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Re-Gen

Risk Assessment of Ageing Infrastructure

Review of the most critical existing structures under growing traffic and advice for precise assessment

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Executive Summary

This report examines the effect of increased traffic loading on older bridges. These are structures that have not been designed to support traffic loading according to Eurocode 1. They make up the majority of existing structures and have been designed to various national design guides. In order to account for reserves in the safety of the structure, more realistic loading should be examined using site-specific loading schemes. This allows the current traffic conditions to be assessed but can also be used for predicting the effect of future traffic growth. There are also certain structure types which are particularly sensitive to increased traffic loading. For these structures, the resistance may also need to be better understood and in some cases increased through structural strengthening.

The Eurocode Load Model 1 is used for the design of new bridges but a scaled down version of the model can be used for the assessment of existing bridges. This scaling is usually done by applying so-called α–factors to the load model. In order to obtain an accurate estimate of traffic loading, consideration must be given to traffic growth as road freight in tonne-km is expected to grow at an annual rate of close to 2% until at least 2030. A method which allows for this traffic growth in the site-specific assessment of α–factors is described. The method involves using weigh-in-motion (WIM) data as the basis for traffic simulations which model year-on-year growth of both traffic flow and truck weights. A time-varying generalised extreme value distribution is then fitted to the simulated load effects used to calculate the characteristic load effects and determine the α–factors. The effect of different traffic growth rates on these α–factors can then be examined. Weigh-in-motion data from a site in the Netherlands is used to demonstrate the method. It is found that annual growth in truck weights has the most influence on the α–factors but that increased flow also has a significant effect.

For a more comprehensive evaluation of the level of safety of a structure, reliability analysis can be used to account for the uncertainty of both the load and resistance. Both the load and resistance variables are time dependent and this must be considered in the service life prediction of deteriorating bridges. Reliability analysis is introduced and a description of how to allow for the time dependence of the resistance of the structure and the traffic loading (with and without available WIM data) is given.

Examples of ageing structures which are sensitive to traffic growth are also examined. Very old structures (arch bridges), structures whose design was not ideal (prestressed beam-and-slab bridges in France) and structures sensitive to the increasing frequency of heavy loads (steel bridges with orthotropic decks) are examined and the particular issues which affect each bridge type are discussed.

In order to assess the existing structural capacity of older bridges, load tests can also be performed during which the response of the structure is measured. In other cases, it is possible to reinforce the structure. Examples, such as adding additional prestress, gluing of composite fibres and adding new strengthening structures, are presented here.

1 Introduction

Freight traffic and traffic in general, is increasing in Europe. At the same time, in most European countries, the majority of bridges were designed and built in times when traffic loading was considerably lower. Many of these bridges may need to be strengthened to carry greater load or signposted to limit the weights of vehicles that legally cross them. However, this process can be optimised if the true safety of the bridge is known through better quantification of the risk of overload. This report provides guidance for the assessment of traffic loading on bridges allowing for future traffic growth and also reviews some of the most critical existing types of bridge structures under growing traffic.

One of the key parameters for evaluation of structural safety of existing bridges is the traffic loading. However, traffic flows, as well as truck weights, increase with time. This must be taken into account in order to accurately assess traffic loading on bridges. The Eurocode Load Model 1 (EC1, 2009) is used for the design of new bridges but a scaled down version of the model can be used for the assessment of existing bridges. This scaling is usually done by applying α–factors to the load model. In Section 2 of this report, a method for examining the effect of traffic growth on these α–factors is described. Weigh-in-motion data from the Netherlands is used to provide an example of how traffic growth can be considered in site-specific modelling of traffic loading. Year-on-year growth of both traffic flow and truck weights is modelled and the effect of different traffic growth rates on these α–factors is then examined.

Resistance of the structure is also a time-dependent variable which must be considered in the service life prediction of deteriorating bridges. This is done through reliability analysis which considers the uncertainty associated with factors such as the material properties, workmanship, element dimensions, traffic load, environmental conditions, etc. In Section 3 of this report, reliability analysis is introduced and a methodology is outlined for incorporating the effect of traffic growth into bridge reliability calculations.

In European countries, traffic growth has already been recognized as an issue for infrastructure, and bridges in particular (Godart 2015; de Jong 2004). In Section 4, some examples are provided of types of bridges that have been identified as sensitive to traffic growth and the works that have been done to ensure their safety. The examples given are linked to the age of existing bridges and the design standards in force when they were constructed. The first example presented is the one of arch bridges, which represent historical heritage and should be maintained. These bridges have not been designed according to today's traffic loads. When designed in the early 1900s, the load model consisted of a combination of horses and carts! Other bridge types which were designed when their behaviour was not as well understood as it is today are also vulnerable. Bridges whose early design was still under development are also vulnerable. The example discussed is of multiple isostatic prestressed beam bridges (in English – Beam-and-Slab bridges; in French VIPP – viaducs à ponts à poutres préfabriquées précontraintes par post-tension). The third example outlined is steel bridges with orthotropic decks, recognized as sensitive structures all over Europe (Netherlands, Germany, and France). In this case, the structure undergoes damage not because of the increase of truck weight, but as a result of the increasing number of heavy vehicles (fatigue).

In additional to precise assessment, there are several other approaches that are used to deal with increasing traffic loading. In order to assess the existing structural capacity of the bridge, one can use load tests that are performed with well-known and well-indentified loads and during which the response of the structure is measured. In other cases (when it is not possible to perform load tests or if adequate capacity is not available it is possible to reinforce the structure to provide increased structural capacity. Several means exist to do that; the case of additional prestress or gluing of composite fibres is presented here.

2 Implications of Traffic Growth on Eurocode Alpha-Factors Used in Site-Specific Bridge Assessment

2.1 Introduction

The Eurocode load model for normal traffic, Load Model 1 (LM1), is applicable for the design of new bridges (EC1, 2009). A scaled down version may be a suitable notional load model for the assessment of existing bridges and this scaling is generally done by applying α–factors to the original model. The Eurocode model can be scaled by estimating the characteristic maximum load effects and comparing the results to LM1.

Characteristic maximum load effects can be calculated in a number of ways. A popular approach is to use Weigh-In-Motion (WIM) technology to measure the weights of trucks as they travel along a road in normal traffic. Extrapolation from this measured WIM data to estimate the characteristic maximum bridge load effects is an established procedure and has been used in many studies, both for site-specific assessment (Getachew & OBrien, 2007; Miao & Chan, 2002) and for the development of bridge design codes (EC1, 2009; Nowak, 1999). One commonly-used approach is to extrapolate using a statistical distribution fitted directly to the measured data. The Normal distribution has often been used for extrapolation, with the measured data being plotted on Normal probability paper (Nowak & Hong, 1991; Nowak, 1993). The Generalized Extreme Value (GEV) distribution is more theoretically justifiable and has also been used by many authors, both for site-specific assessment (Miao & Chan 2002; Getachew & OBrien 2007) and for the development of bridge design codes (EC1 2003; Nowak 1993). This family of distributions contains the Gumbel (type I), Fréchet (type II) and Weibull (type III) distributions. Using this extreme value approach the distribution is fitted to block maxima, e.g., maximum daily or maximum weekly values. Other extrapolation approaches have also been used and the accuracy of different extrapolation approaches is compared by Hajializadeh et al. (2012) and OBrien et al. (2015).

As an alternative to direct extrapolation from measured data, Monte Carlo simulation is very commonly used (Enright & OBrien, 2013; O'Connor & OBrien, 2005; OBrien et al, 2013; Sivakumar et al, 2008). This involves fitting suitable distributions to the various measured parameters – axle weights, axle spacings, inter-vehicle gaps, traffic flow rates, etc. These distributions may be parametric, semi-parametric, or empirical (OBrien et al, 2010). Characteristic values can then be extrapolated from a number of years of simulated traffic, or long-run simulations representing thousands of years of traffic can be used to avoid the need for extrapolation (Enright & OBrien, 2013). These previously mentioned studies generally assume that the characteristics of the traffic (i.e. truck weights, number of axles, flow, etc.) do not change over time, i.e., that future traffic will have the same characteristics as the currently measured traffic. However, road freight transport in the European Union is expected to grow by about 1.8% until 2030 due to economic growth and an increased flow of freight traffic between member states (Capros et al, 2008). This will likely result in an increase in the frequency of trucks and increased pressure on legislators to raise the allowable vehicle weights. Consequently, in order to obtain an accurate estimate of maximum lifetime characteristic load effects, it is important to consider future traffic conditions.

The effect of traffic growth on Eurocode alpha factors is examined here using the traffic modelling approach proposed by OBrien et al. (2014) to predict characteristic load effects while allowing for traffic growth. The alpha factor is defined as the characteristic 1000-year load effect calculated for the site divided by the load effect given by the Eurocode load model 1, Equation 1;

$$\alpha - factor = \frac{Simulated\ 1000yr\ load\ effect}{Basic\ LM1\ load\ effect} \quad (1)$$

This work is presented as a best practice example of how the effect of traffic growth on bridge loading can be assessed. WIM data from a road in the Netherlands is used as the basis for the traffic simulations. Growth in both the weight and flow of trucks is considered. Traffic is simulated on two-lane same-direction bridges as this is a very common lane configuration for bridges on European national roads. Characteristic 1000-year load effects are calculated for a range of load effects and bridge lengths. The 1000-year values are then compared with the corresponding load effects for the Eurocode load model and α-factors calculated. The simulations are repeated without traffic growth and the α-factors for both situations are compared in order to determine the effect of traffic growth on bridge safety.

2.2 WIM data

The WIM data used in this study was collected between February and June 2005 on the A12 near Woerden in the Netherlands. This data was used as it contained two-lane same-direction traffic with time stamp records to an accuracy of one hundredth of a second. This time stamp accuracy is required in order to accurately determine the exact relative location of trucks on the bridge – see Re-Gen Deliverable D3.1.

It should be noted that there is very heavy loading at this site. It has a large average flow rate of 6600 trucks per day and recent studies which compared this site to other sites in Europe showed the loading to be significantly greater than at the other European locations (Enright & OBrien, 2013; OBrien & Enright, 2012). As a result the alpha factors calculated for this site will not be typical of sites across Europe. However, it is believed that the increases in alpha factors with traffic growth should be relatively comparable with other sites.

All WIM databases contain a certain amount of erroneous data (Sivakumar et al., 2008). This data must be identified and removed before any meaningful analysis can be performed. Erroneous data can be as a result of gross errors while weighing certain individual trucks. However, it can also be as a result of calibration drift or loss of calibration of the system which affects all records over a certain period. The cause of individual errors is not always known but errors can sometimes be caused by a vehicle straddling two lanes or by a long vehicle being recorded as two separate vehicles. The quality assurance and cleaning procedures described in Re-gen Deliverable D3.1 are used here to ensure that only good quality data is used in the analysis.

WIM data cleaning rules, as presented in Re-Gen Deliverable D3.1, are used to examine the recorded axle spacings and weights to identify and remove suspicious configurations. Permit trucks are also removed from the WIM data as the Eurocode Load Model 1 (LM1) is being examined and abnormal permit truck loading is not applicable as this is covered by Load Model 3 (EC1, 2009) which is not of interest here. Permit trucks are removed using filtering rules proposed by Enright et al. (2015) and described in Re-Gen Deliverable D3.1.

The database includes weekends and public holiday traffic which is significantly lighter in volume and average Gross Vehicle Weight (GVW), in comparison to normal weekday traffic. Due to these different statistical properties, the weekend and holiday data is removed so the analysis can be applied to a homogeneous dataset. For the A12 site, this results in 569,859 trucks measured over 86 weekdays which are used for the analysis.

Calibration drift or loss of calibration is checked by examining the steer axle weight on five-axle trucks. This axle weight is used as it has been identified as having a fairly constant weight and unlike other axles it does not vary significantly with changes to truck load. Figure 1 shows a plot of this property for the WIM data. It can be seen there is little variation over the measurement period and there is no significant loss of calibration.

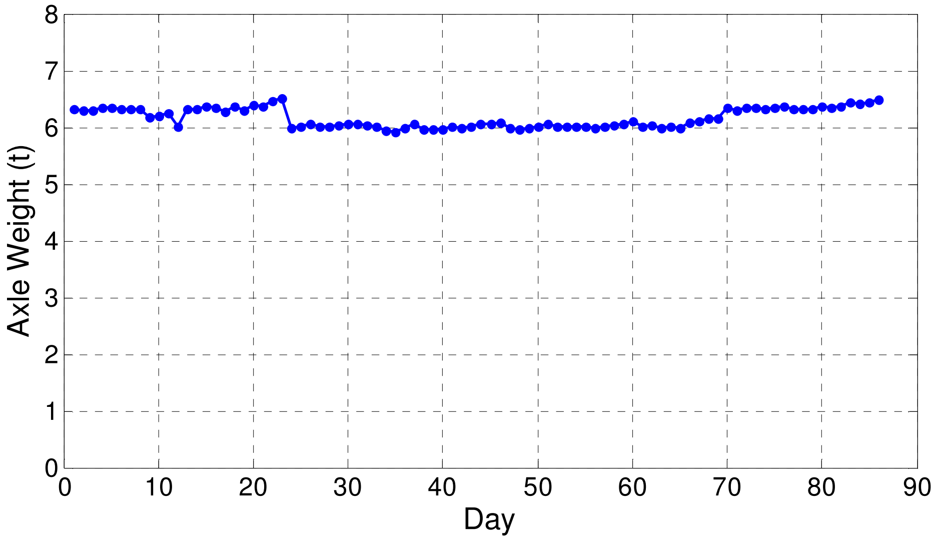


Figure 1. Mean daily steer axle weight on 5-axle trucks.

The histogram of the rearmost axle spacings on 5-axle trucks is also examined, Figure 2, to check that axle spacings are being measured correctly. This spacing is generally part of a tandem/tridem and should be about 1.3 m. As a result, when the distribution of axle spacing is plotted there should be a distinct peak at 1.3 m. The results show that axle spacings are being measured correctly at the site.

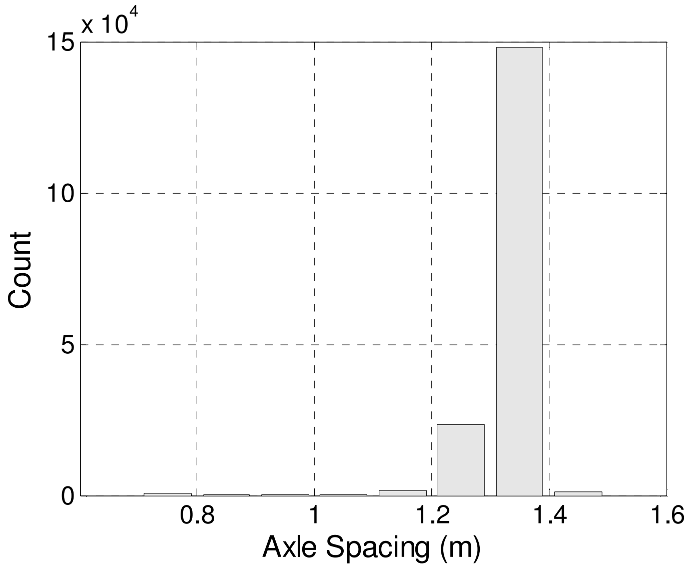


Figure 2. Histogram of rearmost axle spacing on 5-axle trucks.

2.3 Methodology

2.3.1 Traffic simulation model

A Scenario Modelling approach (OBrien & Enright, 2011) is used to perform long run traffic simulations of two-lane same-direction traffic. In Scenario Modelling, the measured data is divided into a series of 'scenarios' – snapshots of approximately 10 trucks, including their properties of weights, configurations and relative spacings. The scenarios are randomly selected from the WIM database and joined together to simulate a stream of traffic. During this process the selected scenarios are perturbed using a 'smoothed bootstrap' approach to generate small changes (OBrien & Enright, 2011). This allows for the occurrence of scenarios a little beyond anything recorded in the traffic. This perturbation process varies the GVW, the in-lane gaps and the inter-lane headway, see Figure 3.

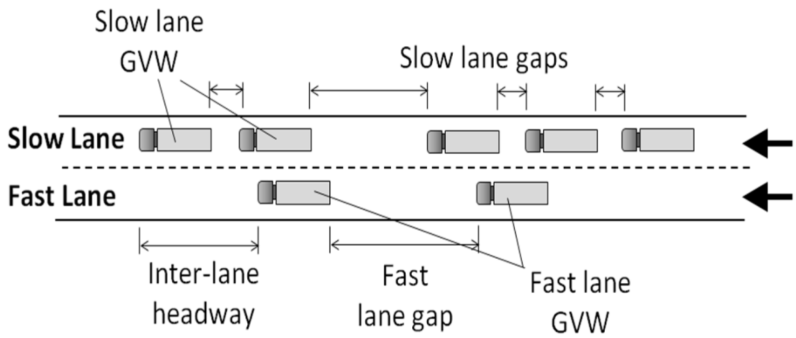


Figure 3. Sample scenario showing the properties which are varied in Scenario Modelling (GVW = gross vehicle weight).

With two-lane same-direction traffic, there are many important correlations between truck weights and inter-truck spacings which influence the results. These correlations must be considered in order to accurately simulate the traffic, something that happens automatically in the Scenario Modelling approach as all relative weights and positions are retained within each scenario. For this reason, Scenario Modelling is recommended over other long run simulation approaches for two-lane same-direction traffic. Scenario Modelling is also only concerned with the trucks in the WIM data, ignoring cars as they are considered insignificant for loading on the bridge lengths considered here (15 m – 45 m).

In the simulation, randomly selected scenarios are placed in the traffic in sequence, with the last truck in one scenario being replaced by the first truck in the following scenario. This preserves the appropriate gap distributions, but care needs to be taken to avoid a very light truck replacing a heavy truck or vice versa. This is achieved when extracting scenarios from the WIM data by specifying that the first and last slow-lane truck in the scenario must have a gross vehicle weight (GVW) less than 30 tonnes. These lighter trucks are assumed not to be significant for critical bridge loading and therefore swapping one of these trucks for another will not affect the characteristic load effects obtained from simulation. Four types of scenario are extracted from the WIM data. The scenarios are based on a minimum of five, six, seven and eight slow-lane trucks per scenario, together with any adjacent fast-lane trucks. All WIM data is scanned for each scenario type. The database of scenarios used for simulation then has the same data stored in four different scenario types, which increases the variability of the simulated traffic. A sample scenario is shown in Figure 3. In each scan, if the last truck is greater than 30 t then more trucks are included until the scenario can finish on a slow-lane truck which is less than 30 t (OBrien & Enright, 2011). The different scans allow for more variation in the scenarios and capture more scenario configurations.

A 40-year simulation of traffic at the Netherlands WIM site is shown in Figure 4 with no traffic growth. The simulated maximum daily load effects for midspan bending moment on a 15 m bridge are plotted alongside those of the measured WIM data. It can be seen that the simulated load effects are a good fit to those of the WIM data and that the trend in the measured data is effectively extrapolated beyond the measured data.

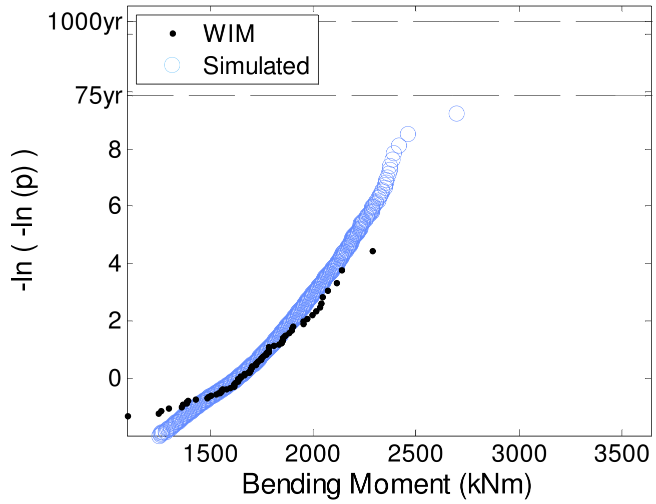


Figure 4. Maximum daily midspan bending moment on a 15 m bridge for a 40 year simulation of traffic at the Netherlands site alongside the maximum daily load effects from the WIM data.

2.3.2 Modelling traffic growth

Increased freight transport can be divided into both a growth in flow and growth in the weights of trucks. As noted previously, an annual growth rate of 1.8% until 2030 has been predicted (Capros et al, 2008). This growth is likely to result in increases in both the frequency and weight of trucks. However, it is not known what proportion of each parameter will contribute to the total growth. As a result, a number of different growth rates and combinations of growth rates are examined for the flow and weight of trucks, see Table 1.

Table 1. Growth Rates Examined.

		Annual Flow Growth		
		0%	1%	2%
Annual Weight Growth	0%	✓[1]	✓	✓
	0.50%	✓	✓	✓
	1%	✓	✓	✓

[1] The reference dataset with which the cases with growth will be compared.

The increase in flow is modelled by assessing each hour of the day independently. This is done to preserve the variations in flow by time of day. Increased flow with time is modelled by first determining the flow associated with the hour of the day and the year being simulated. Scenarios are then randomly selected from scenarios with this flow rate. This results in a gradual increase in flow rate over the simulation period. This increased flow rate, along with the hourly flow, is illustrated in Figure 5 and shows the flow rate at the end of a 40 year simulation period with an annual growth rate in flow of 2%.

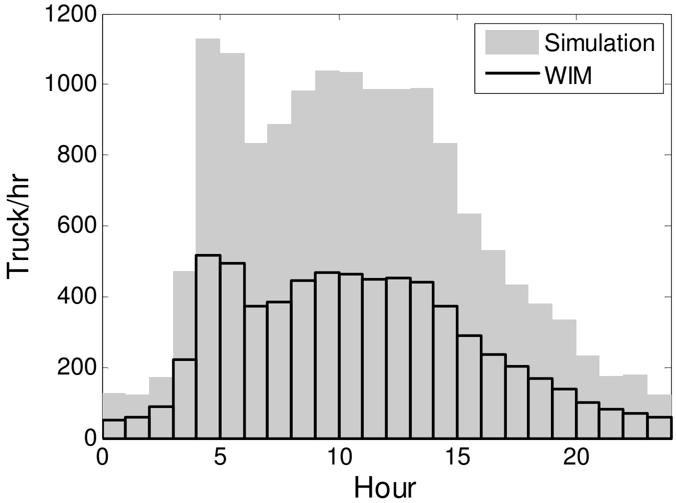


Figure 5. Flow variation by time of day at the WIM site and simulated flow rate at the end of 40 year simulation period with 2% yearly flow growth.

Increases in the weight of trucks with time are also modelled. To realistically increase truck weights, the axle configurations must also change as axle configurations are largely dependent on the load capacity, and hence weight, of the truck. To increase truck weights, the measured traffic is separated into two-tonne weight bands, Figure 6. For each weight band, there is a distribution for the number of axles, with examples shown in Figure 7. It can be seen that, as the gross weight of the truck increases, the number of axles tends to also increase. To increase the weight of a truck in a scenario, it is replaced with another truck whose configuration is randomly selected from the appropriate higher weight band. In general, the new truck will be expected to have the same or more axles than the truck it replaced, but in some cases the heavier trucks may have fewer axles.

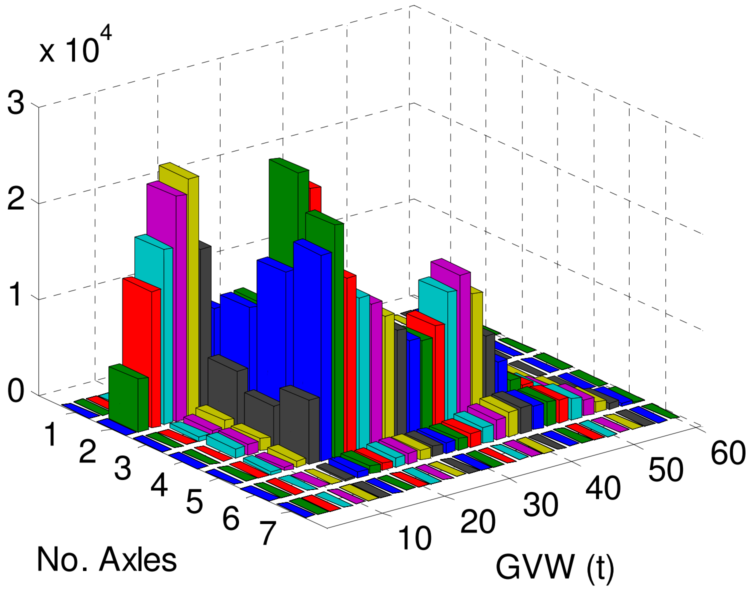


Figure 6. 3D histogram with the number of axles on the trucks in 2 t weight bands.

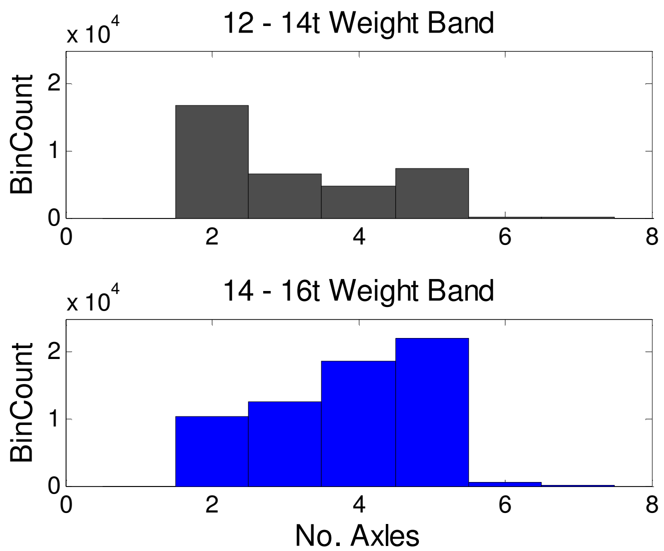


Figure 7. Histogram of the number of axles in the 14-16 t and 16-18 t gross weight bands. The most frequent truck in the 12-14 t band has two axles whereas the most frequent truck in the 14-16 t band has 5 axles.

In the simulation, measured trucks can only be replaced by heavier trucks from the WIM data up to a certain weight limit. Above a certain threshold there will not be any heavier trucks in the WIM database to randomly select to replace the measured truck – see right hand tail of Figure 8. This is particularly relevant towards the end of the simulation period when large increases in truck weight may need to be simulated. As a result, it was decided that the random selection process for simulating heavier trucks would only be applied to trucks with a measured GVW of less than 50 t. This ensures that there is always a selection of heavier trucks to randomly select. Above 50 t, the original axle configuration is kept and the weights on the axles are increased in proportion to simulate growth in weights. However, it is not realistic to continue to increase the weight on an axle without imposing an upper limit. After examination of the trends in axle weights in the WIM data, it was decided to impose an upper limit of 20 t above which the axle weights could not be increased. It should be noted that there were a small number of measured axles which exceeded 20 t and these were allowed to remain in the data.

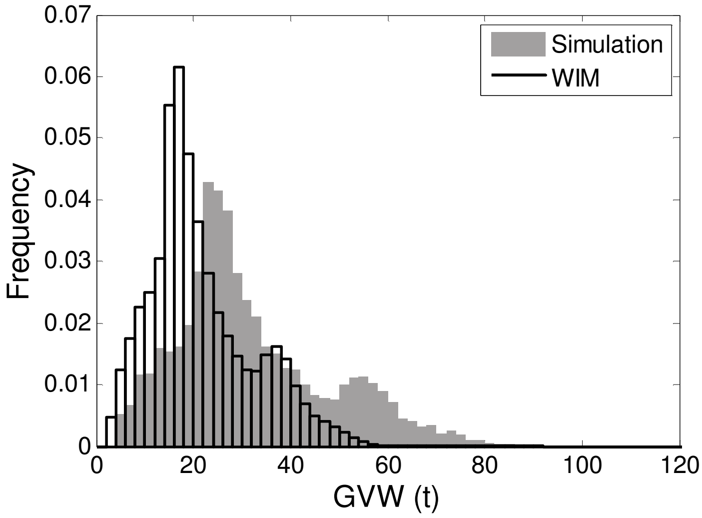


Figure 8. Gross vehicle weight histograms for the WIM data used and for the simulated traffic at the end of the 40-year simulation period with 1.0% yearly growth in weight.

Figure 9 shows a 40-year simulation for the maximum annual growth rate examined (annual increases of 2.0% in flow and 1.0% in weights). The simulated data shows the maximum daily load effects for each day of the 40-year simulation period. Although, at the start of the simulation period the generated traffic matches that of the measured WIM data, the final distribution of load effects is strongly influenced by year-on-year growth and is significantly greater than the WIM data. As a result of this annual growth, during the final year of simulation the truck weights are 1.49 times greater than the first year and the daily truck flow rate is 2.21 times the initial flow. When Figure 9 is compared with the equivalent simulation with zero growth, Figure 4, the effect of traffic growth is clear.

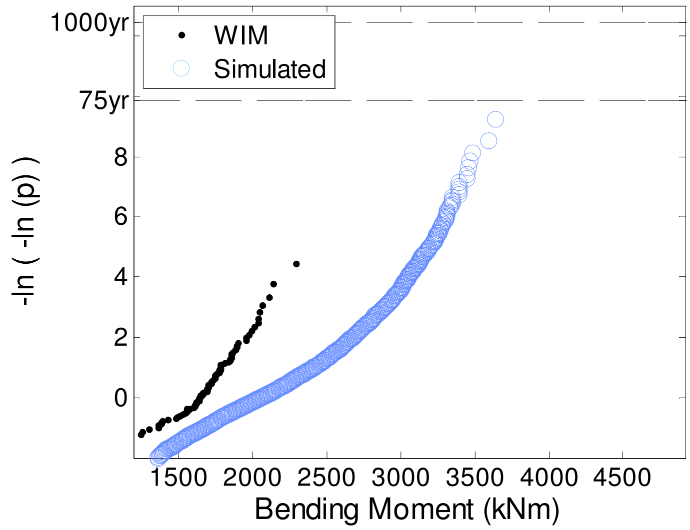


Figure 9. Maximum daily midspan bending moment on a 15 m bridge for a 40 year simulation of traffic at the Netherlands site with annual growth of 1% in weight and 2% in flow.

2.3.3 Influence line analysis

The simulated traffic is ‘passed over’ different influence lines to examine the effect of traffic growth on different bridge types. If a specific bridge was being assessed, then bridge specific influence lines would be used. The following three load effects for bridge lengths of 15, 30 and 45 m are examined here:

- LE1: Midspan bending moment on a simply supported bridge.
- LE2: Shear at the support of a simply supported bridge.
- LE3: Hogging bending moment over the central support of a two span continuous bridge.

It should be noted that for LE3, a 15 m two-span (2 × 7.5 m) continuous bridge is not a common configuration so this case was only considered for 30 and 45 m lengths.

When calculating load effects for two-lane same-direction traffic, each lane is analysed using a simple influence line. The transverse stiffness of the bridge is allowed for by using lane factors (Enright & OBrien, 2013). The primary lane contributes all of its calculated load effect whereas the contribution of the secondary lane is multiplied by a lane factor. The lane factors used are shown in Table 2 and are those which were found by Enright & OBrien (2013) to represent transversely stiff bridges where there is relatively large transverse distribution of load. If a specific bridge is being assessed, the transverse distribution of load for the relevant bridge elements could be assessed, e.g. using a finite element model or onsite measurements to obtain a more accurate lane factor.

Table 2. Lane Factors for overtaking lane with high transverse stiffness.

Load Effect	Lane Factor
LE1: Mid-span bending moment, simply supported	1.0
LE2: Support shear, simply supported	0.45
LE3: Central support hogging moment, 2-span continuous	1.0

2.3.4 Non-stationary GEV method for estimating characteristic load effects

Generalised extreme value (GEV) distributions are often used to extrapolate from the load effects due to measured WIM data to characteristic extremes, e.g. 75 or 1000 year values. This approach assumes that traffic conditions at the site will remain at the same condition as was observed during the measuring period. This does not allow for the fact that traffic flow and vehicle weights may increase with time. To account for this, OBrien et al. (2014) proposed the 'non-stationary' GEV approach for estimating characteristic load effects. With this method, the parameters of the distribution vary with time according to linear equations. This allows the distribution to change with traffic growth. An example of a time varying GEV distribution is shown in Figure 10.

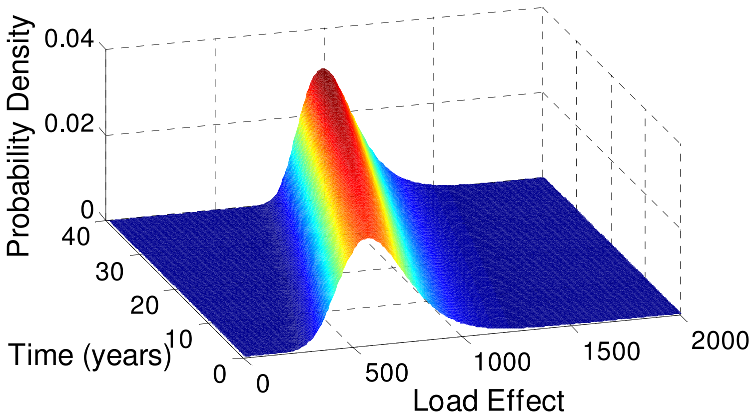
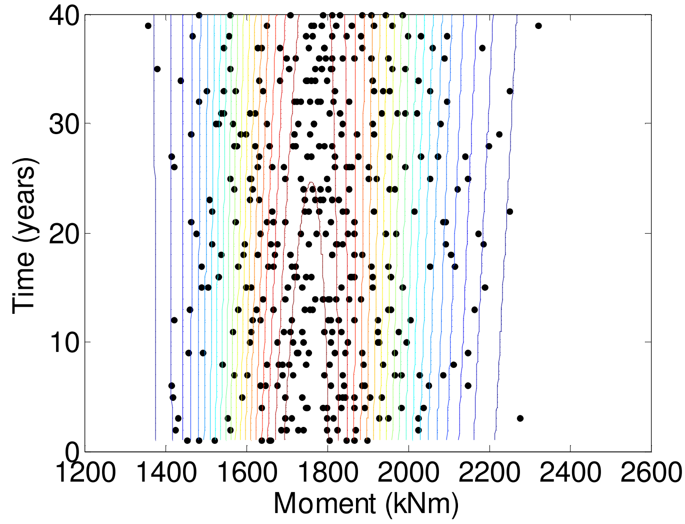
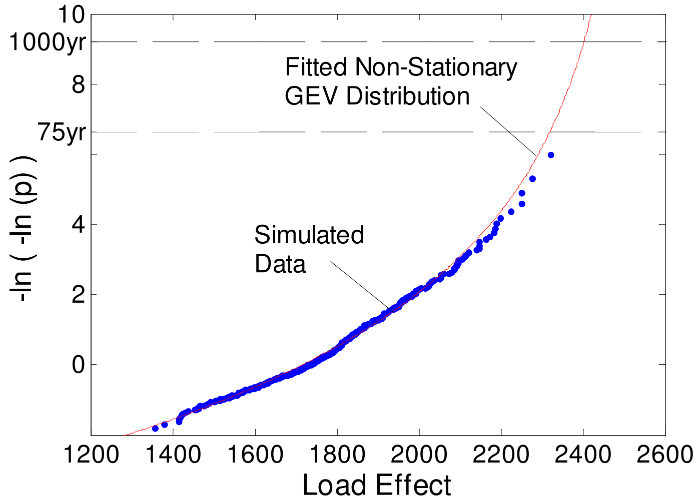


Figure 10. Non-stationary GEV probability distribution (probability density function).

To calculate the characteristic maximum load effects for a bridge at the WIM site, a 40-year traffic simulation is performed for each annual growth rate for weight and flow. 40 years represents the remaining service life being assessed. The non-stationary GEV distribution is then fitted to the maximum-per-25-day load effects using maximum likelihood estimation (25 working days represents one tenth of a 250 working-day year). The 1000-year return period load effect can then be calculated, i.e., the level of load effect that would be expected to be exceeded just once in 1000 years. It is important to differentiate between the service life and return period. Service life is the period over which the traffic growth is occurring while return period, while expressed in units of time, is a level of safety. Figure 11(a) shows the simulated maximum-per-25-day load effects increasing with service life time and a contour plot of the fitted non-stationary GEV distribution. Figure 11(b) shows a Gumbel probability paper plot of the same data and fitted distribution. This plot is effectively a cumulative distribution function (CDF) with the y-axis values plotted on a double log scale to allow the tail of the distribution to be easily examined. It shows that the fitted distribution is a good fit to the measured data and is effective in extrapolating the trend in the data to the 1000-year return period value.



(a) Simulated load effects with contour plot of fitted probability distribution.



(b) Probability paper plot of the same data and distribution as (a).

Figure 11. Non-stationary GEV distribution fitted to maximum-per-25-day load effects for a hogging moment on a 30 m bridge for a 40 year simulation with 1% flow and 1% weight growth.

A 1000-year return period was used in the calibration of the Eurocode LM1. This is predominantly used for the design of new bridges. For the assessment of existing bridges, the remaining service life is generally less than the design life and it may be more appropriate use a shorter return period, i.e., a lower level of safety. The 75-year return period is shown on some of the plots as an example of a lesser return period. A return period of 75 years is used in bridge design in the United States (AASHTO 2012) but an even lesser return period can be used for assessment (Minervino et al. 2003). An appropriate return period should be selected considering the remaining service life required.

2.3.5 Determining α-factors

The simulated 1000-year load effects are compared with those given by the basic Eurocode Load Model 1 (EC1. 2003). The α-factor and is defined as Equation 2:

$$\alpha - factor = \frac{Simulated\ 1000yr\ load\ effect}{Basic\ LM1\ load\ effect} \quad (2)$$

2.4 Results of simulations with traffic growth

Table 3 shows the α-factor for the simulations with no traffic growth. This represents the traffic that was measured at the WIM site.

Table 3. Load effect ratios with no traffic growth (see Table 2 for explanation of IL1, etc.).

Bridge Length (m)	IL 1	IL 2	IL 3
15	0.58	0.66	n/a
30	0.61	0.67	0.60
45	0.57	0.60	0.54

Table 4 shows the increase in the α-factor for the simulation with maximum annual growth (2.0% annual flow growth and 1.0% annual weight growth). This produced an average increase in α-factor of 48% with a maximum increase of 64% occurring for hogging moment on a 45 m bridge. Hogging moment (IL3) appears to be the most sensitive of the load effects to traffic growth.

Table 4. Percentage increases in α-factor with 2% flow + 1% weight annual traffic growth.

Bridge Length (m)	IL 1	IL 2	IL 3
15	36%	45%	n/a
30	44%	49%	52%
45	46%	49%	64%

In general, the percentage change in α-factor increases as the bridge length increases. From examination of some of the critical loading events, this appears to be due to multiple truck presence events being more prevalent on longer spans. Increases in flow will have more influence on these events than on the single truck loading events which tend to govern for short bridge lengths.

Shear at the support (IL2), varies less than the other load effects with bridge length. This appears to be due to the transverse distribution of load on the bridge. IL2 has a lower lane factor as only a smaller proportion of the load in one lane causes shear at the support of the adjacent lane. The critical events in this case are more likely caused by a single truck loading event rather than side-by-side events. As a result, shear will increase in proportion to the growth in truck weights and is less influenced by side-by-side events which become more frequent as bridge length increases.

Table 5 shows the average increase in α-factor across all influence lines examined for each combination of growth rates examined. It can be seen that annual growth in the weight of trucks has a much more significant effect on α-factors than growth in flow. Nevertheless, the effect of growth in flow on α-factors is also significant.

Table 5. Average α-factor increase for each combination of growth.

		Annual Flow Growth		
		0%	1%	2%
Annual Weight Growth	0%	-	6%	9%
	0.5%	19%	27%	31%
	1%	43%	51%	48%

It should be noted that there is a certain degree of random variation in simulations such as those used here as they are based on random number generation. This is apparent in Table 5 where the result for an annual weight growth rate of 1% in flow (51%) is greater than for 2% growth in flow (48%).

3 Implications of Traffic Growth on Bridge Reliability Calculations

Decisions related to bridge design and assessment are based on uncertainty inherent in material properties, workmanship, element dimensions, traffic load, environmental conditions and the deterioration process. Hence, both resistance and load are time-dependent variables which must be considered in the service life prediction of deteriorating bridges. This section starts with an introduction on reliability analysis and subsequently presents guidelines for probabilistic modelling of resistance and load modelling, considering traffic growth.

3.1 Introduction to reliability analysis

Bridge reliability can be defined as a probability that a bridge can function over a specified period of time and under specified service conditions. Bridge response depends on many uncertain factors such as loads, boundary conditions, stiffness and material properties. The response (e.g., stress at a critical location) is considered satisfactory when the design/monitoring requirements imposed on the bridge behaviour are achieved with an acceptable degree of certainty. Each of these requirements can be termed a 'limit-state'. Bridge reliability analysis is concerned with the calculation and prediction of the probability of limit-state violations at any stage during a bridge's life. Generally, the limit-state indicates the margin of safety between the resistance and the load effect on the bridge. In a standard case, when the time factor is neglected in the uncertainty of the variables, the limit state can be written as Equation 3:

$$Z = G(R, S) = R - S \tag{3}$$

where Z in the limit-state margin, G is the limit state function, R is the random variable representing the resistance and S is the random variable representing the corresponding load effect. In this equation, $G(R, S) < 0$, denotes the failure region and $G(R, S) = 0$ and $G(R, S) > 0$ indicate the failure surface and safe region respectively.

Given the limit state function, the probability of failure, P_f, of the bridge can be stated in the following form, Equation 4 (Choi et al (2007) and Melchers (1999)):

$$P_f = P(R \leq S) \tag{4}$$

Assuming that R and S are random variables with density functions f_R and f_S and a joint density function of $f_{RS}(r, s)$, the probability of failure can be represented by Equation 5:

$$P_f = P(R - S \leq 0) = \int_D \int f_{RS}(r, s) drds \tag{5}$$

which is shown as the hatched failure domain, $D,$ in Figure 12.

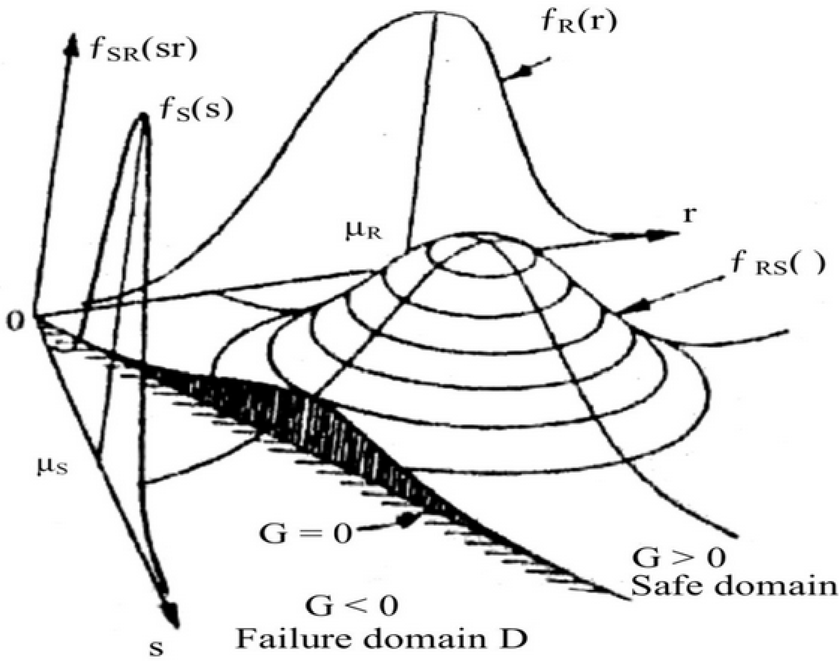


Figure 12. Two-random variable joint distribution function, $f_{RS}(r,s)$, marginal density functions f_R, f_S and failure domain, D (Melchers, 1999).

For a few distributions of R and S, it is possible to integrate this convolution integral analytically, notably when both resistance and load effect are Normally distributed random variables with means μ_R and μ_S and variances σ_R and σ_S respectively. In such a case, given well-known rules of addition/subtraction of normal random variables, the probability of failure can be written as Equation 6:

$$P_f = P(R \leq S) = \phi\left(\frac{0-\mu_z}{\sigma_z}\right) = \phi\left(\frac{-(\mu_R - \mu_S)}{\sqrt{({\sigma_S}^2 + {\sigma_R}^2)}}\right) = \phi(-\beta) \qquad (6)$$

where $\phi()$ is the standard normal distribution function and $\beta = \mu_z/\sigma_z$ is defined as the safety index. In general, the safety index approach is a mathematical optimisation problem for finding the point on the structural response surface (i.e., limit state approximation) that has the shortest distance from the origin to the surface. In other words, the reliability index is a measure of reliability which indicates the distance of the mean margin of safety from the failure surface $G(R,S) = 0$. It is an approximate indicator of the probability of non-compliance with the limit state.

In general, the limit state function is not linear. Therefore the first two moments of $G(x)$ are not readily obtainable. To address this issue, an approximation to linearize the failure surface, $G(x) = 0$ is commonly adopted. Different ways of approximating the limit state function form the basis of different reliability analysis algorithms (i.e., FORM, SORM, etc). From the original idea of second moment approximation, a number of extensions and refinements have grown.

In the FORM (first order reliability method) algorithms, the limit state function is represented by a first-order expansion of the Taylor series. These algorithms change the original complex probability problem into a simple problem and establish the relationship between reliability index and the basic parameters of random variables. This can be done by expanding the first-order Taylor series expansion about some point x^*.

Expansion about the mean value is common in probability theory; however there is no rationale for such an approximation in general and it leads to erroneous estimates of limit state functions with high nonlinearity and large coefficients of variation. A better choice is the point of maximum likelihood on the limit state function, the so called Most Probable failure Point (MPP). Expansion of the Taylor series at the MPP is called the Hasofer-Lind safety index method. In this method, all variables are transformed to the standardized normal distribution with zero mean and unit variance, $N(0,1)$ (i.e., U space, Figure 13).

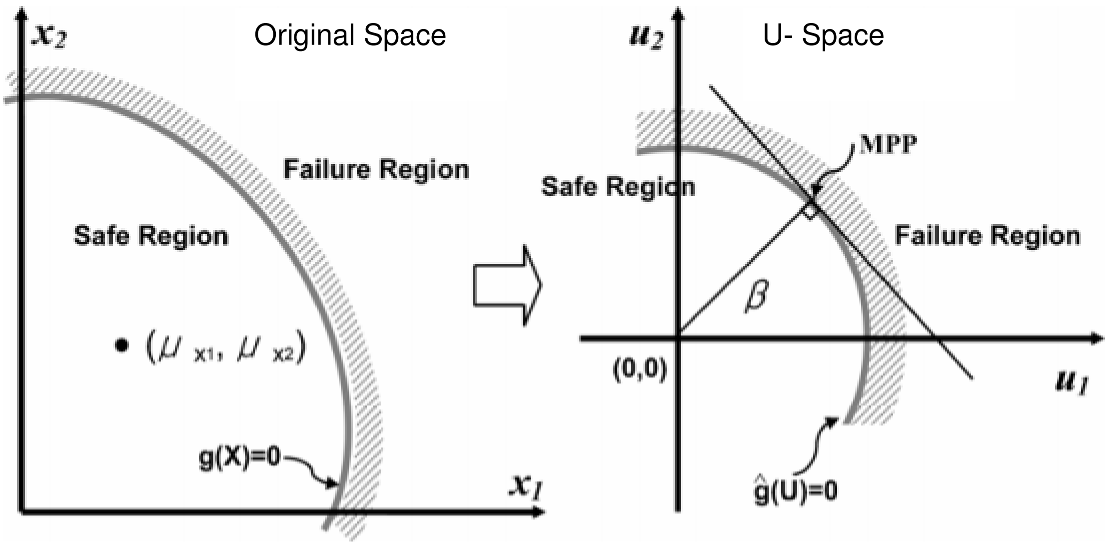


Figure 13. Transformation of failure surface from original space to U-space (Choi et.al, 2007).

In U-space, the joint probability density function is the standardized multivariate Normal. Thus well-known properties of this distribution can be applied and the reliability index can be calculated by Equation 7:

$$\beta(U) = (U^T U)^{1/2} = \|U\|_2 \, , U \in g(U) = 0 \tag{7}$$

The FORM approach usually works well when the limit-state surface is nearly linear in the neighbourhood of the MPP. However in the case of highly nonlinear failure surfaces, the FORM may give unreasonable and inaccurate estimates of the safety index. To resolve this problem, the second-order Taylor series expansion should be considered. Various nonlinear approximations have been proposed in the literature, but the common practice is to use asymptotic concepts. In this method, the failure probability can be estimated from a determination of the limit state surface curvatures, K_i, at the design point and then applying the asymptotic expression, Equation 8 (Choi et.al, 2007):

$$P_f \approx \phi(-\beta) \sum_{j=1}^{k} \left[\prod_{i=1}^{n-1} (1 - \beta . k_i) \right]^{-1/2} \tag{8}$$

where $k_i = -[\frac{\partial^2 y_n}{\partial y_i^2}]$ is the i^{th} principal curvature of the limit state surface at the design point.

Reliability analysis using either FORM or SORM methodologies requires probabilistic modelling of resistance and load. The following sections present the common practices used to simulate load and resistance for a bridge.

3.2 Resistance model

When performing structural safety assessment, it is essential to accurately model bridge resistance parameters. Bridge resistance is a function of time (e.g., deterioration), material properties, strength, stiffness and structure dimensions. This information can be found from the original codes/guidance documents to which the bridge was designed, in situ destructive or non-destructive tests performed on the bridge and quality control or steel mill certificates obtained at the construction stage.

Bridges can be categorised into three main groups from the material point of view;
1. Reinforced concrete (RC);
2. Prestressed concrete;
3. Structural steel.

For the first group, reinforced concrete, the resistance model should include the concrete compressive strength, concrete modulus of elasticity, the ultimate compressive strain of the concrete, information on shrinkage and creep, the variation in the reinforcing steel strength and cross section, the yield strain of the reinforcement and the effect of bar diameter on the properties of the bar.
Table 6 illustrates the distribution parameters for the compressive strength of concrete, as specified by the Danish Road Directorate (DRD) which is based on an extensive study performed on the Danish road network. The other material properties of concrete, such as tensile strength, modulus of elasticity and ultimate strain, can be determined from the compressive strength (OBrien et.al 2015). As can be seen from the table, generally the coefficient of variation of concrete compressive strength is higher for lower strength concrete.

Table 6. Distribution parameters for concrete compressive strength (DRD, 2004).

f_{ck} (N/mm^2)	μ_{fc} (N/mm^2)	COV_{fc}
5	6.76	0.22
10	12.8	0.18
15	18.9	0.17
20	24.8	0.16
25	30.6	0.15
30	36.2	0.14
35	41.7	0.13
40	47	0.12
45	52.8	0.12
50	58.7	0.12

The variation in the strength of the material, the variation in the cross section, the effect of rate of loading, the effect of bar diameter on properties of the bar and the effect of strain at which yield is defined should be considered in the probabilistic model of reinforcing steel strength (Mirza and MacGregor, 1979). By taking account of these parameters,, DRD (2004) recommends a lognormal distribution with the parameters shown in Table 7 for tensile yield strength.

Table 7. Tensile yield strength of non-prestressed reinforcement (*d* is diameter in mm) (DRD, 2004).

Type	Symbol	Diameter (mm)	Characteristic strength, f_{yk} (N/mm^2)	Mean value, μ_{fy} (N/mm^2)	COV_{fy}	Standard Deviation, σ_{fy}
Smooth bars	Fe 360	≤16	235	304	0.082	25
	Fe 360	≥16	225	293	0.085	25
	Fe 430	≤16	275	345	0.072	25
	Fe 430	≥16	265	334	0.075	25
	Fe 510	≤16	355	426	0.059	25
	Fe 510	≥16	345	416	0.06	25
Ribbed bars	Ks 410	-	410	482	0.052	25
	Ks 550	-	550	623	0.04	25
Cold Deformed bars	T	-	550	623	0.04	25

The area of reinforcing bars is recommended to be modelled using a lognormal distribution in which the mean and standard deviation are obtained as a function of bar characteristics (PIARC 1999). In this study the lognormal distribution is found to be a good fit as it models the skewness of the data. The effective depth in a reinforced concrete section is also a random variable. A normal distribution with a mean of nominal effective depth and COV varying from 5% to 20% is recommended for this variable (PIARC 1999). The modulus of elasticity and ultimate strain of the reinforcement is recommended to be deterministic values with no significant effect on the safety calculation. Table 8 illustrates the summary of statistical parameters recommended for reinforced concrete bridges.

Table 8. Statistical properties of reinforced concrete bridges.

Parameter	Distribution	Statistical Parameters
Concrete compressive strength	Lognormal/normal distribution	μ → Table 6 COV → Table 6
Shrinkage strain	Normal distribution	μ → Fib model Code 2010 COV → 0.35
Creep strain	Normal distribution	μ → Fib model Code 2010 COV → 0.20
Tensile yield strength of reinforcement	Lognormal distribution	μ → Table 7 COV → Table 7
Effective depth	Lognormal distribution	μ → nominal value COV → 0.05-0.2

For prestressed structures, documentation from the design or manufacture's documents is the main source for statistical properties. For this group of bridges, it is recommended that the strength of the prestressing steel should be modelled as a lognormal distribution with a low COV of 4%. The ultimate strain and the modulus of elasticity for prestressing steel can be modelled deterministically. To the authors' best knowledge, literature does not exist which proposes specific properties for concrete in this group of bridges; hence, the distributions proposed for RC bridges can be used in prestressed concrete bridges.

For steel bridges, is recommended that the yield strength of structural steel is modelled with a lognormal distribution with a mean value which is a function of steel grade and thickness, *t*, as shown in Table 9. DRD (2004) suggests a constant standard deviation of 25 N/mm^2 for all grades of steel. For the ultimate strength of structural steel, a lognormal distribution is recommended with a constant standard deviation of 25 N/mm^2 for all steel grades. The modulus of elasticity, shear modulus and Poisson's ratio can be taken as a deterministic value.

Table 9. Yield strength of structural steel (DRD, 2004).

Steel type	Characteristic value, f_{yk} (N/mm^2)			Mean value (N/mm^2)			Standard deviation (N/mm^2)
	t≤16	*16<t≤40*	*40<t*	*t≤16*	*16<t≤40*	*40<t*	*All t*
St 37	235	225	215	304	293	283	25
Fe 360	235	225	215	304	293	283	25
St 42 A	260	250	240	328	319	308	25
St 42, -1, -2, -3	260	250	240	328	319	308	25
St44	275	265	255	344	334	324	25
St 42, B, C, D	275	265	255	344	334	324	25
Fe 430	275	265	255	344	334	324	25
St 50	340	330	320	410	400	390	25
St 52	340	330	320	410	400	390	25
Fe 510	355	345	335	426	416	405	25

The resistance model also requires consideration of the model uncertainty. This takes account of the accuracy of the calculation, possible deviations from the material properties and the degree of control on site (i.e., material identity) (OBrien et al 2015). A measurable indicator of model uncertainty is represented by a judgement factor, I_m, which is modelled as a lognormally distributed variable with a mean to 1.0 and CoV, V_{I_m}, calculated as Equation 9 (NKB 1978):

$$V_{I_m} = \sqrt{V_{I_1} + V_{I_2} + V_3 + 2(\rho_1 V_{I_1} + \rho_2 V_{I_2} + \rho_3 V_{I_3}).V_M} \quad (9)$$

where the CoV, V_{I_i} and the coefficient ρ_i $(i = 1,2,3)$, respectively, are related to the accuracy of the calculation model (i.e., V_{I_1}, ρ_1), the material property deviations (i.e., V_{I_2}, ρ_2), the material identity (i.e., V_{I_3}, ρ_3) and V_M is the CoV of the basic material variable, Table (NKB 1978).

Table 10. Model Uncertainty Factors' Parameters.

Accuracy of calculation model			
	Good	Normal	Poor
V_{I1}	0.04	0.06	0.09
ρ_1	-0.3	0.01	0.3
Material Property deviations			
	small	Medium	Large
V_{I2}	0.04	0.06	0.09
ρ_2	-0.3	0.01	0.3
Material density			
	Good	Normal	Poor
V_{I3}	0.04	0.06	0.09
ρ_3	-0.3	0.01	0.3

The cross sectional area of the reinforcement in an RC member is reduced as corrosion progresses, which leads to a reduction in the load carrying capacity of individual segments. Stochastic modelling of this process can be incorporated directly into the probabilistic analysis to study the variation of safety index as a function of time due to deterioration. The current study aims to provide a guideline on the effect of traffic growth on the reliability index only; temporal and spatial variability of the level of deterioration in the bridge is considered beyond the scope of the study. Information on temporal and spatial variable resistance can be found in the work done by Ryan & O'Connor (2013), O'Connor & Kenshel (2012) and Kenshel & O'Connor (2009).

3.3 Load effect model considering traffic growth

The major load components of highway bridges are (1) Dead load, (2) Static and dynamic imposed traffic load, (3) Environmental loads due to temperature, wind and earthquake and (4) Other loads such as collision, emergency braking, etc.

3.3.1 Dead load

Dead load, in general terms, can be divided into the gravity load due to the self weight of the structural and the non structural elements permanently connected to the bridge structure, and the dead load that may be subjected to greater change such as asphalt and ballast. In a probabilistic safety assessment, the first group can be modelled by a normal distribution with a CoV of 5% and the second group with a CoV of 10% to reflect the higher level of uncertainty. In addition to these variations, model uncertainty is also required to be considered due to uncertainties in the load and load effect calculation (OBrien et al 2015).

3.3.2 Imposed Traffic load

The most variable load effects in reliability analysis of bridges are those caused by traffic loads. Traditionally, deterministic loading models derived from practical experience or calibration studies have been used to provide traffic load. This can result in considerable conservatism in the notional loading and unnecessary repair/replacement of safe structures in some instances. To address this issue, the use of site-specific traffic data has been proposed by assessment practitioners and researchers.

Traffic load has two components: static and dynamic. The static component is the self weight of the truck, *W*, and the dynamic component can be represented by an equivalent static load defined by dynamic amplification factor. The dynamic component is a function of road surface roughness, bridge dynamic properties and vehicle speed, configuration and dynamic properties. The current study is only concerned with the static component of traffic load and the dynamic component is considered to be beyond the scope of this study given that the effect of dynamic amplification factors is proven to be small (more information on dynamic amplification can be found in the work done by Gonzalez et. al, 2008 and Ludescher & Bruhwiler, 2009).

The static component of traffic load on a bridge is a function of many parameters, such as truck weight, axle loads, axle configuration, span length, longitudinal and transverse position of vehicle, number of vehicles and bridge geometry. Previous studies have shown that traffic load is highly site-specific; therefore for accurate modelling there is a need for site specific traffic data using weigh-in-motion (WIM) studies. However, it should be noted that this section presents guidelines for performing a reliability analysis which accounts for traffic growth when WIM data may or may not be available.

3.3.2.1 With WIM data

OBrien et al (2014) proposed a methodology using WIM data which is described in Section 2. Growth is addressed by assuming constant linear or quadratic variations in the properties of the best-fit generalized extreme value (GEV) distribution. The load effect distributions derived from this methodology then will be used as input in reliability analysis as S and f_s terms in the limit state function. For example, Figure 11 gives a probability paper plot for a load effect which allows for the effects of traffic growth. A probability paper plot is simply a rescaled plot of the Cumulative Distribution Function (CDF) of load effect and the CDF, with its derivative, probability density function, can be used directly in the reliability calculation.

3.3.2.2 Without WIM data

In the absence of WIM data, Vu and Stewart (2000) suggest a traffic load model which considers increases in weights and volumes. Since this model has been used extensively by researchers, it is suggested as an approach for traffic load modelling in the absence of WIM data.

The main assumption in this model is that the critical load effect generally occurs when heavily loaded trucks are side-by-side and have fully correlated weight. In this model, truck weight is normally distributed with the statistical parameters of μ_w and σ_w, the mean and standard deviation of single truck gross weight respectively. According to the proposed model, the time variant cumulative distribution of the weight of the heaviest truck w (annually) is shown by Equation 10:

$$F_n(w,t) = [\Phi(\frac{w - \mu_w \times (1+\lambda_m)^t}{\sigma_w \times (1+\lambda_m)^t})]^{N\times(1+\lambda_v)^t} \tag{10}$$

in which λ_m and λ_v are constants which are a function of traffic growth. The number of crossings of heavily loaded fully correlated trucks per year is represented by N and t is time in years. In general, the statistical parameters of truck weight should be estimated from traffic information for the bridge under consideration. In the absence of such information, based on the survey data for truck weight reported by Laman and Nowak (1995) are 250 and 0.4 for μ_w and σ_w, respectively. Load distribution between truck axle weights and axle spacings are adopted for the standard HS20 truck (AASHTO 2012) shown in Figure 14.

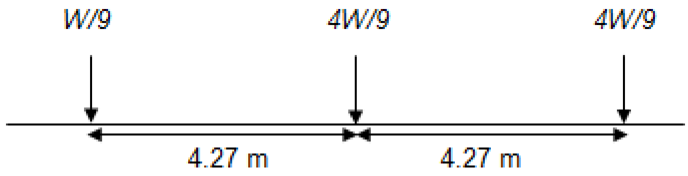


Figure 14. Truck Load Distribution and Axle Spacing.

Using a similar concept, the parameters are revised for the Eurocode LM1 by calibrating the model against 50-year characteristic values obtained from traffic simulations described in Section 2. Table 11 presents the corresponding statistical parameters of the tandem and the UDL in LM1 for two lanes of simulated traffic using the Netherlands WIM data and an annual growth rate of 1% for flow growth and 1% for weight growth over a remaining service life of 40 years.

Table 11. Statistical parameters for LM1 based on Netherlands WIM data.

	μ_w	σ_w	λ_m	λ_v	N
Tandem-Slow lane	260	29	0.00	0.02	1000
Tandem-Fast lane	173	19	0.00	0.02	1000
UDL-Slow lane (kN/m^2)	5.6	1.12	0.00	0.02	1000
UDL-Fast lane(kN/m^2)	1.2	0.25	0.00	0.02	1000

This model is considered to be more appropriate for the purpose of this CEDR project.

4 Implications of Traffic Growth for Older Structures

4.1 Introduction

As explained in the preceding sections, traffic growth implies a higher demand in structural capacity of the bridges. But for existing structures, the structural health of the infrastructure elements and the remaining structural capacity is unknown. Therefore, in addition to the uncertainty associated with the amount of growth of traffic and the extent of climate change, an additional uncertainty is added: the structural health of the existing infrastructure, and in particular the remaining structural capacity.

The target reliability, as described in Section 2, has to be adapted, in particular for the cases where the infrastructure is known to be vulnerable because of its age. Section 4.2 shows the importance of taking into account the age of the infrastructure by giving the histograms of ages of European bridges. Three examples of existing structures whose resistance and thus the target reliability should be assessed carefully are given below:

- arch bridges which are known to be quite old and whose preserving is important as they are numerous and they are part of European cultural heritage;
- prestressed bridges, for which a special framework of risk analysis has been developed in France;
- steel bridges with orthotropic decks which are known to be sensitive to traffic loads if their design is old.

These three types of structures have been analysed because as they have been recognized as vulnerable and various best practice methodologies have been used to deal with them.

- Section 4.3 gives some recommendations for the most demanding cases, of those listed above.

The first recommendation, Section 4.3.1 is to determine the precise traffic loading. This can be done using Weigh-In-Motion (WIM) data, as described in Section 2 and in Re-Gen Deliverable D3.1.

Another recommendation is to assess the residual resistance of the bridge. This can be done through various destructive and non-destructive techniques. Examples of destructive techniques include taking samples and undertaking tests for Young modulus, Poisson's coefficient, creep behaviour, thermal coefficient, etc. The non-destructive techniques include image-based techniques (radar, X-rays, etc) and structural health monitoring through recording of information on the structure (displacements, accelerations, stresses, etc). An extensive state-of-the-art can be found in the PIARC reports, for example (PIARC 2012). A technique which is often used when dealing with traffic loads, and especially abnormal traffic loads, are load tests. This last technique is explained and reviewed in Section 4.3.2.

Section 4.3.3, "Different levels of assessment", gives further advice which is to adapt the response to the issue: indeed, first one seeks to obtain some answers for these demanding cases with simple techniques. If no conclusive answer is given, the techniques are refined until obtaining the answer on the aggressiveness of traffic on this demanding case.

Finally, if refining the knowledge on the load (traffic, Section 4.3.1) and on the structure (Sections 4.3.2 and 4.3.3) does not make it possible to obtain an answer, one can strengthen the structure through various techniques (composites, additional prestressing, additional elements).

4.2 Identifying critical situations

Older bridges that have not been designed to support traffic according to Eurocode 1 (2009) comprise the majority of existing structures. They have been designed to various national design codes and their design capacity does not comply with traffic loading requirements prescribed in modern standards and codes. Moreover, many of these bridges are deteriorated, which further reduces their capacity to withstand the modern traffic. Therefore, keeping these bridges in function and avoiding unnecessary interventions that would result from applying today's design codes for assessment, requires using specific safety assessment procedures that account for realistic evaluation of the remaining capacity and traffic loading.

European bridges are old and have to great extent exceeded their design lifetime. Information about age of bridges was provided by the owners of the state roads from Slovenia, France, Ireland and Germany and is shown in Figure 15. In all countries information about the age of bridges not managed by the state is not available. It should also be noted that this data should be taken with some reserve. For example, in Ireland, just over half the bridges in the database have no age information and the available data is unrealistically biased towards newer bridges. However, in general, the data shows that old bridges represent a significant part of all bridges.

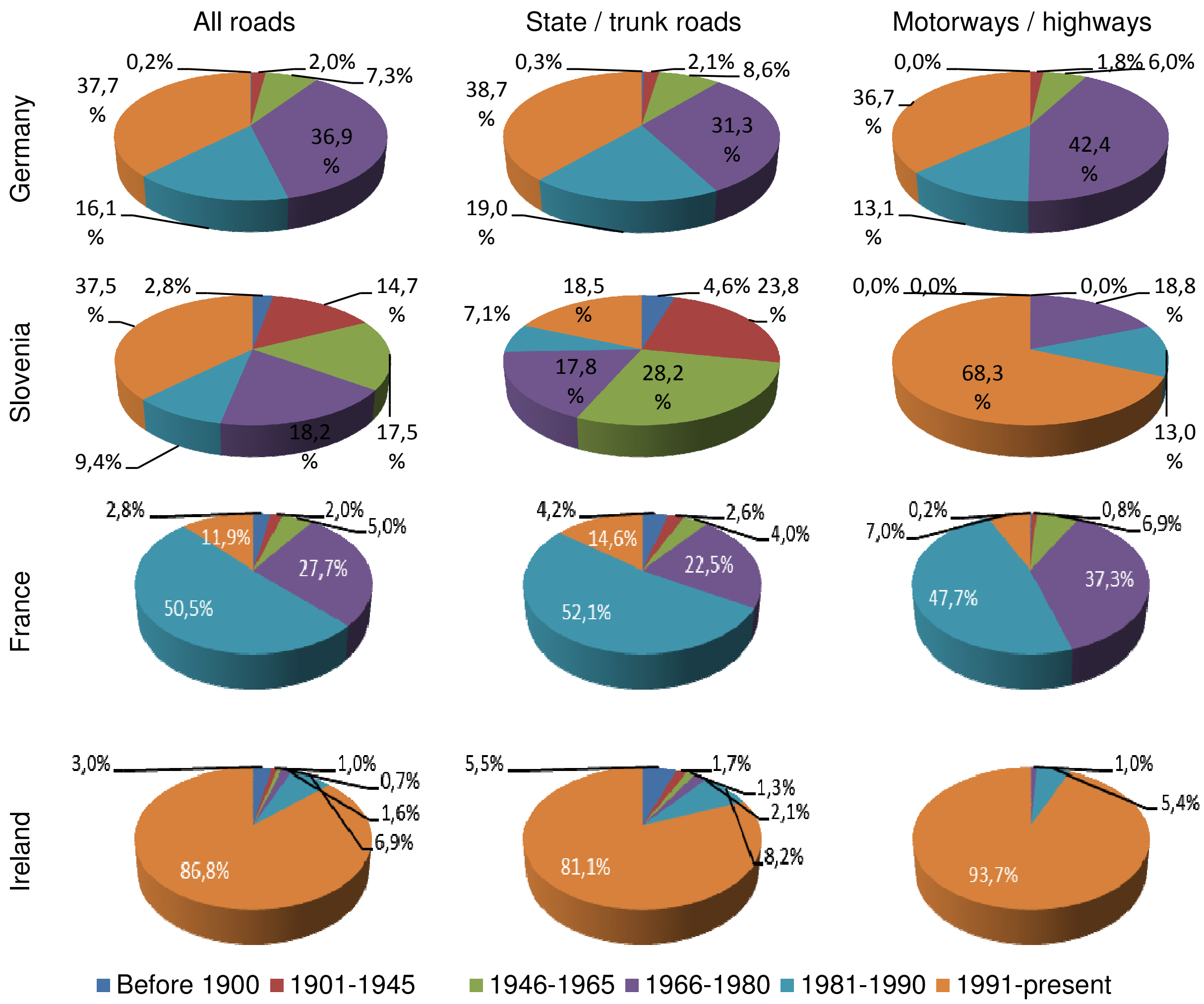


Figure 15. Distribution of age of bridges in European countries.

Old bridges may, and usually do, behave in an entirely different, often more favourable, way than expected. This is often the main reason that bridges which, according to the basic design theory, should already have been damaged can carry traffic without any visible effect. The most common reasons are:

- bearings that do not exist or do not behave in accordance with their theoretical behaviour,
- restricted movements of the expansion joints,
- soil pressure on the abutments,
- lack of knowledge about the design details.

For example, Figure 16 shows a 9.2 m long slab bridge constructed in 1980 and designed as a simply supported bridge, but without physical bearings. Monitoring of the true bridge behaviour under loading reveals that the bridge is not simply supported. Consequently, the actual bending moment at the middle of the span amounts to less than 50% of the theoretical moment.

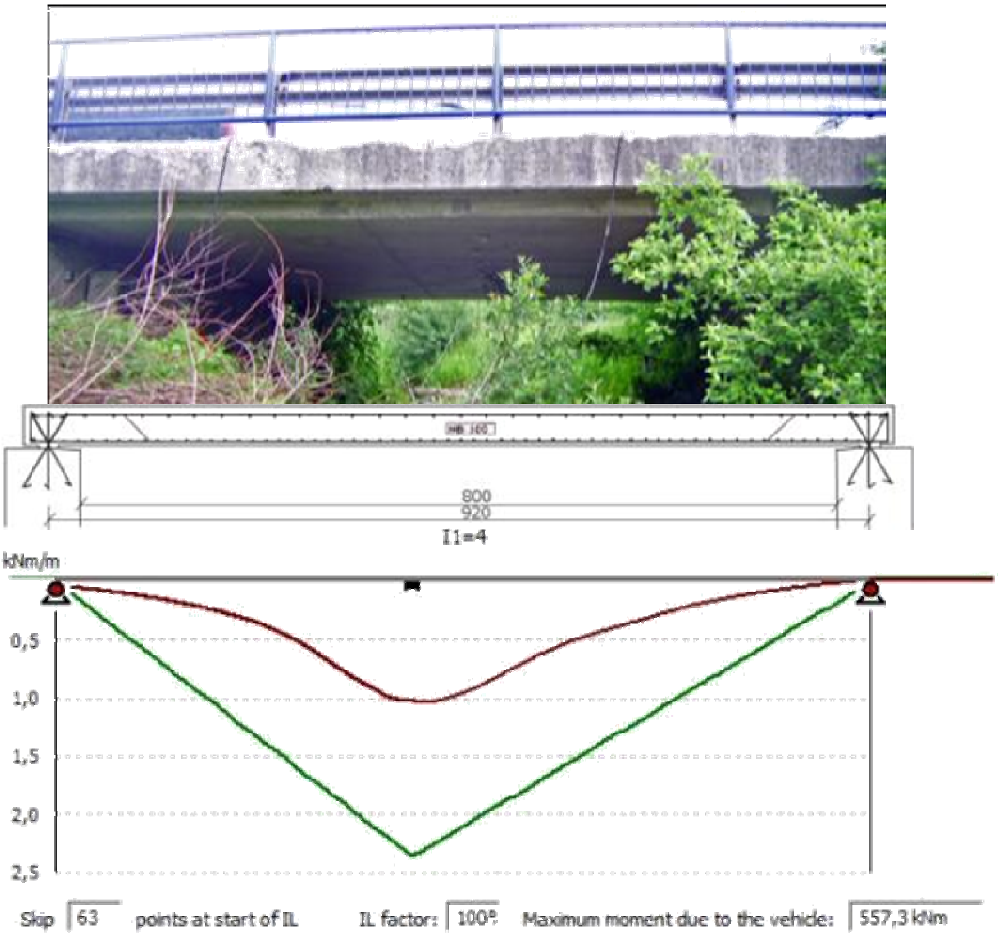


Figure 16. Above: Slab bridge built in 1980. Below: Theoretic influence line (green) and realistic influence line.

4.2.1 Arch bridges

Arch bridges have been a very popular type of bridge throughout history. In the past, most of them were built from brick and stone, Figure 17 (a), whereas today advanced materials enable different modern design of arch bridges, Figure 17 (b). The investigation within the SAMARIS project (2006), where data about the types of bridges in 5 European countries was collected, showed that arch bridges are widespread in the European bridge stock.

Figure 17. (a) Solkan bridge (1913, Slovenia), (b) Chateaubriand bridge (1991, France).

The experience show that masonry (brick and stone) arch bridges are durable and in principle do not need as much maintenance as other types of bridges. However, assessment of these bridges is also needed. They are strong by definition, but their safety may be questionable if:

- curvature of the arch is deformed,
- spandrel walls are not straight, they are detached from the arch or their function is compromised, Figure 18,
- larger stones/bricks have dropped from the arch or are poorly anchored (i.e. loose),
- there are visible signs of settlements, degradation of supports or erosion under them, and
- if the bridge carries extremely heavy or obviously overloaded vehicles.

Figure 18. An example of detached spandrel wall from the arch.

In addition to visual inspection, British standard (DMRB BA16/97 and BD21/01, 2001) provides assessment methods based on the MEXE method for masonry arch bridges. The result of this method is carrying capacity in terms of allowable axle weights. An example of the load capacity calculations for a masonry arch bridge can be found in Annex F of DMRB 34, BA 16/97 (2001).

4.2.2 Prestressed bridges

The beam and slab bridges that are named VIPP in France are viaducts with multiple single spans of prestressed concrete beams that are precast on site and post-tensioned, Figure 19. These structures have been recognized in France as demanding cases, and a whole risk analysis procedure has been elaborated (SETRA 2010, Godart 2015).

Figure 19. Merlebach bridge.

Indeed, a great number of beam and slab bridges were built in France after the end of the Second World War (about 250 between 1945 and 1957, and 450 between 1957 and 1967) for all kind of road networks but mainly national roads and highways. For example, Figure 20 gives the number of these VIPP bridges, on the national road network and built before and after 1975, as 1975 is considered the date where design, construction and maintenance of these bridges was improved. It can be seen that the proportion of VIPP bridges built before 1975 is higher than those after 1975, thus this issue cannot be discarded.

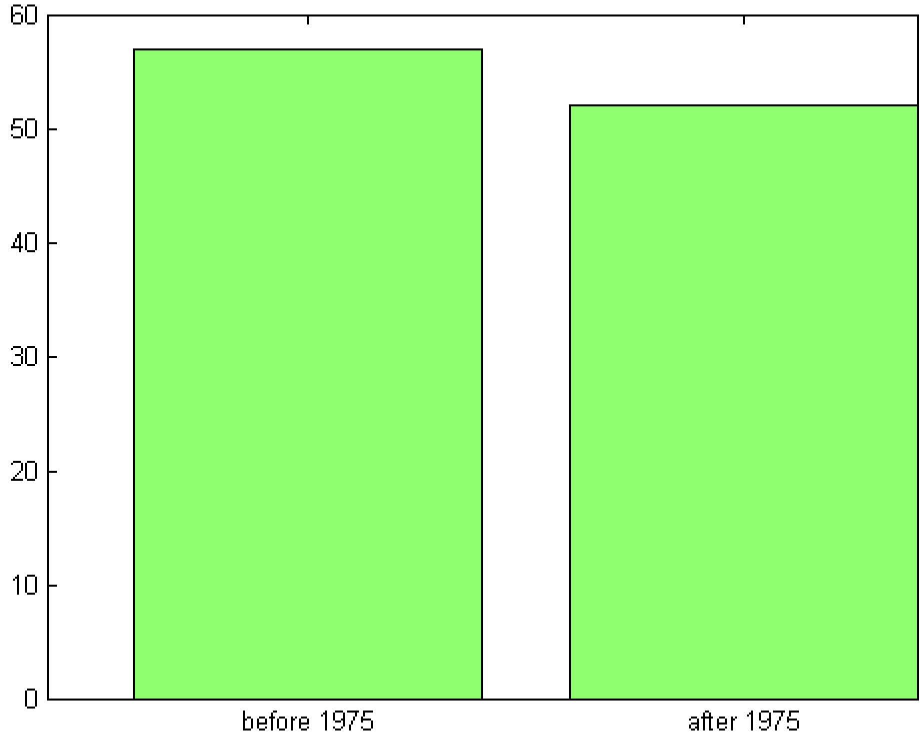


Figure 20. Histogram of VIPP bridges built on national roads before and after 1975.

Most of these structures (including the two longest: the Saint-Waast bridge at Valenciennes, with a span of 64 m, and the Hippodrome bridge in Lille, with a span of 66 m) were built after 1947 in the absence of formal design rules. It was not until 1953 that the first circular on prestressed concrete appeared, and not until 1965 that the Temporary Instruction No. 1 was published.

The enthusiasm which prevailed when the first generation of prestressed structures was being built was reflected in a total confidence in full prestressing (i.e. lack of cracking) and in a belief that concrete under compression was watertight. Consequently, details linked to poor design appeared such as:

- lack of waterproofing (deck waterproofing only became mandatory after 1966);
- lack of sealing behind tendon anchorages;
- lack of provision for drainage. It is often beams located under drainage channels which are most affected by corrosion
- leaking expansion joints;

Damage due to poor construction also appeared: Construction defects which can cause corrosion of the prestressing tendons are unfortunately encountered rather frequently. The two main defects leading to corrosion are poor waterproofing and incomplete grouting of the ducts. It is common to find thin waterproofing layers which do not extend beneath the footways. It is also common to find partly empty or sand-filled ducts in structures built before 1960 when grouts used to contain sand. Grouting techniques have also been poorly applied, and evidence of blockages has frequently been found. In addition to these defects, poor sealing of beam end anchorages, deck anchorages and transverse anchorages are observed.

Finally, defects linked to poor inspection and maintenance were also observed: During the 30 years which followed the end of the second world war, the absence of an inspection policy for structures and the failure to take the necessary action to make the structures watertight are the two main causes of maintenance problems.

Faced with this issue, the French authorities decided to apply a risk analysis to the VIPP bridge stock on national roads with the aim of prioritizing works on the whole stock. This risk analysis has combined threats and consequences in order to determine which of these structures are most at risk (SETRA 2010).

The threats have been classified in several categories: design, execution, maintenance and inspection, environment and general health of the structure. Each of these categories has been detailed in precise facts: for example, existence or absence of a slab, existence of transverse prestressing or only longitudinal prestressing, existence or absence of reinforcement at the ends of the beams, existence or absence of intermediate transverse stiffeners, etc.
These lists of practical facts have been derived by a working group of experts and make it possible, even for non-experts, to assess the vulnerability of the bridge. The extensive lists can be found in SETRA (2010).

The consequences take into account the importance of the bridge, which means the amount of traffic it has to carry each day, the surface of the deck, the consequences of a deviation, etc. As far as the traffic is concerned, the following rating was given to the volume of traffic, Table 12, which appears to be a main criteria for defining the consequence:

Table 12. Importance given to the traffic in the risk analysis underdone on VIPP bridges.

Average daily traffic (ADT)	ADT ≥ 80 K	60K ≤ADT< 80K	35K ≤ADT< 60K	15K ≤ADT< 35K	ADT <1 5K
Rating =	5	4	3	2	1

With this methodology, three classes of VIPP bridges have been determined: those with high consequences and high threats are declared as high risk. Those with low threats and low consequences are said to be low risk. The other ones (high threat and low consequences, or low threat and high consequences) are classified as medium risk. These three classes make it possible to prioritize the actions of inspection, maintenance, and repair.

Thus, VIPP bridges in France are an example of structures that have been recognized as being demanding, not because of traffic growth, but because of bad design, construction and maintenance. However, because of traffic growth, the consequences are higher and thus the risk is also higher.

4.2.3 Orthotropic decks

Steel bridges with orthotropic decks are common in Europe when long spans are required as they are lightweight. Orthotropic decks have been commonly used in Germany from the 1950s, while the first orthotropic deck might be the Jungingen bridge built in 1934. On the other side, France has approximately 40 steel bridges with orthotropic decks on the national road network, with two main examples built recently: the Millau viaduct, Figure 21 (also cited in Deliverable 3.1) and the moveable bridge of Rouen, Figure 22.

Figure 21. Cable-stayed Millau viaduct, whose deck is orthotropic.

Figure 22. Moveable bridge of Rouen, with orthotropic deck.

These decks are constituted by a steel decking with longitudinal stiffeners, Figure 23. Their design has evolved during the last decades as follows;

- the longitudinal stiffener can be continuous or discontinuous (and thus welded),
- the shape of the stiffeners: triangular (Haseltal bridge in Germany), glass-wine shape (Sinntal bridge in Germany),
- The existence of welding between longitudinal stiffener and transverse stiffener,
- Type of welding (penetration of the weld).

A review of these designs can for example be found in (Orthoplus 2009). The issues found on these structures are dependent on the type of design. Nevertheless, common damage are cracks at the weld between the longitudinal stiffener and the deck, Figure 23. These cracks are directly due to traffic loading, more precisely the fatigue phenomenon. This means that in this case, the issue is not the extreme traffic load but the repetition of loads considered as "normal".

Figure 23. Cracks at the weld between the steel deck and the longitudinal stiffener.

Since the 1970's, some rules for design have been published in order to avoid these issues;

- The steel deck should be at least 12 mm-thick,
- The distance between stiffeners should be 30 cm,
- Closed stiffeners should be preferred to open ones.

It should be noted that numerous existing bridges do not comply with these design rules, which means that the damage has to be treated. In the Netherlands, for example, campaigns for repairing the welds are undertaken each year (de Jong 2004). In France, the aim of the Orthoplus project (Orthoplus 2009) was to develop the use of Ultra-high Performance Fibre Concrete (UHPFC) as pavement on the deck, in order to compensate the design weaknesses.

4.3 Recommendations for the most demanding cases

When we consider traffic growth on one side and under-designed and deteriorated existing bridges on the other, then the importance of having an appropriate safety assessment procedure for existing bridges is clear. This procedure is based on the same principles as the design of new bridges but considers different assumptions and input information. The two key differences are:

- When dealing with existing structures, in most cases, a reasonable estimate of the capacity can be determined and the true traffic loading can be measured. As such, the reliability of the input information is much higher than is assumed in the design stage,
- Very often, bridge safety is considered over a shorter time period than the full life span of the bridge. For example, the owner is often interested only in the number of years the life of a bridge can be extended, instead of ensuring sufficient structural safety for the entire design lifetime.

Consequently, using design codes in bridge assessment is unreasonable. Currently we do not have an official bridge assessment standard in Europe, although the present code allows using alternative methods of evaluation as long as the level of structural safety is pursued. As a dedicated Eurocode for bridge assessment is under consideration for many years but still far from realisation, a number of countries have issued related national codes (DMRB BA16/97 and BD21/01, 2001), guidelines (Richtlinie zur Nachrechnung von Straßenbrücken im Bestand, 2011) or recommendations (Žnidarič A., 2010) that take these differences into account. In this section, some advice to find reserves in the resistance of old structures is described.

4.3.1 Realistic traffic loading

Using weigh-in-motion (WIM) data for assessment of old bridges has many positive effects. Primarily, WIM systems provide information about the real traffic situation at specific locations, enabling the current level of traffic loading to be compared to the design load. In other words, WIM systems provide realistic loading information that can be used to develop:

1. Special assessment loading schemes: These schemes are developed from large samples of WIM data and by considering a great number of possible structures (influence lines). As a result, they can be used for bridge assessment in a country or region where the WIM data was collected, even if WIM measurements on the road section where there is concern over a bridge were not performed. An example of development of assessment loading schemes for bridges using WIM data is given in Section 4.3.3 of Re-Gen Deliverable D3.1.
2. Site-specific loading schemes: in an ideal case the WIM data is collected on the road section with the concerned bridge, or even on the bridge itself. Then the most optimal traffic load model can be derived.

These loading schemes *are by definition less conservative* than the design loading schemes from today's standards (e.g. Eurocode 1(2009)), but can result in higher load effects than would be obtained from the loading schemes that were employed in the design stage. In the next stage, when structural safety is assessed, these load effects are compared to the actual bridge capacities, obtained from a detailed inspection of the bridge, to demonstrate whether or not the level of safety is sufficient.

It shall be emphasised, that with respect to collecting WIM data, pavement WIM systems and bridge WIM systems provide the same level and quality of information.

4.3.2 Load tests

Diagnostic load tests have been traditionally used to verify if the true bridge behaviour corresponds to the one foreseen in the design. These tests were compulsory in many European countries to check compliance of new bridges before they were put in service. Diagnostic load tests are typically performed in a way that 70-80% of the design load, usually in the form of fully loaded closely spaced heavy vehicles, is applied on the critical parts of the bridge, usually at mid-span and over the supports. Two examples of diagnostic load tests on a smaller bridge in 1960s and recently on a 1050 m long viaduct are shown in Figure 24.

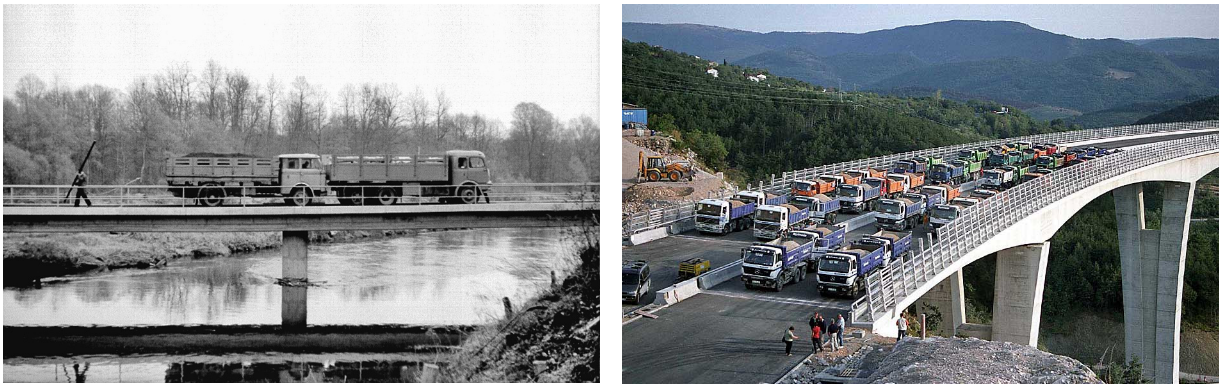


Figure 24. Examples of diagnostic load tests.

Performing load tests on old structures is more challenging as;

1. A traditional load test requires stopping the traffic for a period of hours, which is not always possible,
2. In some cases, due to their age and insufficient information, a reasonably accurate capacity may be difficult to assess, therefore the structure can be overloaded,
3. On old bridges the design load can be considerably below the current traffic load level.

Therefore, the old bridges should preferably be tested with a *proof load test*, where the loading is step-by-step increased to the level that ensures structural safety for the considered level of traffic loading. However, proof load tests on old bridges are extremely rare, primarily because overloading can damage the bridge, which is not acceptable. Today proof load tests are mainly used in studies where old bridges are being replaced and are loaded to collapse in a controlled manner before being removed, Figure 25 (a). The only known exception in Europe is in Germany where they have designed a special vehicle that can incrementally load smaller bridges, Figure 25, (b).

(a) (b)

Figure 25. Examples of proof load tests.

To overcome the limitations of proof load tests, but to still encourage the use of load tests to improve efficiency of safety assessment of old bridges, the SAMARIS (2006) and ARCHES projects (2009) suggested using a *soft load test* (SLT). SLT replaces the loading with pre-weighed heavy vehicles within the random traffic that traverses the bridge. This is considerably more time and cost-efficient. The users are not affected during measurements as the bridge is left opened for traffic and, most importantly, there is no danger to damage the bridge. Results are used to calibrate the structural model *at serviceability limit states*, which is a common situation when attempting to extend the lifetime of a bridge for a number of years. As it has been shown in the case study in Re-Gen Deliverable D3.1, a SLT can demonstrate that a bridge exhibits far lower load effects than those derived from a theoretical analytical model. Consequently, less severe rehabilitation measures can be selected (i.e. repair without strengthening) which results in more efficient spending of the limited bridge management budgets.

Further descriptions and recommendations on how to use diagnostic, proof and soft load testing can be found in the report D16 of the ARCHES project (2009).

4.3.3 Different levels of assessment

Further reserves for older structures may be found by using different levels of assessment analysis. The COST 345 report, for example, specifies five levels of assessment, from the simplest Level 1 to the most sophisticated Level 5. Such a step-by-step approach is common today to most existing bridge assessment methodologies.

The descriptions of the individual assessment levels below are directly summarised from COST 345:

Level 1 assessment
At Level 1, only simple analysis methods are necessary, and partial safety factors from the assessment standards are used. This is the simplest level of assessment, giving a conservative estimate of load capacity.

Level 2 assessment
Level 2 assessment involves the use of more refined analysis, for example grillage analysis, finite element analysis, non-linear or plastic analysis, and better structural idealisation. It also includes the determination of characteristic strengths for materials based on existing available data. No new tests would be carried out on the structures for a Level 2 assessment.

Level 3 assessment
Level 3 assessment may apply the structure-specific loading (see Section 4.3.1). For many bridges, particularly on lightly trafficked roads, the use of bridge-specific traffic loading can be quite beneficial. It also makes use of material testing to determine characteristic strength or yield stress and considers diagnostic load testing (see Section 4.3.2).

Level 4 assessment
Level 4 assessments can take account of any additional safety characteristics for that structure and amend the assessment criteria accordingly. Any changes to the criteria used in this level may be determined through rigorous reliability analysis, or by judgemental changes to the partial safety factors. In the deliberations involving Level 4 assessments, care should be taken not to double count structure-specific benefits which have already been taken into account. For instance, if system analysis based methods such as the yield line method have been used in Level 2 or 3 assessments, system effects should not be utilised in Level 4 assessments.

Level 5 assessment
Level 5 assessment involves reliability analysis of particular structures or types of structure. Such analysis requires statistical data for all the variables defined in the loading and resistance equations. The techniques for determining the probability of the failure from such

data are now available and can be undertaken relatively easily in modest time frames. It provides greater flexibility but the results are very sensitive to statistical parameters and the methods of structural analysis used. Consequently, it requires specialist knowledge and expertise.

4.3.4 Strengthening or repairing the structure to a well-known resistance

When determination of the structural health of the bridge is not feasible or too difficult (financially or technically), one possibility is to repair or strengthen the bridge directly. Three examples will be given here: using composites in order to increase the reinforcement in reinforced concrete, adding prestress in the case of prestressed structures or using additional structures.

For example, gluing composites can be done in order to increase the structural resistance to normal action through confinement: this would be the case for columns and piles. Another case where composites are used is the strengthening for bending moment: in this case, the composite is glued on the bottom flange of the longitudinal beams. Recently strengthening with composites has been done to increase resistance in the transverse direction for longitudinal prestressed beams, see Figure 26.

Figure 26. Bridge near Scorff (Lorient). Transverse strengthening of a bridge which is longitudinally prestressed.

For prestressed structures whose resistance is unknown (and especially if the remaining tension in the cables is unknown), additional prestressing can be used. In this case, the prestressing is calculated in order to compensate for a reduced capacity at a particular section of the structure, but without inducing unwanted effects (in the case of too high prestress). The additional prestressing is then visible, as for the example for the Saint Nazaire bridge (Figure 27).

Figure 27. Additional prestressing on the Saint Nazaire bridge (France) and Ravbarkomanda bridge (Slovenia).

Another example would be the use of additional structures, which is quite rare: this has been used recently in France in the case of abnormal loads (more than 300 t). A provisory bridge has been installed parallel to an existing bridge in order to make it possible for the vehicle to be supported by the bridge transversely.

5 Conclusions

This report examines the effect of increased traffic loading on ageing bridge infrastructure.

Accurate estimates of traffic loading are critical in order to evaluate the true safety of an ageing bridge. Section 2 examines a method for modelling traffic growth with time. As freight traffic is expected to grow significantly until at least 2030, traffic growth needs to be allowed for when modelling traffic loading on bridges. A scenario modelling traffic simulation approach is used to model traffic growth and a time-varying GEV distribution is fitted to the simulation results in order to determine characteristic load effects and the corresponding alpha-factors, which can be used to factor Eurocode LM1 for bridge assessment. As an example of the approach, the effect of traffic growth on bridge loading is modelled for a WIM site in the Netherlands. This example aims to give an indication of how traffic growth affects alpha factors. Different traffic growth rates are assumed over a 40-year service life and the effect on alpha factors is examined. The results show that considering growth has a significant effect on the alpha factors for all bridge lengths and load effects examined. It is shown that certain load effects are more sensitive to traffic growth than others and that the effect of traffic growth is generally more significant as the bridge length increases. Growth in weight has a much more significant effect on alpha factors than growth in flow. For the 40-year service examined for the Netherlands example, a 1% annual growth in flow causes an average increase in load effect of 6% in comparison with a 43% average increase for a 1% annual growth in truck weight. The results highlight the need to consider traffic growth when assessing site-specific traffic loading on an existing bridge.

Section 3 outlines a method for incorporating this traffic growth into a bridge reliability analysis which considers the uncertainty associated with both the load and the resistance when calculating the safety of a structure. It describes how a reliability analysis is performed and provides advice for how to approach the analysis where WIM data is and isn't available.

The issue of increased traffic loading on ageing infrastructure has already been recognized in Europe, for specific types of bridges. Section 4 gives examples of some of these structures, including very old structures (arch bridges), structures whose design was not ideal (VIPP bridges in France) and structures sensitive to the increasing frequency of heavy loads (steel bridges with orthotropic decks). To deal with these bridges, owners or managers try to assess precisely the remaining structural capacity of their structures (load tests) and reinforce them to insure a minimal capacity (additional prestress, composites).

The examples given in Section 4 highlight the necessity of introducing more and more risk analyses on different types of ageing bridges: for example, this has been done in France for VIPP (example in Section 4). Risk analysis makes it possible to prioritize the inspection, maintenance and reinforcement planning for bridge stock on a national or regional level.

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