THE EVALUATION OF SOLID BRIDGES ON BAVARIAN HIGHWAY ROUTES FOR 60 TON HEAVY GOODS TRUCKS

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ABSTRACT

Due to the fact that volumes of heavy traffic will increase until 2050, the European transport industry is asking for higher authorised weights or increased maximum authorised dimensions of trucks in national and international traffic.
In Germany 40-ton (metric) trucks are allowed and so-called “GIGA liners” with authorized weights of up to 60 tons (metric) are proposed.
Within this research project, the effects of these 60-ton (metric) trucks is being examined on nearly 3600 existing highway bridges with a bridge deck surface of about 770,000 m² (921,000 yd²).
Statements are made about the bearing capacities, remaining durability and the consequential costs. 36 % of bridge deck surfaces are in longitudinal direction single beam systems and 45 % are continuous beam systems, therefore the examinations are limited to these systems.
Load combinations according to the German standards for actions on bridges (DIN FB 101, DIN 1072) are compared with traffic situations of 60-ton (metric) trucks to identify critical systems and spans.

Keywords: bridge, load, heavy traffic

INTRODUCTION

A significant factor for economic growth, employment and thus the quality of life for citizens in view of globalisation is the transportation of goods and human mobility, in which an important part is played by the existing road network. Traffic forecasts [1],[2] show a strong rise in average daily heavy goods traffic (ADHGT) in Germany since the 1980s and are predicting a further increase by 2050.

Figure 1: Trend in traffic volume from [2] (left), Modular truck concept from [3] (right) (1 ton$_{(metric)}$ kilometre ≈ 0.69 ton miles)

Based on these forecasts and economic considerations, recent demands by the transport sector for the authorization of 60 ton (metric) trucks have led to the current investigations [3],[4],[5] with reference to the effects of the authorization of heavy trucks according to the modular truck concept [6].

These investigations show that the current load models of the present traffic situation are still legitimate but the increase of permitted laden weights has meant that reserves are to a large extent exhausted, or that computed design loads are exceeded under different constraints.

In accordance with the resolution [7] dated 9th/10th October 2007, the Ministerial Conference on Transport considered "the use of heavy goods trucks that exceed the current general permitted laden weights to be unjustifiable due to the significant increase in risk in traffic safety and the insufficient reserves of bearing capacity in bridge structures. The Ministerial Conference on Transport asks the Federal Government to include the results of this report, including the evaluations of the pilot projects carried out by the states, in discussions at European level in the sense of an EU-wide ruling."

The rejection of the 60 ton (metric) trucks by the Ministerial Conference on Transport can principally be traced back to aspects of the driving dynamics and traffic safety, and for this reason the authorization of higher permitted laden weights cannot conclusively be ruled out. In addition, due to the generally higher permitted laden weights in other EU countries, a harmonisation and thus an increase in permitted laden weights can also be expected in Germany. Disregarding the discussion on increasing the permitted laden weights, these are already frequently exceeded through overloading or through permitted abnormal loads. At the start of the 1980s, there were 5000 permitted abnormal loads registered on Federal network of trunk roads; the number had increased ten-fold by 2004. In addition, the abnormal loads, often permitted through multiple flat-rate assessments, caused complaints at checkpoints with regard to permitted weights.

Existing investigation reports (e.g. [3], [5]) only deal with the higher-level road network, mainly the Federal motorways, and in general only a few structural types are investigated. There is only limited direct transferability of the results to highways. The road traffic count of 2005 [8] also shows differences in traffic trends for different road categories. The average daily traffic volume (ADTV) on Bavarian motorways increased by 176 % in the period of

comparison from 1970 to 2005, whilst goods traffic increased by 172 % and passenger traffic increased by 195 %. In comparison, the ADTV on Bavarian state highways increased by 125 %. However, the average daily heavy goods traffic (ADHGT) only increased by 24 %, whilst at the same time passenger traffic volume increased by 142 %.
From these aspects it can be seen that the results of studies already presented cannot be transferred wholesale to highways. Rather, the majority of the bridge inventory should be examined in view of the expected effects.
The results presented in the following are essentially based on the research report “The evaluation of concrete bridges on Bavarian highway routes for 60 ton (metric) heavy goods trucks” [9]. The report was sponsored by the Supreme Building Authorities in the Bavarian State Ministry of the Interior.

RECORDING THE BRIDGE INVENTORY

The bridge inventory examined on Bavarian state highway routes includes 3532 individual structures with a total bridge area of 769,143 m² (≈ 1005940 yd²). For bridges, examination of the building structure was carried out according to DIN 1076 [10] “Highway Structures – Testing and Inspection”; in this, a main inspection is carried out at 6-yearly intervals and a “simple” inspection at 3-yearly intervals. Damage is recorded by the inspecting engineer and evaluated and/or entered in the “SIB structures” database in accordance with “The Recording and Assessment of Damages according to DIN 1076” (RI-EBW-PRÜF) [11]. Based on the database, for inspections the bridge inventory is classified according to

- Year of construction
- The maximum span
- Static systems
- Designed bridge class
- Traffic volume

amongst others. Evaluation criteria are drawn up using the classification for assessing the effects of a possible increase in load.

AGE PROFILE – DESIGN LOADS

The vast majority of bridges were constructed in the second half of the 20th century. This can primarily be traced back to the reconstruction of destroyed infrastructure after World War II and to the following strong economic growth. About half of the existing bridges were constructed before 1970.
In accordance with the age profile, the design loads meet the DIN 1072 standard series [12],[13],[14] and/or DIN Fachbericht (Technical Report) 101 [15]. Continuous amendments to load standards have been required with the development of transport trucks and the advent of heavy trucks. For the bridge inventory under examination, bridge classes 30, 45, 60 and 60/30 are of prime importance.

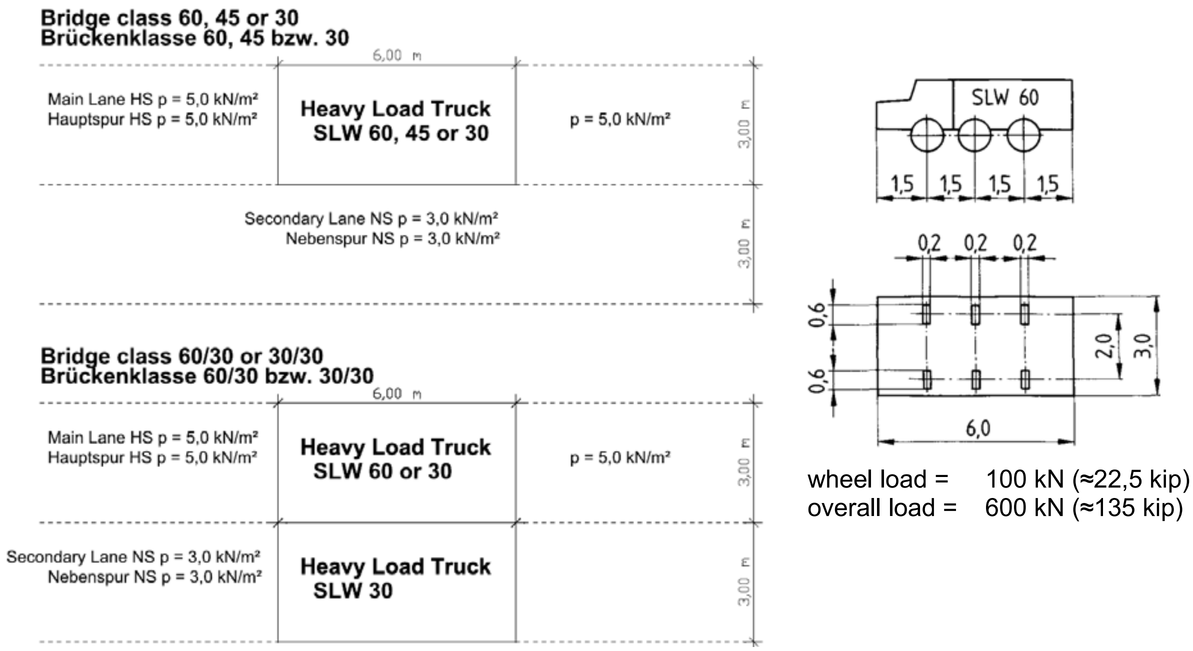


Figure 2: Schematic representation of German bridge classes, plan view
(1 kN/m² = 20.9 lb/ft²)

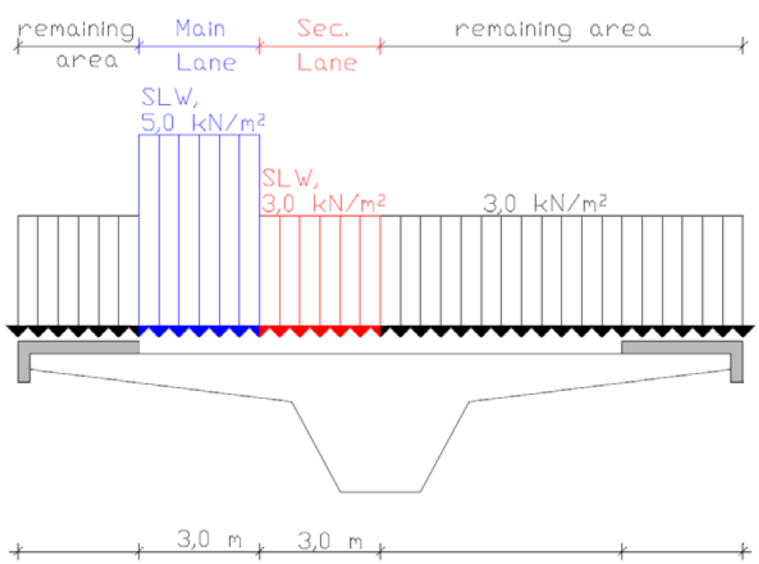


Figure 3: Schematic representation of German bridge classes, cross section

For all bridge classes the bridge deck is unfavourable divided in two lanes (main, secondary lane) in longitudinal direction with a broadness of 3 metres (3.28 ft) per lane and the so called remaining area.

For the main lane of bridge classes 60, 45 and 30, a distributed load of 5.0 kN/m² (104 lb/ft²) was arranged and, instead of the distributed load, an additional reference vehicle, the so-called heavy truck, was arranged in the unfavourable position with a load corresponding to the bridge class in tons (metric). Depending on the span width, the main lane loads were multiplied by a vibration coefficient. A distributed load of 3.0 kN/m² (63 lb/ft²) was arranged on the secondary lane and on the remaining bridge areas. A heavy load truck on the secondary lane was also taken into consideration on the introduction of bridge classes 60/30 and 30/30, as with the main lane.

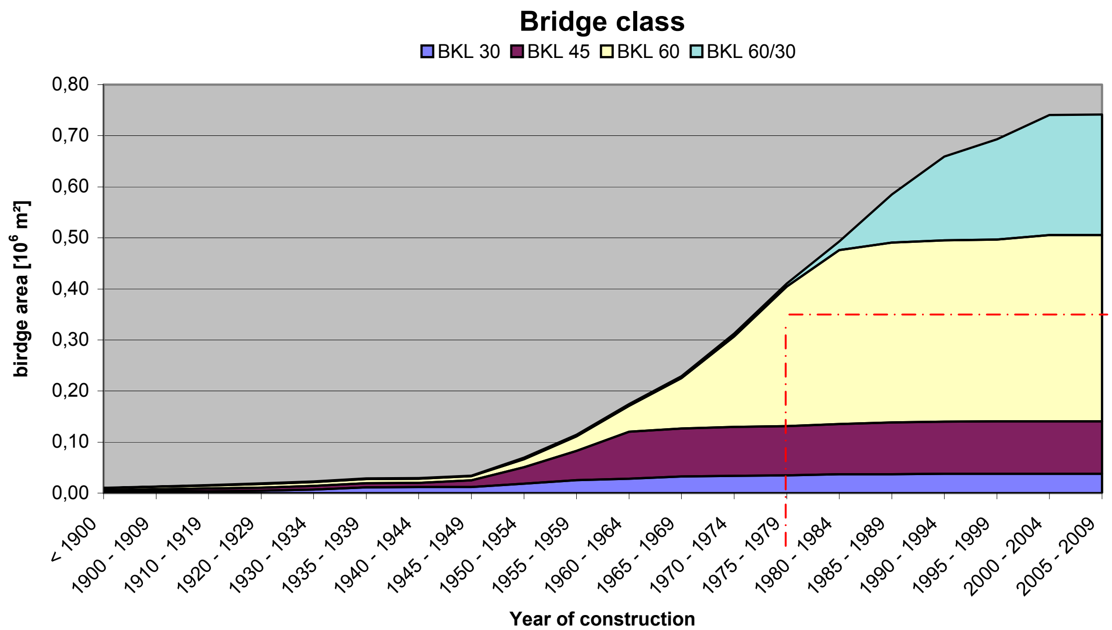


Figure 4: Trend in total bridge area (1 m² ≈ 10,76 ft²)

The distribution of bridge classes in the bridge inventory is shown in Figure 4.

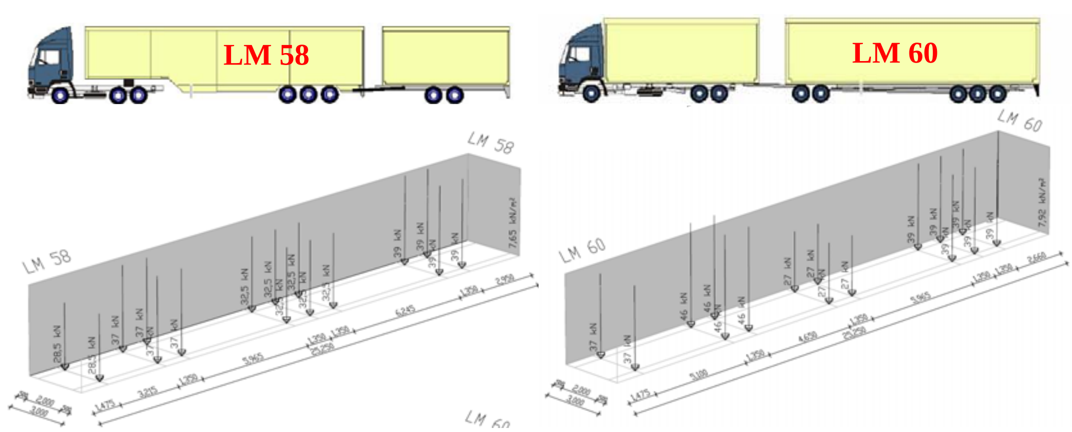


Figure 5: Possible truck-trailer combinations (LM 58, LM 60) (1 kN ≈ 0,22 kip)

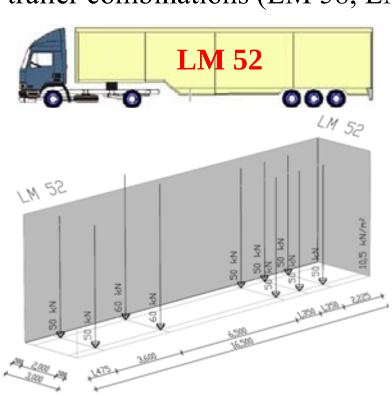


Figure 6: Possible truck-trailer combinations (LM 52) (1 kN ≈ 0,22 kip)

For the investigations, possible truck-trailer combinations as in Figure 5 and Figure 6 were compared with the normative design loads in accordance with Figure 3. The LM58 and LM60 load models were set up according to [5]. Load model LM52 corresponds to the heavy goods truck type most frequently used in Europe, where a permitted laden weight of 52 $t_{(metric)}$ instead of 40 $t_{(metric)}$ was achieved by increasing the permitted axle load.
For load models LM52, LM58 and LM60, three scenarios were assumed:

- Single truck
- Traffic jam situation, i.e. the truck load models stand bumper to bumper
- Flowing traffic (trucks in unfavourable distance, min. distance 30 m (≈ 98 ft))

For these scenarios, passing and overtaking procedures were also considered. If just the distributed loads in the load models are considered, the LM58K and LM60K combinations are each more favourable than a traffic jam situation of permitted 40 $t_{(metric)}$ heavy trucks. To be on the safe side, the probability of occurrence of the individual scenarios is not considered.

In 2005, traffic counts for about 60% of the bridge inventory under investigation were carried out, and the proportion of ADHGT was included. Using the results of the traffic counts, and assuming a certain traffic distribution, bridge loads can be estimated. To determine fatigue, there is a traffic distribution in the form of fatigue load model 4 in DIN EN 1992-2:05-2004 [16] (so-called EUROCODE).

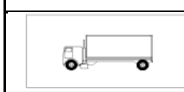
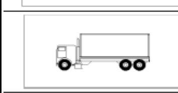
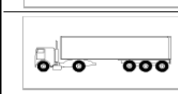
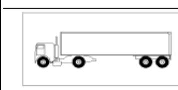
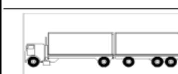
	VEHICLE TYPE			TRAFFIC TYPE			
	1	2	3	4	5	6	7
				Long distance	Medium distance	Local	
	Heavy Trucks	Axle base	Auxiliary axle load	Percentage of heavy goods	Percentage of heavy goods	Percentage of heavy goods	Tyre type
FLM4_1		4,5	70 130	20,0	40,0	80,0	A B
FLM4_2		4,20 1,30	70 120 120	5,0	10,0	5,0	A B B
FLM4_3		3,20 5,20 1,30 1,30	70 150 90 90 90	50,0	30,0	5,0	A B C C C
FLM4_4		3,40 6,00 1,80	70 140 90 90	15,0	15,0	5,0	A B B B
FLM4_5		4,80 3,60 4,40 1,30	70 130 90 80 80	10,0	5,0	5,0	A B C C C

Figure 7: Load model 4 for fatigue calculations in [16]

For highways in the case of the traffic type in column 5 of Figure 7 an “medium distance” is assumed. For considering traffic distribution along with the “new” load models, Figure 7 can be expanded by additional rows corresponding to the “new” load models. The traffic percentages for FLM4_1 to FLM4_5 must then be reduced in favour of the “new” load models.

BRIDGE CONSTRUCTION

In addition to the action, the resistance side of bridge design must also be recorded. The resistances that the bridge must exert against the effects stated above, are regulated in the design standards. Concrete bridges make up the largest proportion of road bridges; on the other hand, wooden, steel or composite bridges make up just a small proportion. Due to the low proportion of other constructions, our examinations will be limited to concrete bridges.
Concrete constructions can be divided into the following main groups, disregarding non-reinforced concrete or stone bridges:

- Prestressed concrete construction and
- Reinforced concrete construction

For economic reasons, reinforced concrete bridge construction is limited to small spans of up to about 25 metres in longitudinal direction. For larger spans usually prestressed constructions are used.

Prestressed concrete bridge building is a “young” discipline in concrete structures. The first prestressed concrete bridge was built in 1936 in Aue, with external prestressing; composite prestressing elements were used for the first time in 1938 in Oelde for a bridge crossing a motorway. The trend in prestressed concrete construction is shown in detail in [17], [18], [19] and [20], amongst others. The components that are significant for the investigation are presented again below.
The design concept for the DIN 4227 ([21], [22], [23], [24], [25], [26]) standard series has remained largely unchanged in principle since its introduction. Proof was to be furnished for the serviceability limit state, as well as to provide sufficient safety for the calculated state of failure.
The calculated state of failure was to be determined for combinations of the following for bridges:

- 1.75 times the dead load,
- 1.75 times the traffic load,
- Prestressing,
- Thermal actions (since the introduction of DIN 4227:12-1979),
- Settlement, and
- Creep and shrinkage

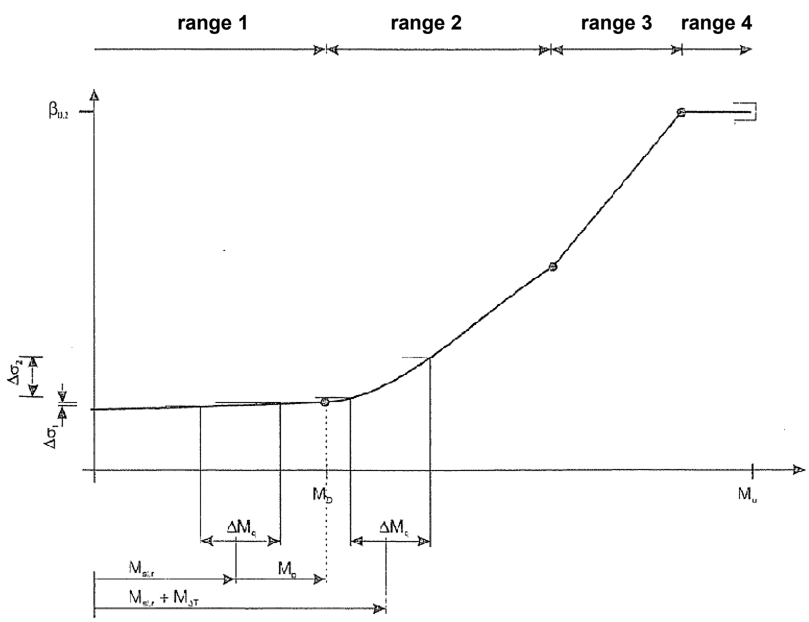
According to DIN 4227, a “state of decompression” was to be furnished in a state of use both for full and limited prestressing. In full prestressing, no tension stresses are allowed with main loads in accordance with DIN 4227.
Main loads in the sense of the standard are:

- Dead load
- Prestressing
- Standard traffic load
- Settlement
- Creep and shrinkage

In the case of limited prestressing, for crack control ([21] 11.2, [26] 10.1.2 (2)), the occurrence of tensile stresses was to be ruled out from prestressing, dead load, creep and shrinkage, and in the case of bridges for half the traffic load (in individual German states, higher proportions of the traffic load were required to prove “decompression”). This means that for a combination of main loads from:

- Dead load
- Prestressing
- 50 % of the standard traffic load
- Creep and shrinkage

the state of decompression must be checked for limited prestressed structures.
It is very significant that the thermal action between the upper side and the lower side of the bridge has to be taken into consideration in the calculated state of failure as an additional load since the introduction of DIN 4227:12-1979.



Range 1:
- Cross-section in compression

Range 2:
- Cross-section cracked
- Strain of reinforcing and prestressing steel

Range 3:
- Reinforcing steel yields
- Strain of prestressing steel up to yield strength

Range 4:
- Prestressing steel yields

Figure 8: M – σ_Z correlation of prestressing steel reinforcement from [27]

In order to determine the stress range dependent only on the internal forces, the moment – tension correlation in Figure 8 is modified in Figure 9.

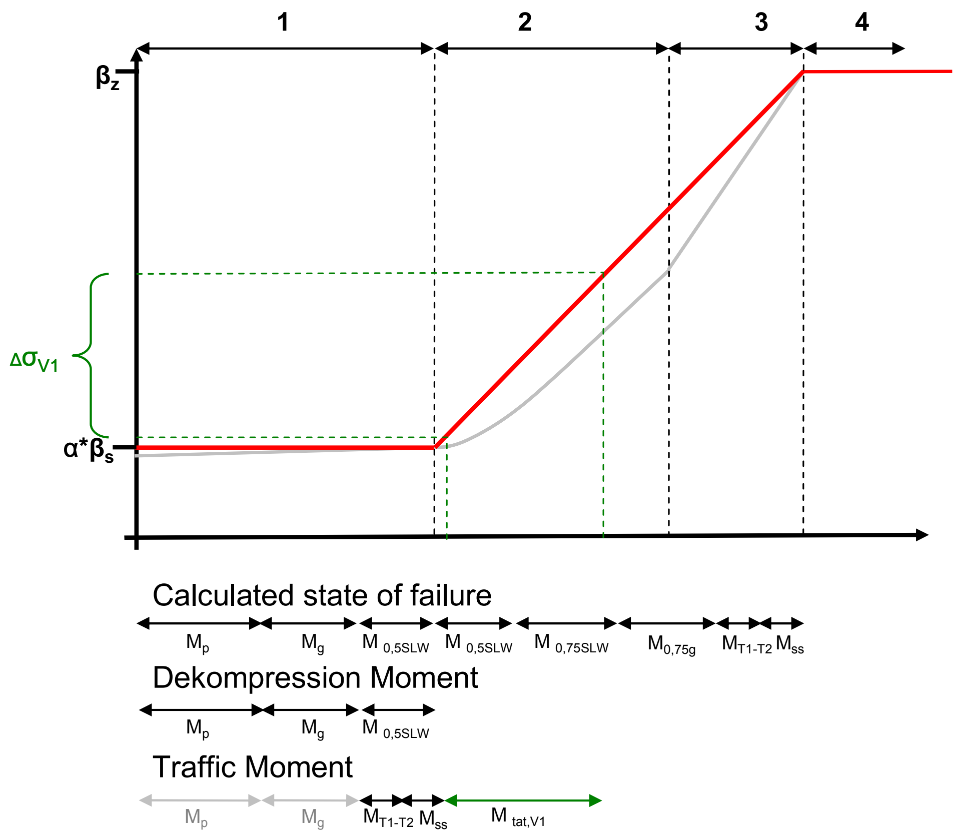


Figure 9: Modified M – σ_z correlation (limited prestressing)

The distribution of the moment – tension correlation in range 1 is presumed to be constant. This simplification is justified as the stress ranges in this range correspond approximately to 7 times ($E_{Prestressed\ concrete}/E_{Concrete}$) the stress ranges of concrete and are therefore negligible for the fatigue. It is assumed that in designing prestressed concrete structures, the state of “decompression” and/or the edge tensions are initially verified. In this, the prestressing steel is already prestressed to its maximum permitted tension; for the calculated state of failure, the prestressing steel is further exploited and the necessary reinforcing steel is added to achieve the required security against fracture. The favourable effect of the reinforcing steel, even in the form of minimum reinforcement, is ignored. This therefore produces a linear relationship for the moment – tension correlation; regarding the slope of the curve, the respective stress range can be inferred from the traffic moments.

The following correlation thus results for $\Delta\sigma$:

$$\Delta\sigma = \frac{(1-\alpha)\cdot\beta_s}{1{,}25\cdot M_{SLW} + 0{,}75\cdot M_g + M_{T_o-T_u} + M_{SS}} \cdot (M_{LM} - (0{,}5\cdot M_{SLW} - (M_{T_o-T_u} + M_{SS})) \qquad (1)$$

with

$-(0{,}5\cdot M_{SLW} - (M_{T_o-T_u} + M_{SS})) \geq 0$

α = $\alpha_{s/z} \cdot \alpha_1$ [-]

$\alpha_{z/s}$ = β_z / β_s (tensile strength/yield strength) [-]

α_1 = $\beta_{s,vor,G} / \beta_s$ utilization factor of prestressing in state of use [-]

M_{LM} = Moment from “new” load (e.g. LM52K) [kNm]

M_{SLW} = Moment from traffic load corresponding to the bridge class (e.g. SLW 60/30) [kNm]
M_g = Moment from dead load [kNm]
$M_{T_o-T_u}$ = Moment from thermal actions [kNm]
M_{SS} = Moment from settlement [kNm]

The first term of the equation (1) describes the increase of the tension – bending moment correlation and thus virtually describes the "resistance side".

$$\frac{(1-\alpha)\cdot\beta_s}{1{,}25\cdot M_{SLW}+0{,}75\cdot M_g+M_{T_o-T_u}+M_{SS}} \tag{1.1}$$

A large proportion of dead load reduces the slope of the curve and the stress ranges are smaller. If the influences of the thermal action and the settlement are taken into account in the design of the bridge, the slope of the curve is smaller and thus also the stress ranges. In the case of structures that were designed before 1979 to DIN 4227 without taking into account thermal actions, the stress caused by thermal actions does not apply here and the slope of the curve is steeper. The second term describes the "action" or the proportion of bending moments that generate stress ranges relevant to fatigue.

$$(M_{LM}-(0{,}5\cdot M_{SLW}-(M_{T_o-T_u}+M_{SS})) \tag{1.2}$$

If the term for evaluating the minimum or maximum moment is negative, the cross-section remains under compression and thus no stress ranges relevant to fatigue occur. The moment from the thermal action must be taken into account here.
The moments from settlement and the thermal action do not generate any stress ranges relevant to fatigue, as in the case of settlement they do not occur periodically or, in the case of the thermal action, only occur with small frequency. However, they increase the basic moment, which means that the cross-section is tensioned earlier and thus a larger proportion of the traffic load becomes relevant for fatigue.

$$-(0{,}5\cdot M_{SLW}-(M_{T_o-T_u}+M_{SS}))\geq 0 \tag{1.3}$$

If the sum of the total of the moments from settlement and thermal action is greater than half of the moment of the designed traffic load, the complete moment of the effective traffic loads becomes relevant to fatigue. As the moments from settlement and the thermal action are not relevant to fatigue, the condition (1.3) must be maintained.

The moments from the traffic loads are known; those from dead loads, settlement and thermal action must be estimated. For the systems being investigated, the beam cross-section is determined depending on the respective maximum span and a slenderness of 25. For a three-span beam with a span of 18 metres for the edge beam and 25 metres for the central beam, a beam cross-section with a height of one meter is assumed, for example. The dead load is determined depending on the beam height, whereby the superstructure and the compensating gradients are taken into account. In order to be able to refer the assumed cross-section weight to the width of the bridge, factor G is introduced. A factor G of 6 means a solid slab with a width of 6 metres. If a slab and beam cross-section of the same width is to be taken into account, for example, factor G is adjusted accordingly (e.g. $G_{\text{Solid slab}}/G_{\text{Slab and beam}}$ = 4/1 → Factor $G_{\text{Slab and beam}}$ = 1.5 instead of 6 for the solid slab).

The bending moments from the thermal actions and settlement are determined through the bending stiffness of the assumed cross-section. A thermal difference between the upper and lower sides of 5 Kelvin and -2 Kelvin is assumed. The possible settlement is estimated at 1 cm.

In addition to the actions, the resistances must also be estimated. For prestressing steel of St 1550/1770 $\alpha_{z/s}$ = 1770/1550 = 1.14. The maximum possible degree of prestressing in state of use of 0.55 β_z produces a α of $\alpha = 1.14 \cdot 0.55 = 0.63$.

The resultant damage from individual crossings can be determined from the stress ranges in accordance with the Palmgren-Miner damage accumulation hypothesis [28], [29] and thus the permitted number of crossings can be indicated. Using the traffic volume of the bridge and taking the estimated traffic distribution as a basis, the residual life time can be indicated.

For this, equation (2) from [30] is rearranged and evaluated for the crossings of individual load models:

$$D = \frac{1}{K} \cdot \sum_{i=1}^{N} [\Delta\sigma_i]^m, \; mit \; K = N^* \cdot [\Delta\sigma(N^*)]^m \qquad (2)$$

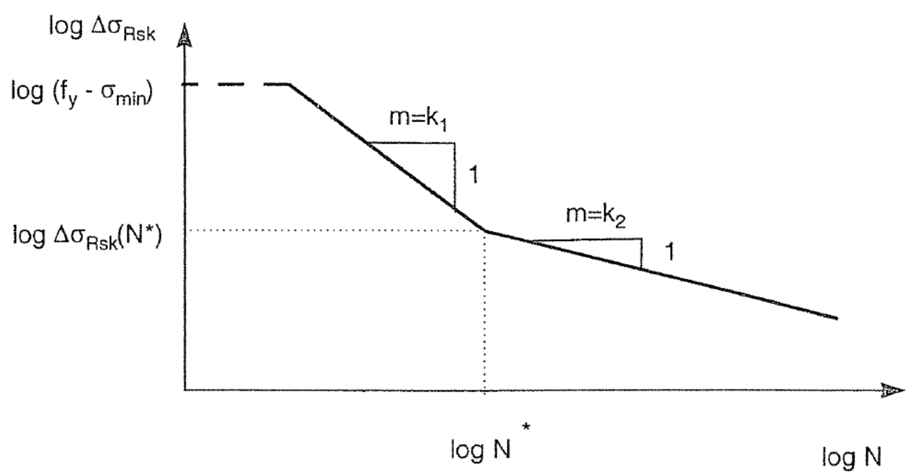


Figure 10: Stress-number curve from [27]

$$N = N^* \cdot \sum_{i=1}^{i} n_i \left[\frac{\Delta\sigma(N^*)}{\Delta\sigma_i} \right]^m, \; mit \; D = 1 \qquad (3)$$

n_i = Proportion of individual load models in heavy goods traffic
k_1, k_2 cf. Figure 11

Prestressing with	Norm	N^*	K_1	K_2	$\Delta\sigma_{Rs,k}$ [MN/mm²] $N = 10^6$	$\Delta\sigma_{Rs,k}$ [MN/mm²] $N = 10^8$	$\Delta\sigma_{Rs,d}$ $N = 10^8$
pre-tensioned	MC 90 EC2 T2 DIN 1045-1	10^6 10^6 10^6	5 5 5	9 9 9	160 185 185	96 111 111	83 96 96
post-tensioned							
straight	MC 90	10^6	5	9	160	96	83
curved	MC 90	10^6	3	7	120	62	54
single stands in plastic ducts	EC2 T2 DIN 1045-1	10^6	5	9	185	111	96
straight tendons or curved tendons in plastic ducts	EC2 T2 DIN 1045-1	10^6 10^6	5 5	9 10	160 150	96 95	83 82
Curved tendons in steel ducts	EC2 T2 DIN 1045-1	10^6	3	7	120	62	54
splicing devices	Alle	10^6	3	5	80	30	26

Figure 11: Stress-number curve of prestressing steel in international standards from [27] (1MN/m² ≈ 0.145 ksi)

For $\Delta\sigma(N^*)$, $\Delta\sigma_{Rs,k}(N^* = 10^6)$ = 160 N/mm² (≈ 23.8 ksi) is estimated for straight tendons in steel ducts, taking into account the safety factor $\gamma_{s,fat}$ = 1.15 according to DIN 1045-1.

STATIC SYSTEMS

According to the distributions in Figure 12, the inventory can be divided into two essentially static systems:

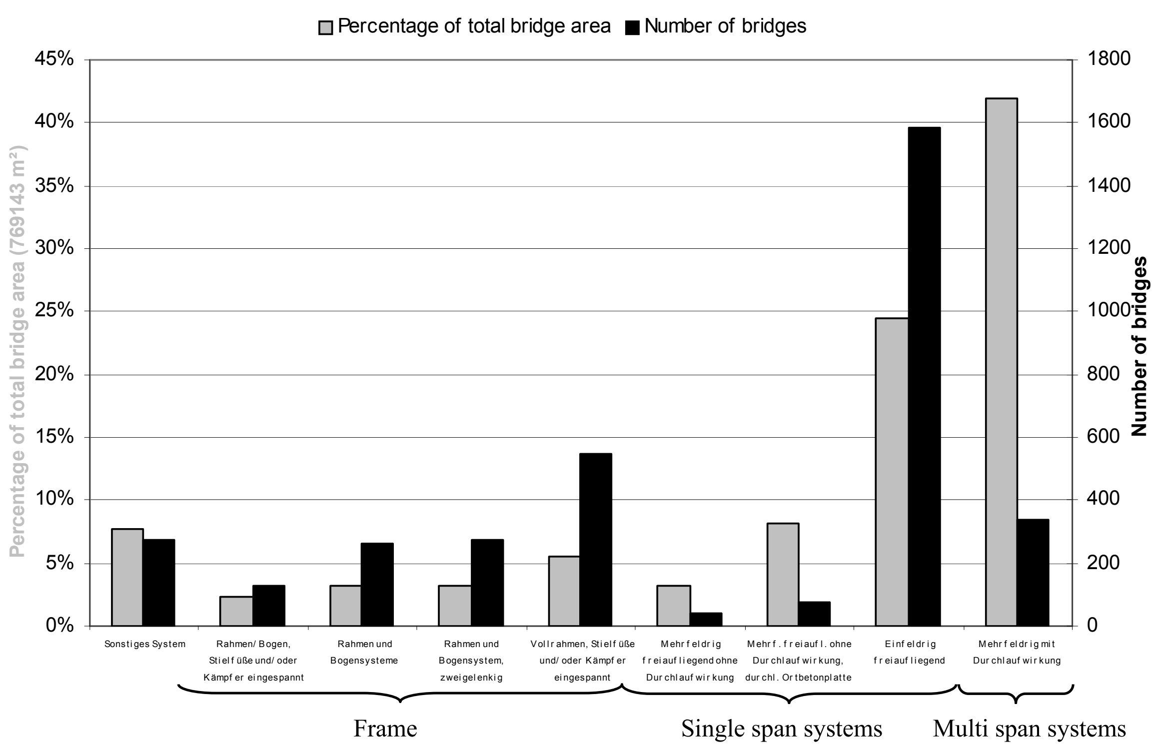


Figure 12: Static systems in longitudinal direction

Single-span systems (SSS) provide 36 % of the total bridge area (≈ 275,000 m², 329,000 yd²) and 48 % of bridge structures (≈ 1700 bridges). This also includes multi-span systems without the effect of continuity. The maximum spans of single-span systems are between 0 and 90 metres (0 and 295 ft), whereby 87 % of single-span girder bridges with a bridge area of 158,000 m² (189,000 ft²) have a span smaller than 20 metres (65.5 ft).
Multi-span systems with effect of continuity (2FT, 3FT, etc.) provide 42 % of the total bridge area (≈ 320,000 m², 383,000 ft²) and 10 % of bridge structures (≈ 350 bridges). The maximum individual spans of multi-span systems are between 5 metres (16 ft) and 180 metres (1246 ft), whereby the maximum span of multi-span girder bridges in 77 % of bridges, giving a bridge area of 168,000 m² (201,000 yd²), is less than 40 metres (131 ft).

Due to the high proportion of individual continuous beam types, the multi-span systems are further broken down into sub-systems according to their number of spans. The largest proportion is made up of 3-span girders at 20% of total bridge area.
The multi-span systems can also be differentiated regarding the ratio of distances between individual supports. If the ratio of individual support distances of two-span girders is given, then these do not vary from each other by more than 15% in 80% of the bridge area investigated, and the span lengths are generally about the same size.

Considering the 3-span girders above, the support distances of spans 1 and 3 only differ 10% from each other in 90% of the spans examined, i.e. the spans of field 1 and 3 are generally the same size. Cases where there are three individual spans of almost equal size, i.e. where the ratios of individual spans only differ from each other by about 10%, occur in about 7% of the bridge areas examined. Assuming that the end spans have about the same spans, then in about 50% of the bridge areas examined, the span of the central span is 1.2 to 1.5 times as large as the end spans.

EVALUATION

The systems described above were examined in respect of their effects on calculated load bearing capacity and fatigue.

CALCULATED LOAD BEARING – A COMPARISON OF BENDING MOMENTS

For an examination of the calculated load bearing capacity, the effects of the "new" load models were compared with the normative traffic loads. Due to the asymmetric position of the normative design loads on the two lanes, the traffic moments can be compared directly with each other to be on the safe side. For the evaluation, only bridges of at least bridge class 45 were examined.
The evaluation of single-span systems has shown that when considering the worst case, a traffic jam situation on both lanes using load model LM52, the calculated load bearing capacity is exceeded in 15 % of the bridge areas of single-span systems examined. When examining the situation of flowing traffic (LM52), only 3 % are affected.
In the case of the three-span girders examined, the support moment is decisive compared to the field moment. Considering the traffic jam situation from LM52 for both lanes, then in 76% of bridge area of three-span girders, the calculated load bearing capacity is not achieved. Assuming the traffic situation, however, the calculated load bearing capacity is achieved in 86% of the bridge area of three-span girders. If load model LM60 is drawn on for the evaluation, then the calculated load bearing capacity is no longer given when considering the

traffic jam situation in 39% of the bridge areas, or when considering flowing traffic in 3% of the bridge areas.

1	4	5	6	7	8	9	10
Longitudinal static system	**Multi-span with effect of continuity**						
Number of fields	***3***						
Bridge class	**DIN: 45**	**DIN: 60**	**DIN: 60/30**	**FB 101 LM1**	**Summe**	**Percentage of bridge area or number of two-span girders**	**Percentage of total bridge area or number**
LM 52K (traffic jam 0 m)							
Critical span [m]	5	10	10	20			
Number of bridges [items]	37	86	38	0	161	91%	5%
Bridge area [m^2]	22920	83041	36065	0	142027	94%	18%
LM 52K (traffic)							
Critical span [m]	35	90	90	90			
Number of bridges [items]	10	2	1	0	13	7%	0%
Bridge area [m^2]	12142	6001	4060	0	22204	15%	3%
LM 60 (traffic jam 0 m)							
Critical span [m]	15	20	25	55			
Number of bridges [items]	22	58	16	0	96	55%	3%
Bridge area [m^2]	19191	70696	23966	0	113854	75%	15%
LM 60K (traffic)							
Critical span [m]	40	90	90	90			
Number of bridges [items]	9	2	1	0	12	7%	0%
Bridge area [m^2]	11331	6001	4060	0	21392	14%	3%

Figure 13: Three-span girder – calculated load bearing capacity exceeded (≥ bridge class 45) (1 m = 3,28 ft)

FATIGUE – PRESTRESSED CONCRETE STRUCTURES

If the calculated load bearing capacity of the bridge is achieved, fatigue must also be considered in order to obtain a statement on the design working life of the bridge under examination. For the evaluations, a traffic combination as in Figure 7 is assumed, where the percentage of ELM4_3 is reduced to 10% in favour of LM52K_30 under the formulation of additional loads for passing traffic (see also legend for Figure 14ff). In addition to the load for passing traffic, only traffic in one lane is considered for ELM4_1 to ELM4_5, LM52 and LM60, although loads are always arranged in two lanes for the respective bridge class. In order not to underestimate the loads from traffic in one lane, the "f(Spur)" factor was determined and drawn on for the evaluation.

Figure 14 to Figure 17 show the stress ranges created by the "new" load models and the sustainable heavy goods traffic, i.e. the number of possible crossings of heavy goods trucks, assuming the selected traffic distribution. The design working life or residual design working life of the bridge can be inferred from the sustainable heavy goods traffic if the ADHGT is known.

In Figure 14 to Figure 17, the stress range of the support moments (dashed line) and that of the field moments (continuous line) of load models LM 52 and LM 60 is presented for all spans of the respective three-span beam (system 1: spans 7 – 11 – 7 m (23.1 - 36.3 - 23.1 ft), system 2: spans 11 – 15 – 11 m (36.3 – 49.5 – 36.3 ft), etc.). The bars represent the sustainable heavy goods traffic for that support moment (SV M_{min}) or field moment (SV M_{max}).

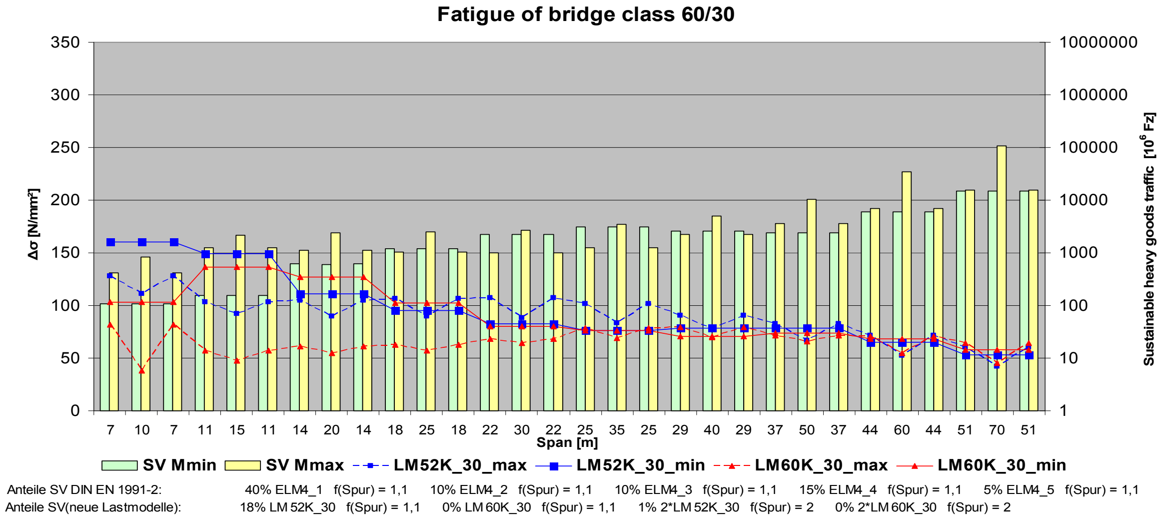


Figure 14: Δσ from the comparison of "new" load models with bridge class 60/30 (1 m ≈ 3,28 ft, 1N/mm² ≈ 0,145 ksi)

In Figure 14, a solid slab is assumed (G factor = 6), and the distribution of the stress ranges refers to the case of passing trucks (lane factor = 2). The 60/30 bridge class was introduced in 1985; the restraint moments (thermal action) were taken into account. The "new" load models are not relevant for the design in view of fatigue, this results in a sustainable heavy goods traffic of 10^8 trucks (in the case of a 10 m central span). This means that in the case of an assumed lifespan of 80 years, an ADHGT of about 3500 trucks is possible.

In order to investigate the influences on bridges of bridge class 60, the constructions must be divided in accordance with the design standard on which they are based. In the case of bridges built according to DIN 4227:12-1979, the restraint moments from the thermal action are already taken into consideration, if an older edition is used the restraint moments from thermal action are not considered.

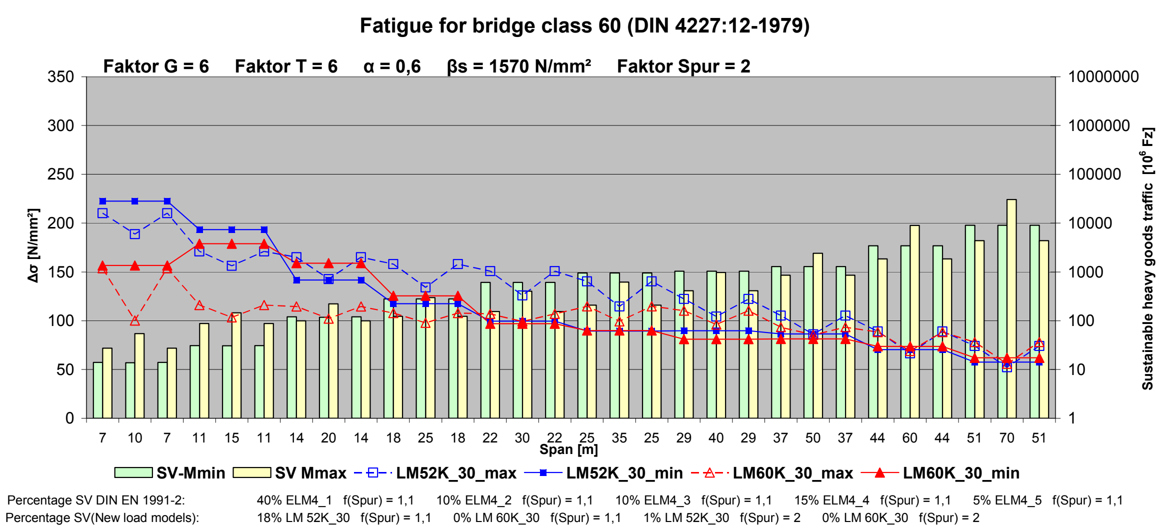


Figure 15: Δσ from the comparison of the "new" load models with bridge class L 60 (DIN 4227:12-1979) (1 m ≈ 3,28 ft, 1N/mm² ≈ 0,145 ksi)

Compared to bridge class 60/30, only heavy goods traffic of $14*10^6$ trucks is sustainable for bridge class 60 with the same spans (see also Figure 15), i.e. an ADHGT of 480 trucks over 80 years. However, it must be taken into account that the bridge was built in around 1985 (the introduction of bridge class 60/30) and has thus already been in operation for more than 20 years. In order to determine the actual heavy good traffic that can still be sustained, the damage must be known, i.e. the number of crossings already made during the period of operation.

If the restraint moment from the thermal action was not taken into account until the introduction of DIN 4227:12-1979, the sustainable heavy goods traffic for the same system falls to $8*10^6$ trucks (see also Figure 16), equivalent to an ADHGT of 275 trucks. Here, too, the period of operation of the bridges of at least 29 years must be taken into account.

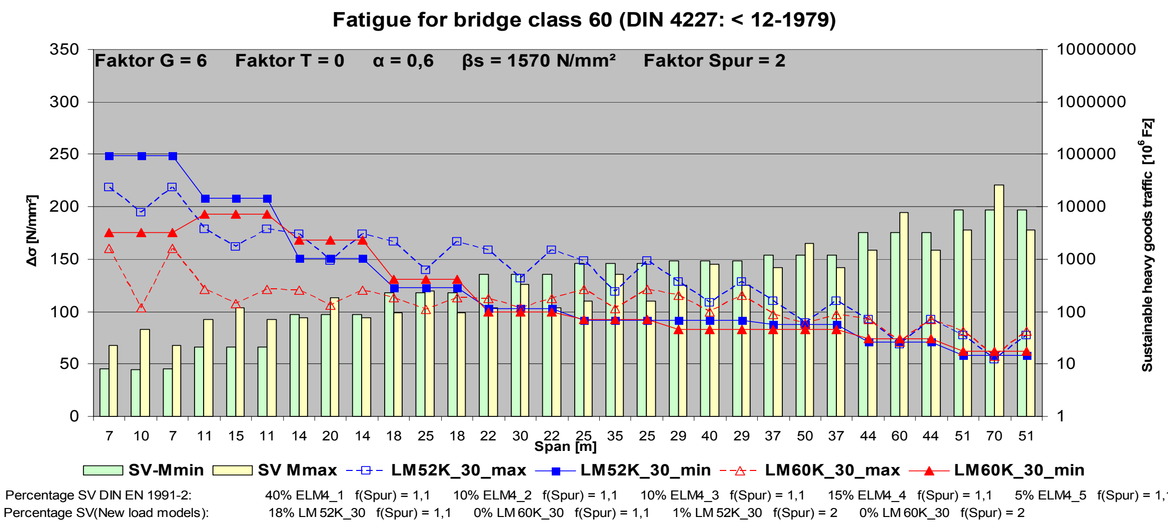


Figure 16: Δσ from comparisons of the “new” load models with bridge class 60 (DIN 4227: < 12-1979) (1 m ≈ 3,28 ft, 1N/mm² ≈ 0,145 ksi)

Compared to bridge classes 60/30 and 60, the new load models have the greatest effect on bridge class 45 regarding fatigue. For the system already investigated with a central span of 10 metres, this results in sustainable heavy goods traffic of 10^6 trucks, equivalent to an ADHGT of 35 trucks. The majority of bridges of bridge class 45 were built before 1965 according to Figure 3 and have thus already been in operation for 42 years.

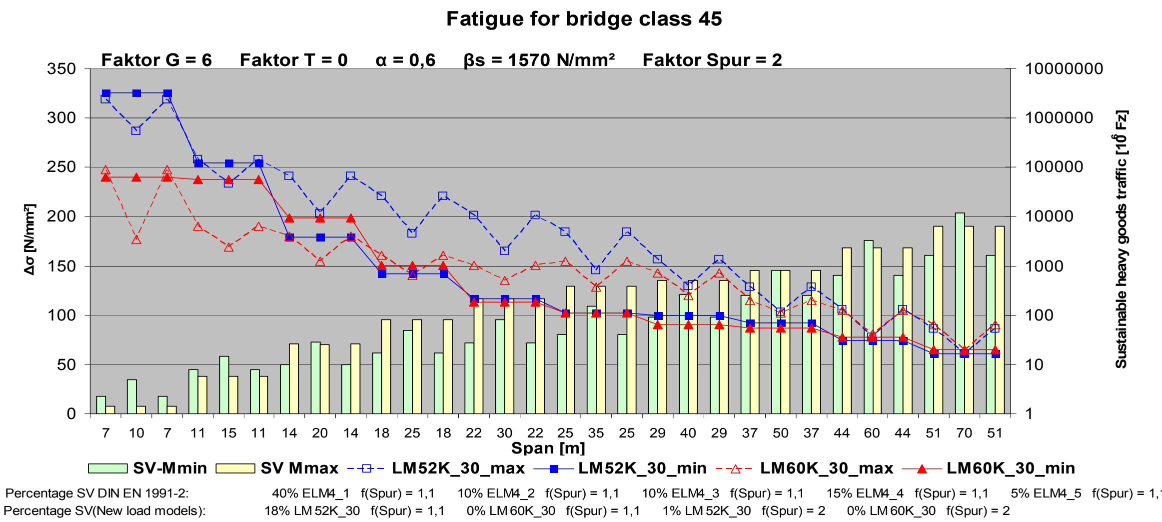


Figure 17: From the comparison of “new” load models with bridge class 45 (1 m ≈ 3,28 ft, 1N/mm² ≈ 0,145 ksi)

CONCLUSIONS

The bridge inventory on Bavarian state highway routes was examined in view of the calculated bending capacity and the fatigue of the bending tensile reinforcement. In the investigations into bending capacity, the internal force stresses of traffic loads caused by the new truck combinations can be compared directly with the internal force stresses of the respective bridge class. The maximum support and field moments were used as evaluation criteria.

The calculated capacity is given for 80% of the bridge area of single-span girders, assuming a worst-case scenario. Looking at the situation of flowing traffic, then the calculated capacity is exceeded in about 10% of the bridge area of single-span girders. If the calculated capacity for three-span girders is evaluated, assuming a central span that is 35% larger than the end spans, then the calculated capacity is not achieved in about 80% of the bridge areas of three-span beams, taking traffic jam situations into account. In the case of flowing traffic, however, sufficient calculated capacity is obtained for 80% of the bridge area of three-span girders.

For the investigations, extremely unfavourable traffic situations were consciously examined without taking into account the probability of their occurrence. For this reason, the next step is to examine the probability of occurrence of the situations in order to check whether the evaluation criteria are at all suitable for the bridges ranked as not bearing the calculated load.

In view of fatigue, the systems examined indicated that they behaved well. For statically determined systems, the probability of fatigue is considered to be low. In the case of statically non-determined systems, the influence of restraint due to thermal action taken into account or not is clear. In the case of bridges that were designed using DIN 4227:12-1979 or a later version, the thermal action that was taken into account in the state of failure has a positive effect on fatigue safety. Here the thermal action taken into account causes a smaller slope on the curve in Figure 9 and thus smaller stress ranges relevant to fatigue.

If the thermal action was not taken into account until the introduction of DIN 4227:12-1979, the basic moment increases and thus smaller loads become relevant to fatigue. In examining fatigue, the traffic composition in accordance with Figure 7 was taken as a basis in considering the new load models and presupposed as applicable for the whole inventory.

In order to determine the actual degree of utilization of the bridges under the new load models, the actual weight of the respective bridges must still be taken into account. According to the proportion of dead loads, this produces a degree of utilization of:

$$\alpha = \frac{M_g + M_{LM}}{M_g + M_{SLW}} \tag{4}$$

with

M_{LM} = Moment from “new” load (e.g. LM52K) [kNm]
M_{SLW} = Moment from traffic load corresponding to the bridge class (e.g. SLW 60/30) [kNm]
M_g = Moment from dead load [kNm]

Analogous to [5], an indication can be made of the costs of reinforcement or a replacement through the degree of utilization, whereby an additional fixed sum should be estimated for each bridge requiring reinforcement depending on the bridge area.

In order to be able to record the proportion of dead load more accurately, details of the actual cross-section, e.g. the average cross-sectional area of the superstructure, should be recorded in a database, as the estimate of the proportion of dead load can lead to significant misinterpretations. In addition to a more accurate examination of cross-sectional values of the structure, the shear of the cross-sections should primarily be investigated in more detail.

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