proposal of the assessment rules

Parameter	Crossing Condition		
	Normal	Caution	Risk
Permanent Load	Yes	Yes	Yes
Variable Load - MLC	Yes	Yes	Yes
Dynamic Effect	Yes	No	No
Vehicle spacing	30.5 m	30.5 m	Single vehicle
Crossing velocity	MLC ≤ 30 : 40 km/h MLC > 30: 25 km/h	MLC ≤ 30 : 25 km/h MLC > 30: 15 km/h	All MLC: 5 km/h
One Way/Two way traffic	Two Way	One Way	One Way
Convoy eccentricity	Worst case	Centerline	Centerline
Variable Load partial factor	$\gamma_{Q.normal}$	$\gamma_{Q.caution}$	$\gamma_{Q.risk}$

The use of different partial factors for variable loading is reflected by the specifics of the different crossing conditions. It is important to recognize that the different partial factors are considered for each crossing mode. This is a deviation from current structural codes. It allows for a more accurate account of loading, including the description in terms of dynamic amplification and target reliability.

4.3 Semi-probabilistic Safety Concept

In a semi-probabilistic safety concept a set of partial factors γ resulting from probabilistic analysis serves to achieve certain reliability level for any structure of interest. Considering Section 3.2 it should be clear, that the development of modified partial factors in semi-probabilistic safety format is the suitable approach for the assessment of existing structures. It is proposed in this work to adapt the semi-probabilistic safety concept as provided in EN 1990 [45] in order to maintain continuity of structural calculations according to the Eurocodes but at the same time to reflect the military traffic and existing nature of the considered bridges.

The factors from EN 1990 Appendix A2 [45] according to the ARS 22/2012 [8] listed as:

- γ_G = 1.35 for permanent action,
- γ_Q = 1.50 for other variable action such as military loading,

are to be modified to reflect the characteristic of existing bridges assessment under the military loading according to STANAG as proposed by LENNER AT AL. [84].

The military loading along with respective partial factors is considered by MANAS & ROTTER [88]. A method is developed for an assessment of specifically fabricated steel bridge TMS designated for only military traffic and potential deployment in emergency situation.

Table 11: Expert judgment based partial factors for military loadings in [88]

Variable	Civilian loading	Military loadings according to STANAG 2021 [93]		
		Normal Crossing	Caution Crossing	Risk Crossing
Permanent loading	γ_G = 1.05	γ_G = 1.05	γ_G = 1.00	γ_G = 1.00
Variable loading	γ_Q = 1.35	γ_Q = 1.10	γ_Q = 1.00	γ_Q = 1.00
Resistance of materials	γ_M = 1.05- 1.10	γ_M = 1.05	γ_M = 1.00	γ_M = 0.95

The partial factors in [88] are based on an expert judgment, when permanent load is dictated by the controlled fabrication of steel members, and therefore a minimal variation of the loading. Variable loading partial factor is based on the assumption that the loading is regulated in terms of vehicle position. It yields partial factors much lower than those listed in EC. The numerical quantification for the selected partial factors is not provided, as the engineering judgment and experiences are the solely indicators. This shows that the proposed method of adapting partial factor to reflect specific needs or characteristics is not unique, however, the topic of military loading on bridges has been up to this point somewhat neglected and there has been limited work accomplished in this field.

This safety concept as summarized by Equations 4-1, 4-2 and 4-3 is utilized and re-formulated for the purposes of military assessment. It is proposed to formulate the safety concept as an evaluation of the resistance and loading [117] :

$$R_d \geq E_d = \sum_j \gamma_{G,j} \cdot G_{k,j} + \gamma_Q \cdot Q_k \qquad \text{Eq. 4-4}$$

where R denotes resistance, E load effect, γ partial factor, G permanent action effect, and Q military variable action effect.. The subscripts "d" and "k" denote design and characteristic values respectively. The symbol "+" implies "to be combined with" and Σ "the combined effect of". The evaluation of different leading actions as in EN 1990 [45] Eq. 6-10a und 6-10b is not foreseen in this study.

According to STANAG, the permanent action shall consist of the self-weight of all structural elements. The variable action for the calculations is represented by point loads or uniform load of the hypothetical MLC vehicle, or by defined the maximum single axle load. There are currently no provisions for distributed load or secondary variable actions (see Section 4.2).

This work is mainly concerned with the actions and in particular the variable action represented by the military traffic loading. For the resistance of existing structures and development of partial factors or resistance models refer to for example [15], [53], [67] and developments listed in Section 3.2.

4.3.1 Partial Factor for Permanent Action

The partial factor for permanent action should reflect the existing state of the bridge. The design value of permanent load is expressed as:

$$G_d = \gamma_G \cdot G_k \qquad \text{Eq. 4-5}$$

and the partial factor is in turn obtained as [117]:

$$\gamma_G = \gamma_{Ed,g} \cdot \gamma_g \qquad \text{Eq. 4-6}$$

where $\gamma_{Ed,g}$ stands for partial factor accounting for the model uncertainty in estimation of load effect from the load model and γ_q is reliability-based partial factor accounting for variability of the permanent action, statistical uncertainty and uncertainties related to the model of permanent action.

The model uncertainty factor can be assumed in structural design and verification as $\gamma_{Ed,g}$ = 1.07 for an unfavorable action and $\gamma_{Ed,g}$ = 1.05 for a favorable action [117], [122]. Alternatively this factor can be calculated in general terms assuming a lognormal distribution as follows [114]:

$$\gamma_{Ed} = \frac{\mu_{\theta E}}{\theta_{Ek}} \exp(-\alpha_E \cdot \beta \cdot V_{\theta E}) \qquad \text{Eq. 4-7}$$

where the first terms $\mu_{\theta E}/\theta_{Ek}$ denotes the ratio of mean to characteristic value of the load effect model uncertainty, β is the target reliability index, α_E stands for the sensitivity factor and V_θ denotes the coefficient of variation of the model uncertainty θ_E .Note that this definition is valid not only for uncertainty in estimation of permanent action, but also in estimation of variable action.

Assuming a normal distribution of the permanent action and the characteristic value equal to mean, the partial factor γ_g can be written as:

$$\gamma_g = 1 - \alpha_E \cdot \beta \cdot V_G \qquad \text{Eq. 4-8}$$

where α_E ≈ -0.7 denotes the FORM sensitivity factor approximated in accordance with EN 1990 [45] and V_g stands for the coefficient of variation for the permanent action *G*.

When assessing permanent actions on existing concrete bridges the following is to be considered:

- Existing bridges can be generally described with higher accuracy and therefore with reduced uncertainty tied to permanent loads [86].
- In accordance with ISO 13822 the target reliability index β for assessment of existing structures can be adjusted by optimization of the total cost related to an assumed remaining working life, topic discussed in detail in Chapter 6.
- The target reliability index β can be further adjusted for relevant crossing conditions.

This suggests that the partial factor for permanent action can be accordingly adjusted to fit the needs of military tailored semi-probabilistic safety concept by adapting coefficient of variation V_g for permanent loads to reflect the existing nature of bridges and defining proper target reliability level especially with consideration to crossing conditions.

4.3.2 Partial Factor for Variable Action

The design value for variable action Q_d, or the value required for assessment in the ultimate limit state, can be obtained from the characteristic value Q_k as follows:

$$Q_d = \gamma_Q \cdot Q_k \qquad \text{Eq. 4-9}$$

where γ_Q is the partial factor for variable action that can in turn be defined as [31]:

$$\gamma_Q = \gamma_{Ed,q} \cdot \gamma_q \qquad \text{Eq. 4-10}$$

where $\gamma_{Ed,q}$ stands for partial factor accounting for the model uncertainty in estimation of the load effect from the load model, for definition see Eq. 4-7; γ_q is reliability-based partial factor accounting for variability of the variable action, statistical uncertainty and uncertainties related to the model of variable action and can be defined as [67] :

$$\gamma_q = \frac{F_{qtref}^{-1}\left[\Phi(-\alpha_E \cdot \beta), t_{ref}\right]}{q_k} \qquad \text{Eq.4-11}$$

where $F_{q,tref}^{-1}$denotes the inverse cumulative distribution function of the variable loading maxima during the considered reference period t_{ref}. The load distribution should be based on the same reference period as considered for selection of target reliability β.

HOLICKÝ & SÝKORA [67] further define the time dependent variable load as:

$$q_{tref} = C_0 \cdot \max_{tref}\left[q_0(t)\right] = C_0 \cdot q_{0,tref} \qquad \text{Eq. 4-12}$$

with $q_0(t)$ equal to the time-variant component and C_0 time in-variant component of the variable load. Probabilistic models for the two components as well as additional details are given in fib SAG7 [67].

As discussed previously, during the military assessment, γ_Q is generally taken from the current bridge standards. Problematic is that the factor was developed using substantially different properties and assumptions on effects of traffic loads, such as dynamic amplification, characteristic load and time variance of the loading. It is therefore proposed by LENNER ET AL. [83] to assess the design load effect of military traffic Q_d on different terms as follows:

$$Q_d = \gamma_Q \cdot Q \qquad \text{Eq. 4-13}$$

where the load effect Q is the total load effect due military traffic.

It is hereafter assumed according to that the total load effect due to the passage of military vehicle(s) Q can be obtained as follows:

$$Q = \theta_E \cdot \delta \cdot Q_{MLC} \qquad \text{Eq. 4-14}$$

where θ_E denotes the model uncertainty in estimation of the load effect from the load model, δ is a dynamic amplification factor and Q_{MLC} is a static load effect (including uncertainties in measurements of weights and spacing) equal to the nominal STANAG value as defined in Section 5.4.

It is further realistically assumed and later proved that mean values of the basic variables included in Eq. 4-14 equal to their characteristic values. Assuming lognormally distributed θ_E and δ and a normal distribution of Q_{MLC}, a lognormal distribution can be considered for total the load effect Q since greater variability is associated with both θ_E and δ rather than with a well-described Q_{MLC}.

Based on these assumptions partial factor γ_Q is proposed to be written as:

$$\gamma_Q = \exp(-\alpha_E \cdot \beta \cdot V_Q) \qquad \text{Eq. 4-15}$$

where α_E denotes the FORM sensitivity factor, β target reliability index and V_Q coefficient of variation of Q obtained as follows:

$$V_Q \approx \sqrt{V_\theta{}^2 + V_\delta{}^2 + V_{\mathrm{QMLC}}{}^2} \qquad \text{Eq. 4-16}$$

where V_θ, V_δ and V_{QMLC} are the coefficients of variation of model uncertainty, dynamic amplification and of military static load effect, respectively.

The aim of this work is therefore to investigate the factors such as stochastic properties of military static load in terms of Q_{MLC} and V_{QMLC}, dynamic amplification in terms of δ and V_δ and model uncertainty in terms of θ_E and V_θ. These factors are required for the proposed modification of variable partial factor. Additionally, it is necessary to properly define the required target reliability level for each crossing condition as this is also decisive for the determination of actual values of the partial factors.

4.3.3 Sensitivity factor considerations

Selection of α_E deserves additional attention. It is the sensitivity factor resulting from FORM analysis indicating the influence of variable on the resulting reliability index. Both EN 1990 Annex C [45] and ISO 2394 [72] allow for two approximations for the specific values:

1. $\alpha_E \approx -0.7$ for the leading action
2. $\alpha_E \approx -0.28$ for the accompanying action

The exact values for α_E should be in principle calculated using FORM analysis for an equation describing the considered limited state, see Section 2.2.2. The application of variable α-factors would require much iteration and be in effect be very impractical. When only the loading is considered, the ratio of permanent load to variable load and their probabilistic models have decisive influence on the sensitivity factors. The load ratio is unknown prior to the design or assessment. Therefore, the above values defined for the leading action are often used in code definitions and partial factor calculations. Section 5.7.2 provides additional details regarding the sensitivity factors.

5 Military Load Effect Considerations

Previous section showed the proposed safety concept format for military vehicles. The partial factor γ_Q is a key element of the proposed concept. This chapter aims at investigating all the parameters required for its calculation and for the establishment of load stochastic properties. With the development of probabilistic load model for military traffic, it is possible not only to redefine the partial factor, but also to engage in reliability based assessment of existing bridges.

Currently, there is no direct guidance on probabilistic parameters of military traffic. The description of military vehicles in terms of STANAG as described in Section 4.1 is the only reference available for engineering and academic purposes. RAY & STANTON [107] list additional details regarding the loading, but provide no information about the probabilistic load model of military traffic or partial factors. They in fact suggest that AASHTO LRFR load factors [2] should be utilized within their method.

It is necessary to obtain data about military vehicles and loading for development of stochastic properties. Direct contact with the army engineers of Germany, Belgium and USA within Land Capability Group 7 [124] produced an insufficient list of vehicles with approximate gross weights and dimensions designated for shipping allowances on rail and by air. Each nation poses a database of vehicles with assigned MLC numbers, total weights and total lengths; however, not a single agency collects data about the axle loads and axle spacing. Statistical data from the classification process are also unavailable. Moreover, the classification is often performed by the manufacturer of each delivered vehicle.

A comparison to civilian traffic shows the military traffic as better described in terms of expected loading due to the differentiation in many MLCs, although numerical quantification of the description is missing [76]. It is therefore necessary to develop a new method for description of military traffic loading. Extensive numerical simulations are employed in order to quantify the expected static load effect due to passage of military vehicles over bridges. In another words, simulations are used to show, how well can be described the static load effect in terms of variation and how can be extracted the statistical data.

5.1 Static Load Effect of Military Vehicles and Numerical Simulations

The goal of this section is to determine a characteristic static load Q_{MLC}, the corresponding coefficient of variation V_{QMLC} and a suitable distribution function for the static load due to military traffic on bridges. Due to the general lack of data, an extensive numerical analysis, with chosen statistical distributions of weights and spacings, is determined as a suitable method to study the potential factors influencing the estimated load effect. Traditionally, for estimation of traffic models, civilian vehicular traffic is measured by using for example weight-in-motion (WIM) technology [12], [36], [39]. Stochastic traffic models can thus be extrapolated from the measured and filtered data. For example

ENRIGHT [49] in his paper describes various methods for estimation of the lifetime maximum loading from WIM. However, this method is difficult to implement since only crossings of military vehicles may be allowed during the time required to collect sufficient data. Additionally, military vehicles are divided in many different classes and it is difficult to assign stochastic data to a single class based on general data samples.

A comparison of rail traffic is to certain extent comparable to military loading, due to its division in load classes and relatively well described loading. JAMES [73] developed statistical model for description of the train axle loading on the basis of the weight measurements. A number of trains were measured with the help of strain gauges placed on the rail. The data analysis was performed separately for locomotives and empty wagons but produced approximately the same 5% to 7% coefficient of variation for the axle loads with a negligible bias. These results are supported by a case study of a large riveted truss railway bridge [99].

It is proposed to numerically simulate the effect of an uncertainty tied to the axle loads and geometrical properties of a military vehicle in order to quantify the variation of resulting MLC and corresponding internal forces.

The numerical process simulates the classification of a vehicle, where the maximum resulting bending moment and shear are calculated on the basis of axle loads and axle configuration. The simulations generate a large number of the same class vehicles with randomly assigned properties for axle loads and spacings. In other words, artificial vehicles with random characteristics are created and their load effect is assessed. The bending moment and shear reactions are calculated for different static system and conditions. Statistical analysis of the results yields the desired information about the static load effect, such as mean value Q_{MLC} and the corresponding coefficient of variation V_{QMLC} – properties that may be used in a reliability analysis and the partial factor development.

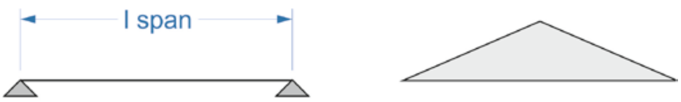
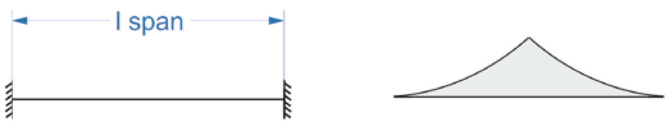
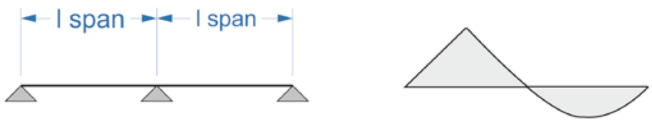
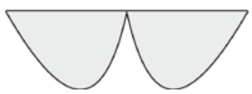
The main parameters considered in the numerical simulations are:

- variation of vehicle load and length,
- response of different static systems determined from simple influence lines,
- various Military Load Classes,
- short span response,
- wheeled and tracked vehicles.

As any random variable, the total or axle load is expected to be expressed with mean value, standard deviation and distribution function, the same applies to the total length or individual spacing between the axles. The variation of load besides natural randomness could be accounted to, for example, physical measurements of the loads. The variation of axle load and spacing is investigated in detail as to quantify its influence on the static load.

Response of different static systems is a deviation from STANAG where only simple beam is considered. This is to investigate the static load variation for additional static systems as to also quantify the expected loading for other commonly used structural systems, such as fixed-end beam or continuous beam. It therefore encompasses the usual bridges encountered on the roads. It is particularly important if the reliability assessment is considered. An approach, using influence lines for calculation of maximum load effects, as inspired by CRESPO-MINGUILLÓN & CASAS [38] and later used by O´CONNOR ET AL [97] and PRATT [103] for the re-assessment of the traffic loads in Eurocode, is adopted with some modifications. The selected influence lines for the different static systems are shown in Table 12. The numerical definition of each was developed within the scope of this work and tailored for the needs of load effect simulations. The definition of limit equations for each of the influence lines is provided in Appendix A.

Table 12: Influence lines used in the numerical simulation

Influence Line Number	Representation	Description of the Influence Line
IL0		Maximum bending moment of a simply supported beam
IL1		Maximum bending moment in midspan of a fixed beam
IL2		Maximum fixed end moment of a fixed beam
IL3		Maximum bending moment in the first span of a continuous beam
IL4		Maximum support moment of a continuous beam

A selection of MLC vehicles is used during the simulations as to include and illustrate the effect of different configurations of axles, total length and total weight. The goal is to develop a single probabilistic model for all military vehicles, even though the differences exist between the classes.

Short span response is a parameter of the study since it limits the number of governing axles of longer vehicles that can physically fit on the bridge. It can therefore influence the load effect of the whole vehicle. The short span response is particularly important for a single span, since continuous bridges are likely to be composed of longer spans in order to maximize the effectiveness.

Wheeled and tracked vehicles are defined in STANAG on different terms, and therefore this study considers both of them for a comparison. Since wheeled traffic is more likely to be expected on bridges, it is studied in a larger detail and serves as the base line for further developments. The numerical simulations of tracked traffic are checked and compared in order to ensure the developed stochastic properties are valid.

For the simulations, it is expected that a single vehicle in a critical position on the span governs the maximum resulting load effect. In view of the minimum spacing between the vehicles as 30.5m and accounting for an additional length of the vehicles itself, the span length becomes excessive of common span lengths of existing concrete bridges, if two or more vehicles are to be considered. Simplified check to find out the exact span length *l* governing the single axle response is provided by the comparison of a single load at midspan and two point loads with spacing of 30.5m and 3.66m vehicle length (equal to MLC4),

$$\frac{P \cdot l}{4} = \frac{P \cdot (l - x)}{2} \qquad \text{Eq. 5-1}$$

$$\frac{P \cdot l}{4} = \frac{P \cdot (l - 30.5m - 3.66m)}{2}$$

the evaluation of Eq. 5-1 delivers the following span length:

$$l = 68.32m$$

The calculated distance of 68m for a single span is seldom encountered in bridge engineering and therefore a single vehicle analysis is justified for most of the cases. The maximum support moment is additionally considered by the use of appropriate influence line, a relevant topic for the longer continuous beams.

The numerical simulations are performed using MathCAD calculation software [89] that allows for a convenient generation of random variables based on the chosen statistical data and an appropriate distribution function. It is possible to generate as many vehicles as possible by assigning random loads and lengths for the calculation of internal forces.

5.2 Wheeled Vehicles

The main parameter under the investigation throughout this section is the bending moment while shear reactions are checked separately. Wheeled vehicles are defined by their axle loads and axle spacings. It is necessary to simulate each of the related axle load *L* and each spacing *S* as a random variable with properties assigned according to the simulated hypothetical vehicle. See Figure 20 for an example input of MLC 40.

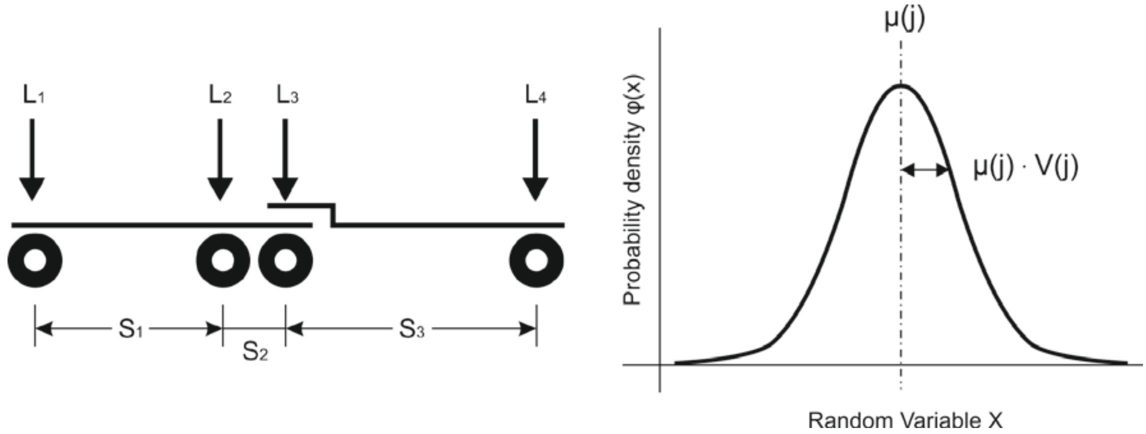


Figure 20: Example of required input for vehicle generation, *j* denotes the axles or spacing number of each considered vehicle.

The following input is required for the numerical simulations:

- mean value μ_L and coefficient of variation V_L of all axle loads,
- mean value μ_S and coefficient of variation V_S of all axle spacings,
- distribution function for *L* and *S*,
- number of generated vehicles/ reference period dictated by n_{sim},
- particular influence line *IL(X)*.

Axles and spacings are generated according to the selected n_{sim} (number of simulations) representing *i* number of MLC vehicles. This number should be selected in accordance to the number of vehicles that can potentially pass the bridge during a period considered for the reliability analysis. It leads to an accumulated set of *i* load effects *Q* (such as bending moments or shear forces). The number of generated vehicles partially influences the accuracy of the resulting coefficient of variation. If it is assumed, that coefficient of variation of the static load effect V_{QMLC} has analytical solution, then the value of V_{QMLC} approaches this analytical solution as n_{sim} approaches the infinity. However, in this study, a quite low number of vehicles is fully sufficient for the determination of stochastic parameters, because mean values and normal distribution are considered. It is essentially irrelevant how the end tail of distribution looks like.

Calculation of load effects resulting from the passage of vehicles over a bridge is summarized in the following steps:

1. Definition of mean values for axle loads $\mu_L(j)$ and axle spacing $\mu_S(j)$ according to a selected hypothetical vehicle in STANAG for each j axle, see Figure 24 for example.
2. Definition of coefficient of variations $V_L(j)$ and $V_S(j)$ for each j axle; coefficients of variation are generally selected as the same for whole vehicle, i.e. each axle load is generated with the same V_L, each spacing with the equivalent V_S for the given vehicle.
3. n_{sim} is established to represent desired number of randomly generated vehicles.
4. Each axle load $L_{i,j}$ and axle spacing $S_{i,j}$ (i = 0..n_{sim}) are n_{sim} times generated according to Eq.

   5-2 and 5-3 as normal distributed random variables with (j=0.. n_L) representing the number of n_L axles and (j=0.. n_S) the number of n_S spacings,

   $$L_{i,j} = rnorm(n_{sim}, \mu_L(j), \sigma_L(j)) \quad \text{Eq. 5-2}$$

   $$S_{i,j} = rnorm(n_{sim}, \mu_S(j), \sigma_S(j)) \quad \text{Eq. 5-3}$$

   where *rnorm* is a MathCAD built-in function that generates a vector of n_{sim} length.

5. Matrix is assembled from vector set of $L_{i,j}$ and of $S_{i,j}$ to represent i generated vehicles
6. Centre of gravity CG_i calculation for each generated vehicle according to Eq. 5-4 for each i vehicle is performed.

   $$CG_i = \sum_{j=0}^{n_L-1} \left(\frac{L_{i,j+1} \cdot S_{i,j}}{\sum L_{i,j}} \right) \quad \text{Eq. 5-4}$$

7. Definition of span length l for the investigated scenario.
8. Definition of Influence line and the respective position of $CG_{i.}$ for maximum load effect according to the influence line.

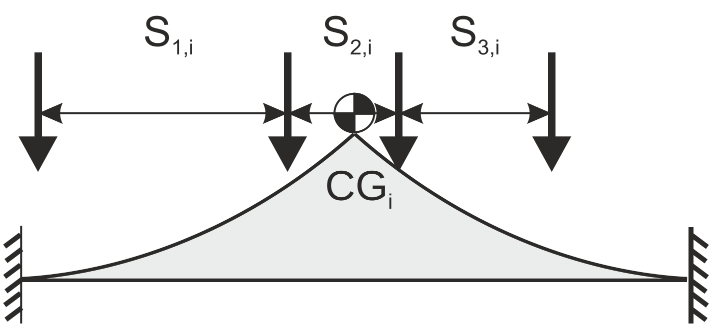


Figure 21: Vehicle positioned in a critical position.

9. Calculation of influence value for each axle position of each generated vehicle according to Eq. 5-4,

$$IL(L) = f_{static.system}(L) \qquad \text{Eq. 5-5}$$

where $f_{static.system}(x)$ is a function of x position along the span length in accordance to the selected static system, see Appendix A.

10. Calculation of resulting bending moment M_i for each generated vehicle with the data from Step 8 by multiplying axle load by its influence value and assembling the sum for the whole vehicle.

$$M_i = \sum_{j=0}^{n_L-1} IL(L_{i,j}) \cdot L_{i,j} \qquad \text{Eq. 5-6}$$

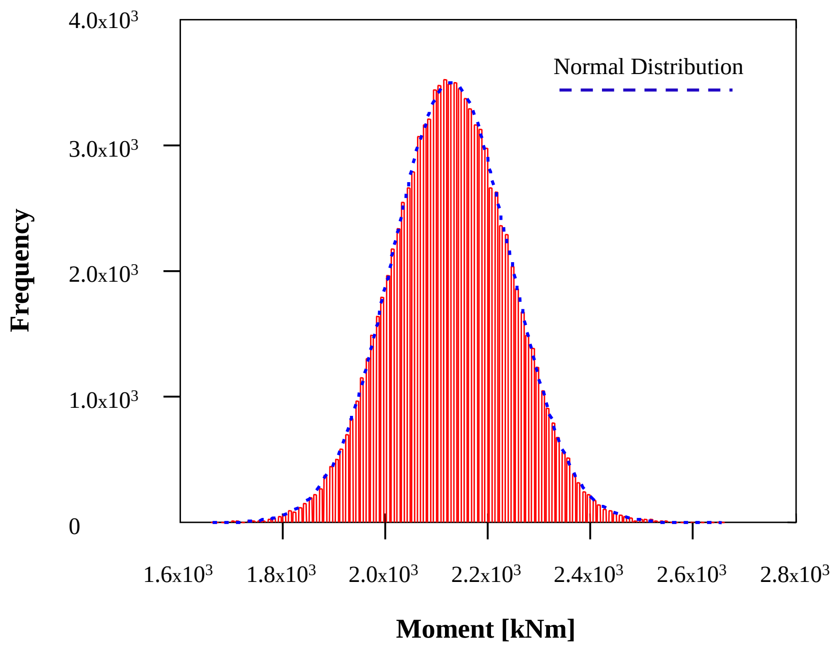


Figure 22: Generated set of bending moments for *i* vehicles for *l* span length.

11. Mean value μ_M and coefficient of variation V_{QMLC} are calculated from the resulting M data set by simple statistical evaluation of the generated sample.

$$\mu_M = \frac{M_1 + M_2 + ... + M_i}{i} \qquad \text{Eq. 5-7}$$

$$V_{QMLC} = \frac{\sigma_M}{\mu_M} = \frac{\sqrt{\frac{1}{i}\left[(M_1 - \mu_M)^2 + (M_2 - \mu_M)^2 + ... + (M_i - \mu_M)^2\right]}}{\mu_M} \qquad \text{Eq. 5-8}$$

An overview of the simulations step-by-step routine is provided in a schematic overview in Figure 23. The detailed whole process of the simulation is presented in the Appendix B. The code is specifically written so that any desired MLC with its configuration can be simulated.

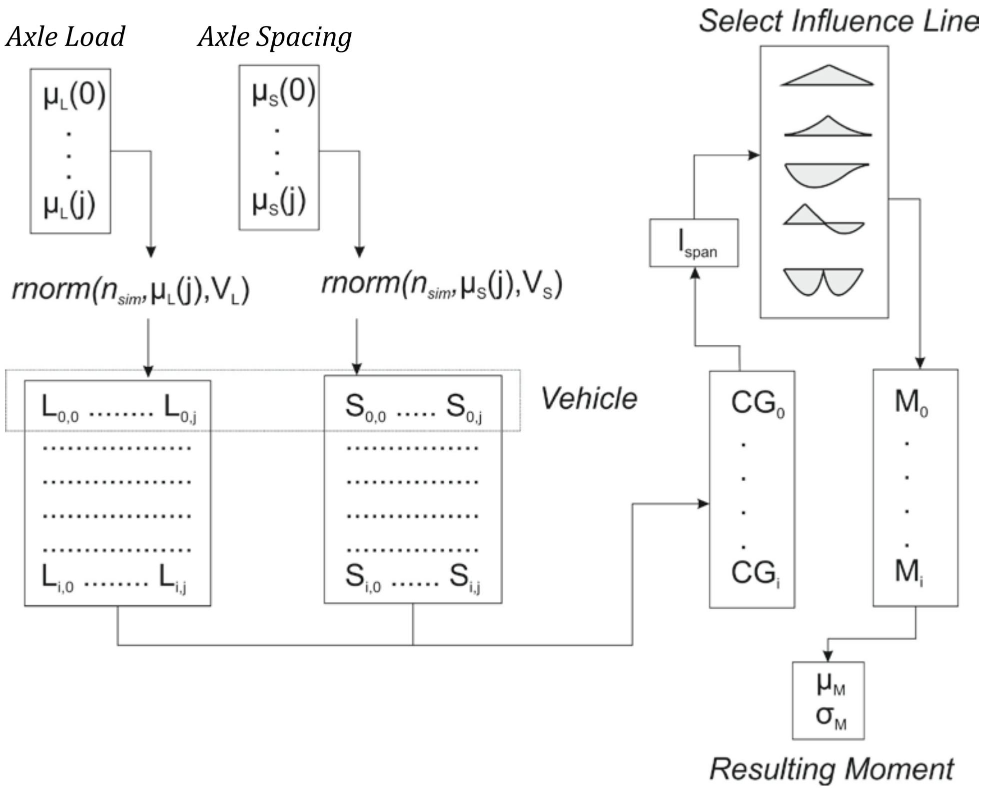


Figure 23: Flowchart of numerical simulation.

Note that normal distribution of the axle load and spacing is deemed as appropriate, since weights of vehicles are normally distributed variables [73]. This is supported by the evaluation of traffic loading and measurements of axle loads by BOGATH & BERGMEISTER [14] where all the applied vehicular loading followed a normal distribution family. Given the normally distributed loads, the resulting set of bending moments is therefore normally distributed as well.

5.2.1 Coefficient of Variation

Main parameters under investigation in this section are V_L and V_S. An example vehicle MLC 40 with four axle loads and three axle spacing is selected as a benchmark vehicle for the first set of simulations in order to establish control values. This vehicle with total weight of 42.63t is a suitable representation of a frequent vehicle used in the military. Moreover, it fits with its parameters a random truck with trailer present in the civilian traffic.

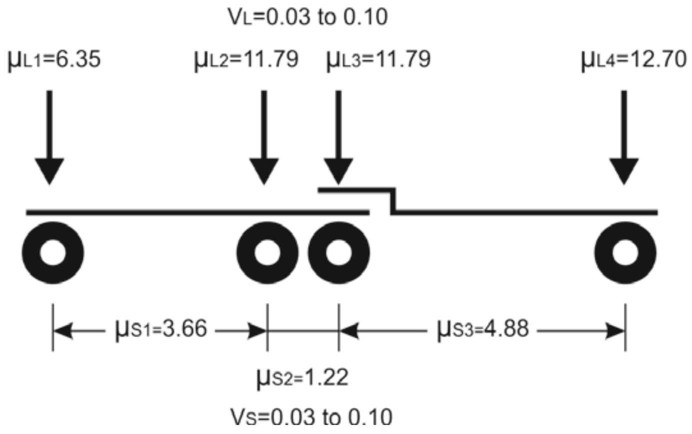


Figure 24: Model of MLC40 with load values in [t] and spacing in [m] according to STANAG.

Mean values are selected in accordance to STANAG. Coefficients of variation V_L for axle load and V_S for axle spacing are selected ranging from 3 % to 10 %, refer to Table 13. These values are similar to the investigated coefficients for axle loads of rail wagons and locomotives [73]. Generally, less variation is expected for V_s due to the fact, that the axle spacing is relatively easily measured. A simple tape or meter stick is fully sufficient. Weight measurements are in comparison much more difficult to carry out and the accuracy often depends on the weighing scale and the fact that weight might shift itself.

Table 13: Input values for wheeled MLC40

Nr.	μ_L [kN]	μ_S [m]	V_L [-]	V_S [-]
1	63.5	3.66	0.03 to 0.10	0.03 to 0.10
2	117.9	1.22	0.03 to 0.10	0.03 to 0.10
3	117.9	4.88	0.03 to 0.10	0.03 to 0.10
4	127.0	--	0.03 to 0.10	--

These limits on V_L and V_S are selected in order to study the sensitivity of results and to introduce realistic values that would represent uncertainty associated with axle loads and spacing. The maximum resulting bending moments M_i for each set of $L_{i,j}$ and $S_{i,j}$ are obtained for different span lengths ranging from 5 to 50 meters. Results are shown in Figure 25, Figure 26 and Figure 27, with the coefficient of variation of static load V_{QMLC} on the vertical axis and the range of span length in meters on the horizontal axis.

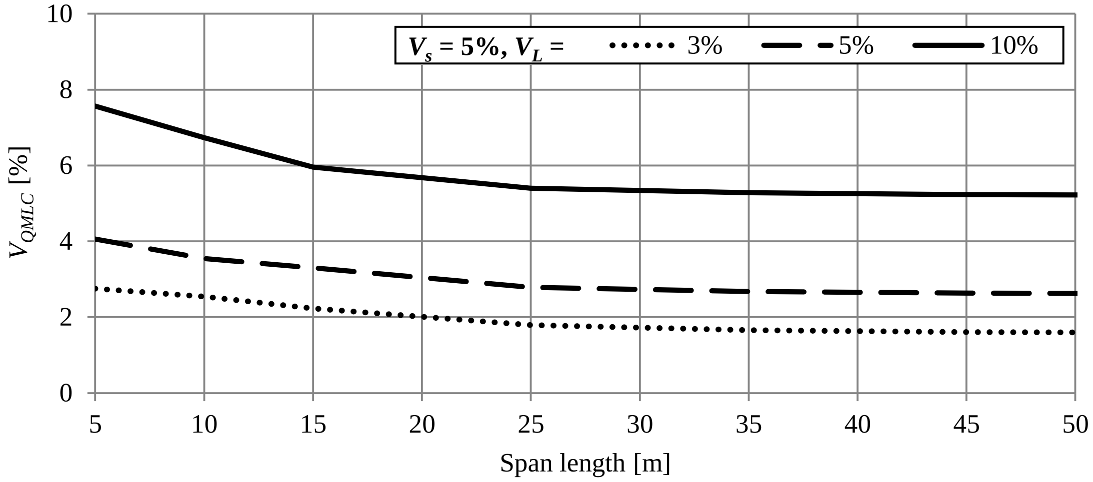


Figure 25: V_{QMLC} for constant V_S = 5% and variable V_L.

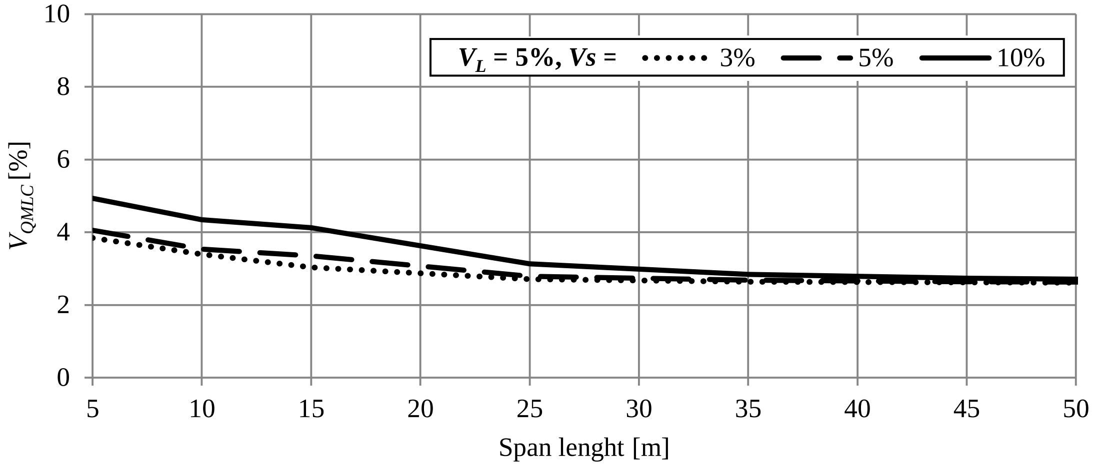


Figure 26: V_{QMLC} for constant V_L = 5% and variable V_S.

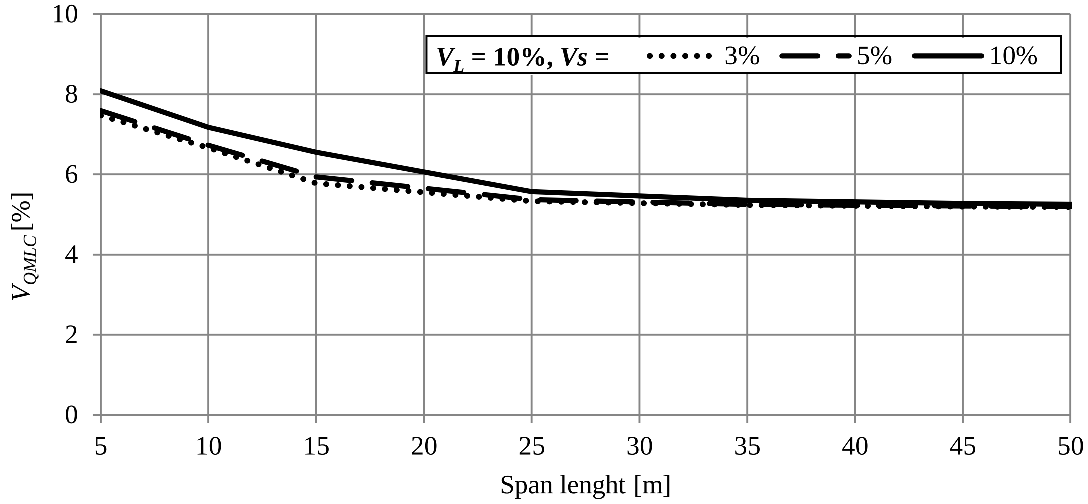


Figure 27: V_{QMLC} for constant V_L = 10% and variable V_S.

It can be clearly observed that the influence of V_L on the resulting V_{QMLC} is more significant than influence of the V_S. At the same time, the maximum value of V_{QMLC} is noticeably larger at shorter span lengths, because the variation of bending moment tends to be much smaller at larger spans due to the span length factor l^2 in the bending moment calculation. The maximum resulting variation of static load at short span lengths can be generally taken as slightly less than the value defined for V_L. V_{QMLC} afterwards decreases and at 25m span length assumes a seemingly constant value of approximately 50% V_L, however in slight dependence on the selected V_S.

5.2.2 Static system influence

The same MLC 40 vehicle as described in Section 5.2.1. is used for the simulations in this section. Since the main parameter of interest is the response of different static systems and its effect on the expected load effect, only the following two combinations of coefficients of variation spacing are considered in this section, V_S is again considered as better described due to easier measurements:

- V_S = 5% and V_L= 10%,
- V_S = 5% and V_L= 5%.

Span lengths for the simulation of static system response are selected between 10m to 50m. The influence lines representing each of the considered static systems are listed in the short form in .

Table 14.

Table 14: Short description of utilized influence lines, see Table 12 for a full definition

Nr.	Description of the Influence Line
IL0	Bending moment of a simply supported beam
IL1	bending moment in midspan of a fixed beam
IL2	Fixed end moment of a fixed beam
IL3	Bending moment in the first span of a continuous beam
IL4	Support moment of a continuous beam

The generated vehicle is always positioned in the most critical position along the bridge length according to the respective influence line. This means that the centre of gravity of a considered vehicle corresponds to the peak value of the influence line and the axle loads are in turn multiplied by influence line value indicating the bending moment at each of the positions. For the case of simple influence lines is the maximum obvious. A search algorithm is employed for the more complicated polynomial influence lines in order to identify the most critical position along the beam length. The axles are again placed accordingly to the vehicle definition and the centre of gravity. The results are plotted in following Figures for the coefficient of variation V_{QMLC} in relationship to the span lengths and different considered static systems.

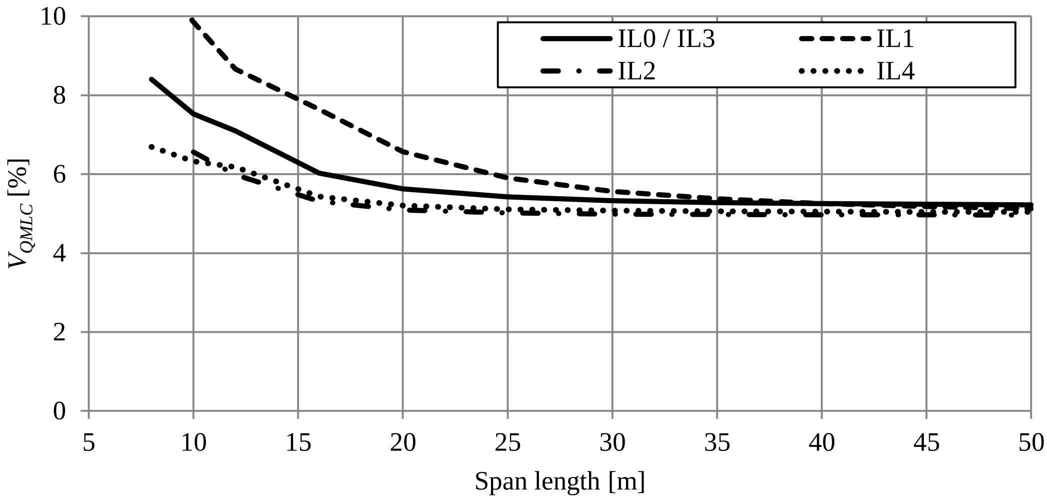


Figure 28: V_{QMLC} for different static systems; with $V_L = 10\%$ and $V_S = 5\%$.

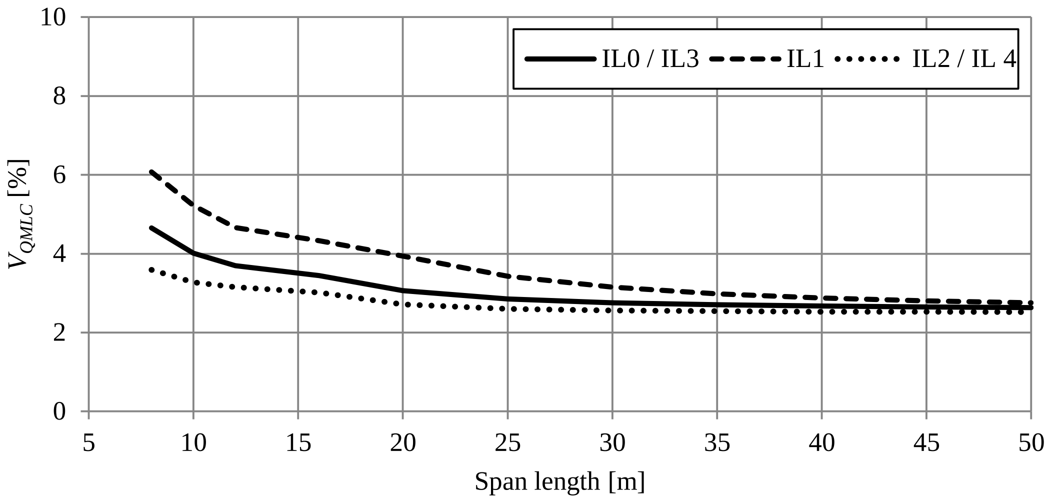


Figure 29: V_{QMLC} for different static systems; with $V_L = 5\%$ and $V_S = 5\%$.

The observed variation of static load is the largest in the case of midspan moment of a fix-end beam (IL1), while the support moment of fixed beam exhibits minimal values. As the span length increases, the resulting coefficient of variation decreases for all static systems and becomes eventually constant at approximately 35m, however once again depending on the particular values of V_L and V_S. The largest differences in static load effect are observed in short to medium span lengths up to approximately 25m. It must be noted, that the absolute difference in not significant, as the maximum results are in the range from 7% to 10% at 10m span length for V_L =10% and V_S =5%. Along with the results from Section 5.2.1 this is pointing to the fact, that the response is more sensitive to the selected values terms of V_L and V_S rather than to the various static systems.

5.2.3 Various MLC Vehicles

The main parameter of interest in this section is the static load variation due to different hypothetical MLC vehicles. The selected wheeled vehicles are specifically MLC 4, MLC 16, MLC 40 and MLC 80. The vehicles are modelled in the same manner as the benchmark MLC 40 with the mean values equal to the tabulated values in STANAG. The numerical investigation is again limited to the two combinations of coefficients of variation:

- V_S = 5% and V_L= 10%,
- V_S = 5% and V_L= 5%.

Table 13 provides an overview of some of the properties used during the evaluation of resulting V_{QMLC}, such as the total number of considered axles or the total length of vehicle. The selected vehicles provide a wide range of study parameters in form of number of axles or total length/ load of each vehicle.

Table 15: Properties of MLC used in simulations

MLC	Number of Axles [-]	Total vehicle Length [m]	Total vehicle Weight [t]
4	3	3.66	4.09
16	4	7.93	16.79
40	4	9.76	42.63
80	5	12.19	84.45

Span lengths for the simulation of a simple beam response are selected between 10m to 60m, with each vehicle positioned in the most critical position. It should be mentioned, that only contributing axles are considered, since vehicles such as MLC 80 with 12.19m length are simply too long for shorter span lengths.

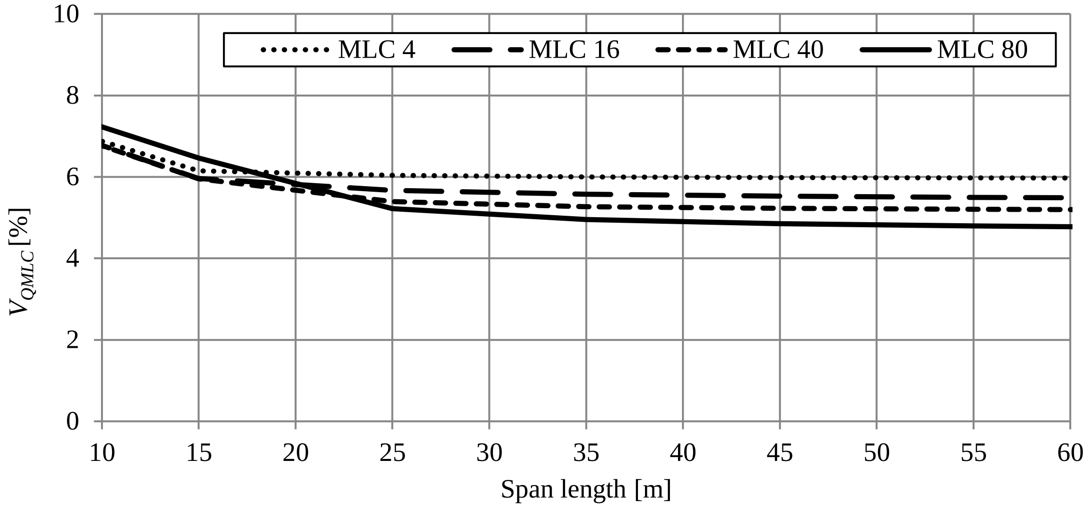


Figure 30: V_{QMLC} for V_S = 5% V_L= 10%.

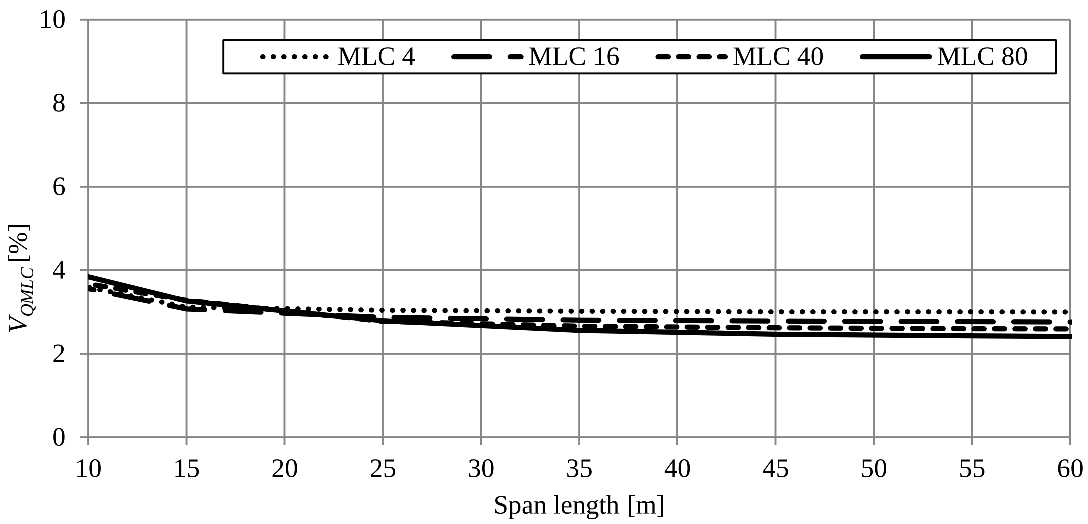


Figure 31: V_{QMLC} for V_S = 5% V_L= 5%.

The results are plotted in Figure 30 and Figure 31 showing the maximum resulting coefficient of variation V_{QMLC} for each vehicle within the considered span lengths for the two cases of V_L and V_S.

It can be clearly observed that the heavier MLCs are dominant in short spans lengths while the lighter MLCs tend to govern the variation starting at approximately 20m. This can be contributed to the total vehicle length and the number of axles. Long vehicles tend to show a larger variation on short spans. Additionally, the number of axles decreases the coefficient of variation in longer span lengths when the independently generated axle loads tend to exhibit a “cancelling effect” of potential maxima or minima of generated loads. However, all of the vehicles show seemingly constant results with negligible differences at short span at maximum difference between 6% and 3.8% at 60m span length for the case of V_S = 5% V_L= 10%.

The most important observation is that the results from MLC 40, which is selected as the benchmark vehicle for the thorough investigation of all aspects such as static systems, are very similar to the rest of the selected vehicles. No major differences are observed and therefore all the obtained results and characteristics of static load variation, due to the various parameters, are fully transferable and can be used in general terms for all Military Load Classes.

5.2.4 Short span response

The routine for calculation of M sets of bending moment has to be slightly modified in determination of short span response because the long vehicles have to be physically limited. Short spans are in the most cases simple beams, since continuous and fixed beams do not generally consists of sufficiently short span lengths. The vehicles are defined with its STANAG axle loads and spacings, furthermore, a maximum single axle load is considered in the determination of maximum bending moment. Various vehicles are checked for the short span response and the benchmark vehicle MLC 40 shows the influence of limited axles on the resulting coefficient of variation.

It can be observed, that the larger values of V_{QMLC} are associated with the situations where the vehicle length is either approaching or is just above the span length. That is due to the nature of the routine where sample of moments is evaluated and axles not contributing to the maximum load effect are disregarded. Therefore it could be the case that vehicle *i* is contributing with all its axles, while vehicle *i+1* with one axle less, because in this case, the random spacing variables added up to excessive length and therefore is one axle omitted in estimation of the load effect, see Figure 32 for an illustration.

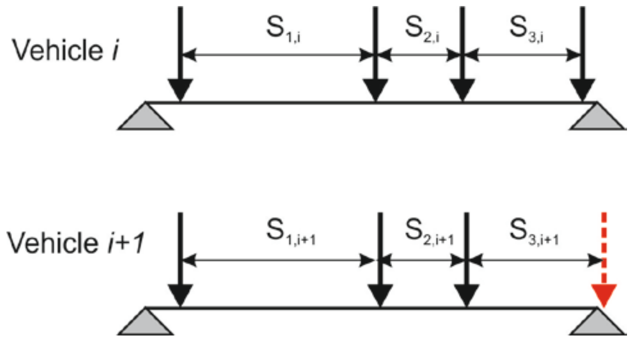


Figure 32: Example of vehicle approaching/exceeding the span length.

Figure 33 shows the results of MLC 40 detailed analysis for span lengths ranging from 0m to 20m, where one to four axles are considered in the determination of maximum static response. The resulting bending moments are plotted for the different vehicle configurations, i.e. for the configuration where only some of the axles are considered. It can be clearly observed, that the single axle is dominating the response up to 4m, afterwards is the double-axle decisive. The three axle configuration is briefly governing the maximum bending moment between 7m to 11m. Afterwards the whole vehicle is relevant for the maximum load effect. This is interesting in the respect, that the variation of static load should be considered only in those cases where the related response in terms of bending moment is maximal.

Clearly, the vehicle with fewer axles, but rather in the critical position on the bridge, is producing larger static load for the given range of span length in comparison to a longer vehicle and its axles in non-critical positions. Therefore in this case, only the relevant coefficients of variation should be considered. At the same time, more axles lead to a larger number of random variables *L* and therefore less of the variation in load effect – or explained in different terms - cancelling effect of loading increases with the number of axles. For example, the randomly generated value of axle *j* is quite small, with increased number of axles increases the chance for a different axle to be quite large and therefore mitigate the effect of axle *j*.

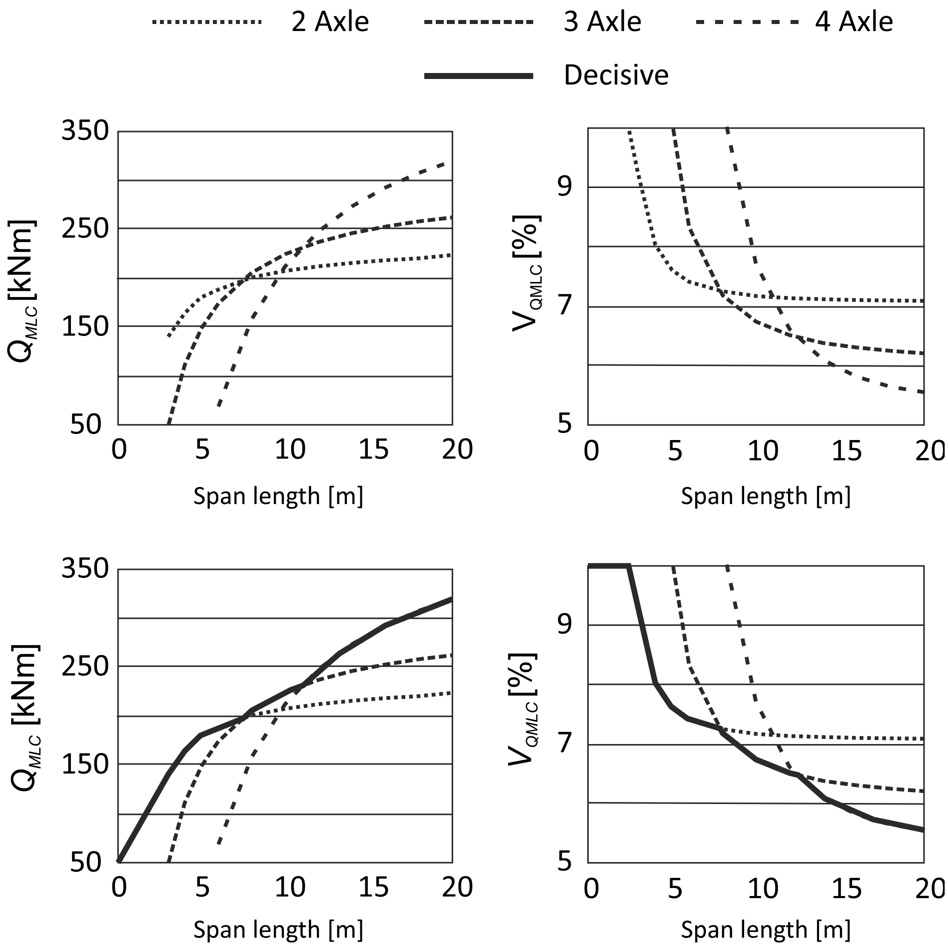


Figure 33: Comparison of Q_{MLC} load effects and V_{QMLC} resulting from different number of contributing axles; with V_L = 10% and V_S = 5%.

For the case of single axle, V_{QMLC} is always equal to the selected V_L. As can be observed in with single heavy dark line defining the decisive response, the V_{QMLC} rapidly decreases as more axles become contributing to the load effect. At 15m span length the variation essentially follows the results from Section 5.2.1. where the influence of various V_L and V_S was investigated. In this section, only one case of V_L = 10% and V_S = 5% is considered, but it provides enough of general insight into the short span response and static load variation.

Since the single axle governs the maximum V_{QMLC} only at certain span lengths, the exact decisive length should be defined for all MLC vehicles. Following graphs in Figure 34 show the ratio of bending moments due to the single axle and due to the double-axle. The lighter MLCs tend to exhibit larger disparity of the bending ratio, but at approximately 5m, most of the hypothetical vehicles approach the 1.0 factor, which means the double-axle begins to govern the response. All of the heavier MLCs are below the 1.0 factor at 4.5m span length. It can be therefore concluded, that V_{QMLC} is equal to the selected coefficient of variation of axle load V_L only in the span lengths up to 5m.

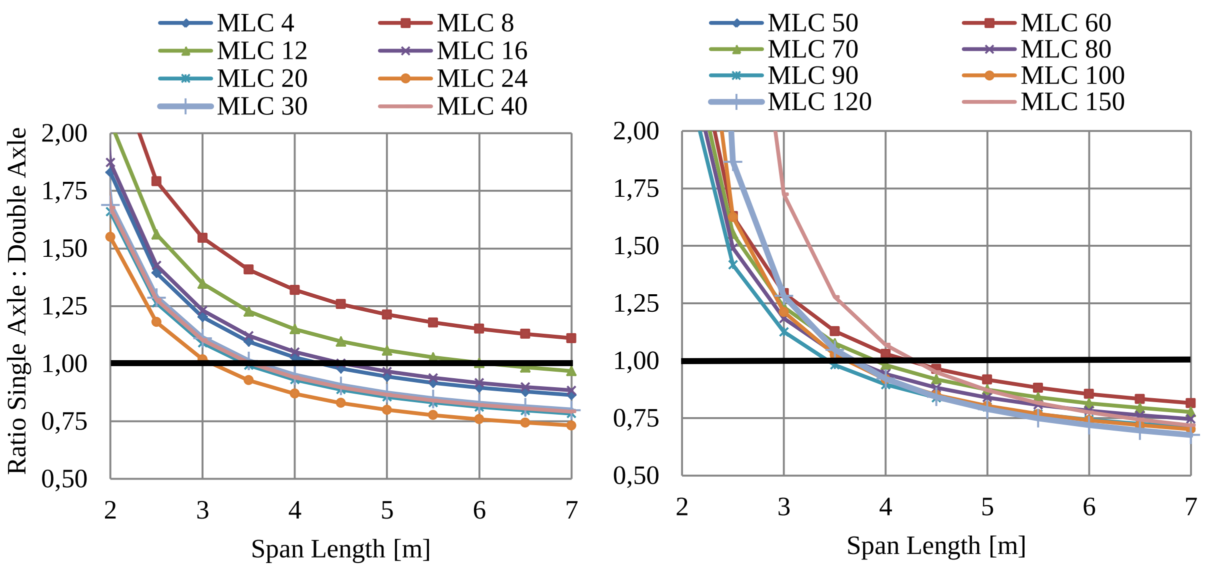


Figure 34: Bending moments ratio of Single Axle to Double Axle for MLC 4 to MLC 150.

5.2.5 Random Vehicular Traffic Flow

When the military traffic is considered as a whole, it would be prudent to consider and account for a random vehicular flow. In reality, there is a mix of various MLC vehicles crossing a bridge. Considering for example a MLC 40 bridge, then there are obviously going to be crossing vehicles with lower MLC value then the bridge MLC_{max} 40. This is simply due to the fact that the real vehicles are classified not only in the standard Military Load Classes 30 or 40, but also as for example MLC 32, 36 or 38. If MLC 40 loading on a bridge is considered, then the actual traffic considered for establishment of the maximal load effect should be composed of vehicles within the limit range of the adjacent lower MLC 30 and maximum allowable MLC 40 – simply a mixture of vehicles from MLC 30 through MLC 40.

The simulation of random vehicle flow is accomplished by taking in account static load coefficient of variation V_{QMLC} for a single vehicle and tabulated bending moments. During the period considered for reliability analysis can *i* potentially different MLC vehicles pass the bridge. The made up of the traffic in terms of exact MLC classes is however unknown prior to actual loading. Two examples are considered in the analytical investigation. That is a bridge rated as MLC 40, where the maximum load class traffic is the MLC 40 and the traffic flow is considered as random from MLC 31 to MLC 40, with MLC 31 representing the lower limit. Anything below MLC 31 is disregarded as it belongs to a lower bridge class MLC 30. Similarly, a bridge rated as MLC 16 with traffic composed of MLC 13 to MLC 16 is evaluated. The main parameter considered is V_{QMLC} for a single vehicle, assuming that all the vehicles within military have the same probabilistic model. A simplistic assumption but seen as a reasonable one with regards to the section investigating different MLC vehicles.

If it is assumed, that the traffic is made up of random vehicles with unknown distribution in the sample, then the expected load effect can be formulated as a vector form assembly of all MLC load effects and their respective simulated random frequency. V_{QMLC} is simply a statistical matter of evaluating the vector sample of all effects for given span length. Table 16 shows two considered examples and the resulting V_{QMLC} for random flow in dependence on selected V_{QMLC} of vehicle.

Table 16: Coefficient of variation for single vehicle and random column of vehicles

MLC 31 - 40			**MLC 13 - 16**	
Single Class	Random Flow		Single Class	Random Flow
V_{QMLC}	V_{QMLC}		V_{QMLC}	V_{QMLC}
3%	8,50%		3%	8,30%
5%	9,00%		5%	8,80%
10%	12,80%		10%	12,80%

As expected, the random column of vehicles exhibits a larger variation of the expected static load due to the presence of various MLC vehicles. It should be obvious, that the mean value of random flow traffic is lower than MLC_{max} considered for the bridge, in fact with sufficiently large i and random uncorrelated traffic, it equals to the actual mean value of analytically considered traffic flow. For the previous two examples it is then 35.5 for MLC 40 bridge and 14.5 for MLC 16 bridge. These are not real loadings, only numerical expression of the mean static load of the considered traffic flow. However, the assumption of random uncorrelated traffic is not always valid, because the randomness of military traffic can be largely limited, as operational needs may allow only certain vehicles to cross the bridge in consideration. In that case, the mean value is not equal to the general assumption as described above, but is rather dependent on the actual traffic made up. These considerations are in larger detail investigated in 5.4 where the load model and the characteristic static load are discussed.

5.2.6 Shear Response

Shear needs to be mentioned, since MLC number can be assigned on the basis of bending moment or shear reaction. Therefore, the variation of the shear load effect due to variations in axle loads and axle spacing should be investigated. Bearing the results from previous section in mind, the investigation of shear variation can be simplified and V_{QMLC} relatively quickly estimated on the basis of following assumptions:

- the variation in axle loads and axle spacings are selected under the same considerations as for the bending moment,
- short span response is dictated by the single axle, where V_{QMLC} equals to the selected V_L as shown in bending moment investigations,
- response of different static system for shear can by represented by a single influence line (Figure 35),

- random vehicular flow results are valid for shear.

Numerical investigations are performed using a single vehicle positioned at the critical position to produce the maximum shear. One axle is directly at the support and the rest is spaced according to the vehicle configuration. Maximum number of axles that potentially fit the span always governs the maximal response. The same selected coefficients of variation V_S and V_L are considered as for the bending moment investigations; see Table 13: Input values for wheeled MLC40.

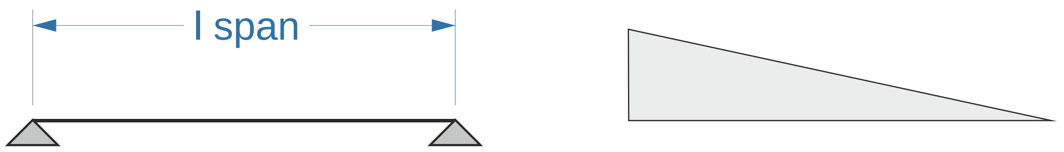


Figure 35: Simple span and influence line for shear response.

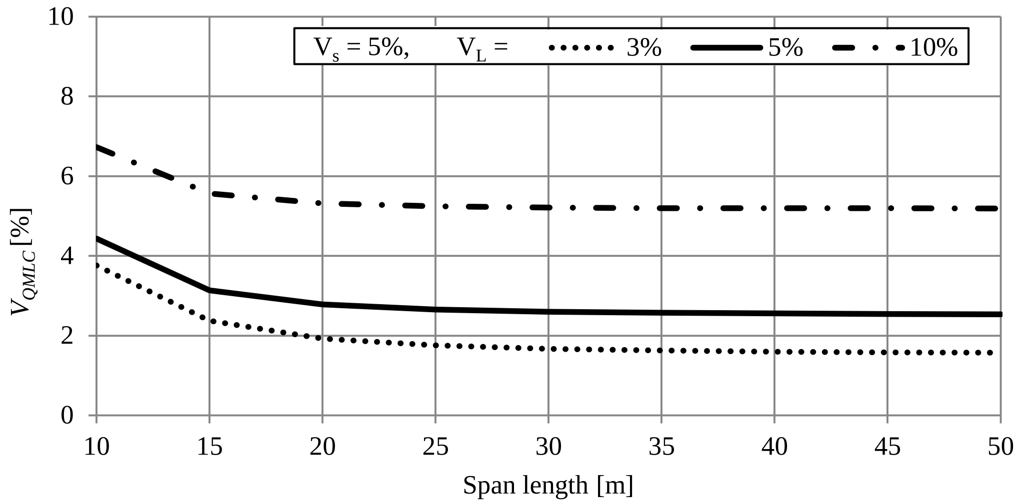


Figure 36: V_{QMLC} for constant $V_S = 5\%$ and variable V_L.

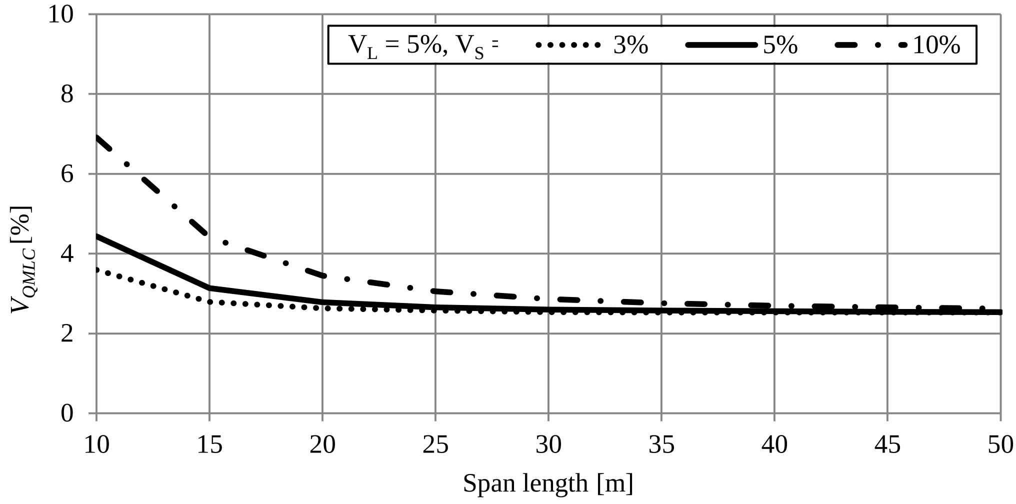


Figure 37: V_{QMLC} for constant $V_L = 5\%$ and variable V_S.

The results are showing that the major influence on V_{QMLC} is the selected V_L. For the case of constant V_S and varying V_L the resulting V_{QMLC} is directly proportional to the selected V_L in short span. It continues to decreases to approximately 25m it equals to about 50% of the original value and becomes seemingly constant. The original selection V_L influences for results for the whole spectra of considered span lengths.

5.3 Tracked Vehicles

Tracked vehicles are defined by their lengths and total weights. The considerations of tracked vehicle static load coefficient of variation V_{QMLC} can be in comparison to wheeled vehicles much simplified. Without any simulation it should be clear that the maximum response for both bending and shear is mainly dictated by the coefficient of variation of the total load V_L (uniform load). According to the simple statics, the total length S variation has a marginal role in the calculation of internal forces given a constant uniform load, except for shorter span lengths just above the selected vehicle length, where the sensitivity may be larger. Nevertheless, numerical simulations are performed in order to quantify the influence of the total length variation and to visually compare the graphical results to the previous section concerned with wheeled vehicles.

The main parameters for the investigation are therefore set as the coefficients of variation of both total load V_L and total length V_S as summarized in Table 13. The vehicles are generated in the same manner as the wheeled vehicles and the calculations steps from Section 5.1.1 are adopted.

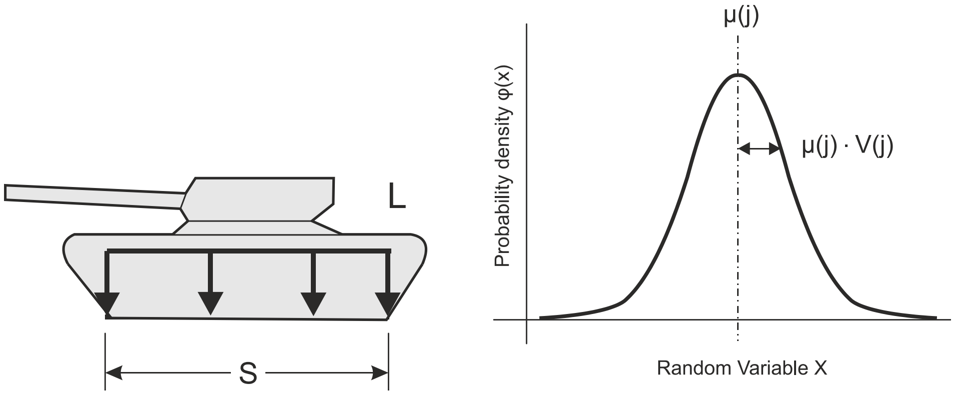


Figure 38: Example of required input for tracked vehicle generation, *j* denotes the either the total load *L* or total length *S*.

It is however only necessary to simulate the uniform load *L* and total length *S* as a random variables. MLC 40 is selected for the analysis (Figure 13). Only simple span “IL0” and fixed end beam “IL1” static systems are considered during the numerical investigations.

Table 17: Input values for tracked MLC40

Configuration	μ_L [kN]	μ_S [m]	V_L [-]	V_S [-]
1	362.9	3.66	0.03	0.05
2	362.9	3.66	0.05	0.05
3	362.9	3.66	0.10	0.05
4	362.9	3.66	0.05	0.03
5	362.9	3.66	0.05	0.10

The total load is assumed to be uniformly distributed along the vehicle length (Figure 39a). This results in the center of gravity corresponding to the center of vehicle. In reality, the weight distribution might be slightly skewed or pitched around the center of gravity (CG) as shown in Figure 39b. However, it is expected that the results will still hold, since the non-uniform load can be idealized as a set of point loads and therefore resembling a form of a wheeled vehicle. Moreover, STANAG assumes a perfectly uniform loading of tracked vehicles.

The results of numerical simulations are plotted for the static load coefficient of variation V_{QMLC} in relationship to the span lengths. Two static systems are considered - simple beam results are shown in Figure 40 and Figure 41, while the results of a fix-end beam are summarized in Figure 42 and Figure 43.

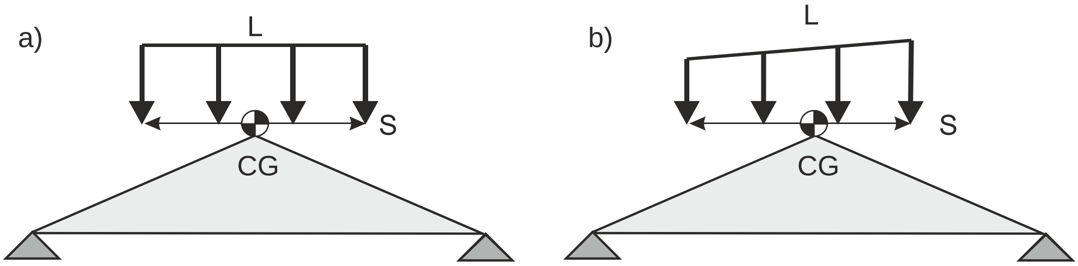


Figure 39: Generated tracked vehicle in a critical position with a) uniform load b) skewed distributed load.

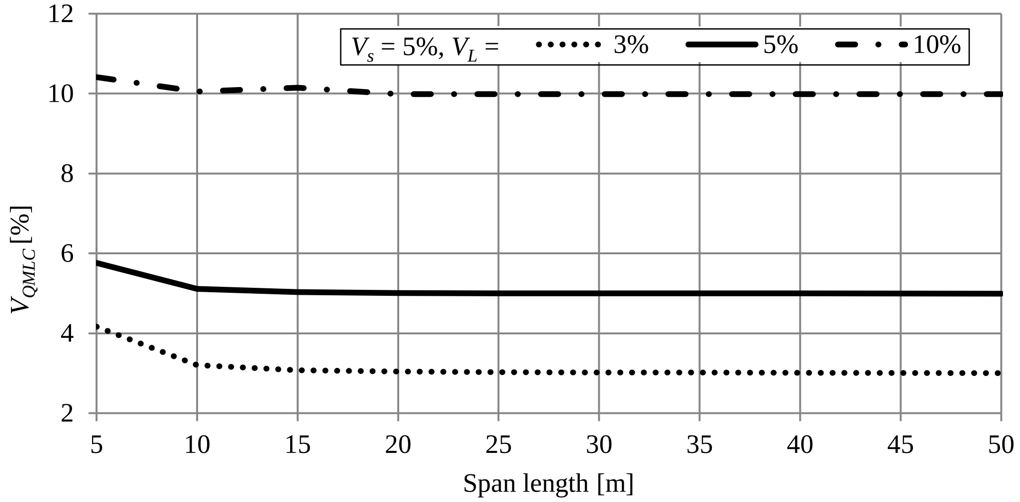


Figure 40: V_{QMLC} for constant $V_S = 5\%$ and variable V_L; simple span IL0.

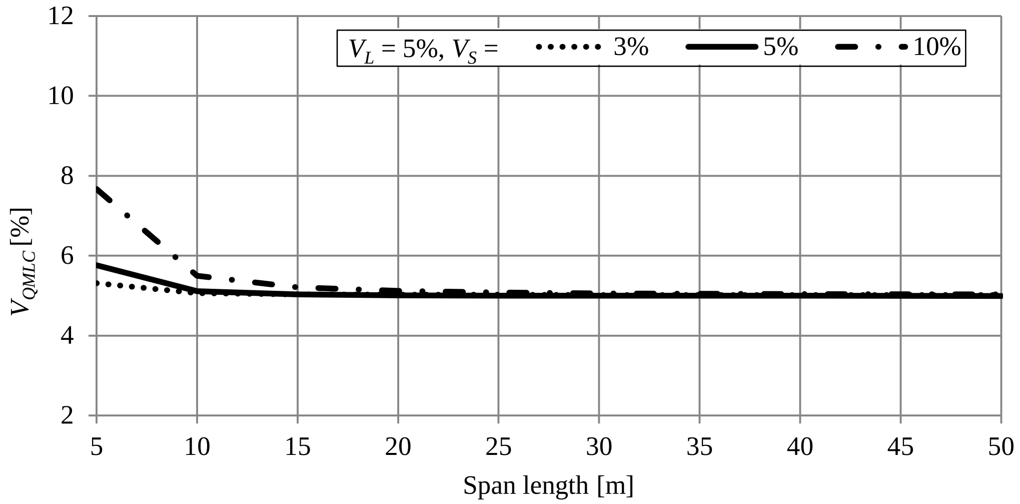


Figure 41: V_{QMLC} for constant $V_L = 5\%$ and variable V_S; simple span IL0.

As expected, the particular response of a simple beam in terms of V_{QMLC} is clearly dictated by the selected V_L. The seemingly constant V_{QMLC} for the whole range of considered span lengths corresponds to the selected V_L when V_S is held constant, with minor increase at 5m (Figure 40). The influence of V_S at constant V_L in Figure 41 clearly shows that at span lengths > 15m the particular selection of the coefficient is insignificant; however at 5m, the variation of V_{QMLC} is quite high due to the sensitivity of bending moment calculations relative to the span length and the considered vehicle length. Short span topic is investigated in further detail. It can be concluded, that for most of the relevant situations with long enough spans, V_{QMLC} is directly tied to the value of V_L.

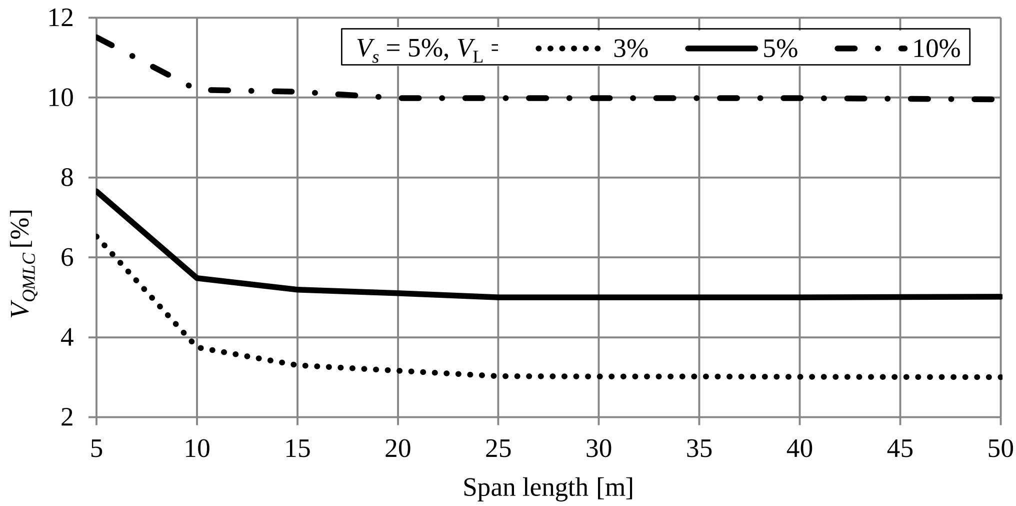


Figure 42: V_{QMLC} for constant V_S = 5% and variable V_L; fixed end beam IL1.

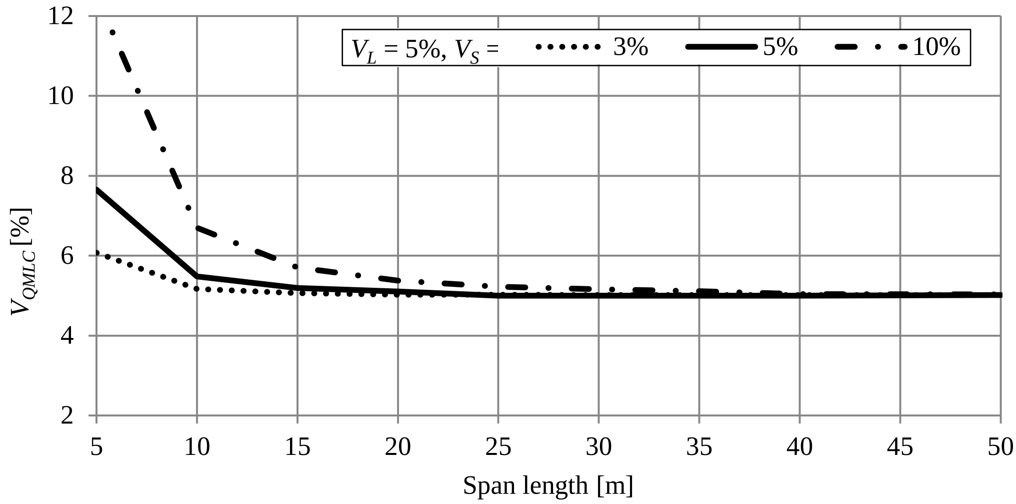


Figure 43: V_{QMLC} for constant V_L = 5% and variable V_S; fixed end beam IL1.

The results considering the fix-end beam show a similar tendency, although the static influence of a fix-end beam on the resulting V_{QMLC} is more apparent at the shorter span lengths, due to the increased sensitivity of V_S, which can be contributed to the nonlinear curvature of the influence line.

The results in Figure 42 support quite clearly the tendency of V_{QMLC} to be dominated by selected V_L as is the case of simple beam. The influence of V_S on V_{QMLC} is much more evident in short span as the difference between V_{QMLC} ranges from 6% to 12%. However, with increasing span length this high sensitivity is quickly dissipated as the influence of a short span phenomenon diminishes and span length *l* factor becomes the major variable in bending moment calculations. Even though that at larger span lengths the V_{QMLC} is largely influenced by V_L, it has to be kept in mind, that the particular selection of V_S is decisive at 5m to 10m. The short span response is in comparison to wheeled vehicles determined by both the total load and length.

Apparently, the applied uniform load random variable q on a short beam is determined as:

$$q(\mu,\sigma)=\frac{L(\mu,\sigma)}{S(\mu,\sigma)}$$ Eq. 5-9

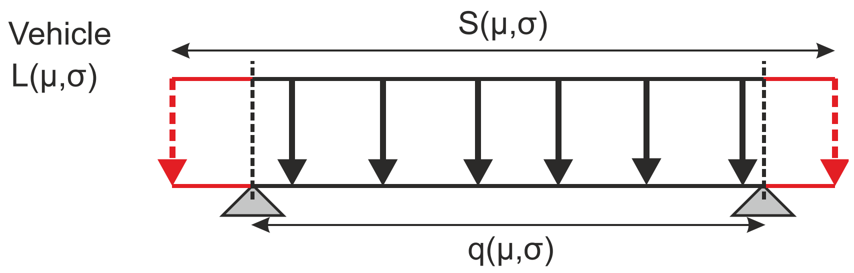


Figure 44: Example of tracked vehicle approaching/exceeding the span length.

The bending moment and shear reaction is then a directly tied to the properties of the uniform load q and V_{QMLC} can be calculated analytically as follows:

$$V_{QMLC}=\sqrt{V_L^2+V_S^2}$$ Eq. 5-10

The resulting coefficient of variation V_{QMLC} is summarized in Table 18 for all of the selected V_L and V_S including both low and high combinations. In spirit of the previous section regarding the wheeled vehicles, the relevant range of V_L and V_S yields a coefficient V_{QMLC} at approximately 0.04 to 0.07 for the short span response.

Maximal length of a wheeled vehicle in STANAG is given as 7.32m for MLC150. Common heavy tracked vehicles (MLC 40 to MLC 80) are approximately from 3.66m to 4.88m long. It can therefore concluded, that the results of short span response are applicable up to approximately 5m. The response beyond these lengths is heavily influenced by the selected V_L.

Table 18: Short Span Coefficient of Variation of Static Load

Nr.	V_L [-]	V_S [-]	V_{QMLC} [-]
1	0.03	0.03	0.04
2	0.05	0.03	0.06
3	0.10	0.03	0.10
4	0.03	0.05	0.06
5	0.05	0.05	0.07
6	0.10	0.05	0.11
7	0.03	0.10	0.10
8	0.05	0.10	0.11
9	0.10	0.10	0.14

5.4 Characteristic Load

Previous sections developed stochastic properties for military traffic on bridges. Especially the coefficient of variation V_{QMLC} was investigated in large detail. This was accomplished for a single MLC class corresponding to the maximum allowable STANAG class and the random traffic flow of real MLC vehicles. It was clearly shown on an example, that a bridge rated MLC 40 can be considered under STANAG loading with mean value equal to MLC 40 and narrow description in terms of coefficient of variation. The bridge could be additionally considered under an independent distribution of real vehicle random flow. Given the same probability for MLC 31 to crosses the bridge as the probability of for example MLC 35 or MLC 40, then the mean value equals to the numerical evaluation of vehicles that have crossed. In this case it equals to MLC 35.5. Naturally, larger coefficient of variation of the static loading is associated with the random flow.

It is necessary to define the characteristic load of military traffic for the successful definition of a safety format. Generally, the characteristic value should be considered on the basis of *i*-passages of vehicles, representing the number of vehicles passing during the reference time period or the service life of the bridge. This leads to a set of *i* load effects *Q*, such as bending moments or shear forces. The characteristic load is expressed in terms of quantile value of the whole sample *Q*. This value is often between 90% to 98% quantile for the traffic load effect. Self-weight (permanent action) is generally considered as 50% quantile and therefore mean. The characteristic value of civilian traffic load corresponds to a 1000-year return period (EN 1991-2 [4]) while generally a nominal (mean) value is considered for military vehicles as defined STANAG standard vehicle. The random traffic flow has certainly a lower mean value. It is questionable whether the lowered mean with appropriate characteristic value should be considered and if it brings any advantage to the evaluation of bridges under military traffic. The purpose of this section is to determine whether the considerations of a random traffic are appropriate.

A conservative approach for the determination of characteristic load is provided by the following comparison of design load effects resulting from stochastic properties according to allowable maximum class and a random vehicular flow.

The indicative design load $Q_{MLC.id}$ not including model uncertainties and dynamic amplification is calculated assuming normal distribution of this variable as:

$$Q_{MLC.id} = \mu_{MLC} - \alpha_E \cdot \beta \cdot \sigma_{MLC} \qquad \text{Eq. 5-11}$$

with selected $\alpha_E \approx -0.7$ as the approximated FORM sensitivity factor, $\beta = 3.8$ as target reliability index in accordance with EN 1990 [45], and the mean value μ_{MLC} and the standard deviation σ_{MLC}. This simplified expression delivers only an indication of design static loading for comparison purposes.

Left hand side of Table 19 corresponds to a single STANAG MLC class representing the hypothetical vehicle and its mean value μ_{MLC} = 40 and the standard deviation corresponding to the results from simulations, where three values are selected for study purposes. Right hand side then represents a random traffic flow with a sufficiently large number of vehicles corresponding to μ_{MLC} = 35.5 and standard deviation selected in accordance to Section 5.2.5.

Table 19: Comparison of Design Load Effects

	MLC 40				**MLC 31-40**			
No.	μ_{MLC}	V_{QMLC}	σ_{MLC}	$Q_{MLC.id}$	μ_{MLC}	V_{QMLC}	σ_{MLC}	$Q_{MLC.id}$
1	40	3.00%	1.2	43.19	35.5	8.50%	3.02	43.53
2	40	5.00%	2.0	45.32	35.5	9.00%	3.20	43.99
3	40	10.00%	4.0	50.64	35.5	12.00%	4.26	46.83

It can be observed from Table 19 that selection of MLC 40 with mean values corresponding to STANAG is a conservative solution in terms of the maximum value of design load effect $Q_{MLC.d}$; the marginal difference in design load effect at V_{QMLC} 3%, respectively 8.50% for the random flow, can be neglected.

It must be noted that the particular selection of exact quantile value selected for expression of the characteristic load is secondary in the ultimate limit state as it has larger influence in the serviceability limit state. The design value in ULS is affected by the selected mean value and standard deviation. This in turn affects the particular value of the partial factor.

Moreover, the assumption of a random vehicular traffic is not always valid, as military operates differently than civilian sector and only a single class of vehicle might utilize the bridge, or the traffic is heavily skewed due to presence of certain MLC. Considerations of random vehicular flow are therefore to be treated on a case-specific basis.

It is therefore consistent to consider the characteristic values as mean values according to STANAG within the proposed safety concept. A general approach is conservatively settled with Q_{MLC} equal to Q_{max} of the considered bridge class. It is proven reliable in the desired range of load effect variation.

Determination of characteristic values as quantile values is usually associated with extrapolation of expected loads due to passage of vehicles as for example 98% of all loads are lower than selected characteristic load. This becomes even more complicated with time variable loading. However, the suggested approach with mean values according to STANAG definition of hypothetical vehicles simplifies the evaluation of expected loads, and the fact, that STANAG loadings are time invariant supports this decision.

5.5 Dynamic Amplification Factor

Bridges as flexible structural members have the characteristics of vibrating under the dynamic action of a vehicle and thus introducing additional loading. This should be considered in the appropriate loading model or in the development of partial factors. There is currently no specification in STANAG regarding either the deterministic dynamic amplification factor or the probabilistic model of dynamic amplification.

In general, the dynamic effect of traffic load is influenced by a number of factors, such as maximal bridge span length, bridge natural frequency, vehicle weight, axle loads, axle configuration, position of a vehicle on the bridge, quality of pavement, stiffness of structural members, etc. Considerable differences exist between different approaches and no consensus seems to be reached among the scientific community. However, large contribution may be attributed to vibrations of the vehicle induced by the road profile roughness depending on the velocity and surface unevenness between the approach and the bridge deck [10], [30], [103]. For heavy loads and smooth roadway the amplification factors remain typically below 1.1 [11], [77].

The most accurate way to determine a dynamic amplification factor δ (DAF) is to use full-scale dynamic bridge testing under controlled or normal traffic conditions. Yet, this approach is unsuitable for the purposes of military traffic assessment, since it is aimed at a single specific bridge and usually can envelop only limited vehicle dynamic characteristics, thus is difficult to be related to general bridge assessment under military loading. A general approach used in the earlier years was to tie the bridge span length to the DAF, but PAULTRE ET AL. [102] note that it has been recently replaced by the relationship between the dynamic response and the natural frequency of bridge. The problem cannot however be reduced to a simple comparison of excitation and fundamental natural frequencies [30].

The review of literature does not provide a single value for dynamic amplification that could be used for military vehicles in general terms. The dynamic amplification factor varies from country to country or even agency to agency due to different assumptions and test outcomes.

Recent studies proved that increasing static loading leads to a lower mean value of δ [35], [68], [77] and to a reduced coefficient of variation [58]. At the maximum (critical) loading level the dynamic component of the total load effect is very small; light vehicles may produce comparably higher dynamic amplifications, but at a low static load effect and thus can be neglected when determining the total loading.

Dynamic factors are already built into the Eurocode traffic loading model LM1. The model is based on the numerical simulations of one, two and four loaded lanes, see Figure 45. These factors are naturally quite conservative in order to capture an entire range of bridges under various loading scenarios with a large number of uncertainties associated with the vehicle-bridge interaction [11].

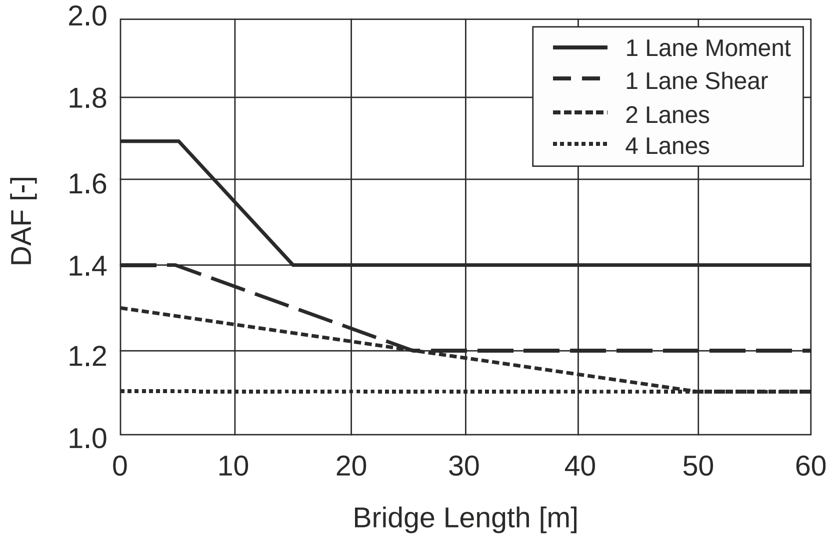


Figure 45: Dynamic Amplification Factors built into Loading Model in Eurocode [11].

It can be observed, that the factors are quite high in short span range and the factor for one lane moment remains relatively high even at lengths beyond 15m. In comparison, AASHTO [1], for example, operates with a 1.33 dynamic amplification for design and load rating.

5.5.1 Military Recommendations

Since this work is concerned with military vehicles crossing over the bridges, relevant dynamic interaction with the bridge should be investigated from this point of view. It must be noted that the following references are mainly concerned with the deterministic value of the dynamic amplification factor and no regard is given to the stochastic properties.

A form of a military standard – the Trilateral Code [127] – provides the dynamic amplification factors for the use in a deterministic verification (Table 20). Possible differences in dynamic properties of tracked and wheeled vehicles are disregarded.

Table 20: Dynamic amplification (impact) factors for clear span bridges

Location	Bending Moment and Deflection	Shear Force
Interior	1.15	--
Ramp	1.20	1.2

The explanation about the provided factors mentions that the selected values cover modern suspension vehicles up to maximum speed of 40 km/h for vehicles < MLC 30 and up to 25 km/h for > MLC 30. Older suspension and high pitch inertia vehicles should be limited to 25 km/h and 16 km/h for < MLC 30 and > MLC 30 respectively. There is no mention of particular span lengths of bridge frequencies in consideration and it is therefore applied for all cases.

Another mention of deterministic dynamic amplification can be found in [88] for the assessment of the specifically fabricated steel bridge under specified military loading. There are essentially three types of prefabricated bridges for given range of span lengths with well-known and analyzed properties. The normal crossing is somewhat regulated by the bridge geometry and approach yielding a factor δ = 1.1, unfortunately with no mention of stochastic properties. Caution and risk crossing are listed with δ = 1.0.

HOMBERG [63] in his work regarding the military classification of bridges on the basis of calculating the equivalent civilian design load notes the difference between tracked and wheeled vehicles by assigning maximum different dynamic amplification factor according to DIN 1072 [40] for each of the vehicle type, see Eq. 5-12. Figure 46 shows the results plotted for various span lengths.

$$1.0 \leq \delta = 1.4 - 0.0081 \cdot l \leq \begin{Bmatrix} 1.10\,for\ \ tracked \\ 1.25\,for\ \ wheeled \end{Bmatrix} \qquad \text{Eq. 5-12}$$

It can be observed that the wheeled vehicles are to be multiplied by a higher dynamic factor at shorter span length. With increasing span length the factors become equal at approximately 38m and decrease to a unity value of dynamic amplification for span lengths larger than 50m. It must be however noted, that these dynamic amplification factors are based on the outdated German structural norm DIN 1072 [40].

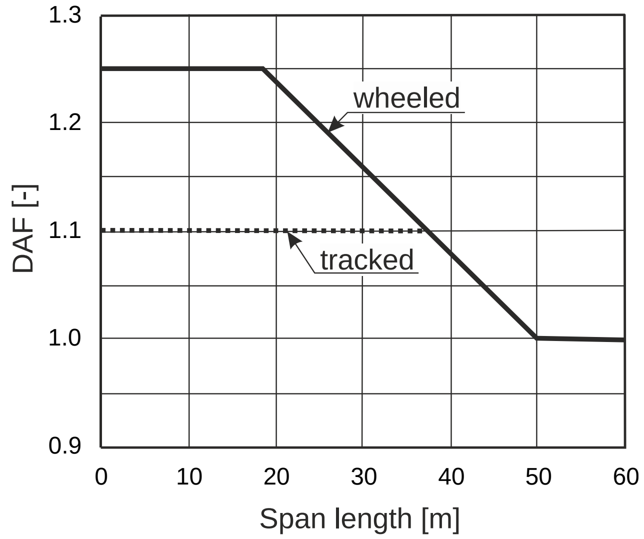


Figure 46: Dynamic amplification factor [63].

The field study of the dynamic factors on reinforced concrete T-beam bridges was performed by TRIMBLE AT AL. [128]. Two different vehicles were used in the field testing, one representing a common civilian truck and one selected military vehicle. The vehicles crossed over two T-beam bridges (12m and 10.5 span lengths) in various locations with respect to the center line. Static loading was performed with approximately 3km/h, dynamic testing with 50km/h. The results of dynamic amplification were unfortunately inconclusive since it was difficult to control the position of the military vehicle in transverse direction, a particularly important consideration for the exterior girder. Due to these difficulties, it was impossible the sufficiently describe the resulting DAF. However, it was noted that for successful identification of dynamic amplification factor it is necessary to perform a parameter study influencing the dynamic interaction between the vehicle and the bridge.

5.5.2 Assessment and Rehabilitation of Central European Highway Structures

Latest testing and calibration of variable loading was accomplished in the Assessment and Rehabilitation of Central European Highway Structures (ARCHES) project [10]. The results can be partially applied towards military loading, since it was concerned with a number of facts directly applicable to dynamic interaction of military vehicles and bridges. These include pre-existing vibrations as it is in the case of convoys, a comparison of normal to heavy traffic, and an influence of road profile on the dynamic amplification and its variation. It must be noted, that generally, military vehicles possess stiffer shocks when compared to their civilian equivalents from the same manufacturer (for example Land Rover Defender). Not all vehicles have however their civilian equivalents, such as DINGO or L2A1, and vice-versa. It is therefore somewhat difficult to directly adapt the ARCHES recommendations. It is especially apparent in the case of tracked vehicles or wheeled vehicles with shocks specifically designed for heavy terrain. Large difficulties dwell in portraying the realistic behavior of leaf spring and therefore the interaction between the bridge and a vehicle [30]. Additionally, military vehicles can be quite heavy, but do travel at slower velocities.

Within the ARCHES project a number of bridges were experimentally tested under civilian loading [11]. For heavy loads and smooth roadway the amplification factors remained typically below 1.1, but unevenness at the bridge approach or a damaged roadway surface may lead to higher values.

Therefore, a number of factors are pointed out that can affect the dynamic interaction between the bridge and a vehicle and may lead to a smaller/larger mean value of dynamic amplification and coefficient of variation. These factors or conditions include:

- deterioration of the bridge,
- pre-existing vibration,
- road profile smoothness,
- normal traffic vs. exceptionally heavy traffic.

Numerical modelling techniques were employed in order to investigate the influence of the above parameters on the dynamic amplification within the ARCHES project [12]. Some of the results are quite intuitive, such as the presence of a crack in the midspan representing the model situation of a deterioration of the bridge and yielding a larger deflection under a moving load. Other damage modes are more difficult to be quantified. Pre-existing vibration is particularly interesting topic for military loading, since military vehicles often operate in convoys with a prescribed 30.5m minimum spacing. Clearly, a bridge that is excited by a vehicle and is in the state of vibration is expected to exhibit a different response under another crossing vehicle when compared to a stationary bridge [11]. The main factors of interest are the spacing between vehicles, the velocity and the road profile smoothness. Using numerical models, two identical 5-axle trucks traveling at the same velocity were used to simulate within the project the relative importance of the preceding truck on the response due to the following truck. The second truck was selected with a high GVW (gross vehicle weight) to simulate the effect of critical loading, since high dynamic amplification is only of interest when it occurs in conjunction with such loading. Figure 47 shows the results of the simulation. It is evident, that at low velocity the amplification is marginal, supported by the fact that even single vehicle at low velocity exhibits low dynamic amplification [58], [102].

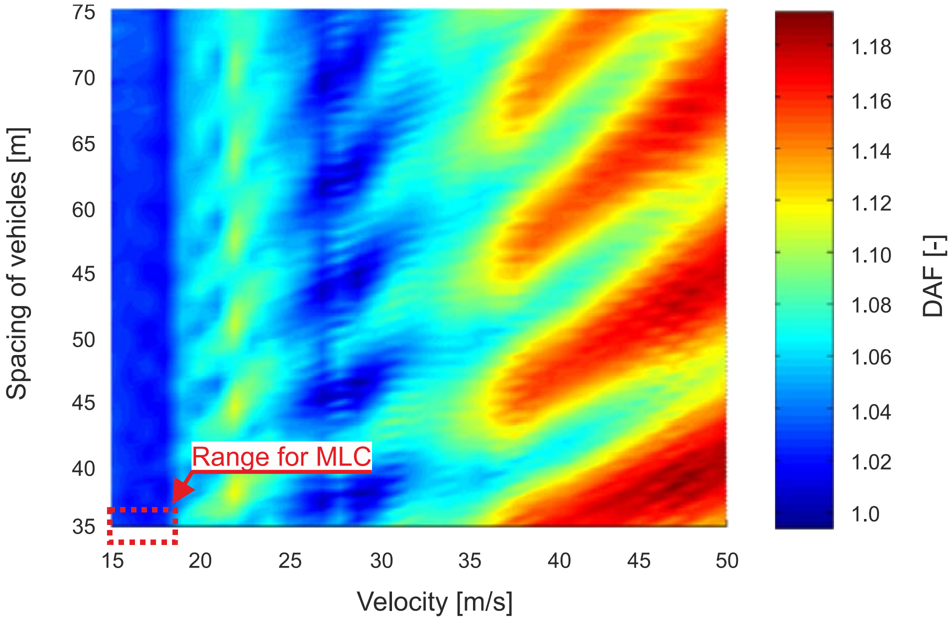


Figure 47: DAF for varying values of velocity and vehicle spacing at a 25m long bridge [11].

The presence of optimal and critical spacing between the vehicles, determining the respective decrease or increase of the particular DAF value, is also observable for different velocities. This can be contributed to periodic time between successive vibration peeks. Military convoys operate at 30.5m spacing and maximum velocities commonly less than 40 km/h for heavy vehicles and 60 km/h for light vehicles, an equivalent to 11.66 m/s and 16.7 m/s respectively [93], [125]. It is therefore apparent, as shown in Figure 47 by the red dashed line, that there is no significant influence of pre-existing vibrations on the dynamic amplification at the given speeds in the investigated bridge scenario.

The smoothness of road profile also plays a major role in the assessment of dynamic amplification and its variation. This is particularly important for short and medium bridges where the road profile appears to dominate the dynamic response [10]. Unevenness at the bridge approach or a damaged roadway surface may lead to a considerably higher dynamic amplification values especially at shorter span lengths (< 10m). The influence of a bump for span lengths in range 5m to 40m considering the most common 5-axle truck and a heavy crane truck was also numerically investigated in detail [11]. Mean values and standard deviations were obtained for bending moments at the midspan.

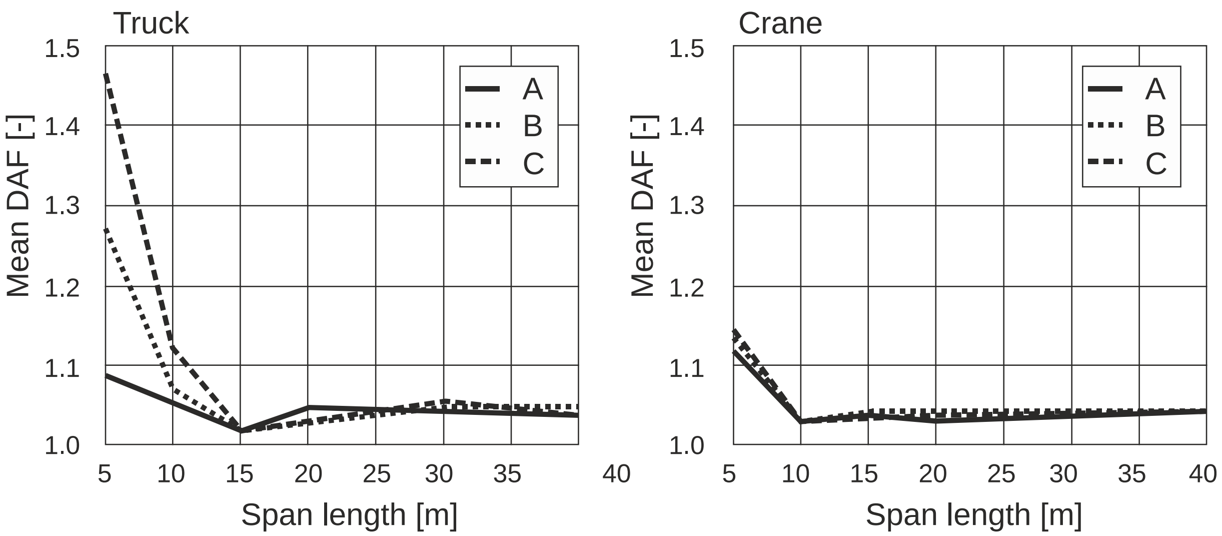


Figure 48: Mean value of dynamic amplification for bending at midspan: A – smooth surface, B – 2cm bump and C – 4cm bump.

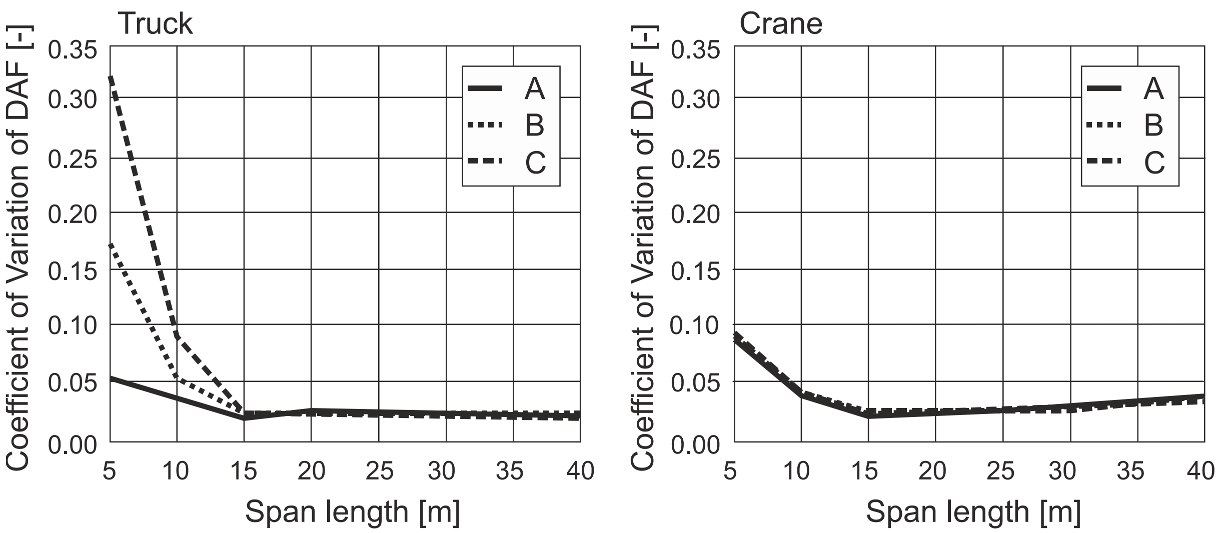


Figure 49: Coefficient of variation of dynamic amplification for bending in midspan: A – smooth surface, B – 2cm bump and C – 4cm bump.

The smooth profile A exhibits a quite constant course for both the 5-axle truck and the crane with the maximum value of δ at approximately 1.1. However, for the truck, a bump present prior to the bridge, as represented by profiles B and C, produces high dynamic response and high coefficient of variation for short span bridges up to 1.5 and 0.35 for mean and coefficient of variation respectively. Rapid decrease of these values and approximately constant progress with δ < 1.1 and V_δ < 0.05 can be observed in span lengths beyond 15m. The crane, a representation of heavy loading, remains in comparison to the truck in relatively low regions for the whole range span lengths and road profiles, although a marginal decrease at 15m can also be observed. Shear response dynamic amplification in comparison exhibited similar properties with slightly lower mean values and lower variation.

5.5.3 Probabilistic modelling

Stochastic properties of dynamic amplification are of the main interest in development of the partial factor. The Danish Road Directorate published one of the first guideline available for probabilistic assessment of existing bridges in 2004 [109]. Dynamic effects for trucks are accounted for in the form of a simple equation where static loads are multiplied by the dynamic factor K_s:

$$K_s = (1 + S_t) \qquad \text{Eq. 5-13}$$

where the so-called dynamic supplement S_t is expressed as an independent normally distributed variable $N(\mu,\sigma)$ for two types of crossing, considering global and local effects with W defining the total vehicle weight in kN:

Table 21: Dynamic supplement S_t [109]

Location	Normal passage	Conditional passage (< 10km/h)
Global effects (influence length $l \geq 2.5$m)	$N\left(\frac{41.5}{W}, \frac{41.5}{W}\right)$	Negligible
Global effects (influence length $l < 2.5$m)	$N\left(\frac{83}{W} - \frac{16.6 \cdot l}{W}, \frac{83}{W} - \frac{16.6 \cdot l}{W}\right)$	Negligible

Normal passage is deemed without any limitation on traffic and it allows a multiple vehicle presence or a transverse positioning of vehicles on the bridge. Conditional passage is limited to a single vehicle in a specified lane with velocity lower than 10km/h.

O´CONNOR [97] summarizes the probabilistic guideline by implicitly by identifying two dynamic amplification characteristics:

1. Inverse proportionality between the dynamic amplification and vehicle weight.
2. Reduction of the coefficient of variation with increasing weight.

As already mentioned in Section 5.5, a number of recent studies was able to relate the increased static loading to the lower value of the maximum (critical) loading level δ, when the dynamic component of the total load effect is small and well below the levels specified in the design codes. It compares well with GONZALES ET AL. [58] where the fact regarding the increase of static loading reducing the variability of the dynamic amplification is additionally confirmed.

This means that the critical loading (or the decisive MLC in the verification) of a bridge is generally tied to low dynamic amplification with lower variation. Lightly loaded vehicles may produce comparably higher dynamic amplifications, but at low static load effect and therefore are generally disregarded in determination of δ. Application of the rules according to [109] for dynamic amplification would produce the following results for the military vehicles, only few MLC are selected for comparison purposes. The dynamic supplement S_t is expressed as normal distribution $N(\mu,\sigma)$ according to Table 21 where the total weight W of a considered MLC vehicle is expressed in kN according to STANAG 2021 [93]:

$$S_t = \left(\frac{41.5}{W}, \frac{41.5}{W}\right) \qquad \text{Eq. 5-14}$$

The dynamic amplification K_s is calculated according to the Eq. 5-13 and the coefficient of variation of K_s (or δ) is expressed as:

$$V_{Ks} = \frac{\sigma_{St}}{K_s} \qquad \text{Eq. 5-15}$$

Consideration of MLC 4, MLC 16, MLC 40 and MLC 80 yields the following results:

Table 22: Stochastic properties of dynamic amplification for MLC considering Global Effects according to Danish Probabilistic Guideline [109]

MLC No.	Total weight W [kN]	S_t		K_S^*	V_{KS}
		μ	σ		
MLC 4	40.9	1.01	1.01	2.01	0.50
MLC 16	167.9	0.25	0.25	1.25	0.20
MLC 40	426.3	0.10	0.10	1.10	0.09
MLC 80	834.5	0.05	0.05	1.05	0.05

*$K_s = \delta$ dynamic amplification factor in this work

It must be mentioned that the used probabilistic guideline is aimed at bridges with certain span length carrying heavy transports. It is therefore difficult to directly apply the suggested values for dynamic amplification for the whole spectra of MLC vehicles. This can be observed in the dynamic amplification value of MLC 4 essentially stating that the static load should be multiplied by factor of two. At the same time, similar amplifications were observed for very light vehicles in measurements at Vransko Bridge in Slovenia [11], and therefore it is not unrealistic for light vehicles to exhibit such values. It should be however kept in mind, that the both absolute value and variation decrease as the loading approaches the critical limit – a state that is normally considered in the ULS verification of bridge load carrying capacity.

5.5.4 Recommended stochastic parameters

A site specific assessment of the total load remains the best available option for determination of the dynamic increment.

The partial factor γ_Q, a primary concern of this study, is affected by the ratio of the mean to the nominal (characteristic) value of δ and its coefficient of variation rather than by an absolute magnitude of δ. That is why it may be acceptable to assess δ in a simplified manner on the basis of the ratio between the critical static loading and total load effect. Couple of points can be assumed regarding the dynamic amplification of military loading:

- deterministic dynamic amplification factor is obtained for each bridge type or span length independently,
- the mean value of δ equals to its nominal value as used in a deterministic structural verification, for example [64], [127],
- maximum allowable static loading is tied to critical bridge loading,
- military vehicles generally cross with low speeds and lower dynamic response is expected.

With reference to ARCHES [11] and considering lower variability associated with maximum static loading, the coefficient of variation V_δ = 0.05 to 0.10 is accepted here for the normal crossing condition. The low V_δ can be regarded for bridges with an exceptionally smooth profile or for all bridges with span lengths over 15m. The medium degree of V_δ takes into account some variation due profile roughness or a bump between the approach and the deck, however, should be carefully evaluated. Any serious rough profile conditions or exceptionally short span lengths should be evaluated in the caution crossing condition, and therefore mitigating the resulting dynamic effects

STANAG 2021 [86] indicates that different amplification factors δ may be utilized for different crossing modes. GONZALES [58] suggests that vehicular speeds between 5-15 km/h are sufficiently low to consider the loading as quasi-static. The similar applies in [109] for 10 km/h. Therefore at these speeds the dynamic amplification factor needs not to be applied. This is relevant for the two controlled crossing conditions – caution and risk. STANAG 2021 [86] also supports this by stating that the “impact factor is not required” for these two types of conditions.

5.6 Model Uncertainty of Load Effect

According to JCSS [74] the model uncertainty is generally a random variable accounting for effects neglected in the models and simplifications in the mathematical relations. Model uncertainty in the load effect θ_E should cover numerous aspects including idealization of supports, composite actions of structural members, computational options (e.g. in FE analysis), description of input data etc. JCSS provides some guidance regarding the selection of mean values and coefficient of variation.

Table 23: JCSS Recommended probabilistic models for model uncertainties [74]

Model Type	Distribution	Mean value	Coefficient of variation
Moment in frames	LN	1.00	0.10
Axial force in frames	LN	1.00	0.05
Shear force in frames	LN	1.00	0.10
Moments in plates	LN	1.00	0.20
Forces in plates	LN	1.00	0.10

Slightly more detailed suggestions for stochastic modeling of uncertainties is provided in the recent work by BRAML ET AL. [16], where the values from JCSS are enhanced according to the FABER [52] and HANSEN [59].

Table 24: Probabilistic models for model uncertainties according to [16]

Model Type	Distribution	Mean value	Coefficient of variation
Axial force	LN	1.00	0.05
Bending (beams)	LN	1.00	0.07
Bending (plates)	LN	1.00	0.10
Shear (beams)	LN	1.00	0.12 – 0.17
Shear (plates)	LN	1.00	0.10

Danish Reliability-Based Classification [109] lists case-specific uncertainties for variable loading in dependence the level of confidence in modeling depending on a structural system, geometric properties and crossing mode, where conditional passage (at speeds < 10 km/h and in specified lane) is usually associated with a high level of confidence and therefore a low uncertainty in the loading model.

Table 25: Coefficient of variation for model uncertainty for variable loads [109]

Uncertainty in loading model	Coefficient of variation
Low	0.10
Medium	0.15
High	0.20

STEENBERGEN & VROUWENVELDER [122] recommend in their work regarding the development of partial factors based on economic optimization a unit mean and $V_\theta = 0.07$ for permanent action and $V_\theta = 0.10$ for traffic load.

Crossing conditions may certainly influence the model uncertainty with the respect of static response of the superstructure. More controlled crossing along the centerline of the way at a lower speed should provide more predictable response and therefore reduce the model uncertainty in same way as it conditional passage handled in [109].

An appropriate model for the model uncertainty should be selected considering bridge-specific conditions. For bridges with apparent static behavior the model uncertainty θ_E can be even neglected. The model uncertainty variation should not certainly lie below the limit associated with permanent action. In further numerical studies V_θ is considered in the range from 0.05 to 0.1 to represent both low and high limits.

5.7 Reliability Analysis

FORM analysis is another tool used for the investigation of all parameters necessary for the definition of the partial factor. The analysis is performed by a software package Comrel [34]. The purpose is to study the influence and sensitivity of the factors composing complete military load effect on the reliability index based on the preliminary results from LENNER [80]. The stochastic properties are investigated as to quantify the effects of chosen properties to ensure a conservative, yet realistic, selection of respective models. Structural reliability calculations are performed in this section considering a reinforced concrete beam.

5.7.1 Stochastic Properties in Analysis

Simple limit state equation is considered:

$$Z = R - E \qquad \text{Eq. 5-16}$$

with R representing the resistance and E the load effect.

General bending limit state of a reinforced concrete rectangular section is selected as a representative situation, and can by described by the Equation 5-17 based on [136]. A slight change in the format of model uncertainty for the purposes of sensitivity evaluation is implemented. Up to this point, load model uncertainty θ_E was regarded a uniform for the load effect side of equation. A division between model uncertainty for permanent action $\theta_{E,G}$ and model uncertainty for permanent action $\theta_{E,Q}$ enables the use of different stochastic models for the respective uncertainty variables. This only means different properties can be assigned for study purposes to permanent and variable model uncertainty.

$$Z(M_R) = \theta_R \cdot \left(A_s f_y d \cdot \left(1 - \frac{k_a}{\alpha_R} \cdot \frac{A_s f_y}{bd \cdot 0.85 \cdot f_c} \right) \right) - \left(\theta_{E,G} \cdot M_G + \theta_{E,Q} \cdot \delta \cdot M_{QMLC} \right) \qquad \text{Eq. 5-17}$$

where θ_R = resistance model uncertainty; A_s = reinforcement quantity; f_y = yield strength; h = section height; d = depth to reinforcement; k_a = height coefficient; α_R = compression block reduction factor; b = section width, f_c = concrete compressive strength; $\theta_{E,G}$ = permanent action model uncertainty; $\theta_{E,G}$ = variable action model uncertainty; M_G = permanent load bending moment; δ = dynamic amplification; and M_{QMLC} = variable load bending moment due to static load Q_{MLC}.

The starting point of the analysis is a concrete beam designed according to EN 1992-2 [48]. The loads are represented by the self-weight (permanent action) and variable load. In order to cover wide spectra of loading scenarios, the load ratio κ, describing the relationship between the characteristic permanent and variable action, is utilized.

$$\kappa = \frac{G}{G+Q} \qquad \text{Eq. 5-18}$$

It is assumed that the design resistance R_d is equal in value to the total design load effect E_d. The relationship between the design resistance and characteristic (mean) permanent and variable load with respective partial factors according to EN 1990 (γ_G = 1.35 and γ_Q = 1.50 selected according to ARS 22/2012 [8]) is utilized and described by:

$$R_d = \gamma_G M_G + \gamma_Q M_Q \qquad \text{Eq. 5-19}$$

$$R_d = \gamma_G M_G + \gamma_Q \left(\frac{M_G}{\kappa} - M_G \right)$$

Only a single geometric configuration (Figure 50) is considered and therefore the permanent load effect remains constant. The load ratio is varied from variable load dominant 0.3 to permanent load dominant 0.8 yielding the respective value of variable load and, in turn, the design resistance in terms of reinforcement A_s. It should be noted, that for the local verification of bridges even a much lower κ might be relevant.

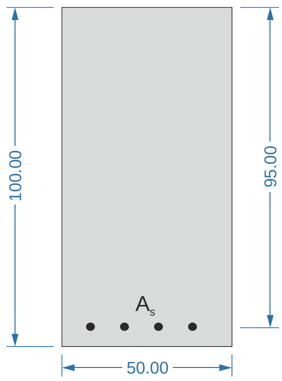


Figure 50: Section considered in the reliability analysis.

The permanent action is calculated assuming a randomly selected 10m length. The properties of analyzed beams are summarized in the following Table 26. Stochastic properties for all the variables describing the selected limit state (Eq. 5-17) are chosen in accordance with BRAML [15] and SÝKORA ET AL. [117] to represent an existing bridge and approximately 50 years remaining service life (see Table 27). All variables are expressed with the mean value equal to characteristic value, except for f_y and f_c where the characteristic value equals the relevant quantile value.

Table 26: List of beams used in the reliability analysis for bending limit state

Beam	κ	d	b	M_G	M_Q	A_s
[No.]	[-]	[m]	[m]	[kNm]	[kNm]	[cm^2]
1	0.3	0.95	0.50	156.25	364.58	19.38
2	0.4	0.95	0.50	156.25	234.38	14.19
3	0.5	0.95	0.50	156.25	156.25	11.15
4	0.6	0.95	0.50	156.25	104.17	9.15
5	0.7	0.95	0.50	156.25	66.96	7.73
6	0.8	0.95	0.50	156.25	39.06	6.68

Table 27: Random variables and stochastic properties for bending limit state

Variable	Distribution function	Unit	μ_x	Coefficient of Variation
f_y	LN	MN/m^2	560	0.054
f_c	LN	MN/m^2	30	0.15
k_a	Constant	-	0.4	-
α_r	Constant	-	0.8	-
b	Constant	m	0.50	-
d	N	m	0.95	0.02
θ_R	LN	-	1	0.05
$\theta_{E,G}$	LN	-	1	0.1
$\theta_{E,Q}$	LN	-	1	0.05-0.1
A_s	N	m^2	A)	0.02
M_G	N	kNm	A)	0.05
M_Q	N	kNm	A)	0.03-0.07
δ	LN	-	1.0	0.05-0.15

A) See Table 26 for values

The results from previous sections 5.1, 5.5 and 5.6 suggest the use of particular stochastic properties of the variables composing the total load effect:

- Coefficient of variation of military static load effect V_{QMLC} – 0.03 to 0.07
- Coefficient of variation of dynamic amplification V_δ – 0.05 to 0.15
- Coefficient of variation of model uncertainty V_θ for variable action – 0.05 to 0.10 for study purposes. Note, this particular value may generally not be set below the coefficient of variation for model uncertainty of permanent action.

The dynamic amplification factor is regarded as 1.0 in the analysis. It should be noted, that if the total load effect M_Q (according to Table 26) is held constant, then it the exact selection of dynamic amplification is insignificant. The reliability index is affected by the ratio of the mean to the nominal (characteristic) value of δ and its coefficient of variation rather than by an absolute magnitude of δ. Note that the model uncertainty term in the Eq. 5-20 θ_E is replaced by a different symbolization $\theta_{E,Q}$ due to the above described reasons. This has only notational effect as the variable remains essentially the same and the coefficient of variation is still denoted as V_θ.

$$Q = \theta_{E,Q} \cdot \delta \cdot Q_{MLC} \qquad \text{Eq. 5-20}$$

$$Q(\mu,\sigma) = \theta_{E,Q}(\mu,\sigma) \cdot \delta(\mu,\sigma) \cdot Q_{MLC}(\mu,\sigma) \qquad \text{Eq. 5-21}$$

It is apparent, from Eq. 5-20 and 5-21, that the stochastic properties of total load effect are not influenced by the particular value of dynamic amplification but rather by the standard deviation or in this respect the coefficient of variation of the present variables. The exact selection of δ is important during the actual assessment of the load carrying capacity and calculation of maximum allowable static load in terms of STANAG hypothetical vehicle. A higher dynamic effect naturally reduces the maximum static loading, but the value of partial factor remains unaffected.

FORM analysis was performed for the six beams summarized in Table 26 that represent the different load ratios. The results are shown in the following three figures, where the reliability index β is plotted for various selected coefficients of variation and load ratios. The following scenarios are considered in order to investigate each random variable separately:

- variable V_{QMLC}; constant V_δ and V_θ,
- variable V_δ; constant V_{QMLC} and V_θ,
- variable V_θ; constant V_{QMLC} and V_δ.

The overall results for all three cases show a quite high reliability index β in the region above 4.0. This can be contributed to the selected high partial factor for variable action and the fact that the stochastic models are representative for existing structures rather than design.

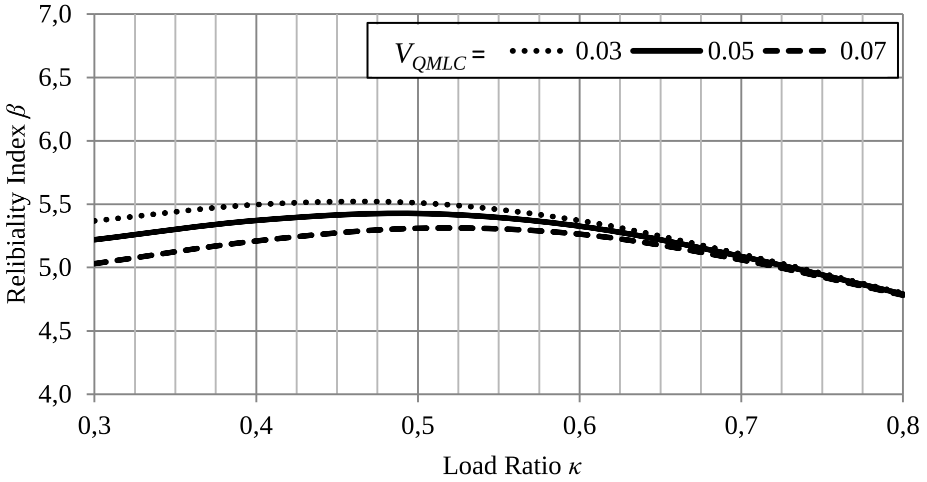


Figure 51: V_{QMLC} ranging from 0.03 to 0.07, $V_\delta = 0.05$, $V_\theta = 0.10$.

The relatively small influence of V_{QMLC} on the resulting reliability is observable in the region of variable action dominance (Figure 51). As the load ratio approaches the upper limit 0.8 commonly considered in bridge engineering the apparent influence of different coefficients of variation diminishes. In fact, already at the load ratio of 0.6 is the difference negligible. At the same time, the maximum difference in β equals to approximately 0.3 at the lower limit of load ratio.

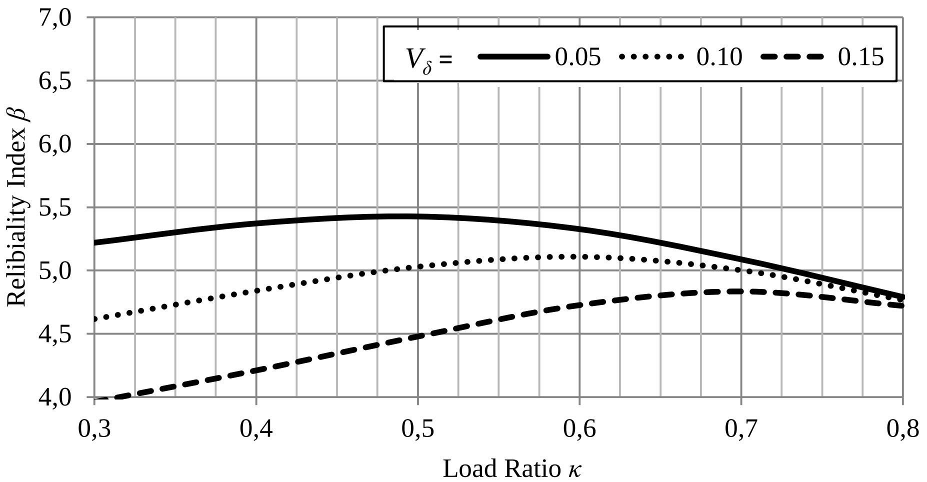


Figure 52: V_δ ranging from 0.05 to 0.15, $V_{QMLC} = 0.05$, $V_\theta = 0.10$.

The observed differences in variation of V_δ are much larger when compared to previous results of the V_{QMLC} parameter (Figure 52). This can be contributed to the higher selection of the coefficients of variation. Once again, the influence on the reliability index is high in the regions of low load ratio; however it decreases at much slower rate towards the high load ratio. The apparent influence is much larger, as the maximum difference in β is approximately 1.4 at 0.3 load ratio, and remains still of a large influence at higher ratios with 0.3 difference in β at 0.7 load ratio.

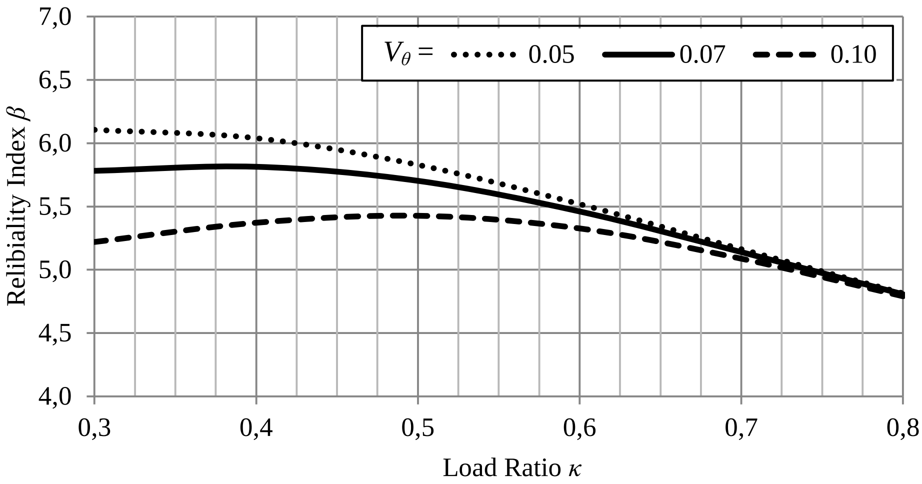


Figure 53: V_θ ranging from 0.05 to 0.10, V_{QMLC} = 0.05, V_δ = 0.05.

Model uncertainty is generally not calibrated and only a single factor is used when considering, for example, beams in bending [74]. Still, under considerations of different crossing conditions, the model uncertainty expressed in term of confidence of structural response may be potentially modified. The observed influence of the reliability index β is again highest in the low regions of load ratio and the significance of different value of coefficient of variation becomes negligible at approximately 0.65 value of load ratio (Figure 53). The selection of different coefficients of variation is still significant as the maximum difference in reliability index amounts to 0.9 at the lowest considered load ratio.

It can be concluded that the particular selection of V_{QMLC} is of the least importance when compared to the influence of selected V_θ and V_δ on the reliability index, and the resulting reliability is relatively insensitive to the exact selection of V_{QMLC}. With the results from Section 5.1 in mind, V_{QMLC} can be regarded as 0.05 for most of the situations.

At the same time, the considered values for the coefficient of variation of dynamic amplification and model uncertainty have quite large influence on the reliability index, especially in the regions of low load ratio, that is where the variable action dominates the loading.

5.7.2 Sensitivity factor α

The sensitivity factor is an important result from the FORM analysis, since it indicates the relative importance of each individual random variable from the limit state equation on the reliability level. The sensitivity factor is used in the definition of partial factor. Up to this point, it was suggested to accept the approximation for α_E as listed in EN 1990 [45]. Since the stochastic models for military loading have been updated in previous sections involving numerical simulations and an analysis of the influence on the reliability index, the aim is to study α-factors as to decide whether the values in EN 1990 [45] are adequate for military loading in terms of delivering conservative results.

A FORM estimation of α-values can be accomplished on the basis of same limit state equation as described by Eq. 5-17 in the previous section and variable load ratio κ describing the usual relationship between the permanent action and variable action in bridge engineering.

The limit state equation is repeated here for convenience:

$$Z(M_R) = \theta_R \cdot \left(A_s f_y d \cdot \left(1 - \frac{k_a}{\alpha_R} \cdot \frac{A_s f_y}{bd \cdot 0.85 \cdot f_c} \right) \right) - \left(\theta_{E,G} \cdot M_G + \theta_{E,Q} \cdot \delta \cdot M_{QMLC} \right) \qquad \text{Eq. 5-22}$$

The model uncertainty for loading is generally not treated discretely, while the separation in this case helps to interpret the values.

The basis for the FORM analysis is the concrete beam designed in the previous section. The same geometry, reinforcement ratios and stochastic properties are utilized as described by Table 26 and Table 27. Previous section investigated the influence of coefficients of variation of static load effect V_{QMLC}, dynamic amplification V_δ and model uncertainty of load effect $\theta_{E,Q}$ denoted simplistically as V_θ. Similar process is adopted here, but only four scenarios are considered to represent the situations with high, medium and low accuracy/ confidence of the described variables. An overview is shown in Table 28. It should be noted, that V_{QMLC} is considered as constant 0.05 accordingly to previous section and only the V_δ and V_θ are varied within the suggested boundaries. The coefficients of variations V_δ and V_θ for the *Case B* are assigned the values of 0.05 and 0.10 respectively. These are essentially interchangeable with no numerical effect. Mirroring of these coefficients would lead to exactly same results, with exception of the mirrored α-factors for $\theta_{E,Q}$ and δ.

Table 28: Considered scenarios in the reliability analysis

Case	V_{QMLC}	V_δ	V_θ
A	0.05	0.05	0.05
B	0.05	0.05	0.10
C	0.05	0.10	0.10

Some of α-factors for selected random variables are shown tabulated as numerical values. Subsequently, the factors are combined together to express the collective sensitivity factor of both the permanent $\alpha_{E,G}$ (Eq. 5-23) and the variable action $\alpha_{E,Q}$ (Eq. 5-24) as if they were represented by a single random variable. The whole α_E for loading is expressed by Eq. 5-25. It is then possible to study the influence of each of the loadings at the specified load ratio and potentially judge the influence on the proposed partial factor.

$$\alpha_{E,G} = \sqrt{{\alpha_{M_G}}^2 + {\alpha_{\theta E,G}}^2} \qquad \text{Eq. 5-23}$$

$$\alpha_{E,Q} = \sqrt{{\alpha_{M_{QMLC}}}^2 + {\alpha_{\theta E,Q}}^2 + {\alpha_\delta}^2} \qquad \text{Eq. 5-24}$$

$$\alpha_E = \sqrt{{\alpha_{E,G}}^2 + {\alpha_{E,Q}}^2} \qquad \text{Eq. 5-25}$$

Since the loading is the main interest of this work, the detailed sensitivity factors are only presented for the variables composing of the load effect. However, the collective sensitivity factor for resistance α_R can be easily obtained due to the following relationship:

$$\sqrt{{\alpha_E}^2 + {\alpha_R}^2} = 1 \qquad \text{Eq. 5-26}$$

The overall results are numerically shown in Table 29, while the sensitivity factor of permanent action $\alpha_{E,G}$ and the sensitivity factor of variable action $\alpha_{E,Q}$ are plotted for various load ratios in Figure 54. As expected, the particular influence of each random variable is to the largest extent subject the load ratio. At low load ratios indicating variable load dominance is clearly $\alpha_{E,Q}$ the largest while $\alpha_{E,G}$ the lowest. The exact selection of coefficient of variation dictates at which load ratio level the influence of both variable and permanent load is equal.

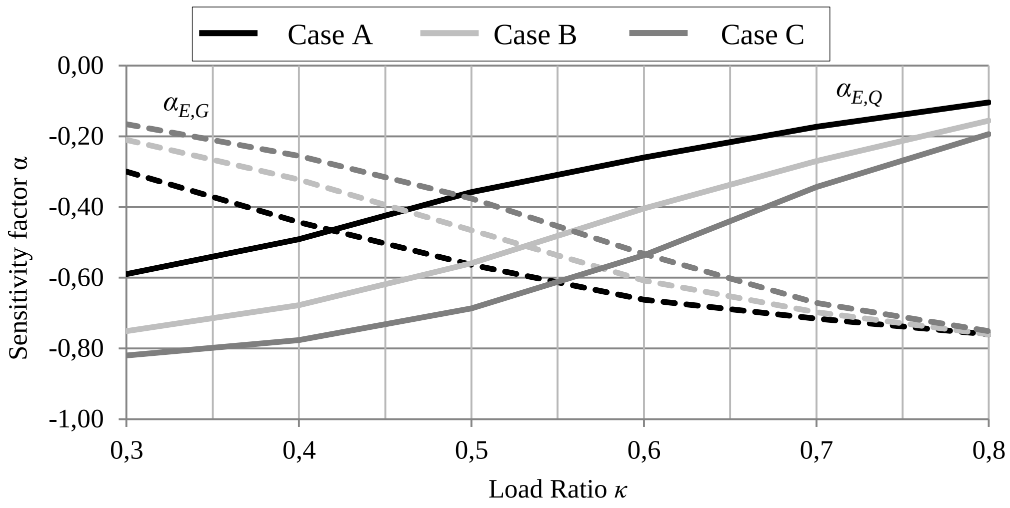


Figure 54: Values of $\alpha_{E,G}$ and $\alpha_{E,Q}$ at various load ratios.

Table 29: Resulting α factors for loading variables

	Beam	Load ratio κ	α					$\alpha_{E,G}$	$\alpha_{E,Q}$	α_E	α_R
			M_G	M_{QMLC}	$\theta_{E,G}$	$\theta_{E,Q}$	δ				
Case A	1	0.3	-0.13	-0.32	-0.27	-0.35	-0.35	**-0.30**	**-0.59**	-0.66	0.75
	2	0.4	-0.19	-0.27	-0.4	-0.29	-0.29	**-0.44**	**-0.49**	-0.66	0.75
	3	0.5	-0.24	-0.2	-0.51	-0.21	-0.21	**-0.56**	**-0.36**	-0.67	0.74
	4	0.6	-0.28	-0.15	-0.6	-0.15	-0.15	**-0.66**	**-0.26**	-0.71	0.70
	5	0.7	-0.3	-0.1	-0.65	-0.1	-0.1	**-0.72**	**-0.17**	-0.74	0.68
	6	0.8	-0.32	-0.06	-0.69	-0.06	-0.06	**-0.76**	**-0.10**	-0.77	0.64
Case B	1	0.3	-0.09	-0.29	-0.19	-0.62	-0.31	**-0.21**	**-0.75**	-0.78	0.63
	2	0.4	-0.14	-0.26	-0.29	-0.56	-0.28	**-0.32**	**-0.68**	-0.75	0.66
	3	0.5	-0.2	-0.22	-0.42	-0.46	-0.23	**-0.47**	**-0.56**	-0.73	0.69
	4	0.6	-0.26	-0.16	-0.55	-0.33	-0.17	**-0.61**	**-0.40**	-0.73	0.68
	5	0.7	-0.3	-0.11	-0.63	-0.22	-0.11	**-0.70**	**-0.27**	-0.75	0.66
	6	0.8	-0.32	-0.06	-0.69	-0.13	-0.06	**-0.76**	**-0.16**	-0.78	0.63
Case C	1	0.3	-0.07	-0.26	-0.15	-0.55	-0.55	**-0.17**	**-0.82**	-0.84	-0.54
	2	0.4	-0.11	-0.25	-0.23	-0.52	-0.52	**-0.25**	**-0.78**	-0.82	-0.58
	3	0.5	-0.16	-0.22	-0.34	-0.46	-0.46	**-0.38**	**-0.69**	-0.78	-0.63
	4	0.6	-0.23	-0.17	-0.48	-0.36	-0.36	**-0.53**	**-0.54**	-0.76	-0.65
	5	0.7	-0.28	-0.11	-0.61	-0.23	-0.23	**-0.67**	**-0.34**	-0.75	-0.65
	6	0.8	-0.32	-0.06	-0.68	-0.13	-0.13	**-0.75**	**-0.19**	-0.78	-0.64

The point of equal influence is at approximately κ = 0.42 for the narrowly described variable load (Case A), while, as expected, the broader stochastic description of variable loading extends the equality point into higher load ratio regions. Permanent action begins to have larger influence at κ = 0.60 for Case C.

The influence of individual factors composing the total variable load effect is determined by the selection of respective coefficients of variation, when clearly for the Case A ($V_{QMLC} = V_\theta = V_\delta$) the α-factors are almost equal. Case C shows a comparably higher variation of both $\theta_{E,Q}$ and δ (Table 28) and the sensitivity of M_{QMLC} is clearly reduced, while for model uncertainty and dynamic amplification is sensitivity increased when compared to the Case A.

At the same time, the $\alpha_{E,G}$ seems to reach a constant -0.75 at the highest considered load ratio, yielding the listed value $\alpha_E \approx$ -0.7 in EN 1990 [45] as slightly lower in this case. This could be contributed mainly to the selection of $\theta_{E,G}$. The precise minimum value of $\alpha_{E,Q}$ is largely dependent on the selected stochastic models, but can be specified from -0.60 to -0.80 for narrowly and broadly described variables respectively. It can be again slightly higher than the listed value in EN 1990 [45]. On contrary, the maximum value of $\alpha_{E,Q}$ reaches only about -0.10 to - 0.20 for the case of high load ratio and therefore remains well below the approximated limit.

Generally, the results indicate that decreasing the variability of traffic loading decreases the respective sensitivity factor and increases sensitivity factor for permanent loading. The numerical investigation confirms that the leading action is likely to have larger influence on the reliability although better described variables are likely to show smaller value of α, as shown by BRAML [15] in his work regarding evaluation of the reliability of concrete bridges based on inspection data. During the assessment of existing bridges the uncertainty related to resistance and permanent action effect is often reduced and therefore their sensitivity factors decrease. In turn, the absolute value of sensitivity factor for variable action increases, since the summation of α^2 is always equal to one. The particular values of α should be considered on a case-specific base since full probabilistic approach is recommended due to the fact, that there are many factors that are either unknown, or would complicate the deterministic verification of capacity.

The exact value of α is particularly important for the calculation of partial factors since it defines, along with the reliability index and standard deviation, the distance from design point to the mean value of the considered variable. Too conservative selection results in an unnecessarily high partial factor. At the same time, the α factor needs to be sensibly selected as to secure adequate performance under various loading scenarios.

5.8 Summary Military Load Effect

A number of factors has to be kept in mind for the summarization of stochastic properties of military vehicles,. The static load effect variation is largely dependent on the selected axle load coefficient of variation, while the coefficient of variation of spacing V_S has a marginal role. At the same time, it is easier to obtain the V_S from 0.03 to 0.05 due to easier obtainable geometrical measurements. A consideration of slightly broader definition of V_L between 0.05 – 0.10 yields for all vehicles and all investigated static systems seemingly constant results. The span length is besides V_L a decisive factor in the determination of coefficient of variation of static load V_{QMLC}. Short span response is clearly tied to the original selection of V_L, but at longer span lengths the resulting coefficient of static load moderately decreases in all the cases. Moreover, it is shown in that the particular selection of V_{QMLC} from the range 0.03 to 0.07 has only a marginal influence on the reliability. This is also supported by the FORM analysis results. It is therefore proposed to accept $V_{QMLC} = 0.05$.

Dynamic amplification stochastic properties are also to certain extent dependent on the considered span length. The characteristic (mean) value of dynamic amplification δ is clearly tied to the bridge length or natural frequency. The largest amplification of the static load and its variation V_δ can be observed for common civilian loading at short span lengths. The profile roughness or unevenness between the approach and bridge deck is the second most important factor in the assessment of stochastic properties. Rough road profile or a small bump can in some cases produce significant increase of dynamic effects. At the same time, critical loading of the bridge exhibits the lowest values of amplification and lowest variation of these effects. This is an important fact, since the maximum allowable MLC resulting from the assessment represents the critical loading. In this work it is proposed to assess the coefficient of variation only as it affects the particular value of the proposed partial factor. There are some listed suggestions for the δ, but it is proposed to assess the dynamic amplification on a case-specific basis. Crossing conditions are certainly essential in the determination of proper stochastic properties, as caution and risk crossing is to be associated with the lack of dynamic amplification due to the limited speed.

Model uncertainty is largely influenced by the static system and description, or level of confidence in applied loading. The particular selection shall not be lower than the model uncertainty for permanent action. The controlled crossing conditions is therefore regarded with model uncertainty unit mean and V_θ = 0.07, while normal crossing with vehicle anywhere on the bridge deck is considered with V_θ = 0.10.

It is therefore proposed in this work to further accept the coefficients of variation as summarized in Table 30 for the respective crossing condition. This is accomplished on the basis of numerical simulations, literature review and reliability analysis results.

Table 30: Summary of coefficients of variations for static load effect, dynamic amplification, model uncertainty and resulting total load effect

Variable	Normal	Caution	Risk
Coefficient of variation V_{QMLC} Coefficient of variation V_θ Coefficient of variation V_δ	0.05 0.1 0.05 – 0.10	0.05 0.07 -	0.05 0.07 -
Coefficient of variation V_Q	0.12-0.15	0.09	0.09

6 Target Reliability Index

The target reliability has a major role within the semi-probabilistic safety concept. It specifies the desired structural performance in terms of limiting the probability of failure. The acceptable risk for structures has been traditionally established by calibration of past practice methods assuming their optimal performance [105]. There are differences between specification of a reliability level for new structures and for existing structures as discussed in larger detail in Chapter 3.

The newest approach is to adjust the target reliability based on the cost-benefit ratio by the economic optimization. This section aims at developing a suitable target reliability index for military vehicles crossing over existing bridges. Moreover, it should be accomplished for the three crossing conditions: normal, caution and risk.

In the probabilistic framework the target reliability levels should be compared with “nominal” structural reliabilities resulting from randomness of basic variables (resistance and load effect variables, model uncertainties) rather than from actual failure frequencies that are dominantly affected by human errors [88], [131]. The target reliability levels as recommended in EN 1990 [45] are primarily intended for design of new structures. The evaluation of β for existing bridges and the remaining service life is not provided in the Eurocodes, but it is currently an urgent research topic [118]. Some of the basic guidance is provided in ISO Standards ISO 2394 [72] and ISO 13822 [71] . The optimization of target reliability is achieved by balancing the costs.

With regards to the military loading and crossing conditions, it is possible to modify the target reliability to reflect:

- existing condition of a fixed civilian bridge,
- implicit reliability mandated by crossing conditions,
- minimum required human safety (regarding users of the bridge as well as safety of people endangered by closure of the bridge).

STANAG 2021 requires that normal and cautious crossing reflect the same degree of safety, or another words – are based on the same reliability level. Risk crossing can be associated with a higher probability failure. This suggests that the β should be adjusted – decreased considering case-specific conditions. Existing condition of the bridge is regarded from the bridge owner’s view – or the public, as bridges on national networks often belong to the society. In that respect, it is proposed to accept developed concepts for target reliability when considering normal and caution crossing conditions. Risk crossing may be mandated on different terms due to a very short possible duration. High consequences of permitting risk crossing can be potentially mitigated by a high benefit in a risk situation.

The definition of risk situation is missing in STANAG, but it is assumed in this work that the risk crossing is allowed in response to natural, terrorist and wartime threats and therefore yields a potentially very high benefit and at the same time can lead to a range of failure consequences, as in the case of strategically important bridges. It is therefore extremely difficult to quantify the potential benefits and costs for all possible situations. In this respect, the bridge and reliability index are treated from the civilian point of view in terms of cost optimization with particular care devoted to human safety and differences between normal and risk situation. Newly developed approach is to consider structural performance and safety on cost optimization terms. SÝKORA, HOLICKÝ & LENNER [115], [119] show in their work the cost optimization and development of an optimum target reliability for an emergency situation. Large portion of this work is utilized here during the framework development for target reliability indices pertaining to the military traffic and safety concept.

6.1 Target Reliability in Eurocodes, ISO and JCSS

An overview of the current structural codes is provided in this section. The target reliability levels as recommended in EN 1990 [45] are primarily intended for design of structures, where the defined reliability classes are associated with consequences of failure. EN 1990 [45] recommends the target reliability index for two reference periods (1 and 50 years), see Table 31.

The couples of β-values given in Table 31 for each reliability class correspond approximately to the same reliability level. For a bridge of Reliability Class 2 (RC2), the reliability index β = 3.8 should be thus used, provided that probabilistic models of basic variables are related to the reference period of 50 years. The same reliability level should be reached when β = 4.7 is applied using the theoretical models for one year. Note that the couples of β-values correspond to the same reliability level only when failure probabilities in individual time intervals (basic reference periods for variable loads) are independent. Target reliability index β = 3.8 could better be interpreted as corresponding to about 4.5 per year as complete independency of resistance and loads in subsequent years is not realistic [130].

Considering a reference period t_{ref}, it might be understood from EN 1990 [45] that the related reliability level can be derived as follows:

$$\beta_{tref} = \Phi^{-1}\left\{\Phi(\beta_1)^{tref}\right\} \quad \text{Eq. 6-1}$$

where β_1 = target reliability index taken from Table 31 for a relevant reliability class and the reference period t_{ref} = 1 year; Φ and Φ^{-1} = cumulative distribution function of the standardized normal variable and its inverse function respectively.

Table 31: Reliability classification for different reference periods in accordance with EN 1990 [45]

Reliability class	Failure consequences	β (1 y.)	β (50 y.)	Examples
RC3	high	5.2	4.3	significant bridges, public buildings
RC2	medium	4.7	3.8	bridges, residences, offices
RC1	low	4.2	3.3	agricultural buildings

The graphical interpretation, of the respective target reliability indices β as shown on the vertical axis with t_{ref} time in years on the horizontal axis, is portrayed in Figure 55. It can be observed, that particularly short reference periods are sensitive to the reliability index.

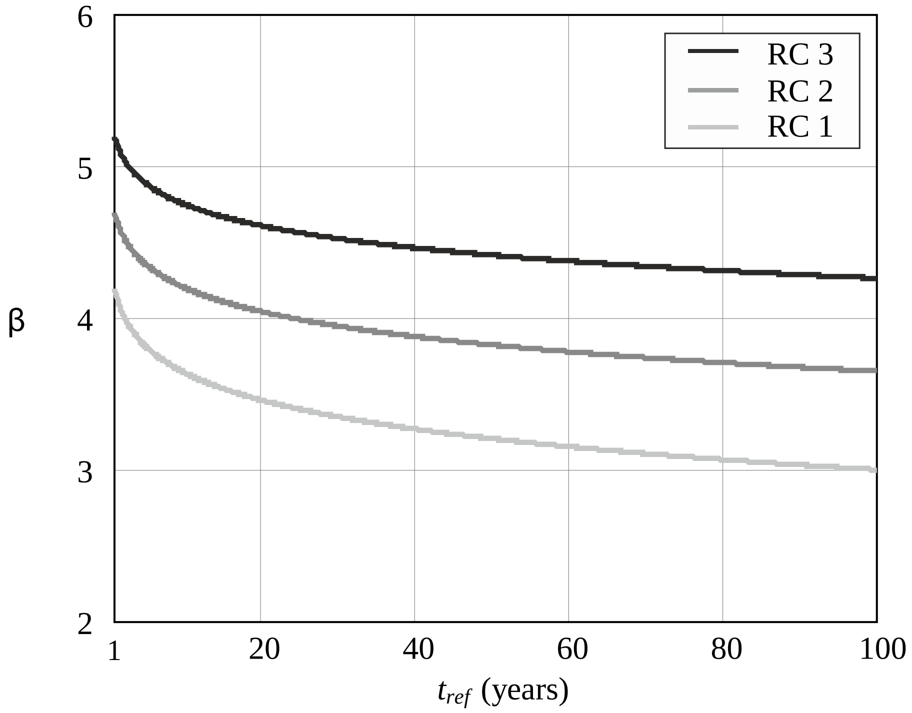


Figure 55: Reliability levels for different reference periods in accordance with EN 1990 [45].

However, the concept in EN 1990 [45] seems to be hardly applicable for the situations where the reference period can be very short, as in the case of an emergency situation and risk crossing, and the reliability level excessively increases. For instance according to Eq. 6-1 $\beta \approx 5.5$ should be considered for t_{ref} = 1/52 year = 1 week and RC2. Note, this value is not shown in Figure 55 due to the limits on horizontal axis.

A more detailed recommendation is provided by ISO 2394 [72] where the target reliability index is given for the working life and is related not only to the consequences but also to the relative costs of safety measures (Table 32). The target reliability might thus be selected independently of the reference period which seems to be a more appropriate approach than provided by EN 1990 [45]. Using Table 32 for existing structures the target level usually decreases as it takes more effort to increase the reliability level [130]. So for a couple of similar new and existing structures, e.g. moderate costs of safety measures can be considered at a design stage while high costs may apply when assessing the existing structure. It is much easier to implement for example a deeper beam during the design, then to retrofit an existing one to carry the same design load.

Table 32: Target reliability index (life-time, examples) in accordance with ISO 2394 [72]

Relative costs of safety measures	Failure consequences			
	small	some	moderate	great
High	0	1.5[A]	2.3	3.1[B]
Moderate	1.3	2.3	3.1	3.8[C]
Low	2.3	3.1	3.8	4.3

Some suggestions regarding the recommended values in ISO 2394 [72]:

- A – for serviceability limit states, use the safety classes β = 0 for reversible and β = 1.5 for irreversible limit states.
- B – for fatigue limit states, use the safety classes β = 2.3 to 3.1, depending on the possibility of inspection.
- C – for ultimate limit states design, use the safety classes β = 3.1, 3.8 and 4.3.

Similar recommendations are provided by the Probabilistic Model Code of the Joint Committee on Structural Safety (JCSS) [74]. Recommended target reliability indices are also related to both the consequences and to the relative costs of safety measures, however for the reference period of one year instead of life time. Additional guidance regarding the quantification of consequences and cost is provided. For most common design situations β = 4.2 should be utilized, this corresponds to β = 3.2 in 50 years.

Table 33: Target reliability index for year according to JCSS [74]

Relative costs of safety measures	Failure consequences		
	minor	moderate	large
Large	3.1	3.3	3.7
Normal	3.7	4.2	4.4
Small	4.2	4.4	4.7

ISO 13822 [71] indicates four target reliability levels for different consequences of failure at the ultimate limit state as illustrated in Table 34.

Table 34: Illustration of target reliability level [71]

Limit State	Target reliability index β	Reference Period
Serviceability		
reversible	0.0	remaining working life
irreversible	1.5	remaining working life
Fatigue		
can be inspected	2.3	remaining working life
cannot be inspected	3.1	remaining working life
Ultimate		
very low consequences of failure	2.3	L_s years[a]
low consequences of failure	3.1	L_s years[a]
medium consequences of failure	3.8	L_s years[a]
high consequences of failure	4.3	L_s years[a]

L_s is a minimum standard period for safety (e.g. 50 years)

Lower target reliability levels can be used if justified on the basis of social, cultural, economic and sustainable considerations as indicated in ISO 13822 [71]. ISO 2394 [72] shows that the target level of reliability should depend on a balance between the consequences of failure and the costs of safety measures.

The following additional notes are made concerning available approaches to the target reliabilities:

- Costs of safety measures might be perceived as an unacceptable factor for the target reliability particularly of new structures.
- Several empirical models for the assessment of target reliabilities have been proposed in previous studies; SÝKORA & HOLICKÝ [114] provided a brief overview.

6.1.1 Target Reliability Index for Existing Structures

As KOTES & VICAN [79] note in their work, increase effort has been aimed at securing satisfactory reliability and durability of the transportation infrastructures, especially because a large portion of the bridges is more than 50 to 60 years old. It has been recognized that it would be uneconomical to specify for all existing buildings and bridges the same reliability levels as for new structures [132]. A higher reliability level of new structures generally requires more material, whereas the design and construction costs are affected marginally. Strengthening or upgrade of existing structures required to achieve the necessary higher reliability might be comparably much more expensive in terms of direct and indirect costs, and in many cases, the bridge would require complete replacement in order to comply with the required safety level. The balance of safety and economy is demonstrated by for example the practice in the USA, where β = 3.5 for design of structures is replaced by β = 2.5 for load rating calibration of partial factors [55]. The approach for reduction of target reliability levels, based on the inspection and system evaluation, is additionally demonstrated by the present practice in Austria [101], Denmark [109], the Netherlands [94], and Switzerland [112], where the target reliability indices for existing structures decrease by about 0.5-1.7 when compared with indices for new structures [33], [86], [135]. Germany is still debating over the introduction of reduced target reliability levels [134]. Lower reliability level might be justified assuming proper and regular bridge inspection and maintenance [135]. A detailed proposal for the target reliability levels based on the monitoring and inspection cycle, ductility of critical system components, system behavior and loading is provided by BERGMEISTER & SANTA [13] for a RC2 structure as shown in Table 35. Similar proposal for the reduction of β for the assessment of existing buildings is provided by ALLEN [4] only with different Δ adjustment factors.

Table 35: Proposal for Target Reliability Index in ULS and SLS (1 year reference period)

$\beta = 4.7 - (\Delta_M + \Delta_D + \Delta_S + \Delta_L) \geq 3.5$ for Ultimate Limit State $\beta = 3.0 - (\Delta_M + \Delta_D + \Delta_S + \Delta_L) \geq 1.7$ for Serviceability Limit State	
Monitoring	Δ_M
Continuous monitoring of critical elements	0.5
Annual inspection of critical elements showing visual warning signs of distress	0.25
Annual inspection of critical elements with no visual warning signs of distress	0.1
Bi-annual inspection	0
Ductility	Δ_D
High ductility	0.5
No ductility	0
System behavior – Robustness	Δ_S
High robustness, member failure leads to system redistribution	0.5
Medium robustness, system collapse requires more individual members to fail	0.25
Low robustness, systems collapse with an element failure	0
Loading	Δ_L
Design loading	0
Exceptional loading – seldom occurrence (annual); maximum 20% of design load	0.1
Exceptional loading with additional loading (wind or snow)	0.2

Quite comprehensive approach is offered in the Canadian CSA-S6-06 [29].The target reliability there is specified from 2.50 to 4.0 in 0.25 increments and the particular value of β is dependent on the system redundancy *S*, element ductility *E* and inspection level *INSP*. Additionally, partial factors are listed for the permanent and variable loading according to one of the selected reliability indices.

Table 36: Target reliability index for normal traffic [29]

System behavior	Element behavior	Inspection Level		
		INSP1	INSP2	INSP3
S1	E1	4.00	3.75	3.75
	E2	3.75	3.50	3.25
	E3	3.50	3.25	3.00
S2	E1	3.75	3.50	3.50
	E2	3.50	3.25	3.00
	E3	3.25	3.00	2.75
S3	E1	3.50	3.25	3.25
	E2	3.25	3.00	2.75
	E3	3.00	2.75	2.50

The explanation regarding the performance and inspection level is provided as follows:

- System behavior – S1, element failure leads to total collapse; S2, element failure does not cause total collapse; S3, local failure only.
- Element behavior – E1, sudden loss of capacity with no warning; E2, sudden failure with no warning but with some post-failure capacity; E3, gradual failure.
- Inspection level – INSP1, component not inspectable; INSP2, inspection records available to the evaluator; INSP3, inspection of critical and substandard members inspected by the evaluator

Some of the aspects regarding the proposed reductions of the reliability index in Table 35 and

Table 36 36 are difficult to be applied to the military traffic due to the fact, the military is not responsible for the monitoring of bridges (they belong to the civilian authorities) and the design loading is often unknown. Only the system robustness/behaviour and the ductility of members can be accounted for. The re-evaluation of the target reliability is then somewhat limited.

6.1.2 Approach in Netherlands according to NEN 8700 [94]

An appropriate approach for military vehicles should utilize their characteristics. The differentiation is made between normal along with caution crossing and risk crossing. However, the main facts – such as time invariability of loading, or shorter reference period when compared to design life are still appropriate. A quite attractive approach is then ISO 2394, where the relative cost of safety measures are implemented and target reliability may be selected on terms of cost of safety and consequences independent of the reference period. It is however somewhat difficult to quantify the costs related to normal crossing of military vehicles. Particularly appealing proposal for target reliabilities is then provided by STEENBERGEN & VROUWENVELDER [122]. The described approach there has been implemented for the National Annex A2 Bridge in the Netherlands. An extension of this approach yields a concept applicable to the military traffic. It is argued that the purely economic optimization makes sense, when with an assumed linear increase of the failure probability, the target reliability remains constant regardless the design life time. Example is presented with β = 3.8 yielding probability of failure approximately 10^{-4}. For a design life of one year this would yield the probability of failure $P_{f.tref} \approx 10^{-4}$, however, a structure designed for 50 years (t_{ref}) has in each year much smaller arbitrary probability of failure $P_{f.annual} \approx 2\cdot10^{-6}$ according to Equation 6-2. It is reasoned that such approach makes sense for economic investment into the structural safety measures. Higher initial investment for longer reference period (remaining work life) is compensated by a longer utilization.

$$(1 - P_{f.tref}) = (1 - P_{f.annual})^{tref} \qquad \text{Eq. 6-2}$$

Economic arguments along with minimum criteria for human safety are employed in establishment of target reliability levels for assessment, and that is β_r that corresponds to a safety level for repair, and β_u representing the safety level at which the structures is unfit for use.

The presented results based on optimization are summarized as:

- $\Delta\beta_r$ = 0.5 corresponding to the difference between the safety levels in old code and new code
- $\Delta\beta_u$ = 1.5 based on crude economic optimization

Human safety criteria are selected based on the probability to die as a result of an accident in Netherlands, where the rate is approximately 10^{-4}. It is argued, that for society it is unacceptable to have a larger probability of becoming a victim of a structural failure rather than traffic accident and therefore is the maximum probability of life loss determined as 10^{-5}. Conditional probabilities p_1 for a loss of human life are also established:

- CC3 $\rightarrow$ $p_1 = 0.3$
- CC2 $\rightarrow$ $p_1 = 0.03$
- CC1 $\rightarrow$ $p_1 = 0.001$

Essentially, the design target reliability index can be reduced by 0.5 before the repairs are necessary, while a reduction beyond 1.5 calls for the closure of the bridge. It must be noted, the minimal human safety in higher consequence classes governs the limits for unfit for use. Table 37 provides a summary of considered values for non-dominant wind limit equations. The time reference of 15 years is dictated by the minimum required human safety.

Table 37: β values for repair and unfitness for use; based on economic arguments [122]

Consequence Class	Minimum reference period	β_n new	β_r repair	β_u unfit for use
CC1	15 year	3.3	2.8	1.8
CC2	15 year	3.8	3.3	2.5*
CC3	15 year	4.3	3.8	3.3*

* minimum limit for human safety is decisive

However, if t_{ref} is less than 15 years, human safety criteria should be checked and different β-values may be needed. This could be particularly important for risk crossing conditions where the time reference can be extremely short. Additionally, military might set different criteria on human safety.

6.2 Optimization for Military Loading

It is proposed for the safety concept of military vehicles to accept the results from previous section regarding the target reliability for normal and caution crossing. The normal condition represents an unlimited use of a civilian bridge and in that respect it should probably be governed from the civilian perspective. Caution crossing condition requires the same safety level as normal crossing. The suggested target reliability indices in previous Section are accepted and checked for the limit according to the human safety criteria mandated by the military.

Optimization for structures in emergency situations has been presented by SÝKORA,HOLICKÝ & LENNER [115], [119]. Their work concerns the cost optimization assuming an immediate response to a natural or industrial disaster, when for example an exceptionally heavy vehicles need to cross a bridge. A presented case study includes examples of transportation of flood barriers or decontamination units in case of an industrial explosion. The developed concepts are transferred to this work concerning the military crossing, as it is essentially similar in nature. It means that a heavy load needs to cross a bridge. The required duration for the situation of crossing may be in many cases quite short, especially when risk crossing is considered. According to HOLICKÝ [66] almost no recommendations are available for temporary structures and this holds likewise for structures under temporary conditions including emergency situations requiring risk crossing. In general, ISO 2394 [72] seems to provide the most appropriate reliability differentiation for existing bridges in emergency and crisis situations since costs of safety measures are taken into account and the reliability levels are associated with a full working life, in this case defined by the duration of the emergency or crisis situation mandating risk crossing. ISO 13822 [71] indicates a possibility to specify the target reliability levels for existing structures by optimization of the total cost related to an assumed remaining working life. This can be in turn tied to the dependence on the required duration for the assessment, as might be dictated by different crossing conditions. This approach in conjunction with the criteria for human safety in accordance with ISO 2394 [72] is further developed here.

6.2.1 Human Safety

The cost optimization is commonly perceived to aim at finding the optimum decision from the perspective of a bridge owner. However, society commonly establishes limits for human safety. General guidelines for the assessment of the target reliabilities with respect to human safety are provided in ISO 2394 [72]. In principle structural design and assessment of existing bridges are not distinguished.

ISO 2394 [72] states that structural reliability is important first and foremost if people may be killed or sustain injuries as a result of the collapse. An acceptable maximum value for the failure probability might be found from a comparison of risks resulting from other activities. Individual lethal accident rates ranging between 10^{-6} - 10^{-5} per year [114], [122] , seem to be reasonable for structures in persistent design situations, when compared to the typical rates in industries, e.g.:

- 10^{-4} per year for work in all industries ($2\cdot10^{-4}$ for users of motor vehicles),
- 10^{-5} per year for third parties in ship industry (passengers or public ashore).

The human safety during military assessment can be regarded somewhat differently. It is assumed that during the crossing of military vehicles there is no civilian traffic or pedestrians and therefore the structural failure affects the military personnel only. In the case of risk crossing it is the crew of a single vehicle that is allowed on the bridge. Naturally, the occupational risk for military personnel has to be considered here. Normal and caution crossing require an equal level of safety and it seems appropriate to consider the 10^{-4} rate. However, during risk crossing higher probability of failure may be acceptable since it may be compensated by mitigation of consequences in endangered areas. Therefore, a tentative value of 10^{-3} per year is considered hereafter [19].

A comparison of U.S. military deaths is provided in a DMDC document [44] , where all the deaths of military personnel since 1980 are divided according to the cause: accident, hostile action, illness or self-inflicted. The resulting rates are following:

- Average death rate a year due to accident: $4\cdot10^{-4}$
- Maximum death rate a year due to accident: $7.2\cdot10^{-4}$
- Average death rate a year due to hostile action: $9\cdot10^{-5}$
- Maximum death rate a year due to hostile action: $5\cdot10^{-4}$

The death rate due to hostile action is an inappropriate measure here, since it does not account for the rate of military personnel actually exposed to the hostile action, but rather includes the overall number of listed service members.

The actual death rate of military personnel in Iraq between 2003 and 2006 is estimated by PRESTON & BUZZEL [104] as $4 \cdot 10^{-3}$ per year. GOLDBERG [57] provides a more detailed casualty overlook for the Operation Iraqi Freedom where both the death and injury rates are considered. It is also mentioned, that death rates are significantly reduced due to the rapid and available medical care.

Table 38: U.S. Military Casualties Sustained in Operation Iraqi Freedom [57]

	Number	Rate per 100,000 per year	Individual rate per year
Person-Years of Exposure	721,220	--	--
Deaths			
Hostile	2417	335	0.003
Non-hostile	584	81	0.001
Total	3001	416	0.004
Wounded in Action			
Returned to duty <72 hours	12643	1753	0.018
Not Returned to duty	10191	1413	0.014
Total wounded	22834	3166	0.032

It can be therefore seen, that the considered 10^{-4} rate for normal crossing and 10^{-3} for risk crossing may be deemed as acceptable limit for the safety of military personnel when crossing of bridges.

The concept of individual risk provided in ISO 2394 [72] then yields the following relationship between the target failure probability $p_{\mathrm{ft,hs}}$ and the conditional probability of occupant fatality p_1, given the structural failure for normal crossing and risk crossing for one year respectively:

$$p_{f.hs.normal} \leq \frac{10^{-4}}{p_1} \qquad \text{Eq. 6-3}$$

$$p_{f.hs.risk} \leq \frac{10^{-3}}{p_1} \qquad \text{Eq. 6-4}$$

With respect to the loss of human life, EN 1990 [45] distinguishes among low, medium, or high consequences (Consequence Classes CC1-CC3, respectively). Note that the class CC3 means that there is a high conditional probability. In such case, the failure can occur without previous warning (e.g. shear failure of reinforced concrete beam or buckling of bridge piers) and subsequent collapse is likely. Consequence Classes may be associated with Reliability Classes (see Table 31).

Based on a SÝKORA AT EL. [116] the following conditional probabilities for assessment of bridges might be accepted:

- CC3 → $p_1 = 0.05$
- CC2 → $p_1 = 0.01$
- CC1 → $p_1 = 0.001$

It is mentioned that probability of casualty given a structural failure is tentatively assumed based on the review of structural failures, where the upper bound $p_1 = 0.05$ is suggested [50]. STEENBERGEN & VROUWENVELDER [122] considered $p_1 = 0.3$ for the CC3 and perhaps they compensated this by a conservative assumption of lower fatality rate 10^{-5}. It is apparent, that p_1 should be better defined on collected data of casualties given a structural failure. The required target reliability index β_{hs} for human safety in one year for normal crossing is determined from Eq. 6-5 as:

$$\beta_{hs} = -\Phi^{-1}(p_{f.hs}) \qquad \text{Eq. 6-5}$$

- CC3 → $p_{ft,hs} = 2 \cdot 10^{-3}$ corresponding to $\beta_{hs} \approx 2.9$,
- CC2 → $p_{ft,hs} = 1 \cdot 10^{-2}$ corresponding to $\beta_{hs} \approx 2.3$,
- CC1 → $p_{ft,hs} = 1 \cdot 10^{-1}$ corresponding to $\beta_{hs} \approx 1.3$.

It is obvious that values of β_{hs} are lower than suggested in Section 6.2 due to the modified selection of p_1 and, more importantly, the accepted rate for individual death per year. It is therefore apparent, that the target reliability should be dictated by the minimum structural safety based on economic optimization rather than by the safety of military personnel.

For risk crossing situations, the target failure probabilities of structural members are related to a reference period t_{ref} shorter than a year. It is necessary to adjust for the shorter reference period by multiplying Eq. 6-4 by the required time reference in years [115].

$$p_{f.hs.risk} \leq \frac{10^{-3}}{p_1} \cdot t_{ref} \qquad \text{Eq. 6-6}$$

Table 39 indicates the target reliability index β_{hs} for the different consequence classes and reference period according to Eq. 6-5.

Table 39: Target reliability β_{hs} for human safety in risk crossing situations and different reference periods

Consequence Class	t_{ref} = 1 week	t_{ref} = 2 weeks	t_{ref} = 4 weeks
CC3	3.4	3.2	3.0
CC2	2.9	2.6	2.4
CC1	2.1	1.7	1.4

It should be noted, that given the high probability of life loss at risk crossing situation, the period of risk crossing should be limited to a reasonable time frame. Assuming a longer period for class CC2, for example a year, the resulting β_{hs} equals to 1.3 and minimum structural safety dominates the target reliability.

6.2.2 Structural Safety for Risk Crossing

From an economic point of view the objective is to minimize the total structural cost. It is expected in the cost optimization analysis, that a bridge may be upgraded immediately before the risk crossing would take place. This may increase the total cost and increase the target reliability index depending on the particular decision parameter *d* defined as a variable in the economic optimization. The expected total costs C_{tot} may be generally considered as the sum of the expected costs of inspections, maintenance, upgrades and costs related to failure of a bridge [9], [96]. The objective is to optimize relevant decision parameters *d*, represented by factors affecting the resistance, actions, serviceability, durability, maintenance, inspection, upgrade strategies, etc. Examples of *d* include:

- during design phase: sectional area of a steel beam, shear reinforcement ratio of reinforced concrete beam, concrete cover in durability design,
- for assessment of an existing bridge: strategies to upgrade bridge resistance in a governing failure mode (local strengthening by fibre-reinforced polymers, construction of a secondary load bearing structure), limits on traffic load (restrictions of vehicle weights, reduction of traffic lanes) etc.

The decision parameter is assumed to concern mainly the immediate upgrade while the inspection, maintenance and future repair or upgrade strategies are influenced only marginally. This may be a reasonable assumption in many practical cases. Implications for the assessment in emergency and risk crossing situations are clarified in the following.

An upgrade of the bridge, immediately undertaken during or before the risk crossing situation, may in general lead to the following costs:

- Cost C_0 independent of the decision parameter - economic losses and potential societal consequences (injuries or fatalities) caused by temporary bridge closure in the emergency or crisis situation due to upgrade works immediately resulting from the decision to enhance bridge resistance.
- Marginal cost C_m per unit of the decision parameter.
- Estimation of the cost C_0 may be a difficult task and expert judgments may be necessary. However, it is further assumed that the upgrade costs C_0 and C_m can be reasonably estimated.

The main reason for the existence of civil infrastructures is the public interest. Therefore, all related societal aspects should be considered when assessing the failure consequences C_f. Depending on a bridge concerned, failure may be associated with the following consequences [115]:

- potential societal consequences directly caused by the failure (collapse),
- cost of repair or replacement,
- economic losses and potential societal consequences caused by bridge closure due to repair works taken after the failure (possibly including also losses due to damage on detour routes),
- possible other consequences such as unfavorable environmental or psychological effects.

Estimation of the failure cost is a very important, but likely the most difficult, step in the cost optimization. It is important to include not only direct consequences of failure (those resulting from the failures of individual components), but also indirect consequences related to a loss of the functionality of a whole bridge. Background information for the consequence analysis is provided in IMAM & CHRYSSANTHOPOULOS [69], THOFT-CHRISTENSEN [126] and by the outcomes of SeRoN project [113] focused on the security of road transport network.

In cost optimizations, discounting is commonly applied to express the upgrade and failure costs on a common basis [66]. Apparently such considerations are not needed in the case of situations of short-term durations such as emergency situation.

Based on these assumptions, the expected total costs can be determined for the case of upgrade by Eq. 6-7 and the case of no upgrade (accepting a present state) by Eq. 6-8 [119]:

$$C_{tot}(d) = C_0 + C_m d + C_f\, p_f(d) \quad \text{Eq. 6-7}$$

$$C_{tot}(d_0) = C_f\, p_f(d_0) \quad \text{Eq. 6-8}$$

where $p_f(\cdot)$ = failure probability related to a reference period; and d_0 = value of the decision parameter before an upgrade such as flexural resistance or cross-sectional area.

From Eq. 6-7, the optimum value of the decision parameter d_{opt}, defined as the parameter indicating the optimum upgrade strategy, can be assessed on the basis minimum total cost as:

$$\min_d C_{tot}(d) = C_{tot}(d_{opt}) \quad \text{Eq. 6-9}$$

From an economic point of view, no upgrade is undertaken when the total cost according to Eq. 6-7 is less than the total cost of the optimum upgrade Eq. 6-8. It follows from equations that d_{opt} is independent of C_0.

The optimum upgrade strategy should aim at the target reliability β_{up} corresponding to:

$$\beta_{up} = -\Phi^{-1}(p_f(d_{opt})) \quad \text{Eq. 6-10}$$

However, the total costs given in Eq. 6-7 and 6-8 should be evaluated in order to determine whether to upgrade the bridge or not.

The limiting value $d_{0\lim}$ of the decision parameter before the upgrade is then found as follows:

$$C_f p_f(d_{0\lim}) = C_0 + C_m d_{opt} + C_f p_f(d_{opt}) \qquad \text{Eq. 6-11}$$

$$p_f(d_{0\lim}) = \frac{C_0}{C_f} + \frac{C_m d_{opt}}{C_f} + p_f(d_{opt})$$

For initial conditions lower than the limiting value $d_0 < d_{0\lim}$ the reliability level of an existing bridge is too low, failure consequences become high and the decision to upgrade the bridge is the optimum strategy yielding a lower total cost. For $d_0 > d_{0\lim}$ the present state is accepted from an economic point of view, the no-upgrade strategy is the optimum solution leading to a lower total cost.

The minimum reliability index β_0 below which the bridge should be upgraded then corresponds to:

$$\beta_0 = -\Phi^{-1}(p_f(d_{0\lim})) \qquad \text{Eq. 6-12}$$

Realistically assuming C_f is substantially larger then $C_m\, d_{opt}$ in emergency and risk crossing situations, the minimum reliability index β_0 crystalizes simplified as:

$$\beta_0 \approx -\Phi^{-1}\left[\frac{C_0}{C_f} + p_f(d_{opt})\right] \qquad \text{Eq. 6-13}$$

It is however problematic to deliver a general procedure for the assessment of the optimum repair strategy (d_{opt}) as it generally requires a case-specific approach due to the broad definition of the decision parameter *d*. For instance, the cross-section or limitation of traffic load may be optimized as the result of the selected strategy minimizing the cost.

A set of obtained results [114] is adopted for the estimation of d_{opt} and subsequently β_{up} for purposes of this work. The cost optimization was performed assuming some limits of estimated maximum $C_{f,max}$ and minimum $C_{f,min}$ failure cost for the respective consequence class according to KANDA & SHAH [75] and an estimated upgrade cost C_0 reflecting the disruption due to temporary closure of a bridge.

The result is a comparison of total cost to target reliability at different values of arbitral d_{opt}. Graphical representation for CC2 and CC3 is offered in Figure 56 and Figure 57. CC1 can be found in [114].

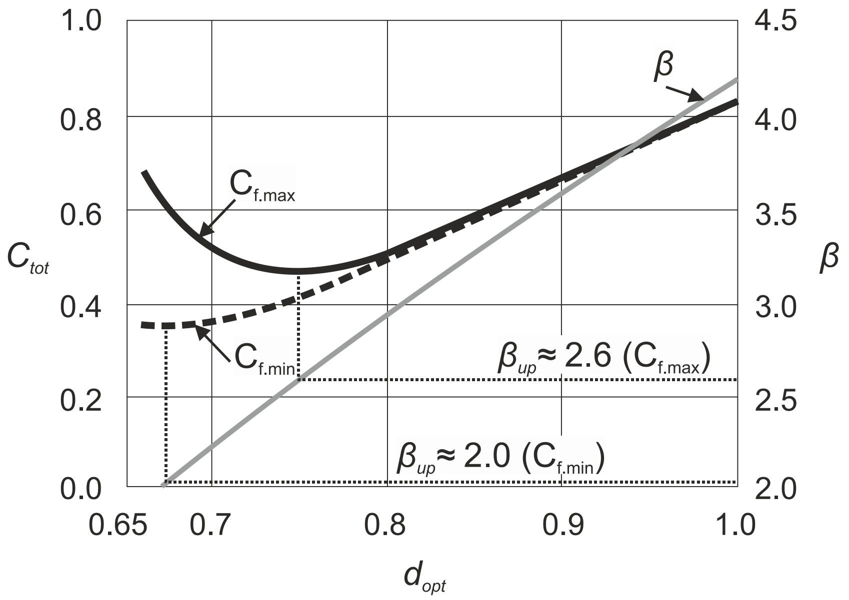


Figure 56: Variation of total cost and reliability index indicating the optimum reliability index β_{up} for CC2 [114].

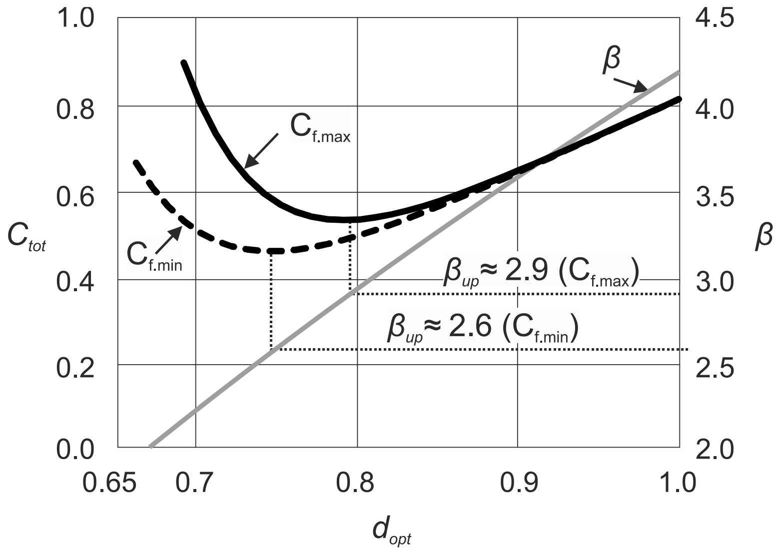


Figure 57: Variation of total cost and reliability index indicating the optimum reliability index β_{up} for CC3 [114].

Approximation of the C_f costs by the middle values from $C_{f.max}$ and $C_{f.min}$ (from Figure 56 and Figure 57) yields the following results for each consequence class:

- CC1: Failure consequences small/some: $p_f(d_{opt}) \approx 0.03$ corresponding to $\beta_{up} \approx 1.9$,
- CC2: Failure consequences medium: $p_f(d_{opt}) \approx 0.008$ corresponding to $\beta_{up} \approx 2.4$,
- CC3: Failure consequences high: $p_f(d_{opt}) \approx 0.003$ corresponding to $\beta_{up} \approx 2.8$.

However, the minimum reliability index β_0 is additionally dependent on the cost ratio C_0 / C_f that has to be accounted for. It essentially dictates, whether β_{up} may be reduced as justified by the cost of closure and failure consequences. The results in Figure 58 show β_0 on the vertical axis in relation to the cost ratio. This section is concerned with risk crossing of vehicles and during an emergency situation. It can be assumed that if upgrade is to be undertaken, the target reliability will be mostly governed by the cost ratio C_0 / C_f. As these become comparable in an emergency situations, the exact evaluation of the $p_f(d_{opt})$ becomes of a lower importance [119]. This can be observed from Figure 58 in the region beyond $C_0 / C_f > 0.01$ where the influence of initially selected $p_f(d_{opt})$ is quite low. High cost of upgrade C_0 in essence decreases the minimum accepted structural reliability.

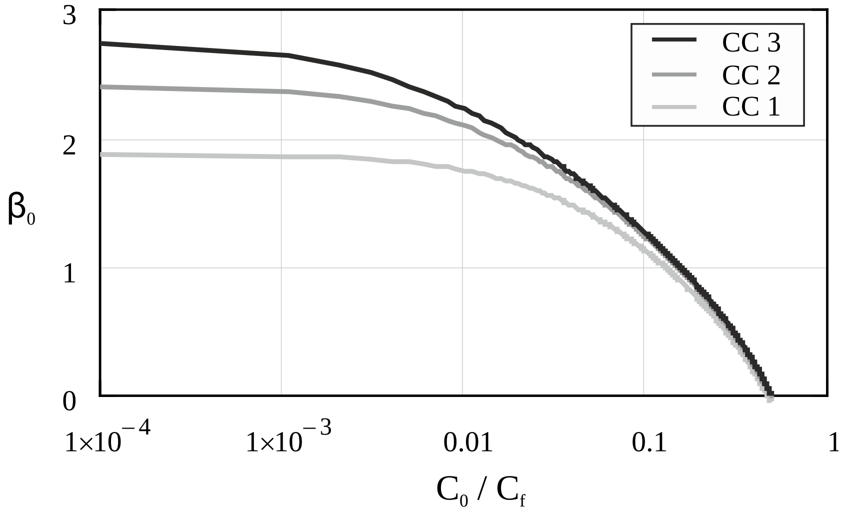


Figure 58: Variation of the minimum target reliability index β_0 based cost ratio C_0 / C_f.

Considerations of mainly CC2 (common bridges) and CC3 (large and important structures) and a low ratio of $C_0 / C_f < 0.001$ dictate the minimum reliability β_0 index essentially equal to the particular value of β_{up}, the optimum reliability index for upgrade. In this case, the minimum reliability index is about 2.4 and 2.8 for CC2 and CC3 respectively. The reliability level drops below 2.0 at ratios C_0 / C_f higher than 0.01, a commonly considered limit for serviceability limit state. It is interesting to note that for high relative costs of safety measures, ISO 2394 [72] indicates $\beta = 2.3$ and 3.1 for moderate and great failure consequences, respectively.

However, it is questionable whether the target level should be selected on the basis of the human safety criterion since it regards only safety of users of a bridge and fails to consider additional costs in form of losses related to temporary bridge closure (cost C_0). The decision depends on case-specific conditions and in general should aim at balancing risks of users and risks of people endangered when the crossing of heavy freights is not permitted during emergency situation. The people may be actually put in danger by the decision not to cross, as shown by the example in SÝKORA, HOLICKÝ & LENNER [115].

6.3 Optimised Reliability Index

6.3.1 Normal and Caution Crossing

Since the assessment of existing bridges for military loading should produce the highest allowable loading and not a decision whether the specified loading requires a repair or a bridge closure, it is proposed to accept the target reliability index values β_r as listed in Section 6.2 and developed by STEENBERGEN & VROUWENVELDER [122] on the basis of economic optimization. As could be observed, these are clearly not mandated by the human safety criteria of military personnel but rather structural safety. The target reliability may be regarded as constant for the selected time reference. The particular values for normal and caution crossing may be therefore regarded as:

- CC3; $\beta_r = 3.8$
- CC2; $\beta_r = 3.3$

6.3.2 Risk Crossing

Risk crossing condition may utilize lower target reliability. It is expected in this work that risk crossing is associated with extremely short durations, where the human safety dictates the target reliability. The structural safety becomes more decisive at longer periods of time.

Very high costs of closure due to upgrade in terms of C_0 increase the acceptable probability of failure of present state. Small values C_0 compared to failure cost C_f will lead to the upgrade decision. Without any provisions for the particular ratio of C_0 / C_f it is proposed to accept for risk crossing the target reliability equal to β_0 with very low ratio $C_0 / C_f < 0.001$ as it leads to a higher reliability level. This perception additionally assumes, that the target reliability will remain above the optimum target reliability for upgrade, i.e. no upgrade will be necessary before the passage of military vehicles.

However, for a case specific approach, the cost of upgrade C_0 and cost of failure C_f cost should be evaluated more carefully as the reduction in β may significantly influence the outcome of the assessment and may permit significantly higher vehicles. This is particularly important at longer reference periods where the target for human safety is very low and the target reliability is clearly dominated by β_0. Additionally, the cost ratio C_0 / C_f should be considered during situations where the cost of temporary closure C_0 is very high, as could be the case of military response to immediate danger or natural threat. The decision to close the bridge due to an upgrade would reflect in high consequences caused by the lack of response to the threat. It might be therefore a better decision in this case to permit heavier vehicles under reduced target reliability conditions, refer to Section 6.2.2.

Table 41 shows the target reliability index for risk crossing condition β_{risk}. This is essentially an evaluation of maximum required target reliability for human and structural safety. At shorter periods, the human safety clearly dictates the risk crossing target reliability index.

Table 40: Target reliabilities for Risk Crossing Condition

Consequence class	Time reference	β_{hs} human safety	β_0 structural safety	β_{risk} max (β_{hs}, β_0)
CC3				
	1 week	3.4	2.8	3.4
	4 weeks	3.0	2.8	3.0
	1 year	2.0	2.8	2.8
CC2				
	1 week	2.9	2.4	2.9
	4 weeks	2.4	2.4	2.4
	1 year	1.3	2.4	2.4

7 Partial Factors for Actions

This section aims at showing the calibrated partial factors within the proposed semi-probabilistic safety concept for military vehicles crossing over bridges. Stochastic models for static load effect, dynamic amplification and model uncertainty along with target reliabilities for structural and human safety were proposed in previous sections.

For the development of partial factors only two consequence classes are considered, CC3 for major bridges and CC2 for standard bridges. Given the experience in bridge assessment for military loading and the scope of military engineers, it is expected that most of the bridges will fall into CC2 category.

Generally, the partial factor is considered with a single sensitivity factor α_E as regulated by EN 1990 [45]. Such provision should secure an acceptable and conservative solution for all loading situations. Consideration of different α values can be accomplished when a set of partial factors is developed for specific ratios of permanent load to variable load effect. This would however unnecessary complicate matters since the process of calculating partial factors becomes iterative as the load ratio κ is not known prior to the assessment and particular values of model uncertainty and variation of loading should be updated according to the inspection. The partial factors are however exemplarily shown assuming the results from the FORM analysis performed in Section 5.7.

The target reliability is dictated by the minimum structural safety rather the human safety of military personnel for normal and caution crossing condition. This suggests the partial factor for military loading are independent of time reference as the lower human safety requirements of military personnel are not relevant. Economic optimization in this case leads to constant partial factors for the selected design life. Risk crossing condition on other hand may be governed by the minimum safety criteria for human life during the very short time reference periods. The factors should be specified accordingly.

7.1 Partial Factor for Permanent Action

The definition of the partial factor for permanent action is repeated here for the convenience. Essentially, the factor is composed of two components – model uncertainty and reliability based factor (Eq. 7-1).

$$\gamma_G = \gamma_{Ed,g} \cdot \gamma_g \qquad \text{Eq. 7-1}$$

It is assumed from Section 4.3.2 that $\gamma_{Ed,g}$ = 1.07 and the partial factor γ_g can be written as:

$$\gamma_g = 1 - \alpha_E \cdot \beta \cdot V_G \qquad \text{Eq. 7-2}$$

where α_E denotes the FORM sensitivity factor, β stands for the target reliability and V_G stands for the coefficient of variation for the permanent action G.

It is assumed in this section that α_E is approximated as -0.7 in accordance with EN 1990 [45]. The resulting partial factors are then plotted in Figure 59 for two coefficients of variation of permanent action [117]:

- V_G = 0.1 as commonly assumed for the design of new structures.
- V_G = 0.05 is taken as a representative value for an existing structure, assuming verification.

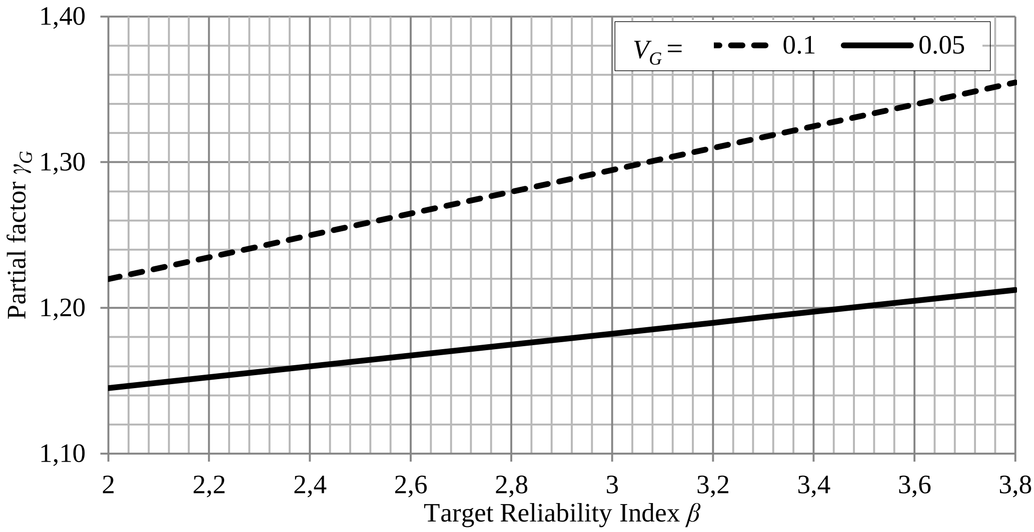


Figure 59: Partial factor γ_G for different safety levels.

A consideration of different crossing conditions reveals the possibility to utilize the target reliability index for each respective condition. The slope dictating the partial factor at V_G = 0.05 is seemingly flat indicating a small sensitivity of partial factor to the reliability index. Assumption of the target reliability level β_r repair for a consequence class CC3 and CC2 (Section 6.3.1) and V_G = 0.05 yields the partial factors for both normal and crossing condition summarized in Table 41.

Table 41: Partial factor γ_G for permanent action; normal and caution crossing

Consequence Class	β_r	γ_G
CC3	3.8	1.21
CC2	3.3	1.19

Risk crossing condition may utilize lowered target reliability as summarized in Table 40. As already mentioned, it is expected in this work that risk crossing is associated with extremely short durations, where the human safety dictates the target reliability. At longer periods of time becomes the structural safety more decisive. The partial factors for different reference periods are summarized in Table 42.

Table 42: Partial factor γ_G for risk crossing

Consequence class	Time reference	β_{risk}	γ_G
CC3			
	1 week	3.4	1.20
	4 weeks	3.0	1.18
	1 year	2.8	1.17
CC2			
	1 week	2.9	1.18
	4 weeks	2.4	1.16
	1 year	2.4	1.16

There are apparently quite minimal differences in the resulting partial factor for permanent action γ_G during risk crossing situation regardless the selected reference period. This can be contributed to the seemingly flat line describing the partial factor in relationship to the reliability index.

7.2 Partial Factor for Variable Action

The definition of the load effect and partial factor for variable action is also repeated here for convenience. Background on the proposal can be found in Section 4.3.3. Essentially, the total load effect is composed of three components (Eq. 7-3):

$$Q = \theta_E \cdot \delta \cdot Q_{MLC} \quad \text{Eq. 7-3}$$

where θ_E denotes the model uncertainty in estimation of the load effect from the load model, δ is a dynamic amplification factor and Q_{MLC} is a static load effect. The partial factor γ_Q is defined as:

$$\gamma_Q = \exp(-\alpha_E \cdot \beta \cdot V_Q), \quad \text{Eq. 7-4}$$

where α_E denotes the FORM sensitivity factor, β target reliability index and V_Q coefficient of variation of Q obtained as follows:

$$V_Q \approx \sqrt{V_\theta{}^2 + V_\delta{}^2 + V_{\text{QMLC}}{}^2}, \quad \text{Eq. 7-5}$$

where V_θ, V_δ and V_{QMLC} are the coefficients of variation of model uncertainty, dynamic amplification and of military static load effect, respectively.

In this section it is assumed that α_E is approximated as -0.7 in accordance to EN 1990 [45]. The resulting partial factors are plotted according to the developed stochastic models from Section 5.8, Table 30. Summary is provided for the resulting coefficient of variation of military load V_Q corresponding to each crossing condition in Table 43. Caution and risk crossing exhibit the same stochastic properties, but risk crossing is commonly associated with a higher probability of failure.

Table 43: Summary of coefficient of variation V_Q, see Table 30.

Variable	Normal	Caution / Risk
Coefficient of variation V_Q	0.12-0.15	0.09

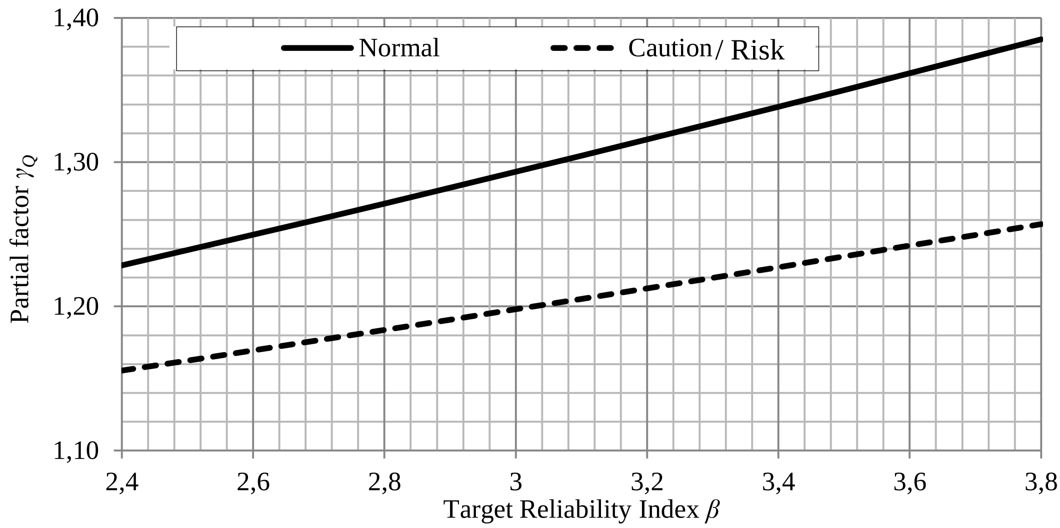


Figure 60: Partial factor γ_Q shown with V_Q = 0.12 for normal; and V_Q = 0.09 for caution and risk crossing.

As can be observed from Figure 60, the target reliability index is not overly significant in the determination of partial factors. It is especially apparent for the case of caution and risk crossing conditions with a quite flat slope. An increase of $\Delta\beta$ = 0.2 produces an increase of the partial factor γ_Q equal to 0.1. This slightly increases for normal crossing where the same change in target reliability index produces an increase of 0.3 in the partial factor value.

Normal Crossing

The influence of selected value for the coefficient of dynamic amplification V_δ is considered for normal crossing. Shorter bridges often exhibit a higher dynamic amplification and a higher variation of the dynamic effects. From Section 5.5 it is apparent that bridges with longer span lengths clearly show a lower coefficient of variation. Regardless, two partial factors are provided here for each Consequence Class to account for low dynamic variation (V_δ = 0.05) and medium dynamic variation (V_δ = 0.10). It is assumed that the bridge under consideration does not exhibit a high road profile roughness or a large bump at the bridge approach and is additionally longer than 5 meters. Should these limits be perceived as not applicable, the structure should be evaluated in caution crossing scenario or on a case specific basis. It must be noted, that particular care should be devoted to the selection of dynamic amplification mean value, since only the coefficient of variation is considered in the development of the partial factor. The factors shown in Table 44 are developed using stochastic properties summarized above and definition according to Equation 7-4.

Table 44: Partial factor γ_Q for normal crossing

Consequence Class	β_r	*low dynamic*		*medium dynamic*	
		V_Q	γ_Q	V_Q	γ_Q
CC3	3.8	0.12	1.40	0.15	1.50
CC2	3.3	0.12	1.33	0.15	1.40

Caution Crossing

Dynamic considerations are not necessary for caution crossing and therefore only a single set of partial factors is developed. Partial factors are however provided for each considered consequence class. It can be observed, that the values are significantly lower when compared to normal crossing condition while maintain the same reliability level. Again, Equation 7-4 and above described stochastic properties are utilized for the calculation of the factors shown in Table 45.

Table 45: Partial factor γ_Q for caution crossing

Consequence Class	β_r	γ_Q
CC3	3.8	1.26
CC2	3.3	1.22

Risk

Risk crossing condition also does not require dynamic considerations and may utilize increased probability of failure as summarized in Table 40. Risk crossing is additionally associated with very short durations and therefore the factor is calculated for different reference periods. The resulting partial factors on the basis of Equation 7-4 are summarized in Table 46. The influence of time reference is as quite low, but with reference to the seemingly flat slope in Figure 60 even a minor change in partial factor yields an observable difference of the reliability index.

Table 46: Partial factor γ_Q for risk crossing

Consequence class	Time reference	β_{risk}	γ_Q
CC3			
	1 week	3.4	1.23
	4 weeks	3.0	1.20
	1 year	2.8	1.18
CC2			
	1 week	2.9	1.19
	4 weeks	2.4	1.16
	1 year	2.4	1.16

7.3 Load ratio considerations

It is proposed to exemplarily investigate the different sensitivity factors α_E as dictated by the load ratio. FORM analysis has shown that the load ratio strongly influences the resulting sensitivity factor. Table 47 summarizes for clarity the results from FORM Analysis *Load Case B* from Section 5.7.2. The results are plotted for selected target reliabilities in Figure 61. Following assumptions are regarded for the definition of factors:

- Partial factor γ_G is calculated in accordance to Eq. 7-1 and Eq. 7-2 with V_G = 0.05 for existing structures and $\alpha_{E,G}$ as the sensitivity factor.
- Partial factor γ_Q is determined for normal crossing condition in accordance to Eq. 7-4 with V_Q = 0.12 for low dynamic conditions and $\alpha_{E,Q}$ as the sensitivity factor.

Table 47: Sensitivity factor α_E at different load ratios; *Load Case B*

Load ratio κ	$\alpha_{E,G}$	$\alpha_{E,Q}$
0.3	-0.21	-0.75
0.4	-0.32	-0.68
0.5	-0.47	-0.56
0.6	-0.61	-0.40
0.7	-0.70	-0.27
0.8	-0.76	-0.16

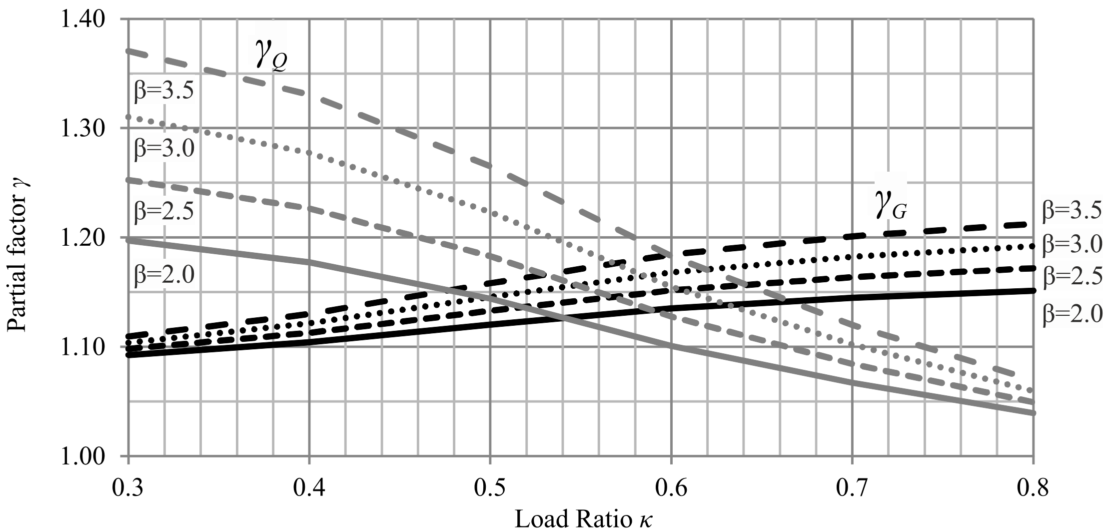


Figure 61: Partial factors γ_G and γ_Q variation with load ratio and target reliability index.

The influence of α_E as dictated by the load ratio can be clearly observed. As expected, the influence of permanent action on the structural reliability is quite low at the low regions of load ratio with a dominance of variable load. The partial factor for permanent action is then reduced in this region. It is compensated by the high influence of variable action and respective high partial factor. This is essentially mirrored at high load ratio with dominating permanent action. It can be observed that the influence of load ratio is quite apparent in comparison to the constant α_E.

The difference of γ_G at the shown target β-values is quite high, as there is an increase of approximately 0.05 to 0.10 in the partial factor value. It is much more apparent for γ_Q where the difference at respective β-values is approximately 0.15 to 0.20. The selected target reliability has somewhat limited influence on the permanent partial factor, the curves remain relatively close to each other for γ_G. The partial factor γ_Q certainly exhibits larger differences. It is especially apparent at the low load ratio and the influence of target reliability is considerable. It indicates a difference of approximately 0.20 of the partial factor value.

It is possible to assess case-specific structures with this approach as it delivers more accurate results than the method with constant $\alpha_E \approx -0.7$ and could be decisive for the evaluation of specific structures, when the partial factors can be much better adjusted to the actual conditions. Should the structure be clearly dominated by either permanent load (long span bridge) or variable load, the corresponding α_E may be used for the adjustment of partial factors to reflect the loading situation and might lead to more economical results. At the same time, such considerations are likely to be better considered by a full probabilistic analysis where the case specifics can be captured more accurately and the time cost and experiences required for such advanced analysis may be mitigated by the favourable results.

7.4 Summary of Partial Factors

It is therefore in this work proposed to consider only $\alpha_E = -0.7$ for the purposes of military safety concept and development of a single set of suitable partial factors. This provides a simple solution and reduces potential demand on engineers. It is however possible to adjust the factors on case-specific basis when clearly the permanent or variable action is dominating the loading, similarly to the provision in EN 1990 [45] for the lead and accompanying action. Additionally, the conditions of a bridge under investigation should be checked as to ensure suitable road profile and bridge approach in order to mitigate an excessive dynamic response of the bridge. In the case of violated profile conditions or exceptionally short bridges, it is advised to utilize caution crossing condition that mitigates the high dynamic response. For details regarding dynamic behavior see Section 5.5. The resulting partial factors are summarized in Table 48 according to the crossing condition for CC3. It is expected, that the majority of bridges under investigation fall into CC2 category. The respective partial factors are shown in Table 49.

Table 48: Partial factors for assessment of CC3 bridges

CC3	Permanent Action γ_G	Variable Action γ_Q
Normal	1.21	--
low dynamic	--	1.40
medium dynamic	--	1.50
Caution	1.21	1.26
Risk		
1 week	1.20	1.23
4 weeks	1.18	1.20
1 year	1.17	1.18

Table 49: Partial factors for assessment of CC2 bridges

CC2	Permanent Action γ_G	Variable Action γ_Q
Normal	1.19	--
low dynamic	--	1.33
medium dynamic	--	1.40
Caution	1.19	1.22
Risk		
1 week	1.18	1.19
4 weeks	1.16	1.16
1 year	1.16	1.16

The suggested factors may be used under the following conditions:

- Thorough inspection of the bridge, including verification of dimensions, is necessary.
- The limitations in Eq. 2-63 must be satisfied with respect to approximation of α for both load and resistance.
- The developed partial factors are suggested for the global system assessment only, local checks are necessary and the listed factors may not be applicable without further verification.
- Case specific approach might be required for structures that do not comply with the suggested limitations.

8 Conclusions

Current partial factors for load effects in Eurocodes are not optimal for the reliability verifications of existing bridges under well-defined loading such as military loads. In addition, considerable differences exist in the definition of the civilian and well-defined military traffic loads exist. Main inconsistence was found in the definition of characteristic loading, load effect variation and dynamic amplification. It was therefore necessary to investigate the safety concept and respective partial factors and to modify them in order to reflect the military traffic and existing nature of the considered bridges while maintaining the continuity of semi-probabilistic safety concept. Simple principles of structural statics, traffic modeling and structural reliability theory were employed to duly account for knowledge about load models, uncertainties, dynamic load effects and crossing conditions.

Numerical simulations served to investigate the static loading and along with an extensive review of dynamic and model uncertainties allowed for the development of stochastic properties for the use in a reliability analysis and partial factors development. Target reliability index definition was considered as a key element of the safety proposal. Required target β-values for respective crossing conditions were delivered based on the cost optimization of criteria for human and structural safety.

The partial factors for military assessment of existing concrete bridges were considered for two consequence classes, but it is expected that the vast majority of encountered bridges fall into CC2. The calculated partial factors for permanent and variable loading are significantly lower than those factors listed in EN 1990 [45] . This can be mainly attributed to the improved description of loading effects and the reduced target reliability. The full overview of calculated factors for each respective crossing condition and time reference is provided for CC2 in Table 49, while a quick overview indicates the following estimates under the previously listed conditions:

- $\gamma_G \approx 1.20$ for normal and caution crossing; $\gamma_G \approx 1.17$ for risk crossing
- $\gamma_Q \approx 1.35$ for normal; $\gamma_Q \approx 1.20$ for caution and $\gamma_Q \approx 1.17$ for risk crossing

The partial factor values developed in this work may be used for the general global assessment of concrete bridges, where military vehicles represent the variable loading according to the STANAG 2021 [93]. Such approach recognizes the particular aspects of military assessment of existing concrete bridges.

A possibility of case-specific considerations in form of a reliability analysis or development of custom partial factors allows for even more accurate assessment of a selected bridge. This can be contributed mainly to:

- potentially improved on-site dynamic amplification characteristics,
- target reliability index reflecting cost optimization,
- estimated load ratio indicating the sensitivity factors.

It might be therefore prudent to investigate some bridges in more detail when higher capacity is required. This however increases the demand on engineer and calculation times.

The topic of military bridge assessment is certainly open to further developments. The dynamic amplification undoubtedly deserves additional work as there is no consensus among the scientific community regarding the specific values of dynamic amplification factor and stochastic properties. Further investigation regarding the assessment may also be aimed at the development of partial or combination factors for mixed military and civilian traffic on bridges. Considering the aging infrastructure, it might be prudent to even investigate the military assessment along with considerations for commonly encountered damage on concrete bridges including the effects of posttensioning. The ultimate goal of any further advances shall be to improve the here developed concepts for safe and reliable military assessment of existing bridges.

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http://www.tankmodell.de/html/bw035.html

Appendix A

Influence Lines

1.0 Define Span Length

$l_{span} := 10$ this is a randomly selected span length,

2.0 Define Influence Lines

2.1 Simple Beam

$$IL(x) := \begin{vmatrix} Value \leftarrow x \cdot \frac{2}{l_{span}} \text{ if } x \leq \frac{l_{span}}{2} \\ Value \leftarrow 2 + x \cdot \frac{-2}{l_{span}} \text{ if } x \geq \frac{l_{span}}{2} \\ Value \leftarrow 0 \text{ if } x < 0 \\ Value \leftarrow 0 \text{ if } x > l_{span} \end{vmatrix}$$

define the portion of influence line, the similar numerical definition is used for the rest of the IL reflecting the static system of each

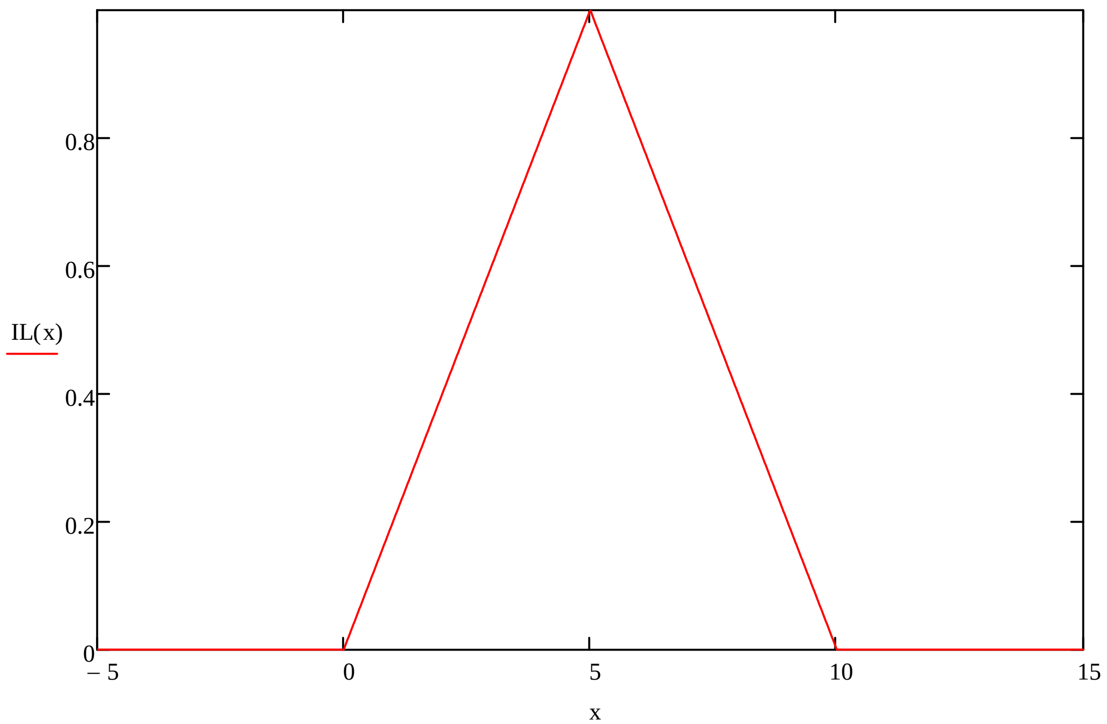


Figure A1: Influence Line for Simple Beam

2.2 Fixed End Beam at Midspan

$$IL(x) := \begin{array}{|l} \text{Value} \leftarrow \dfrac{x^2}{\left(\dfrac{l_{span}}{2}\right)^2} \quad \text{if } x \le \dfrac{l_{span}}{2} \\ \text{Value} \leftarrow \dfrac{\left(x - l_{span}\right)^2}{\left(\dfrac{l_{span}}{2}\right)^2} \quad \text{if } x \ge \dfrac{l_{span}}{2} \\ \text{Value} \leftarrow 0 \quad \text{if } x < 0 \\ \text{Value} \leftarrow 0 \quad \text{if } x > l_{span} \end{array}$$

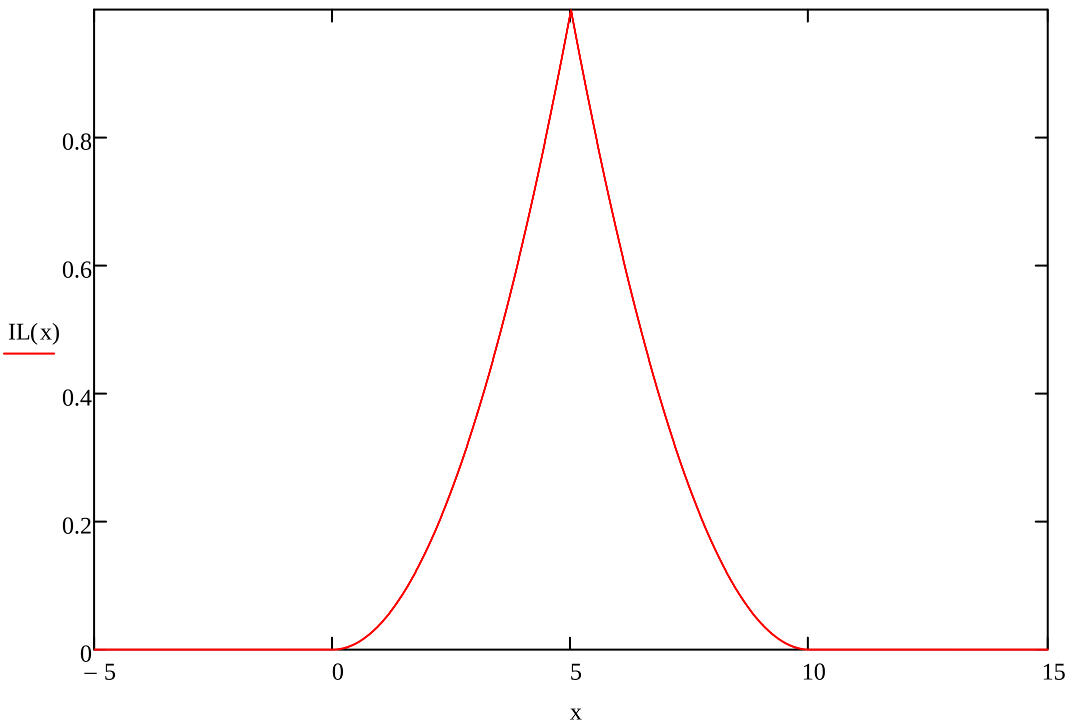


Figure A2: Influence Line for Fixed End Beam at Midspan

2.3 Fixed End Beam at End

$$IL\_1(x) := \begin{vmatrix} Value \leftarrow -x \cdot \left(\frac{l_{span} - x}{l_{span}}\right)^2 \text{ if } x \le l_{span} \\ Value \leftarrow 0 \text{ if } x < 0 \\ Value \leftarrow 0 \text{ if } x > l_{span} \end{vmatrix}$$

$X := 1$

Vorgabe

$X > 0$

$\text{Minimieren}(IL\_1, X) = 3.333$

$Place_{CG} := \text{Minimieren}(IL\_1, X)$

this has to solve for the position of maximum moment in order to determine the center of gravity point for the MLC vehicle

at the same time, the maximum value is used to become unity value of the influence line

$$IL(x) := \begin{vmatrix} Value \leftarrow -x \cdot \frac{\left(\frac{l_{span} - x}{l_{span}}\right)^2}{-IL\_1(Place_{CG})} \text{ if } x \le l_{span} \\ Value \leftarrow 0 \text{ if } x < 0 \\ Value \leftarrow 0 \text{ if } x > l_{span} \end{vmatrix}$$

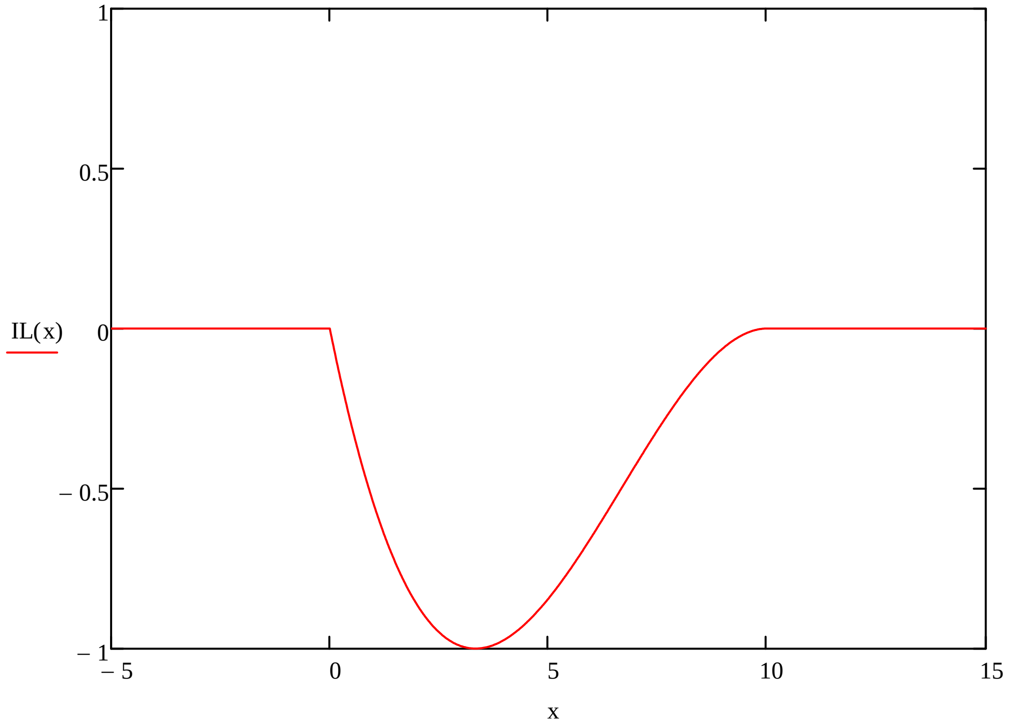


Figure A3: Influence Line for Fixed End Beam at End

2.4 Continous Beam at Support

$$IL\_1(x) := \begin{array}{|l} Value \leftarrow -\left[\frac{x\cdot(l_{span}-x)}{4\cdot l_{span}^{2}}\cdot(l_{span}+x)\right] \text{ if } x \le l_{span} \\ Value \leftarrow 0 \text{ if } x < 0 \\ Value \leftarrow 0 \text{ if } x > l_{span} \end{array}$$

$X := 1$

Vorgabe

$X > 0$

$\text{Minimieren}(IL\_1, X) = 5.774$

$Place_{CG} := \text{Minimieren}(IL\_1, X)$

$$IL(x) := \begin{array}{|l} Value \leftarrow \dfrac{-\left[\frac{x\cdot(l_{span}-x)}{4\cdot l_{span}^{2}}\cdot(l_{span}+x)\right]}{-IL\_1(Place_{CG})} \text{ if } x \le l_{span} \\ Value \leftarrow \dfrac{-\left[\frac{(x-l_{span})\cdot(2l_{span}-x)}{4\cdot l_{span}^{2}}\cdot(3l_{span}-x)\right]}{-IL\_1(Place_{CG})} \text{ if } x \ge l_{span} \\ Value \leftarrow 0 \text{ if } x < 0 \\ Value \leftarrow 0 \text{ if } x > 2l_{span} \end{array}$$

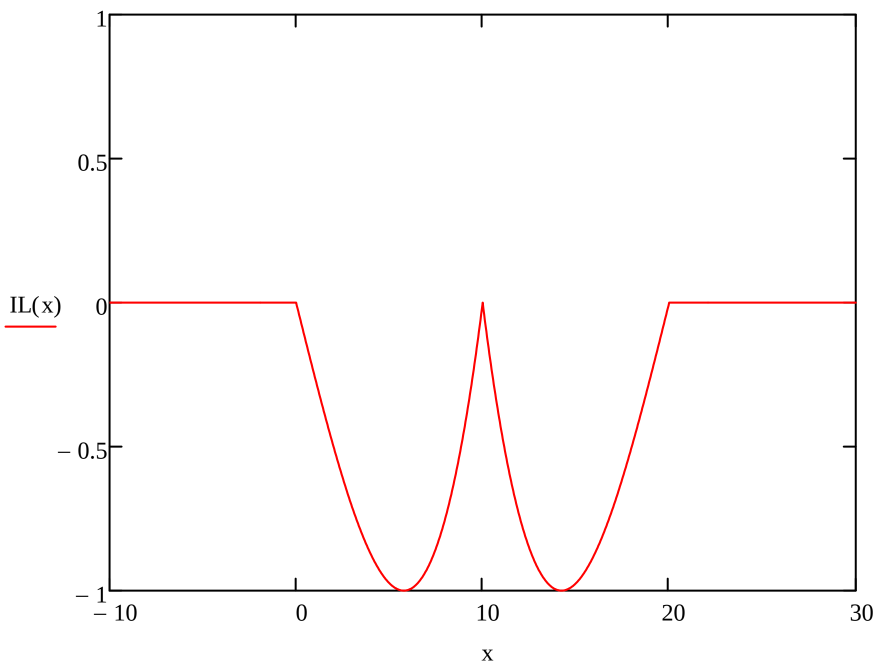


Figure A4: Influence Line for Continous Beam at Support

2.4 Continous Beam at Midspan

$$IL\_1(x) := \left| \begin{array}{l} V_1 \leftarrow \frac{(l_{span} - x)}{4 \cdot l_{span}^{3}} \cdot \left[4 \cdot l_{span}^{2} - x \cdot (l_{span} - x)\right] \\ Value \leftarrow V_1 \cdot x - (1 - V_1) \cdot \left(\frac{l_{span}}{2} - x\right) \ \text{if} \ x \le \frac{l_{span}}{2} \\ Value \leftarrow V_1 \cdot \frac{l_{span}}{2} \ \text{if} \ x \ge \frac{l_{span}}{2} \\ Value \leftarrow 0 \ \text{if} \ x \ge l_{span} \\ Value \leftarrow 0 \ \text{if} \ x < 0 \end{array} \right.$$

$X := 1$

Vorgabe

$X > 0$

Maximieren(IL_1, X) = 5

$Place_{CG} :=$ Maximieren(IL_1, X) = 5

$$IL(x) := \left| \begin{array}{l} V_1 \leftarrow \frac{(l_{span} - x)}{4 \cdot l_{span}^{3}} \cdot \left[4 \cdot l_{span}^{2} - x \cdot (l_{span} - x)\right] \\ Value \leftarrow \frac{V_1 \cdot x - (1 - V_1) \cdot \left(\frac{l_{span}}{2} - x\right)}{IL\_1(Place_{CG})} \ \text{if} \ x \le \frac{l_{span}}{2} \\ Value \leftarrow \frac{V_1 \cdot \frac{l_{span}}{2}}{IL\_1(Place_{CG})} \ \text{if} \ x \ge \frac{l_{span}}{2} \\ Value \leftarrow 0 \ \text{if} \ x \ge l_{span} \\ Value \leftarrow 0 \ \text{if} \ x < 0 \end{array} \right.$$

Here should be noted, that the negative portion of influence line in second span is neglected since it does not contribute to the maximum load effect

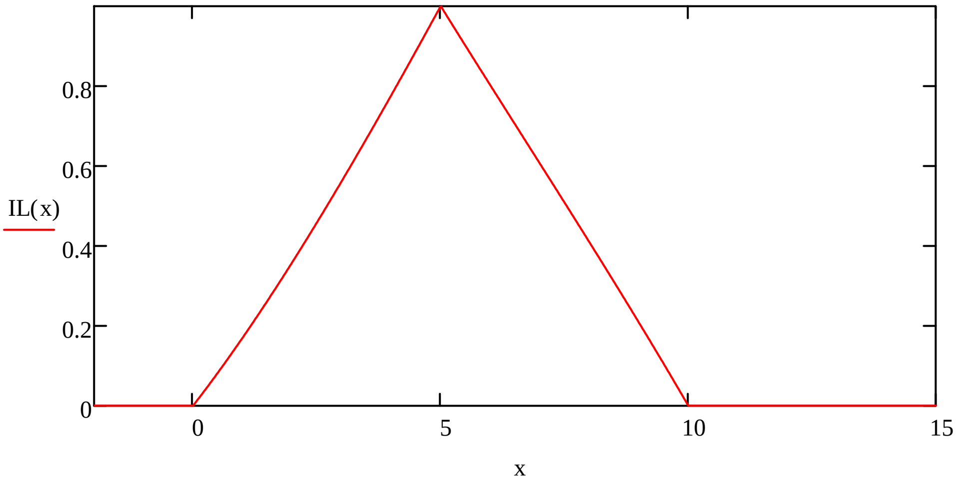


Figure A5: Influence Line for Continous Beam at Midspan

Appendix B

Numerical Simulations

Job:	**Partial Factors for Military Variable Loading**	Date:	05.09.2013
Calc for:	**Numerical Simulation - Simple Span**	By:	RLE

1.0 Define vehicle

Define the MLC40 Vehicle

Axle Loads:

$$L_{def} := \begin{pmatrix} 63.5 \\ 117.9 \\ 117.9 \\ 127 \end{pmatrix} \qquad \sum L_{def} = 426.3$$

Axle Spacings:

$$S_{def} := \begin{pmatrix} 3.66 \\ 1.22 \\ 4.88 \end{pmatrix} \qquad \sum S_{def} = 9.76$$

Define the variation coefficients for each variable

$COV_L := 5\%$ $\qquad$ $COV_S := 5\%$

$$\sigma_L := L_{def} \cdot COV_L = \begin{pmatrix} 3.175 \\ 5.895 \\ 5.895 \\ 6.35 \end{pmatrix} \qquad \sigma_S := S_{def} \cdot COV_S = \begin{pmatrix} 0.183 \\ 0.061 \\ 0.244 \end{pmatrix}$$

Generate n_{sim} number of theoretical vehicles by normal distribution of the values

$Nr := 10000$ $\qquad$ n_{sim} number selected

Axle Loads

$$AL(x) := \left| \begin{array}{l} \text{for } y \in 0..(Nr-1) \\ \quad Lx \leftarrow rnorm\left[Nr, L_{def\,(x)}, \sigma_{L(x)}\right] \\ Lx \end{array} \right.$$

Axle Spacings

$$AS(x) := \left| \begin{array}{l} \text{for } y \in 0..(Nr-1) \\ \quad Sx \leftarrow rnorm\left[Nr, S_{def\,(x)}, \sigma_{S(x)}\right] \\ Sx \end{array} \right.$$

Assemble values into a vectors

$L := erweitern(AL(0), AL(1), AL(2), AL(3))$

$S := erweitern(AS(0), AS(1), AS(2))$

1.1 Check the vehicle

Calculate vehicle weight

$$W_{check} := \left| \begin{array}{l} i \leftarrow 0 \\ \text{for } \; y \in 0..(Nr-1) \\ \quad \left| \begin{array}{l} j \leftarrow 0 \\ \text{for } \; x \in 0..\left(\text{länge}\left(L_{def}\right)-1\right) \\ \quad \left| \begin{array}{l} X_j \leftarrow L_{i,j} \\ j \leftarrow j+1 \end{array} \right. \\ W_i \leftarrow \sum X \\ i \leftarrow i+1 \end{array} \right. \\ W \end{array} \right.$$

$$W_x := \sum L_{def} = 426.3$$

$$\frac{\max\left(W_{check}\right)}{W_x} = 1.104 \qquad \frac{\min\left(W_{check}\right)}{W_x} = 0.918$$

Calculate maximum vehicle length

$$L_{check} := \left| \begin{array}{l} i \leftarrow 0 \\ \text{for } \; y \in 0..(Nr-1) \\ \quad \left| \begin{array}{l} j \leftarrow 0 \\ \text{for } \; x \in 0..\left(\text{länge}\left(S_{def}\right)-1\right) \\ \quad \left| \begin{array}{l} X_j \leftarrow S_{i,j} \\ j \leftarrow j+1 \end{array} \right. \\ S_{check_i} \leftarrow \sum X \\ i \leftarrow i+1 \end{array} \right. \\ S_{check} \end{array} \right.$$

$$S_x := \sum S_{def} = 9.76$$

$$\frac{\max\left(L_{check}\right)}{S_x} = 1.117 \qquad \frac{\min\left(L_{check}\right)}{S_x} = 0.88$$

$$\max\left(L_{check}\right) = 10.906$$

1.2 Center of Gravity

It is necessary to calculate center of gravity for each generated vehicle

$$
CG := \left|
\begin{array}{l}
i \leftarrow 0 \\
\text{for } y \in 0..(Nr-1) \\
\quad \left|
\begin{array}{l}
j \leftarrow 0 \\
\text{for } x \in 0..\left(\text{länge}\left(S_{def}\right)-1\right) \\
\quad \left|
\begin{array}{l}
S_{vector_j} \leftarrow S_{i,j} \\
j \leftarrow j+1
\end{array}\right. \\
k \leftarrow 1 \\
Length_0 \leftarrow S_{vector_0} \\
\text{for } x \in 1..\left(\text{länge}\left(S_{vector}\right)-1\right) \\
\quad \left|
\begin{array}{l}
Length_k \leftarrow Length_{k-1} + S_{vector_k} \\
k \leftarrow k+1
\end{array}\right. \\
l \leftarrow 0 \\
\text{for } x \in 0..\left(\text{länge}\left(L_{def}\right)-1\right) \\
\quad \left|
\begin{array}{l}
L_{vector_l} \leftarrow L_{i,l} \\
l \leftarrow l+1
\end{array}\right. \\
m \leftarrow 0 \\
\text{for } x \in 1..\text{länge}\left(L_{vector}\right)-1 \\
\quad \left|
\begin{array}{l}
C_m \leftarrow \dfrac{L_{vector_{m+1}} \cdot Length_m}{\sum L_{vector}} \\
m \leftarrow m+1
\end{array}\right. \\
CG_i \leftarrow \sum C \\
i \leftarrow i+1
\end{array}\right. \\
CG
\end{array}\right.
$$

Rearrange the axle positions so that they are expressed in the distance from CG.

Center of Gravity is set in the critical position on the span.

```
L_CG := | i ← 0
        | for y ∈ 0..(Nr − 1)
        |   | j ← 1
        |   | Lp_{i,0} ← (−CG)_i
        |   | for x ∈ 1..länge(L_def) − 1
        |   |   | Lp_{i,j} ← Lp_{i,j−1} + S_{i,j−1}
        |   |   | j ← j + 1
        |   | i ← i + 1
        | Lp
```

2.0 Load Effect Calculation

2.1 Span length

It is necessary to define the considered bridge span length

$l_{span} := 25$

2.2 Influence Line and Critical Position

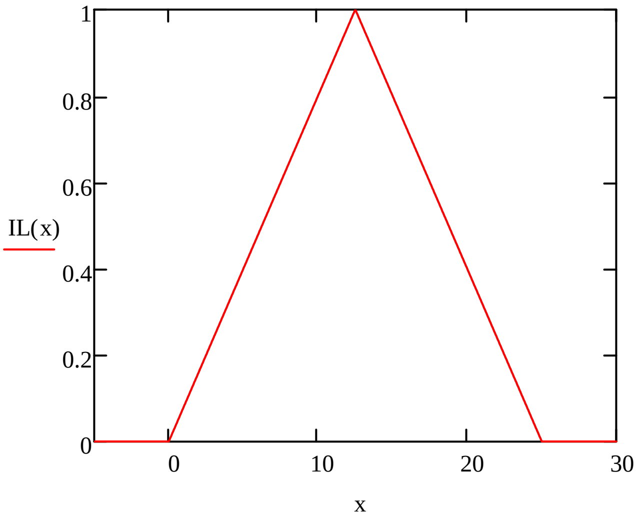
Rearrange the axles in terms of distance from the start

```
L_position := | i ← 0
              | for y ∈ 0..(Nr − 1)
              |   | j ← 0
              |   | for x ∈ 0..länge(L_def) − 1
              |   |   | L_position_{i,j} ← l_span/2 + L_CG_{i,j}
              |   |   | j ← j + 1
              |   | i ← i + 1
              | L_position
```

$$L_{position} := \frac{l_{span}}{2} + L_{CG}$$

Define influence line for simple beam according to Appendix A

$$IL(x) := \left| \begin{array}{l} Value \leftarrow x \cdot \frac{2}{l_{span}} \quad \text{if } x \le \frac{l_{span}}{2} \\ Value \leftarrow 2 + x \cdot \frac{-2}{l_{span}} \quad \text{if } x \ge \frac{l_{span}}{2} \\ Value \leftarrow 0 \quad \text{if } x < 0 \\ Value \leftarrow 0 \quad \text{if } x > l_{span} \end{array} \right.$$



Calculate the influence line value for each of the axle

$$IL_{value} := \left| \begin{array}{l} i \leftarrow 0 \\ \text{for } y \in 0..(Nr-1) \\ \quad \left| \begin{array}{l} j \leftarrow 0 \\ \text{for } x \in 0..\text{länge}(L_{def}) - 1 \\ \quad \left| \begin{array}{l} X \leftarrow L_{position_{i,j}} \\ IL_{value_{i,j}} \leftarrow IL(X) \\ j \leftarrow j + 1 \end{array} \right. \\ i \leftarrow i + 1 \end{array} \right. \\ IL_{value} \end{array} \right.$$

2.3 Bending Moment Calculations

Calculate the resulting moment

$$M := \left| \begin{array}{l} i \leftarrow 0 \\ \text{for } y \in 0..(Nr-1) \\ \quad \left| \begin{array}{l} j \leftarrow 0 \\ \text{for } x \in 0..\text{länge}(L_{def}) - 1 \\ \quad \left| \begin{array}{l} X_j \leftarrow IL_{value_{i,j}} \cdot L_{i,j} \\ j \leftarrow j + 1 \end{array} \right. \\ M_i \leftarrow \sum X \\ i \leftarrow i + 1 \end{array} \right. \\ M \end{array} \right.$$

2.4 Statistical Data

The resulting median and standard deviation are easily obtained

$\text{mittelwert}(M) = 334.998$

$\text{median}(M) = 334.928$

$\text{stdev}(M) = 9.528$

$$COV := \frac{\text{stdev}(M)}{\text{mittelwert}(M)} = 2.844 \cdot \%$$

$H_M := \text{Histogramm}(100, M)$

$h := 0 .. \text{zeilen}(H_M) - 1$

$\text{Norm}_h := \text{dnorm}\left(H_{M_{h,0}}, \text{mittelwert}(M), \text{stdev}(M)\right)$