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Autor: Lebek, Dietrich E.
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Experiences with Orthotropic Steel Deck and Cable Stayed Bridges

Expériences avec tabliers en dalle orthotrope et ponts à haubans

Erfahrungen mit orthotropen Fahrbahnplatten und Schrägseilbrücken

Dietrich E. LEBEK
P. Eng.
Federal Road Research Institute
Köln, Fed. Rep. of Germany

D.E. Lebek graduated from the Technological University of Munich, Federal Republic of Germany. He worked for nine years in Alberta and British Columbia, Canada. On his return to Dusseldorf (FRG), he was appointed Bridge Engineer, and is presently Assistant Head of Bridge Division for the Federal Road Research Institute.

SUMMARY
A survey is made of some important aspects related to maintenance, repair and rehabilitation of orthotropic steel deck and cable stayed steel bridges. Thermal effects due to machine laid resurfacing of orthotropic steel decks can no longer be neglected. Information is presented as an aid to diagnose such effects. The need for inspecting cables and for provisions for replacement of cables is emphasized, and methods for improved corrosion protection of cables are briefly described.

RESUME
Le rapport présente quelques aspects importants de l'entretien et de la réparation de dalles orthotropes et de ponts à haubans. Des effets thermiques lors d'un renouvellement du revêtement ne peuvent plus être négligés. Des informations sont données pour faciliter le diagnostic de ces effets. La nécessité d'inspecter les câbles et de prendre des précautions pour les remplacer est soulignée. Des méthodes de protection anticorrosion améliorée des câbles sont décrites brièvement.

ZUSAMMENFASSUNG
Dieser Beitrag gibt einen Überblick über einige wesentliche Aspekte im Zusammenhang mit Unterhaltung und Reparatur von Brücken mit orthotroper Fahrbahnplatte und seilverspannten Stahlbrücken. Temperaturauswirkungen infolge Belagserneuerungen mit Fertigern können nicht mehr vernachlässigt werden. Dazu werden Angaben gemacht. Die Notwendigkeit einer Überprüfung der Seile und einer Seilauswechselung wird unterstrichen und Angaben für einen verbesserten Korrosionsschutz der Seile gemacht.

1. INTRODUCTION

1.1 Brief historical Background

Orthotropic steel bridge decks have been built since about 35 years, modern cable stayed bridges since about 25 years. In Germany, the first orthotropic steel bridge deck, with reinforced concrete as wearing surface, had been built in 1947. All orthotropic bridge decks built from 1950 have a bituminous wearing surface with an insulating layer of sufficient adhesive and waterproofing qualities.

The basic principle of cable stayed bridges is not at all new. Plans of cable stayed bridges are known from the 17th century. Failures and collapses have apparently hampered further development. Insufficient tensile strength of the materials used, faulty detailing of connections and insufficient vertical and horizontal stiffnesses have certainly contributed to have cable stayed bridges let fallen into oblivion. The development of cables of sufficiently high tensile strength suitable for bridge construction and modern means of structural analysis finally met requirements needed for modern cable stayed bridges.

First designs of modern cable stayed bridges found no acceptance for the time being short after World War II. Today, it may be considered as a curiosity that in 1956 a bridge over the Rhine, with a main span of 256 m, had been erected with the aid of stay-cables but the bridge in it's final state was not a cable stayed bridge. However, the final "break-through" came about shortly afterwards and cable stayed bridges were built in short succession.

1.2 Statement of the Problem

Both, the orthotropic steel deck and the cable stayed bridges have gone through several principal phases of development until they have reached the present state. Naturally, at the begin of the development phase problems such as refinement of analytical methods, shop fabrication, welding and erection techniques had top priority. From the beginning, due consideration was given to the problem of surfacings for orthotropic steel decks. However, the relation between the structural elements of the orthotropic plate regarding their relative stiffnesses and the bridge deck surfacing and the waterproofing was not fully recognized. Furthermore, to what extent the micro-climate affects corrosion of stiffeners was only learned by experience. Economical considerations brought about the change from hand to machine laying of bituminous surfacings. Thermal effects resulting from machine laying have been not always allowed for in the original design of the bridge.

The full-locked-spiral-cable, the type of cable almost exclusively used in Germany for cable stayed bridges, was believed to be completely sealed internally and, with the outside painted, to outlast the assumed life expectancy of the bridge. After about one decade of service in an increasingly worsening environment such an assumption proved to be too optimistic. The extent of the overall corrosion problem including the stress-corrosion problem was again only learned by experience after several years of service.

1.3 Scope of the Report

This report surveys aspects related to maintenance, repair and rehabilitation of orthotropic steel deck and cable stayed steel bridges. A few cable stayed concrete bridges have been built recently and probably more are to come in near future. Because of their short service life, not enough experience could be gained so far. In order to achieve brevity in this report, detailed information on bituminous surfacing and paint systems has been omitted.

Aspects related to the orthotropic steel deck are described in Chapter 2, while aspects related to cable stayed steel bridges are reviewed in Chapter 3. Conclusions and recommendations are listed in Chapter 4.

2. ORTHOTROPIC STEEL DECK

2.1 Present State of Development

The present "standard" orthotropic plate has a 12 mm thick deck plate stiffened with longitudinal trapezoidal stiffeners, 300 to 325 mm wide at 300 to 325 mm transverse spacing. The webs of cross-beams are cut out so that stiffeners are continuous for the full length of the bridge with shop, respectively, field splices at or near the points of contrafluxure. The stiffeners are continuously welded to the deck plate and supported at the cross-beams by web to web stiffener to cross-beam welds. Thicker deck plates and deeper stiffeners are normally only used where structurally necessary, usually in the region of peak negative moments. Such "standard" orthotropic plates show an overall satisfactory behaviour, shop fabrication and assembly are likewise nearly "standardized" procedures and the even surfaces favour relativ easy paint application.

Orthotropic plates, even those with torsionally stiff stiffeners, have a poor transverse load distribution. Results of strain gauge measurements on the bottom flange of stiffeners of a "standard" orthotropic plate are shown in Fig. 1 [1] . Studies made under traffic in motion show that about seventy to eighty per cent of all wheels of commercial vehicles passed within about ± 300 mm of the centre line of the "ideal" wheel track. These two facts combined indicate that orthotropic plates have heavily loaded and less heavily loaded stiffeners.

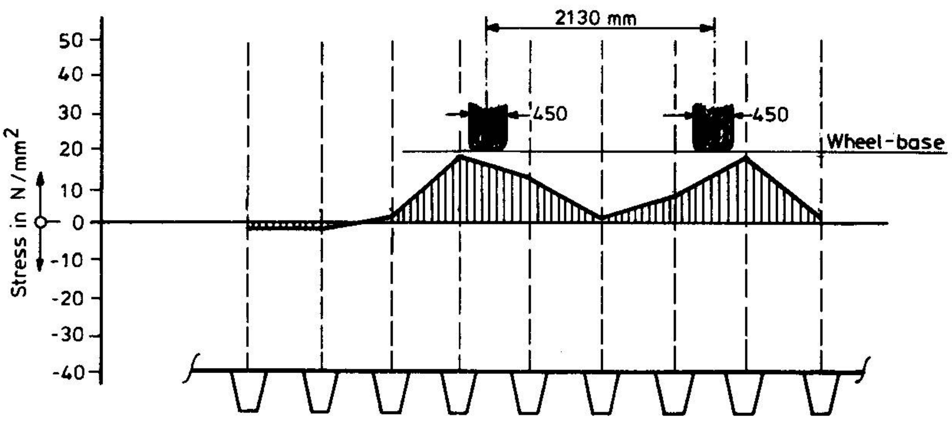


Fig. 1 Transverse Load Distribution

2.2 Inspection

The development of an appropriate procedure and sequence for the inspection of orthotropic steel decks is important in order to ensure the degree of thoroughness and completeness in an inspection that is essential. The scope of inspection may be described in terms of its scale or intensity and its frequency as follows:

- Superficial inspection carried out quickly and frequently by highway maintenance personnel, not necessarily trained in bridge inspection. The purpose of superficial inspection is to report the fairly obvious deficiencies of the bridge deck which might lead to traffic accidents or cause high maintenance costs if not treated promptly.
- Principal inspection is made by a trained inspector under the general supervision of a bridge engineer. This type of inspection usually falls into two categories referred to as general and major defined by frequency and intensity. The general inspection will be primarily a visual inspection supplemented by standard instrumented aids and will entail examination of the bridge deck and the underside of the orthotropic plate. A written report will be made of the condition of the orthotropic plate and its various elements. The major principal inspection will be more intensive and require close examination of all elements, involving setting up of special access facilities where necessary. The intervall between major inspections will vary between 5 to 7 years. A full report containing photographs, drawings, etc. will usually be prepared. A crew whose sole function is to make detailed principal inspection should have a well equipped vehicle stocked with hand tools, with some scaffolding, traffic signs, safety equipment and non -destructive testing equipment for weld inspection.
- Special inspection is made in connection with unusual circumstances, such as exceptional loadings, with occurance of major weaknesses or with reassessment of the orthotropic plate after major repairs. Such inspections may require a good deal of supplementary testing and structural analysis and will invariably require detailed involvement of a bridge engineer and, in some cases, of a specialist.

2.3 Cracks

Cracks in the steel deck plate, in stiffeners and cross-beams have found to be a rare exemption. Cracks have been found especially in the stiffener to cross-beams welds. Fabrication tolerances and welding sequence appear to influnce possible cracking in this welded detail which may be prone to fatique cracking. Some cases are known where rusting took place in closed stiffeners that are nominally sealed by welding.

2.4 Corrosion

The need for a good, durable and longlasting corrosion protection of the deck plate surface is obvious. Not so obvious is, at first sight, the need for a high class corrosion protection of the underside of an orthotropic plate. The temperature gradients, inherent to orthotropic plates, cause condensation which in turn causes rusting. Dewdrops on the underside of a stiffener and condensation stains on the stiffener web are shown in Fig. 2. Experience has clearly demonstrated that only the best paint systems available are capable to give the desired corrosion protection. Since condensation is a permanent occurrance, intervals between application of paint layers should be kept as short as possible.

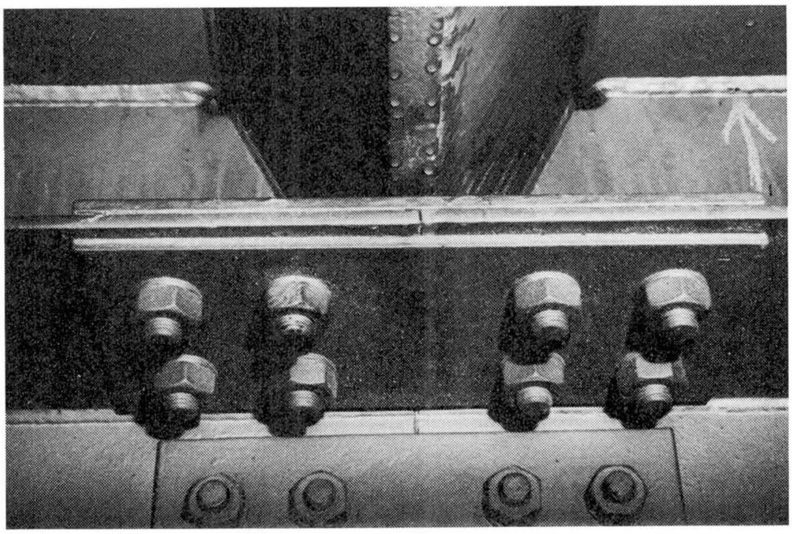


Fig. 2 Dew-Drops on the Underside of Stiffeners

2.5 Connection between Plate Siffness and Surfacing and Waterproofing

Cracking of the surfacing may loosely be grouped under the two categories of cracking over elements much stiffer than the orthotropic plate, e.g. webs of main girders, box girders, stiffening girders, etc. and cracking due to insufficient bond to the insulating layer or to the steel plate. When cracks have formed, water may penetrate and this causes corrosion of the steel which gradually spreads under the surfacing away from the cracks. This results in loss of adhesion which, in turn, leads to the formation of a series of parallel cracks as demonstrated in Fig. 3.

Fig. 3 Severe Cracking of the Surfacing

Ongoing research in Germany into the interaction of plate stiffness factors and surface cracking suggests that it may be advisable to formulate stiffness criteria for orthotropic plates in addition to the traditional strength requirements to achieve a smoother transition between unfavourable stiffness ratios.

2.6 Machine Laying of Surfacings

The service life of surfacings for orthotropic steel decks varies between about 8 and 18 years, excluding of course faulty surfacings or unsuitable material. Consequently, a bridge will have to be resurfaced several times during its service life.

In Germany, "gussasphalt" is the favoured surfacing material used. Gussasphalt is similiar to mastic asphalt which has been used in the case of Severn, Forth and Humber bridges in the U.K.

Traditionally gussasphalt and mastic asphalt have been laid by hand, because they remain fairly fluid for some time after laying and they are also quite dense, so that they tend to trap air within it. The hand trowelling operation would correct the tendency for levels and thicknesses to change after laying and also encourage the escape of trapped air to reduce the risk of blisters and holes forming in the asphalt. However, hand laying is slow, it may give irregular surface, and the number of craftsmen who can do it well is diminishing. Consequently there has been increasing pressure to lay asphalt surfacings by machine to economize on the laying operation and to minimise delays to traffic. Provided necessary precautions are taken, asphalt surfacings can be machine laid successfully. Requirements for a successful machine laying are absolutely dry and clean surfaces free from any oily substances. If those requirements are not observed, the risk of blisters forming in the asphalt and insufficient bond to the surface must be faced with all its consequences.

As already mentioned above, hand laying is a slow operation. Consequently thermal effects were localized and, as a rule, only paint was affected by the heat of the laid asphalt. Thermal effects in structural elements could be neglected. The average machine laying speed is between 1.4 and 2.0 m/min. The minimum width of the spreaders used for the machine laying operation corresponds to the width of one traffic lane (in Germany, that would be about 3.5 m). Thermal effects on the structure can no longer be neglected.

The results of operational research in Germany [2] suggested that existing structures be investigated into thermal effects of machine laying [3] . Four bridges were investigated representing the types of steel bridge most common to modern orthotropic steel deck bridges: plate girder bridge with parallel flanges, haunched plate girder bridge, box girder bridge (curved) and cable stayed bridge. The basic assumption for all four bridges investigated was resurfacing of one half of the total bridge deck under traffic. The investigation was aimed at getting better knowledge of thermal effects on bending stresses and horizontal loads.

The following assumptions were made for the investigation:
- Dead load of the completed structure including weight of the surfacing for one half of the bridge deck, i.e. surfacing on the other half had been taken away in preparation of resurfacing;
- Full design live load for the remaining traffic lanes;
- Design wind load;

- The proportional weight of the machine laid surfacing layer;
- The live load of vehicles and machines needed for the machine laying operation;
- Thermal effects due to machine laying operation.

The results obtained, in comparison to the original design, are as follows:

- Stresses may reverse in some cross-sections, i.e. compression stress in the original design are reversed to tensile stresses;
- Tensile stresses due to the effects of machine laying amount to between 20 per cent and 28 per cent of the total resulting tensile stress. Total resulting tensile stresses can exeed allowable tensile stress up to 16 per cent.
- Compression stresses due to the effects of machine laying amount to between 64 per cent and 81 per cent of the total resulting compression stress. Total resulting compression stresses can to a considerable amount exeed allowable values.
- The stress ratio

  $$\frac{\text{maximum stress due to machine laying}}{\text{maximum original design stress}}$$

  is for tensile stresses 1.00 to 1.31, for compression stresses 1.79 to 3.26. The higher values for compression stresses are partly due to stability requirements. However, instability is normally not a serious problem for orthotropic plates.
- Horizontal loads can be appreciably higher than design horizontal loads.

The investigation clearly demonstrated that reliable results can only by expected if the calculations are based on the actual temperature distribution in the structure. Simplified models of temperature distributions may give incorrect results.

It should be noted that no structural failure in bending due to machine laid surfacings has been observed so far. However, the results of the investigation suggest that thermal effects due to machine laying may partially use up safety reserves which, fortunately, most structures have.

Table 1 shows increase factors f for horizontal loads

$$f = \frac{\text{horizontal load due to machine laying of surfacing}}{\text{horizontal load due to design assumptions}}$$

at the supports of the bridges investigated. The guidebearing on the abutment of the bridge with a calculated increase factor f = 263 sheared off completely from the bearing seat. This may be an indication that safety reserves in a structure are not always inexhaustible.

2.7 Temperature Range of Deck Plate and Insulating Layer

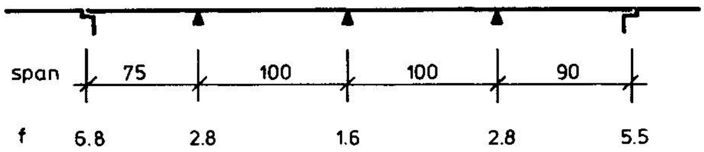
Orthotropic Bridge decks require, as already mentioned above, the best possible corrosion protection of both the deck surface and the underside. Regardless of the base material used for paint systems, e.g. oil, resin, polythene, etc., the degree of temperature is limited which paint systems can endure without damage. In Germany, the material used for insulating layers has changed from pure bituminous material to tar-epoxy-resin which is believed to have better adhesive qualities than pure bituminous material such as mastix.

Plate temperatures are useful as starting point for calculating temperature distributions in a steel bridge and as criterion for

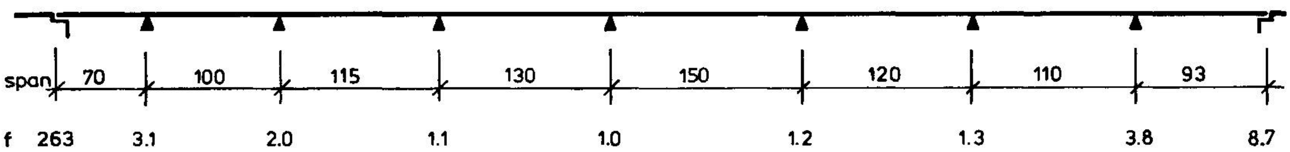
$$f = \frac{\text{horizontal load due to machine laying of surfacing}}{\text{horizontal load according to design assumptions}}$$

$= \text{increase factor against horizontal design load}$

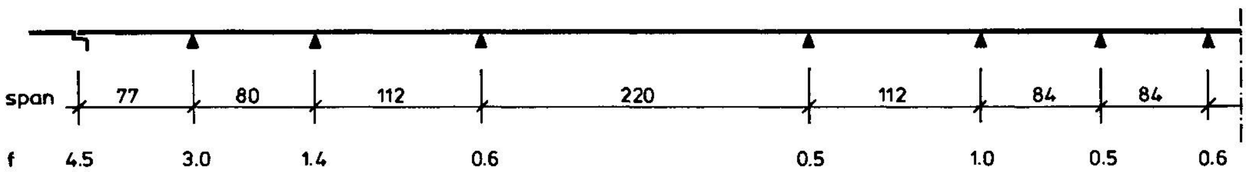
Plate Girder Bridge



Box Girder Bridge (curved)



Haunched Plate Girder Bridge



Cable Stayed Bridge (box girder)

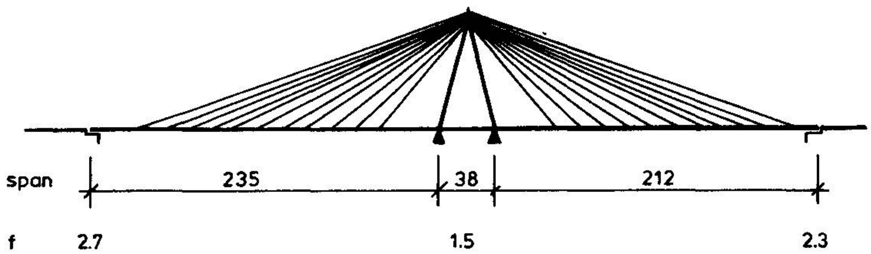


Table 1 : Horizontal loads induced by machine laying of bituminous surfacings on orthotropic steel bridge decks.

heat resistance of paint systems. The maximum plate temperature and the duration of certain temperature ranges, both values are inter-related, define the criterion for heat resistance of paint systems. The results of many temperature measurements in various bridges under various climatic conditions and with varying surfacing systems can be used to predict the maximum deck plate temperature and the duration of temperature ranges. The results are presented in Fig. 4 to 7.

Fig. 4 to 7 may be used as guide to questions arising in connection with resurfacing such as:

- Possibility of damage to existing paint;
- Maximum allowable mixing temperature of surfacing material in order to avoid damage to existing paint, e.g. when there is a choice between surfacing systems;
- Choice of paint system capable to withstand temperature effects without damage, e.g. when only a certain surfacing system has to or can be used;
- Formulation of requirements for corrosion protection systems;
- Prediction of maximum deck plate temperature as starting point for calculating temperature distributions.

Resurfacing may require, according to the condition of the existing surfacing, a total resurfacing, i.e. resurfacing will have to start from the steel deck plate, or a partial resurfacing, i.e. resurfacing of the wearing course only.

From measurements taken the maximum deck plate temperature was empirically found to be

$$t_{max} = \text{variable } (t_S - t_P) + t_P$$

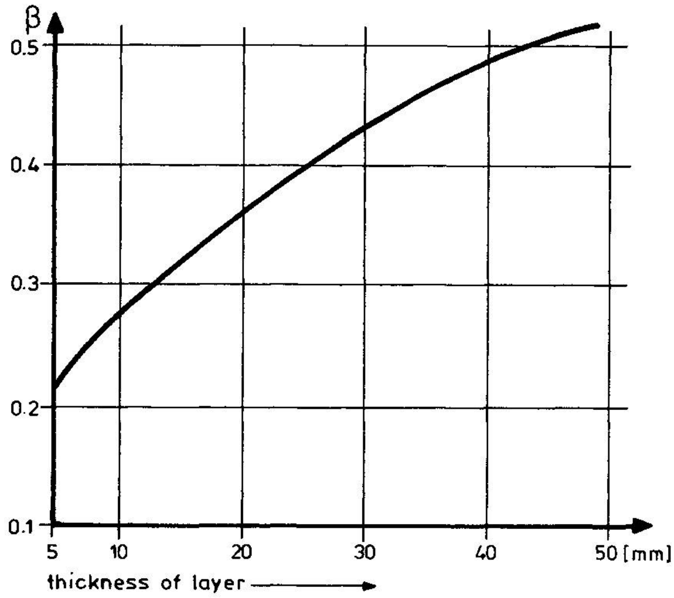
and the maximum surface temperature of the insulating layer

$$t_{max} = 0.44\, t_S + t_P$$

where the variable β is to be used in the case of total resurfacing as presented in Fig. 4 and the variable γ for the case of partial resurfing as presented in Fig. 5; t_S is the recommended mixing temperature of the surfacing material and t_P the existing deck plate temperature at time of laying. The β-values account for an existing layer of a few millimeter thickness. The use of Fig. 4 and Fig. 5 are best illustrated by an example:

- A typical resurfacing operation in Germany would have a first layer of "gussasphalt" with an average thickness of 35 mm laid on an insulating layer about 2 mm thick. The recommended mixing temperature t_S for "gussasphalt" is 250°C. The deck plate temperature t_P shall be 40°C, a value typical for the months May and September. From Fig. 4 a β-value = 0.46 is taken for a thickness of 35 mm. With the above values the predicted maximum plate temperature would be

$$t_{max} = 0.46\ (250 - 40) + 40 = 137°C$$



$$t_{max} = \beta(t_S - t_P) + t_P$$

where

t_{max} = maximum temperature at underside of deck plate

t_S = recommended mixing temperature of surfacing material, e.g. t_S = 250°C for "gussasphalt"

t_P = deck plate temperature at time of laying

β = variable

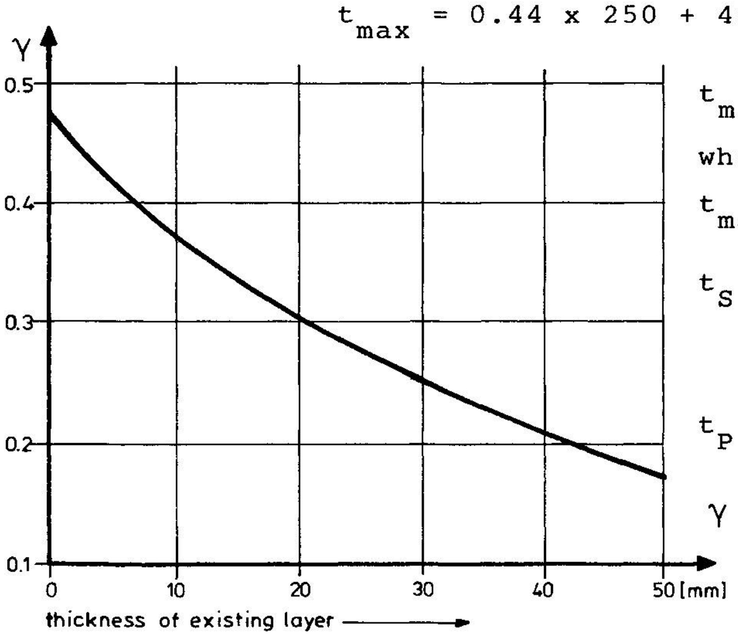
Fig. 4 Maximum Deck Plate Temperature due to Machine Laying of first Layer.

- A typical partial resurfacing operation in Germany would have a wearing course of "gussasphalt" with an average thickness of 35 mm laid on an existing layer about 35 mm thick. The deck plate temperature shall be 40°C. From Fig. 5 a γ-value = 0.23 is taken for an existing thickness of 35 mm. With the above values the predicted maximum plate temperature would be

$$t_{max} = 0.23\ (250 - 40) + 40°C = 88°C$$

- For a typical total resurfacing operation with the condition and values mentioned in the example for Fig. 4, the predicted maximum surface temperature of the insulating layer would be

$$t_{max} = 0.44 \times 250 + 40 = 150°C$$



$$t_{max} = \gamma(t_S - t_P) + t_P$$

where

t_{max} = maximum temperature at underside of deck plate

t_S = recommended mixing temperature of surfacing material, e.g. t_S = 250°C for "gussasphalt"

t_P = deck plate temperature at time of laying

γ = variable

Fig. 5 Maximum Deck Plate Temperature due to Machine Laying on already Existing Layer.

Fig. 6 can be used to predict the duration of certain temperature ranges for the paint on the underside of the deck plate and Fig. 7 for the insulating layer. The use of Fig. 6 will be illustrated by the following example.

Because the critical temperature range for normally used paint systems starts from 80°C, the duration of the temperature range above 80°C shall be predicted. The following data are needed: the recommended mixing temperature t_S, the thickness of the surfacing layer and the existing deck plate temperature t_P.

A layer of 35 mm thick "gussasphalt" is to be laid at a deck plate temperature t_P = 35°C.

In Fig. 6, the temperature range is plotted versus duration (time in min). The temperature range is the difference between an upper and a lower temperature limit. The upper limit is $t_{max} - t_P$ as shown in Fig. 4, the lower limit is the temperature above which the temperature range shall be predicted minus t_P.

In the example, the upper limit is 0.46 (250 - 35) = 100°C, where β= 0.46 is taken from Fig. 4, and the lower limit is 80°C - 35°C = 45°C. Consequently, the temperature range is 100°C - 45°C = 55°C as indicated by arrows in Fig. 6.

The predicted duration for a temperature range above 80°C is for:
the minimum duration 10 min + 75 min = 85 min
the average duration 10 min + 95 min = 105 min
the maximum duration 10 min + 110 min = 120 min

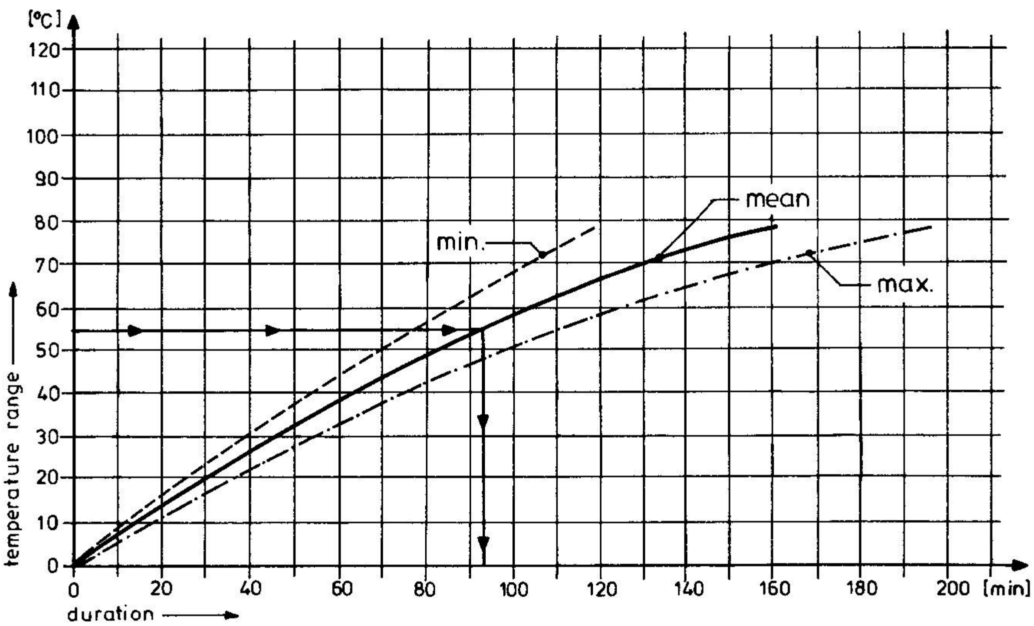


Fig. 6 Duration of Temperature Ranges for Paint Systems

The use of Fig. 7 will be illustrated by a following example. Critical temperatures for tar-epoxy-resin start from above 100°C. The recommended mixing temperature and the existing surface temperature of the insulating layer must be known in order to predict the desired temperature range.

In the example, a layer of 35 mm thick "gussasphalt" is to be laid on an insulating layer with a surface temperature of 35°C. The duration of the temperature range above 100°C shall be predicted.

Required is again the upper and the lower temperature limit. The upper limit is 0.44 x 250°C = 110°C, the lower limit is 100°C - 35°C = 65°C. Consequently, the temperature range is 110°C - 65°C = 45°C as indicated by arrows in Fig. 7. The predicted average duration for temperatures above 100°C is about 60 min.

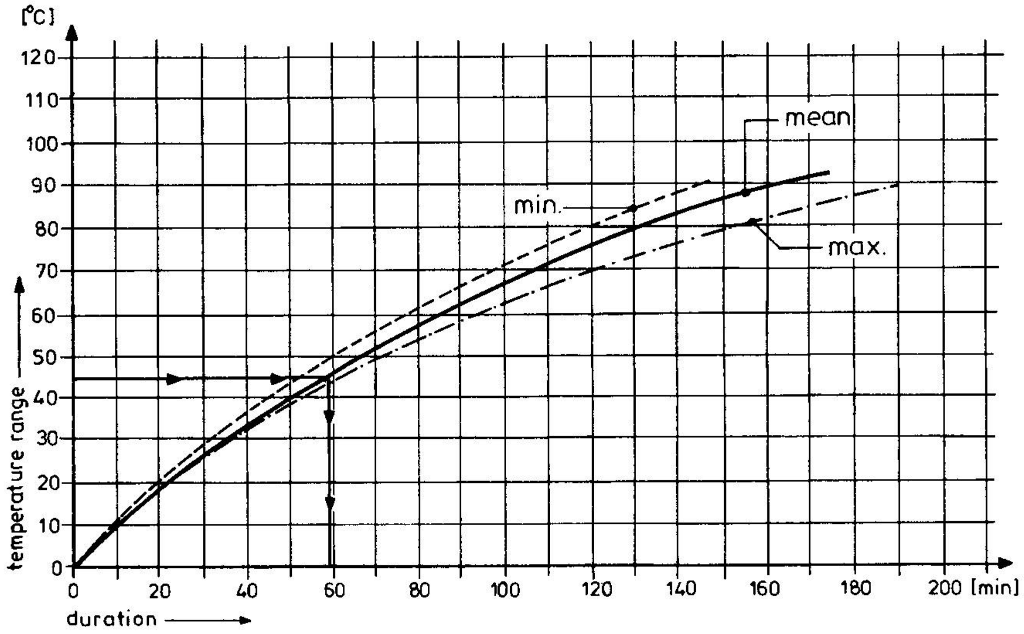


Fig. 7 Duration of temperature Ranges for Insulating Layers

3. CABLE STAYED BRIDGES

3.1 Present state of Development

The present state cable stayed steel bridge uses single cable stays normally either in the form of radiating fan cable stays or harp cable stays depending on bridge aesthetics or structural requirements or a combination of both, aesthetics and structural requirements. Main spans vary between about 175 m and 350 m. Normal span to main girder depth ratios may vary between about 60 and 100. Main girders are either plate girders or box girders with an orthotropic steel plate deck. Cable supports along the bridge centre line require a box girder type main girder. Towers may be either vertical or A-shaped. The type of cable, almost exclusively used in Germany, is the full-locked-spiral-cable. Cable stayed bridges are normally very flexible to aesthetical, structural and geometrical requirements.

3.2 Cable Stays

3.2.1 Wire Strands

From the point of view of tensile strength a wire with the highest possible tensile strength appears to be desirable. From the point of view of durability a wire having the least sensivity appears to be desirable. Unfortunately, an economically feasible material for wire strands to be used in full-locked-spiral-cables combining both qualities does not exist at present. A practical compromise has to be found between tensile strength and durability requirements. Wires with a tensile strength not exeeding about 1600 N/mm^2 are believed to be a practical compromise between required tensile strength and sensivity to corrosion and stress-corrosion.

The "standard" full-locked-spiral-cable is made up of two different wire profiles. Round wires represent the core, Z-shaped wires enclose the core and form the outer layers. All wires in a cable must have the same tensile strength.

3.2.2 Transport of Cables

Full-locked-spiral-cables are prefabricated in the shop to the required lenghts and shipped to the site on reels. Although the diameter of the reel is, in most cases, restricted by clearence limitations along the route, its diameter should not be less than about 35 to 40 times the diameter of the cable. Experience has demonstrated that cables transported on reels having smaller diameters tend to show a permanent "wave-forming" despite of pre-stretching. The "wave-lengths" correspond to the length of the coil around the reel used for the transportation of the cable. Such "waves" may affect the total resulting effective modulus of elasticity of the cable.

3.2.3 Use of Single Cable-Stays

In some of the "older" cable stayed bridges up to 13 full-locked-spiral-cables, erected one at a time, had been connected by special cable bands to form the "main" cable. Each of the cables is, of course, individually socketed and anchored. Grooves formed by the round cables had been puttied up in order to seal the interior of the "main" cable. With the outside painted, the system was believed to give a satisfactory, long lasting corrosion protection. However, this system fell short of the designer's expectations. The "inside" cables can neither be inspected nor re-painted. In addition, erection of cable stayed bridges proved to be easier and more economical, after having gained more experience, if single cable stays at shorter intervals are being used. The system of "main" cables for cable stayed bridges can no longer be recommended.

3.2.4 Cable Replacement

For a period of about 10 to 15 years, bridge designers were convinced, as already mentioned, that full-locked-spiral-cables are a long lasting structural element and would in all probability outlast the expected service life of a cable stayed bridge.

Possible fatigue was already a point of concern from the moment of first designs of cable stayed bridges. Extensive testing of wires and full sized test specimen of full-locked-spiral-cables suggested that fatigue due to live load would, in all probability, not shorten the expected service life of cables. Extensive measurements in a cable stayed bridge in service [4] appear again to support the thesis that fatigue damage due to "static" live load is not to be expected.

Whilst "static" live load appears to be not a serious problem regarding fatigue life expectancy, the same cannot be said for "dynamic" effects, i.e. the sum of all dynamic effects including wind, if at the same time corrosion takes place. Past experience suggests that the combination of dynamic effects and corrosion may present a serious problem to cables especially at or near the anchorage.

Cables had to be replaced in one cable stayed bridge in Germany after only about 5 years of service. The results of rigerous inspection indicate that replacement of cables within expected life time of bridges must seriously be taken into consideration. The German Ministry of Transport considers provisions for cable replacement a must for all future cable stayed bridges.

3.2.5 Damping Devices

As mentioned above, the combination of dynamic effects and corrosion may present a serious problem. In addition, the aerodynamics of cables in service due to the combined actions of wind, dead load, live load and inter-related vibrations of cables, towers and superstructure is not yet fully understood. There are indications that only a certain combination of wind velocity, wind direction and mean stress in a cable cause vibrations where amplitudes in the order of several hundred millimeters have been observed. Normally even such amplitudes are relatively harmless from the stress point of view because of their seldom occurance. However, the psychological effect to road users should not be underestimated.

It stands to reason that damping devices should be installed wherever dynamic effects can be presumed to have adverse effects on the life expectancy of cables or where the behaviour of an existing bridge suggests that damping devices are installed as a rehabilitation measure. Fig. 8 and 9 present examples of possible solutions for the installation of damping devices as a rehabilitation measure. The spring-type dampers as shown on Fig. 9 were fixed to stiffened hand rails.

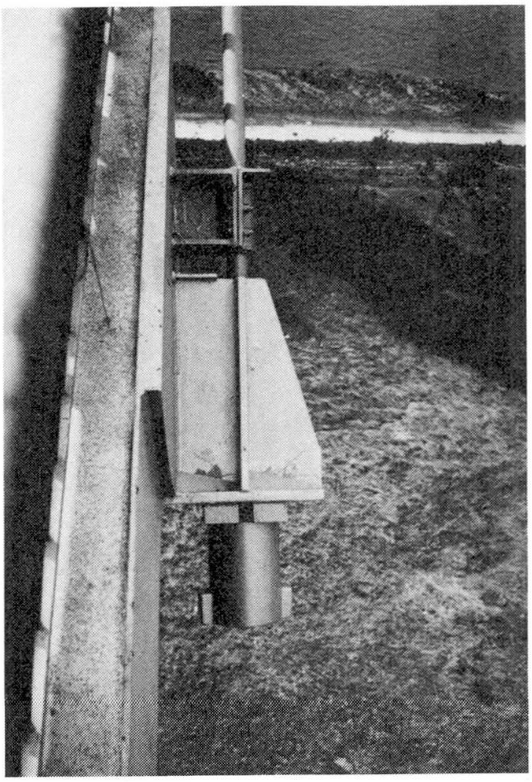


Fig. 8 Elastomeric Damper

Fig. 9 Spring Type Damper

3.3 Inspection

The development of an appropriate procedure and sequence for the inspection of all parts of a cable is important. In the past, the importance of a thorough and complete inspection of cables was often not acknowledged to the required extent by all designers of cable stayed bridges. Even an appropriate access to cable anchorages and cable sockets has not always been provided for in the design. Access to the free cable length between bridge deck and tower must also be possible and would normally require a special type of traveller. Fig. 10 shows a possible solution to this problem.

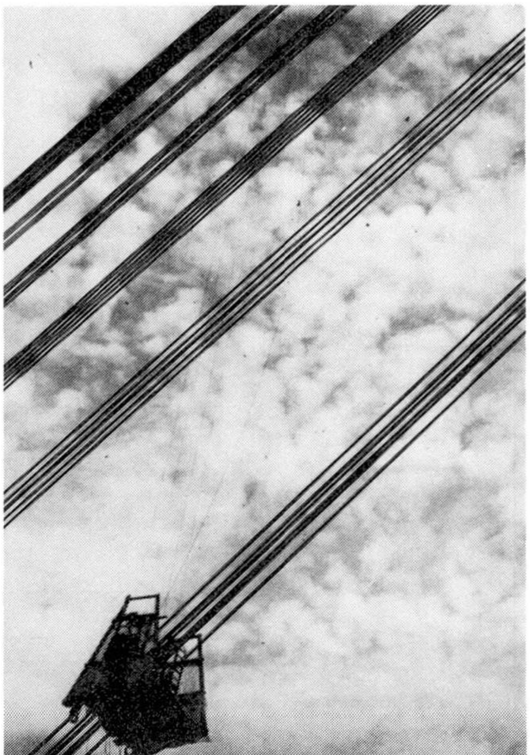


Fig. 10 Inspection of Cables

Reference is made to sub-chapter 2.2 regarding scale or intensity and frequency of inspection. Normally, the principal inspection will be the appropriate form of inspection, special inspection may be required in connection with unusual circumstances.

The general principal inspection will entail examination of the cable anchorages, cable sockets and condition of corrosion protection. An examination of the free cable length from the bridge deck, high quality binokulars will normally suffice, is strongly recommended. The interval between general principal inspections should not exeed three years.

The major principal inspection will be more intensive and require close examination of all parts. Free access to all parts is absolutely necessary. Special attention should be paid to possible wire breakage, cracks in sockets and anchorages including support elements, to a possible slip of wires, displaced shims, misalignement, movements and any kind of corrosion. The interval between major principal inspections will vary between 5 to a maximum of 7 years.

3.4 Corrosion Protection

In sub-chapter 2.4, it had already been stated that experience has clearly demonstrated only the best corrosion protection systems are capable to achieve the desired corrosion protection, so vital for the durability of all cable elements. Experience has also shown success can only be expected from a systematically built up corrosion protection. Since all exposed surfaces will require re-painting or even rehabilitation of the outside protection system during the expected service life of a cable stayed bridge, bridge designers are urgently called up to pay proper attention to a free access to all elements and to make allowances for all necessary provisions in order to accomplish high quality maintenance, repair and rehabilitation work to be done.

A systematically built-up corrosion protection is to start with the fabrication of cables in the shop. All wires should be galvanized firstly as an effective additional corrosion protection and secondly to provide for an additional mechanical protection during transport and erection but also during the following service life in a bridge. Mechanical damage caused by chippings, often applied in order to maintain a good skidding resistance of the bridge deck surfacing, are a well known occurance, and even willful damage can no longer be excluded. In addition to galvanizing a further corrosion protection of all interior wires is strongly recommended. The choice of materials should be dictated by the fact that an interior corrosion protection must last for the entire service life.

The wires of full-locked-spiral-cables are normally secured in the socket by a casting of a special zinc-alloy. The heat of the casting material may damage the interior corrosion protection in the cable ends near the socket. Special injection pipes are now being installed in the sockets in order to inject suitable material into the casting and into the cable ends. Tests have proved that the injected material extends into the undamaged zone of the cable.

"Main" cables in older cable stayed bridges, see also paragraph 3.2.3, are now being encased as a rehabilitation measure. The encasing is designed in such a way that the outside cables can be inspected in a major principal inspection. The encasing is then injected with suitable material in order to achieve a durable long lasting corrosion protection of the entire interior of the "main" cable.

A "standard" rehabilitation measure is now wrapping single cables in the bridge deck area as shown in Fig. 11 in order to provide additional protection in the brine spray zone and against any kind of mechanical damage. Best results can be expected if the wrapping is done before the paint is hardened.

In sum, corrosion protection should only make use of the best paint systems available in order to achieve an economical, long lasting durable protection of all corrosion prone elements against an increasing number of aggressive gases and soluble substances and taking into account mechanical damage that cannot be excluded in the vicinity of the bridge deck area.

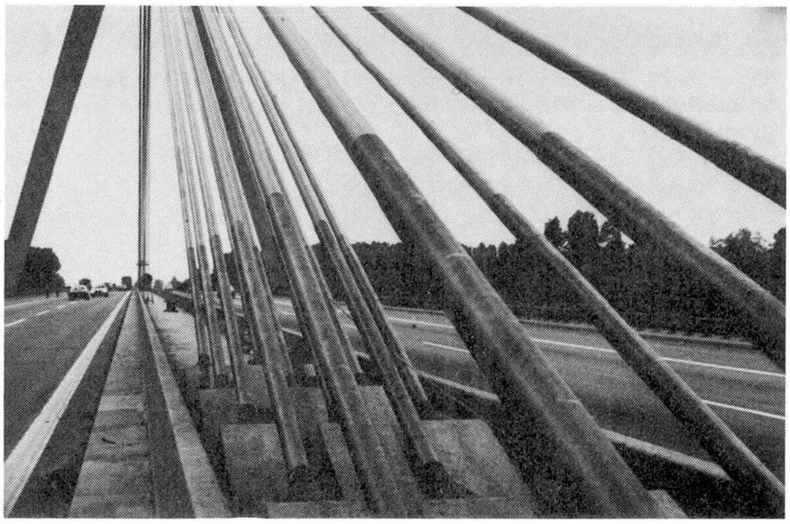


Fig. 11 Additional Wrapping in Splash-Zone

4. CONCLUSIONS AND RECOMMENDATIONS

4.1 Orthotropic Steel Deck

Orthotropic plates have a poor transverse load distribution. Most wheels of commercial traffic pass within a width which corresponds to the normal transverse stiffener spacing. These two facts combined indicate that orthotropic plates have heavily and less heavily loaded stiffener.

The development of an appropriate procedure and sequence for the inspection of orthotropic steel plates is important in order to ensure the degree of thoroughness and completeness in an inspection that is essential. Three forms of inspection can be recommended: the superficial inspection carried out quickly and frequently by highway maintenance personnel; the principal inspection, made by a trained inspector under the general supervision of a bridge engineer, falling into two categories referred to as general principal inspection and major principal inspection defined by frequency and intensity; the special inspection, normally made in connection with unusual circumstances.

Cracks have been found especially in the stiffener to cross-beam welds. Some cases are known where rusting took place in closed stiffeners that are nominally sealed by welding.

Temperature gradients, inherent to orthotropic plates, cause condensation which in turn causes rusting on the underside of stiffeners. It is recommended to use only the best paint systems available. Since condensation is a permanent occurance, it is recommended that intervals between application of paint layers be kept as short as possible.

Resurfacing is nowadays normally machine laid in order to economize on the laying operation and to minimise delays to traffic. Thermal effects on the structure due to machine laying can no longer be neglected. It is recommended that a total reassessment of the structure is made before resurfacing in order to avoid possible damage to the structure. Information is presented in sub-chapters 2.6 and 2.7 as an aid to diagnose thermal effects.

4.2 Cable Stayed Bridges

A tensile strength not exeeding 1600 N/mm² is believed to be a sound compromise between required tensile strength and sensivity to corrosion and stress corrosion.

Single cable stays are recommended. They allow free access to inspection and necessary maintenance work. Since cables may have to be replaced during the service life of a cable stayed bridge, the bridge should be so designed as to allow for cable replacement in a bridge in service.

Damping devices may be required in order to counteract aerodynamic effects and to relieve unfavourable stress situations near the anchorages. Installation of damping devices can in most cases be done as a rehabilitation measure in existing bridges.

Proper inspection of all cable elements is essential. Free access is a presupposition for the required quality of inspection. Normally, the principal inspection will be the appropriate form of inspection.

A thorough and complete corrosion protection is vital for the expected durability of all cable elements. Corrosion protection systems should be built-up systematically and should start with the fabrication of cables in the shop. Corrosion protection should only make use of the best paint systems available. As a rehabilitation measure additional protective wrapping in the vicinity of the bridge deck area is recommended.

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