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INTRODUCTORY REPORT

Rapport introductif

Einführungsbericht

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1. The Orthotropic Plate

Attributing to every structural member an exactly defined function was a common rule during the classical days of bridge building, resulting in plain statical conditions, easily grasped by simple manual methods of computation.

Although the advantages of highly statically undeterminate systems had been known for a long time, a broad application of those systems was blocked at first by the tremendous expenses involved in their solution. In bridge building, the introduction of beam grids may be considered the first effort to achieve a more economical design of bridge decks.

The rapid progress made in pre-stressed concrete constructional methods about 20 years ago led to a reallocation of competive limitations between concrete and steel. That means, in terms of bridge building, the pre-stressed concrete is used for longer spans, previously reserved for steel structures. In order to stop these recession of structural steel in comparison to pre-stressed concrete, the development of new designs was necessary and, therefore, steel construction engineers were compelled to think entirely differently in terms of structural designing. This development was contributed to extensively in Germany by the limited production of steel, which lasted several years after World War II. It is not surprising, therefore, that, particularly in Germany efforts concerned with new solutions for steel bridge construction led to the production of a light-weight deck, called "orthotropic plate", an abbreviation for a rectangular (or orthogonal) shaped, anisotropical plate. At the same time the calculation of highly statically indeterminate systems, practically insolvable by usual manual methods, became possible by the introduction of electronics.

Except during the first years of development the design of steel bridges using the orthotropic plate has altered only to a small extent. Their structural development was brought to a speedy preliminary conclusion with the following determinations:

1. The steel plate should be 10, 12 or 14 mm thick (3/8", 1/2" or 9/16" appr.), the mean value being normal.
2. Normally, the stiffening ribs are spaced at 30 cm = 1 ft centers.
3. To date, the distance of beams is 1,5 m to 3,0 m = 5 to 10 ft. There is a tendency, however, to increase the distance so that the use of orthotropic plate is more economical.
4. At first, mainly flat steel bars and, preferably, bulbs were used for stiffening ribs, and in a few cases angles; later on the closed sections have been preferred increasingly, namely trapeziodals, V-, Y- and, more rarely, half circle-shaped sections. The torsional rigidity of closed sections considerably reduces the deflections of the steel plates, resulting in lower stress of the surfacing.

5. In any case, plate joints are made by welding, because early experience had already demonstrated that a surfacing, permanently stable and crack-resistant but limited in thickness, cannot by used as long as there are doubler plates and rivet heads in the relevant area.

2. Problems Arising

For about two decennia endeavours have been continuing in Germany to find out the most suitable surfacing for orthotropic steel plates as described above. There is a long way to go. In every case progress is to be expected only by long-term observations of the pavements already applied on bridges. Parallel to these, extensive laboratory tests have to be undertaken continously, aimed at an adequate explanation of the observed facts on the one hand and suitable to determine the next steps to clarify the problem on the other.

During recent years several countries have started construction of steel bridges using orthotropic plates:

United Kingdom	:	New Forth and Severn Bridges;
Switzerland	:	St. Albans's Bridge in Basle;
Austria	:	Europe Bridge near Innsbruck;
Hungary	:	Elizabeth Bridge over the Danube in Budapest;
USA	:	Poplar Street Bridge in St. Louis, Miss., spanning the Mississippi River;
		San-Mateo-Hayward Bridge, San Francisco, Bay Toll Crossings, California;
Canada	:	Port Mann Bridge in Vancouver;
India	:	Jamuna River Bridge near Delhi

and several others.

For the time being, orthotropic steel plates have been used mainly on movable bridges, e.g. in Sweden, Denmark, the Netherlands and in USSR. I believe, however, that the problems to be mentioned here are already very well known to the experts concerned. Furthermore, publications dealing with wearing surfaces on orthotropic steel plates have already been used in several countries.

I may limit myself, therefore, to a brief enumeration of requirements t h e a s p h a l t i c w e a r i n g s u r f a c e s on steel decks - exclusively dealt with in Thema I - should fullfill in respect of durability, economy and traffic security.

Most important are:

Surface smoothness (smooth riding),

Skid resistance,

Wear resistance,

Ready and foolproof application
by mechanical equipment,

Reliable protection of the steel plate
against corrosion,

Stable at permanently high temperatures,

High cracking resistance at sudden
drops of and low temperatures,

Good bonding to steel.

3. Existing Wearing Surfaces

In Germany many different wearing surfaces have been tried out since installation of the first orthotropic plate. Generally, all of them consist of several courses or layers, each of which fullfills a special function, except the so called "splittverfestigte Mastix-Decke" (Stabilized Mastix or Stone Filled Mastic Asphalt System). There has been a lively competition in proposing new compositions for asphaltic wearing surfaces - proposals being made by authorities concerned with the planning of forthcoming bridges and by numerous steel bridge and road construction firms as well as by the asphalt industry. Leaving aside any qualifying remarks the following tables show all of the surfacings applied up to day in Germany.

No.	anchoring device	bonding compound or bond coating	bonding layer or insulation	lower course (levelling course)	upper course (wearing course)
1	- - -	Okta-Haftmasse@	Mastix @ 8 - 10 mm	mastic asphalt appr. 2,5 cm	mastic asphalt appr. 2,5 cm
2	- - -	Okta-Haftmasse	Mastix 10 mm	mastic asphalt 3 cm	fine aggregate asphalt concrete 2,5 cm
3	- - -	Okta-Haftmasse	Mastix 8 - 10 mm	Binder @ 3 cm	mastic asphalt appr. 3 cm
4	- - -	Okta-Haftmasse	Mastix 8 - 10 mm with Pulvatex	Binder 2,5 cm	mastic asphalt 3 cm
5	- - -	Okta-Haftmasse	Mastix	stone-filled Mastix 2 - 2,5 cm	asphalt concrete 3 - 4 cm
6	- - -	Okta-Haftmasse	Okta-Mastix	"VABIT" Binder	fine aggregate asphalt concrete "VABIT"
7	- - -	Okta-Haftmasse	aluminum foil	mastic asphalt appr. 3 cm	mastic asphalt appr. 3 cm
8	- - -	Okta-Haftmasse on cold applied zinc paint	- - -	mastic asphalt appr. 3 cm	mastic asphalt appr. 3 cm
9	- - -	Okta-Haftmasse on hot sprayed zinc coating	Mastix 8 - 10 mm	binder course 2,5 cm	mastic asphalt 2,5 cm
10	- - -	Okta-Haftmasse on hot sprayed zinc coating	Mastix 8 - 10 mm	mastic asphalt 2,5 cm	mastic asphalt 2,5 cm
11	- - -	Okta-Haftmasse on hot sprayd zinc coating	- - -	mastic asphalt 2,5 cm	mastic asphalt 2,5 cm

@ See remark on page 9

No.	anchoring device	bonding compound or bond coating	bonding layer or insulation	lower course (levelling course)	upper course (wearing course)
12	- - -	Okta-Haftmasse	- - -	stone-filled Mastix 4 cm in two layers	fine aggregate asphalt concrete 2,5 cm
13	zig-zag-bars	Okta-Haftmasse	- - -	stone-filled Mastix 4 cm in two layers	fine aggregate asphalt concrete 2,5 cm
14	zig-zag-bars	Okta-Haftmasse	- - -	stone-filled Mastix with Okta-additive	
15	- - -	cold applied bitumen coating	- - -	mastic asphalt 2 cm	mastic asphalt 2,5 cm
16	- - -	cold applied bitumen coating	- - -	mastic aspahlt 2 cm	mastic asphalt 2 cm
17	- - -	cold applied bitumen coating	aluminum foil	mastic asphalt 2,3 cm	mastic asphalt 2,7 cm
18	- - -	cold applied bitumen coating	copper foil	mastic asphalt 3,5 cm with Pulvatex	mastic asphalt 3,5 cm with Pulvatex
19	- - -	cold applied bitumen coating	Mastix 10 mm	mastic asphalt 2 - 3 cm	mastic asphalt 2 - 3 cm
20	with and without reinforcing steel fabric	cold applied bitumen coating	Mastix 10 mm	Binder 2 - 3 cm	mastic asphalt 3 cm

No.	anchoring device	bonding compound or bond coating	bonding layer or insulation	lower course (levelling course)	upper course (wearing course)
21	reinforcing steel fabric	hot applied bitumen	Mastix	mastic asphalt 3 cm	mastic asphalt 2 cm
22	- - -	"Proderit"-varnish	Mastix	mastic asphalt 2 cm	mastic asphalt 2,5 cm
23	- - -	"Proderit"-varnish	aluminum foil	mastic asphalt 2 - 3 cm	mastic asphalt 2,5 cm
24	- - -	"Proderit"-varnish	Mastix with Pulvatex	Binder 1,5 - 3 cm	mastic asphalt 2,5 cm
25	- - -	"Colzumix"-painting	Mastix 8 - 10 mm	mastic asphalt 2,5 cm	mastic asphalt 2,5 cm
26	- - -	"Colzumix"-painting	Mastix 8 - 10 mm	Binder 2,5 cm	mastic asphalt 3 - 3,5 cm
27	- - -	"Colzumix"-painting	Mastix 8 - 10 mm	Mastix 2 cm	asphalt concrete 2,5 cm
28	- - -	"Möllerit"-resin	hot applied bitumen	Binder 3 cm	mastic asphalt 3 cm
29	- - -	"Möllerit"-resin	- - -	mastic asphalt 2,5 cm	mastic asphalt 2,5 cm

No.	anchoring device	bonding compound or bond coating	bonding layer or insulation	lower course (levelling course)	upper course (wearing course)
30	- - -	"Vedag"proprietory cement	aluminum foil	mastic asphalt 2,5 cm	mastic asphalt 2,5 cm
31	- - -	2 x zinc-powder painting	"Pulvatex"-grouting	Binder 3 cm	mastic asphalt 3 cm
32	- - -	2 x Zinkplast-painting	Mastix 10 mm	Binder 3 cm	mastic asphalt 3 cm
33	zig-zag-bars	Okta-Haftmasse	- - -	mastic asphalt 3 cm with Pulvatex	fine aggregate asphalt concrete 2 cm with Pulvatex
34	zig-zag-bars	"Isotex" bonding primer	"Isotex"-insulation	mastic asphalt 3 cm	fine aggregate asphalt concrete 2 cm with Pulvatex
35	- - -	"Sempex" 1 mm	- - -	mastic asphalt 3,5 cm	mastic asphalt 3,5 cm
36	- - -	2 x red load paint	impregnated felt	mastic asphalt 3 cm	"Non-Skid" 2,5 cm

No.	anchoring device	bonding compound or bond coating	bonding layer or insulation	lower course (levelling course)	upper course (wearing course)
37	- - -	2 x bituminous Latex	- - -	mastic asphalt 3,0 cm	" Non-Skid " 2,5 cm
38	- - -	plastic bonding cement applied in two layers	tar-epoxy-resin	7 cm " V A B I T " -(proprietory surfacing)	
39	- - -	epoxy-resin mortar	- - -	mastic asphalt 3 cm	mastic asphalt 3,5 cm

@ Remark

In cases, where there is no exact English equivalent for a German technical term, the word is capitalized to distinguish it as a special term. cf. F. F. Fondriest and M. J. Snyder, "Research on and Paving Practice .....", 1966, Battelle Memorial Institute, p. 3 and A-1 / A-2.

It may be pointed out only that "Okta-Haftmasse" (a proprietary bonding compound or adhesive) has been used most frequently; that the insulating layer consisted of Mastix in most cases; and that "Gußasphalt" (mastic asphalt) was chosen predominantly for both layers of the actual surfacing.

4. Nature of Damage and its Causes

Blistering, cracking, shoving and rutting are damages most frequently observed in the course of the years.

Defects found more rarely are inter alia wet spots on the top surface, increase in joint width and breaking-off of the surfacing behind transverse bars. In detail the following may be said:

Blistering

Blistering is caused by small amounts of air entrapped underneath the surface during application, or is due to development of water vapor or gases. When heated, the small amounts of air or gas expand, forcing the surface to warp, not noticeably in the beginning. Subsequent cooling causes a vacuum in water-vapor traps, and a partial vacuum in areas with entrapped air or gases. As a consequence, more water vapor, air or gases are drawn in from the surrounding area. When the surfacing becomes heated again, it is forced farther up until, eventually, visible blisters are formed by this "pumping action". (See figure 1). However, on traffic lanes, such blisters are pressed down again by the live load so as to cause circular cracks. (See figure 2). Then, penetrating water will gradually detoriate the surfacing.

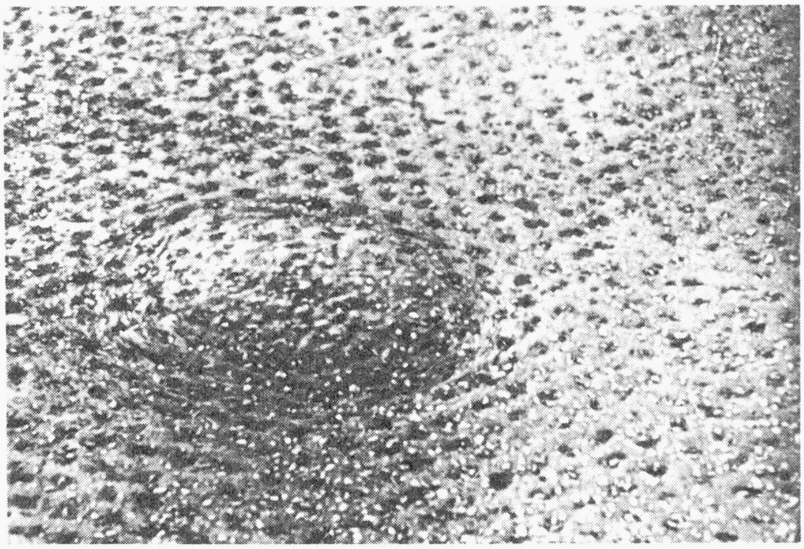


Figure 1: Increasing blister

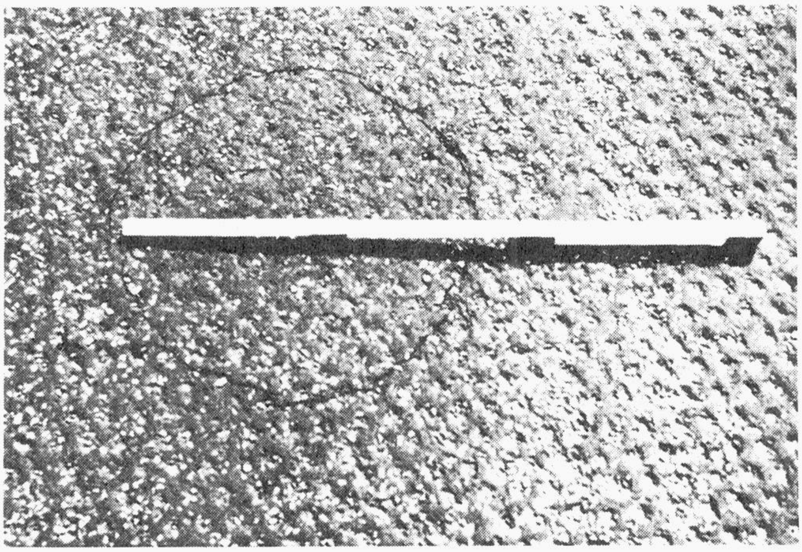


Figure 2: Crack caused by blistering; 35 cm dia.

Development of water vapor is to be expected, when there is m o i s t u r e in the lower courses of the surfacing, e.g. when these are applied in damp air or on steel plates not fully dried after rainfall.

In this respect s e p a r a t o r s of paper or bituminized felt have a very adverse effect, since they prevent escape of entrapped moisture.

The danger of entrapping air, when separators are used, can be avoided by very careful application only.

Cold-applied zinc-paints may cause blistering as well. They contain resin binders, the volatile solvents of which have not entirely escaped when subsequent layers of the surfacing are applied. Furthermore resin binders do not resist to the high temperatures (up to 240°C = 464°F) of mastic asphalt.

Penetration of water occuring later may cause blistering too. This has frequently been observed when, for increased stability of the pavement, a binder course had been applied under an unpermeable wearing course of mastic asphalt. Rain-water, penetrating through unsealed edges or other damages of the surface course may easily spread out in the cavities in the binder course, particularly on sloping steel decks. The water will either emerge at a lower place or will cause blistering here and there when infiltrated into the binder course.

Cracking

There may be a certain relationship between cracking and the thickness of an orthotropic plate together with the spacing of its stiffeners.

However, the consistency of the bitumen used and its percentage in the mixture are much more important factors. As this had not sufficiently known, when the early surfacings were being applied, relevant records have not been drawn up, unfortunately, in many cases. For accurate statements, it is necessary, therefore, to obtain the desired information by making subsequent analyses.

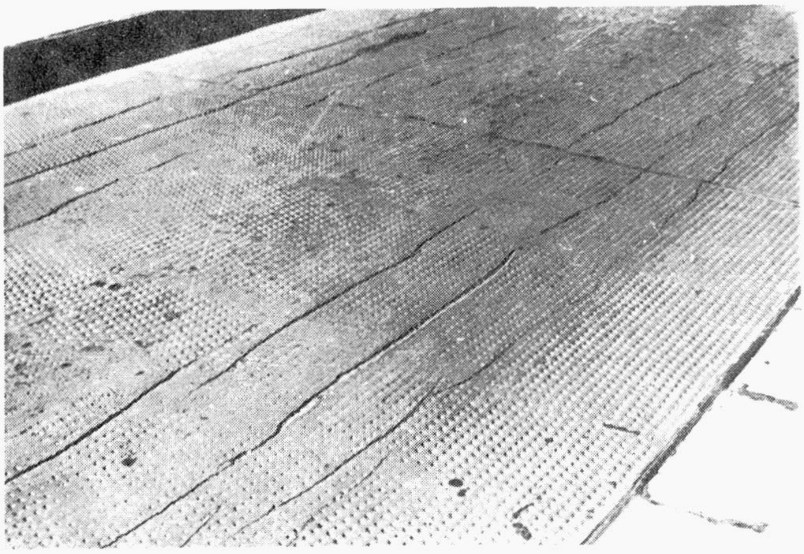


Figure 3: Longitudinal cracking, spaced like the stiffeners

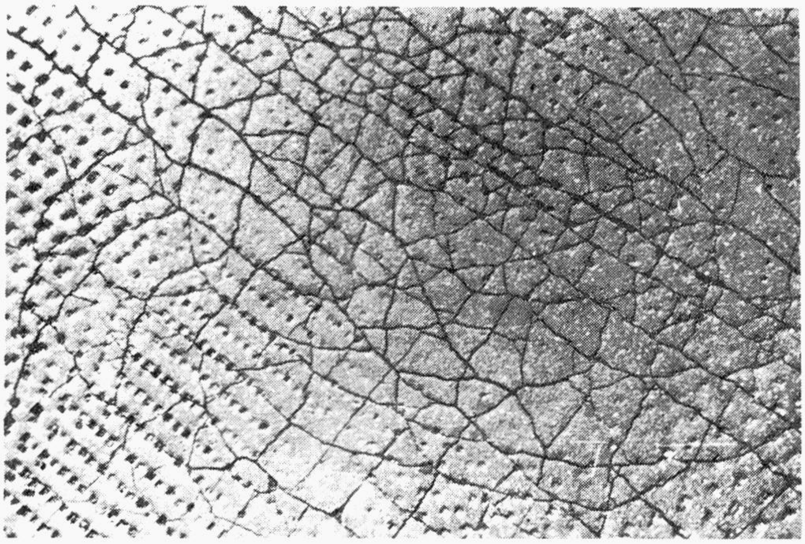


Figure 4: Small-gridded crack area

In most cases, the cracks are in parallel with the stiffeners, both at the top and in between. However, random cracks are not rare; they spread out to a great extent to form distinct crack areas. A destructed binder course, provided for increased stability underneath the mastic asphalt wearing course, has in several cases been determined as the cause of such cracking. (see figures 3, 4 and 5).

Penetrated water if, due to lack of slope, it stays within the binder course, may become aggressive by de-icing agents, and strip the bitumen coating from the mineral aggregates, the latter then being crushed by the live load. Eventually, due to lack of support the mastic asphalt course breaks in.

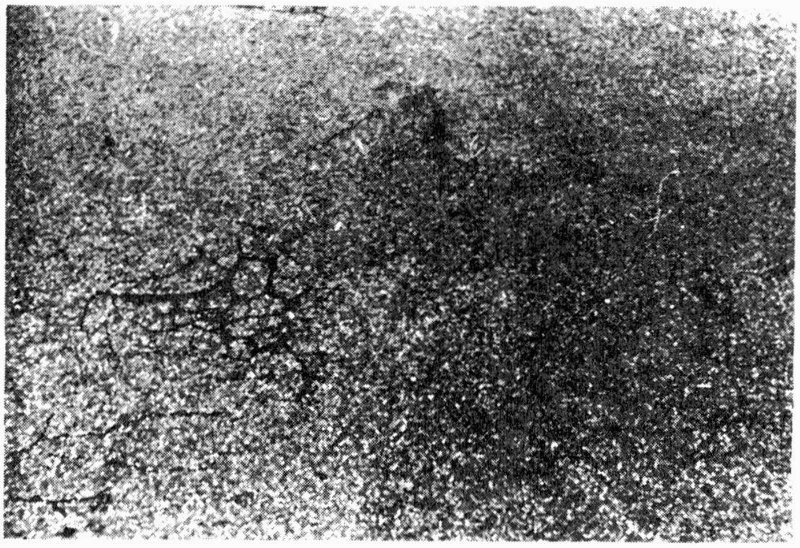


Figure 5: Places broken in over destructed binder course

Rutting

Rutting is a frequent problem, caused at high temperatures by a bitumen content either too rich or too soft. Ruts may be spaced regularly corresponding to the longitudinal stiffeners. See figure 6.

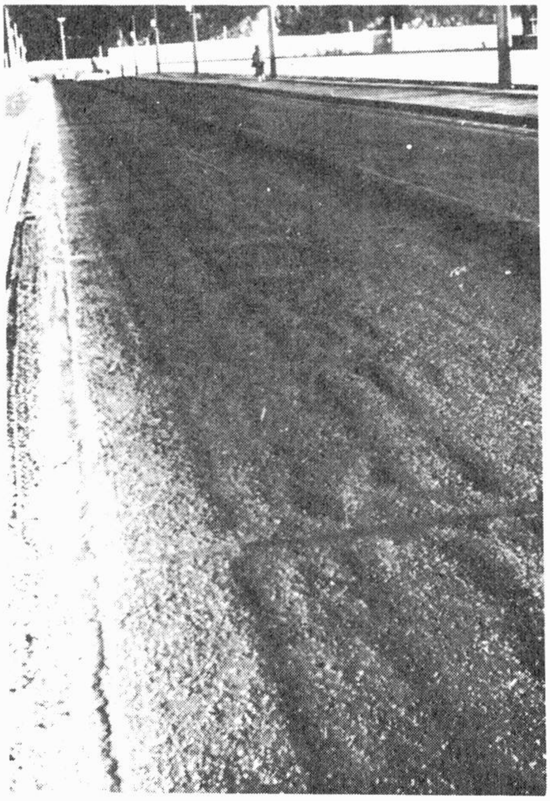


Figure 6: Rutting (transverse waving)

The surfacing may, however, have become so soft that, due to traffic, lateral shoving occours over long distances forming wide ruts superimposed on the aforementioned undulations, spaced like the stiffeners. See figure 7.

Shoving

Shoving, i.e. fulling or waving across the axis, occurs less frequently. Reasons are: faulty composition of the mixture, unsuitable grading of mineral aggregates and a content of bitumen, which is either too rich or too soft. Anchoring devices on steeper longitudinal slopes, made up of flat or angle sections and welded across the steel plate at long distances, increase the number of shovings and cause the surfacing, simultanously, to break loose behind of the anchorings in most cases. See figure 8. and 9.

Figure 7: Big-sized shoving,
up to 7 cm deep

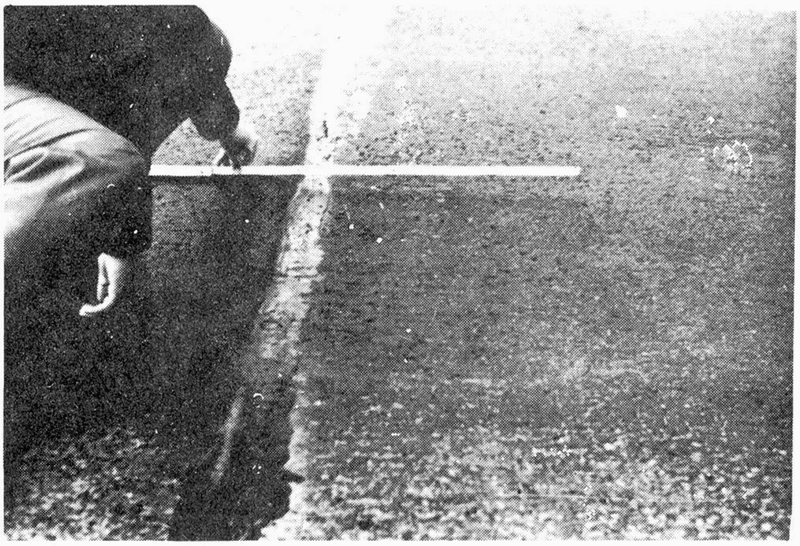


Figure 8: Transverse shoving
up to 4 cm high
ahead of anchor bars

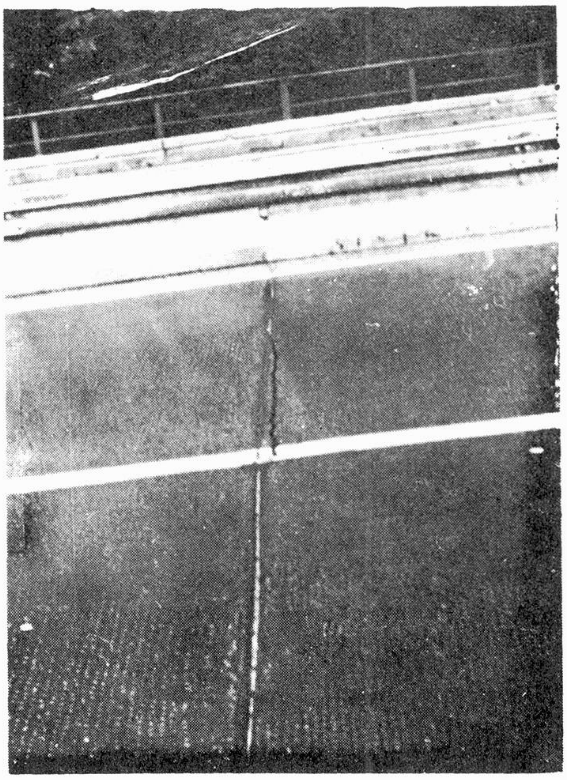


Figure 9: Transverse cracking behind steel anchor bar with joint on top

Even shoving and flowing of big areas has been observed on the top surface. See figure 10.

It may also be mentioned here that surfacings with a rubber-powder-additive contained in the bituminous binder sometimes have been found broken off its lateral supports.

Surfacings using a rubber-powder-additive have been proved to increase the tensile strength of the asphalt. On the other hand our observations have clearly shown that such surfacings tend to shove. We, therefore, believe that there is a connection between the a.m. properties and wide joints:

See figure 11.

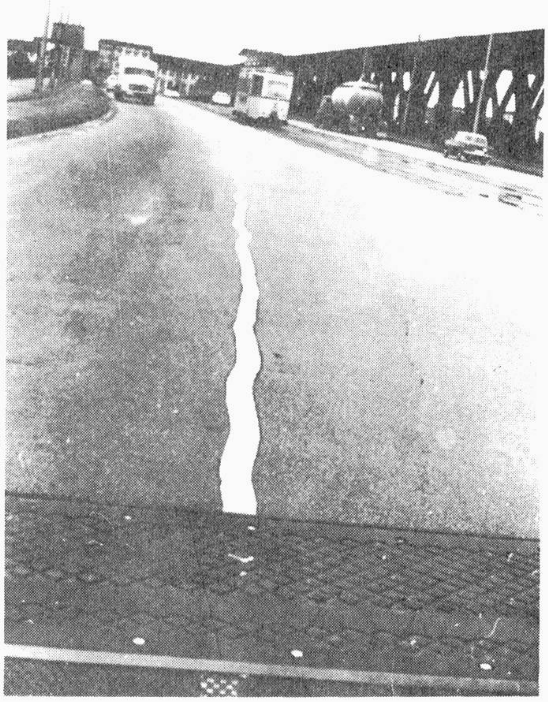


Figure 10: Shoving visible from deformed marking strip

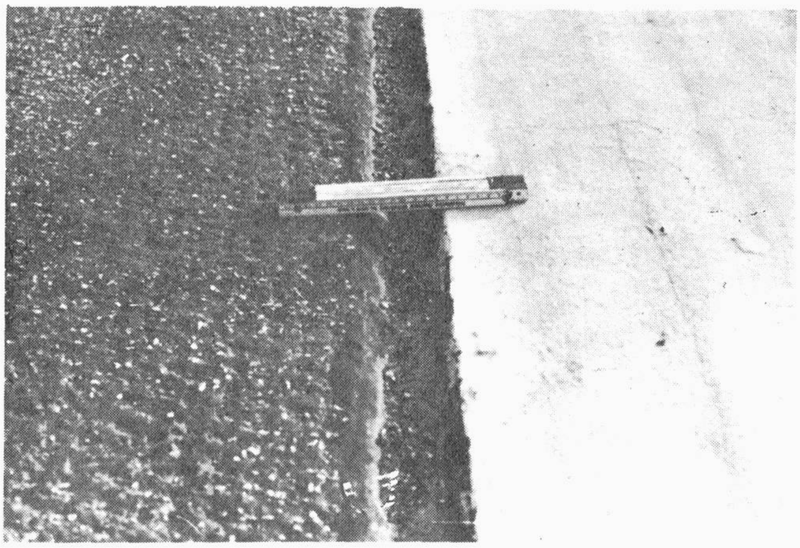


Figure 11: Enlarged joint (8 cm) at longitudinal connection

Across the axis the asphalt will not be prevented from contracting. Hair cracks will be excluded by its increased viscosity and the low stability will not hamper contraction. Warming up the surfacings will not bring back the particles to their original positions but, due to the increased tendency to yielding, the particles will shove away, i.e. upwards.

Wet Spots on Surfacing Top

Wet spots remaining on the surface some time after the last rainfall indicate that some big amounts of moisture have penetrated into the surfacing due to an improperly sealed wearing course. See figure 12. This has been noted on surfacings using a top layer of asphaltic concrete for better skid resistance. A content of de-icing agents will increase the danger of deterioration brought about by penetrated water. However, most of the bridges observed

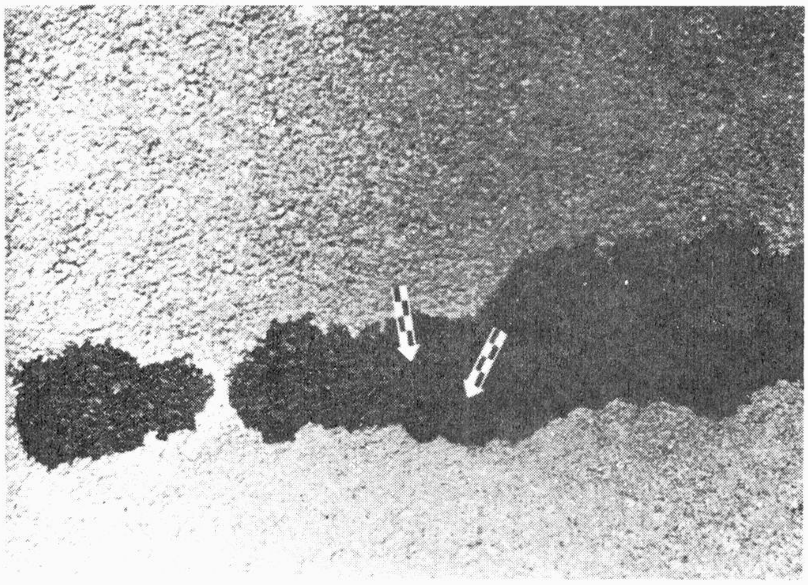


Figure 12: Wet area and visible holes (see arrows !)

have been opened to traffic only for short periods of time and are still performing well. It seems too early, therefore, to draw any final conclusion.

Unsatisfactory Bond or Shear Strength

Any surfacing will be destroyed within short time due to insufficient shear strength or bond. It has been tried, therefore, to dowel the surfacing to the steel plates by mechanical means, e.g. by welding mesh reinforcing steel on studs to the plate. This method has not been successful. Due to different thermal expansions of steel and asphalt square targets in size of the mesh appeared on the top surface, presumably indicating insufficient bond. Small furrows up to 2,5 cm = 1" deep may develop atop of the mesh rods, causing cracks in harder types of asphalt and, eventually, destruction in total. See figure 13.

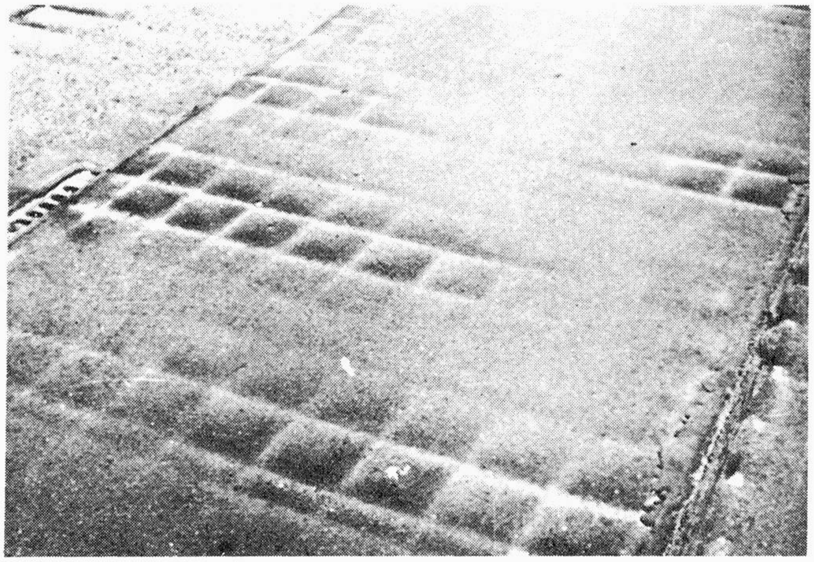


Figure 13: Steel mesh anchoring; plasterlike top surface

Zig-zag-anchors have performed better. They consist of zig-zag-shaped flat steel bars welded normal to the steel plate at 15 cm = 6" centers by staggered fillet welds. In 1951, the zig-zag-steel bars had been 28 x 6 mm = appr. 1-1/8" x 1/4" in cross section and at 8 cm = 3-1/8" centers on the Düsseldorf/Süd Bridge (Southern Bridge) over the Rhine River. Six years later, on the

Düsseldorf/Nord Bridge (Northern Bridge) the cross section was reduced to 22 mm x 6 mm = 7/8" x 1/4", the spacing being increased to 15 cm = 6". In 1964, the cross section was reduced even more to 17 mm x 5 mm = 11/16" x 3/16" on the Danube-Bridge near Wörth. In this case the zig-zags were welded on flat steel bars 20 mm x 3 mm = appr. 13/16" x 1/8", arranged in flat positions corresponding to the distances of stiffeners. The prefabricated grids were laid on the bridge-plate. On these types of anchoring the stone-filled mastic asphalt surfacing has performed well.

Against these devices it has been objected that spot welding will considerably decrease the fatigue strength of the steel plate. This objection does not stand. The actual stresses are less than the allowable fatigue stresses in any case. Furthermore, relevant tests made on a plate with anchoring flats have demonstrated that such objections are unjustified. However, the stresses produced in the steel plate will be closer to the allowable ones.

There are, however, some good reasons why anchoring bars should not be used in general. It is recognized that the welded-on bars do increase the stability of the surfacing and the stiffeness of the deck plate, but the increased dead weight of the deck should not be neglected completely. Furthermore the structure becomes more expensive. We have arrived at good results even without these anchoring bars, and we believe, therefore, that their use should be limited to a few exceptional cases.

5. Preliminary Laboratory Tests

The first effort to give preliminary guidance has been made by publishing, in 1961, a leaflet entitled

"Preliminary Specifications for Bituminous Surfacings on Light-Weight Steel Bridges"

"Vorläufiges Merkblatt für bituminöse Fahrbahnbeläge auf Leichtfahrbahnen im Stahlbrückenbau", Ausgabe 1961, based on the experiences gathered from surfacings on orthotropic plates in Germany during the first ten years. We had good reasons to call this leaflet a "preliminary" one, because we were well aware that our observations had not resulted in any profunded knowledge of the matter at that time.

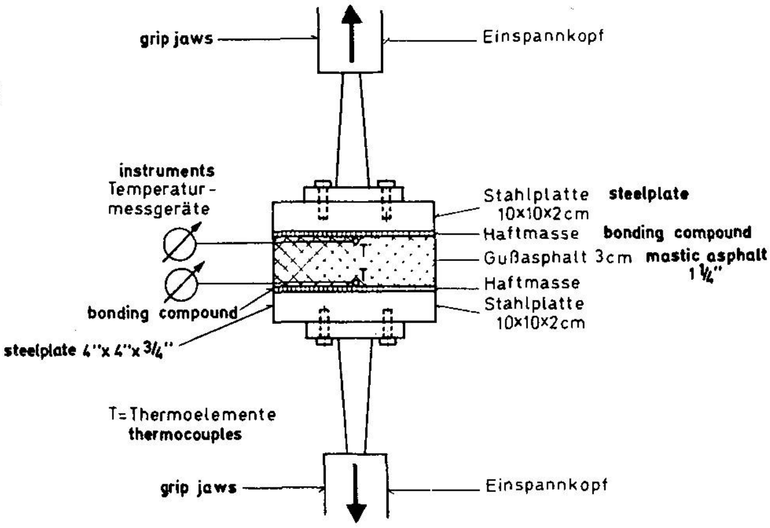
Anyhow, one fact came out clearly even then: It is extremly important to get a b o n d i n g c o m p o u n d having the following properties: permanent reliable adhesion to the steel plate as well as to the surfacing, and a sufficient sealing capacity. The Federal Road Construction Institute ("Bundesanstalt für Straßenwesen") has been carrying out tests aimed at this goal for about ten years. The chief purpose of these endeavours has been to evaluate, by simple mechanical tests, various bonding compounds and adhesive coatings on a comparable basis and to single out suitable materials for additional tests, both in the laboratory and in the field.

The tests conducted comprise the following:

Bond strength, direct pull normal to plate,
Bond shear strength and
Bond strength and ductility on bending around a mandrel

Figure 14 shows the pull-test schematicly.

Figure 14:
Arrangement for testing of bond strength (direct pull)



About 30 different types of surfacings have been tested. Failure stresses ranged from 15 to 38 kp/cm^2 = 210 to 540 lbs/sq.in at -10°C = 14°F and from 10 to 35 kp/cm^2 = 140 to 500 lbs/sq.in at -20°C = -4°F. Great importance has been attached to determinating the level of failure:

between the steel plate and the bonding compound/bond coat,

within the bonding layer,

between the bonding and the upper courses or

within the upper courses.

Again at -10°C = 14°F and -20°C = 4°F in the second test, a shear failure was brought about between the plate and the pavement by one heavy stroke with a hammer, and the line followed by the crack was recorded, but no strength figures were determined.

For the tests of bending over mandrels, carried out in succession at temperatures of +25°C = 77°F, 0°C = 32°F, -10°C = 14°F and -20°C = -4°F, 1 mm-sheet-metal strips, 3 cm wide, were each coated with bonding compounds/coats and bent through 180° maximum around mandrels of 50, 20, 10 and 5 mm dia = appr. 2", 3/4", 3/8" and 3/16". It was determined at which angle cracks occured and which lines they followed. In some cases the bonding compound broke loose from the sheet-metal strip.

As a result of these tests quite a number of bonding compound/bond coating materials were excluded from further considerations. Ultimate loads (bonding strength on direct pull) and the lines followed by the cracks were voted in particular. A comparison of the part results of these tests showed that the conditions of the bending test over mandrel had been too exacting for bonding compounds. The chief purpose of these tests had been to determine the behavior of the compounds at low temperatures, so they don't disclose anything about stability of the surfacings at high temperatures. And they do not show either that a rapid drop of temperature has more influence on cold-cracking of asphaltic surfacings than the lowest temperature arrived at.

The applied testing methods appear to simulate unsufficiently the conditions prevailing on surfacings under traffic and the results should be judged with some reservations, because nothing but relations are shown between the various bonding compounds tested. However, these tests are offering the advantage of a quick, low-cost preselection among the proposed bonding compounds/bond coatings.

6. Inventory of Asphaltic Surfacings on Bridges

In addition to the laboratory tests a general stock-taking of all important practical applications of the various bonding compounds and pavement-compositions became necessary. From this we expected essential findings, which appeared very important for future dealing with the problem. The inadequacies to be determinated should above all indicate the direction for additional tests wich we have started in the meantime.

Checking of the asphaltic wearing surfaces on 56 orthotropic steel deck bridges started late in 1964. The determined damage has been recorded in each case systematically as well as its supposed causes. Some of these bridges had been completed as early as 1949. Two bridges abroad have also been covered , the "Europa-Brücke" in Austria and the St. Alban's Bridge in Switzerland.

The records were screened twice and two basicly different criteria were applied:

First, the structural lay-out of orthotropic steel plates used was the basis and the performances of different pavements were checked accordingly.

Second, the pavement itself was in the centre of the studies. Its specification, time of service, traffic conditions and the effects of climate during application, were the chief criteria for its appreciation.

Bridges and pavements to be studied were classified according to the structural lay-out of its deck plates, the thickness of the plate (10, 12 and 14 mm = appr. 3/8", 1/2" and 9/16"), the spacing of the stiffeners ($\lesseqgtr$ 30 cm; $\lesseqgtr$ 1') and the direction of the ribs (longitudinal or transverse) being taken as chief criteria.

With respect to structural design of orthotropic plates it was first supposed that torsionally stiff full-web steel ribs of V, Y and especially trapezoid cross sections were superior to flat or bulb steel ribs, since latter, for lack of transverse stiffness, could not greatly reduce the elastic deflections of orthotropic plates.

A comparison of pavements supported by plates 10, 12 and 14 mm (appr. 3/8", 1/2" and 9/16") thick, on the one hand, and rib-spacings of less than, equal to and over 30 cm = 1' on the other hand did not permit the conclusion that surfacings had a longer life on thicker plates and more narrowly spaced ribs, i.e. when the elastic deflections of the deck plate are smaller.

There is no reason, in our opinion, to change the design of orthotropic plates hitherto used. We do not think, either, that increasing the stiffness of the deck plate by zigzag-bars will be necessary in general. This should be limited to bridges with steep longitudinal slopes.

We are convinced, that there is the chance of there being errors in the composition of, or more precisely, in the specification for, an asphaltic surfacing, as well as in the preparation and mixing procedures, and in the methods of application. Unsufficient knowledge of these factors did positively cause many surfacings to fail prematurely in former years. The performance of quite a number of newly laid asphaltic surfacings substantiates this supposition indicating that as to the asphalt there are many good chances still unknown and unexploited.

It is not at all aneasy task to find that pavement most suitable for the structural relations between, and for the composition of, its different layers. The requirements to have stability during summer and resistance to cracking in wintertime are contradictory and, therefore, a compromise only will be the best solution. The dominating factors are:

Structural lay-out of pavement,

Consistency of bitumen
(softening and breaking points),

percentage of binder-content in compound,

quality of mineral aggregate,

aggregate grading and

physical and chemical properties
of the surfacing layers.

As observed on 56 bridges certain types of damage are corresponding to special criteria of the surfacings. We believe, that an evaluation of certain materials should be possible to some extent as far as the suitability in general and the special applications they are be used for are concerned. In respect of the causes of damage we are referring to section 4.

7. Laboratory Tests at Stuttgart

Now, for additional systematical laboratory tests, an extensive program had to be fixed based an the results of the preliminary statical tests conducted by the Bundesanstalt für Straßenwesen (Federal Road Construction Institute), and on the knowledge gained in the field in the meantime. The main task consisted above all in various, mostly dynamical, tests subjecting the specimens to loading conditions which would simulate the reality as close as possible. The performance of bonding compounds had to be studied by fatigue tests subjecting them to fatigue loads at different temperatures in order to limit the number of currently available bonding compounds. For we are convinced that the fatigue strength of a wearing surface of good composition in other respects is in accordance to the quality of the bonding compounds.

The Otto-Graf-Institute of the Institute of Technology in Stuttgart has been entrusted by us with the carrying-out of this research program. In doing so we have proceeded on the provisional assumption that all of the other criteria for the pavement have in general already been established on the basis of sufficient experience, unless a special composition of the wearing surface is specified by the respective manufacturers of bonding compounds.

The steel plate must be sand-blasted to metallic white. A bonding compound and subsequently one layer of mastic, 8 mm = 5/16" thick are applied, followed by two courses of Gußasphalt (mastic asphalt) applied at 180°C = 360°F and 220°C = 430°F respectively, totalling 50 mm = 2" in thickness.

The following 8 bonding compounds have been tested:

1. Cold-applied bituminous primer (5 - 10 µ = 0,2 - 0,4 mils) according to AIB (Instructions for the sealing of civil engineering structures - Specification of German Federal Railroad).

2. "Okta-Haftmasse" (800 g/m^2 = 1,5 lbs/sq.yd.) - a bituminous, highly cyclical bonding compound, manufactured by Teerbau, Gesellschaft für Straßenbau mbH, Essen.

3. "Isotex" - an insulating and bonding compound, manufactured by Smid & Hollander, Hoogkerk, Netherlands, using unvulcanized rubber powder ("Pulvatex") of Rubber-Latex-Poeder-Compagnie N.V., Amsterdam. The Isotex system consist of a primer and a special type of Mastix, followed by two layers of mastic asphalt (Gußasphalt).

4. "Colzumix" cold-applied painting (250 g/m^2 = 0,46 lbs/sq.yd.), manufactured by Westfälische Mineral- und Asphaltwerke, W.H. Schmitz KG, Dortmund.

5. Coal-tar pitch epoxy-resin (700 = 28 mils), sprinkled with aggregate.
6. "Tenaxon" - a modified epoxy-resin, manufactured by C.Fr. Duncker & Co., Hamburg, to be applied in two layers sprinkled with basaltic chippings. The mastic asphalt is appplied directly to the second epoxy-resin layer as long as that is not hardened yet; there is no mastic course.
7. "Prodorit" - tar-epoxy-resin combination, manufactured by Th. Goldschmidt AG, Mannheim.
8. "VABIT"-insulation - a multi-stage system consisting of three coats based on tar-epoxy-resin, and of one VABIT insulating layer, 1 cm = 3/8" thick. On this one binder course, 3 cm = 1/4" thick, and one VABIT wearing course, 2 cm = 3/4" thick, are applied.

Apparently the composition of surfacings using the no. 3, 6 and 8 bonding compounds differs from that of the remaining types.

The laboratory equipment and the testing methods were checked by several preliminary tests.

The surfacings to be tested were applied in a width of 15 cm = 6" over the total length of steel plates which were 70 cm = 28" long, 20 cm = 8" wide and 12 mm = appr. 1/2" thick. See figure 15. These strip-shaped samples were supported by three rollers spaced at 2 x 30 cm = 2 x 12" in the form of two span beams. The whole arrangement was quite a realistic spot sample, similar, by sufficient approximation, to a strip-shaped transverse cutting of an orthotropic plate. The fatigue flexural tests, planned for the first series, have been conducted on these specimens up to 2 x 10^6 cycles or until failure by applying two equal, increasing loads, arranged slightly out of midspan and 35 cm = 14" apart.

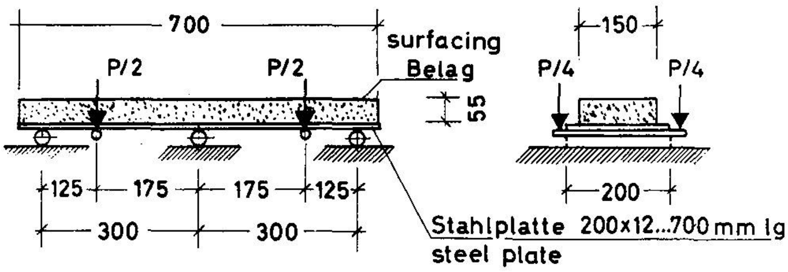


Figure 15: Arrangement for testing fatigue strength by continous flexing

It was determined by a preliminary test that the surfacing did not stand an appropriate load applied directly to its top surface. The loading props penetrated into the surfacing, squeezing it aside at the same time. So we turned to an indirect application of the loads, transmitting them to the protruding edges of the plate. The deflections were measured by dial gages, arranged at the same points and having a read-out precesion of 0,01 mm = 0,4 mils.

In order to determine the composite action of the steel plate and the surfacing, statical tests were run first. For this the applied load was set to a magnitude which, when slowly increased, eventually caused a deflection of 0,5 mm = 2 mils, equal to 1/600 of the span. The same deflection under the same load was reached on a bare plate, without any surfacing, which showed that the surfacing was statically ineffective under slowly applied loads. The bituminous surfacing adapted itself by creeping to the new condition, so that the composite action was lost, when a certain time after load increase had elapsed.

Minimum load and maximum load for the following d y n a - m i c t e s t s were set to such magnitudes that the resulting range of load (increasing load) corresponded to the load applied for the statical test.

At first time all tests were conducted at room temperature (+20°C = 68°F). At the beginning of the test, run at a relatively high frequency of n = 300 cycles per minute, a considerable composite action of the surfacing was observed. The total deflection was only 27% of that measured previously during the statical test (ℓ/600). The surfacing broke loose from the plate at 7 x 10^5 cycles and the test had to be terminated.

It was the purpose of a second test, conducted at about half of the previous frequency (n = 160/min), to reach a deflection of ℓ/600 by increasing the maximum load. The deflection, however, was raised from 27% to 65% only, in spite of a tripled load. The surfacing broke loose after few cycles, when the load had been increased again to get a deflection of 0,5 mm = 20 mils, equal to ℓ/600.

As shown by this test the bituminous surfacing continously loaded at 160 cycles per minute is far from following a deflection of 1/600 of the spacing of the longitudinal stiffeners.

From this fact we concluded that the increasing load, used in the previous test, had been appropriate for our planned flexural fatigue tests except that for the following tests a frequency of n = 160/min should be used. It is doubtfoul, wether or not this frequency corresponds to the actual conditions on bridges. Measurements, therefore, are being conducted now and their results will be taken account of in the next test series.

At room temperature the number 1 to 7 surfacings have been tested twice up to now. The results do not reveal much, because the performance of both surfacings and bonding compounds re-

respectively will still be tested at extreme temperatures of $-20^{o}C = -4^{o}F$ and $+60^{o}C = 140^{o}F$. The results of these test will be much more significant then those obtained at room temperature.

When the testswere started, deflections from 0,12 mm = 5 mils = $\ell/2500$ to 0,29 mm = 12 mils = $\ell/1030$ were obtained. The numbers of cycles sustained by the materials were different as well:

> Whilst the no. 5 surfacing (coal-tar-pitch-epoxy) broke loose from the bond coat immediately after starting pulsation, the no. 6 surfacing (Tenaxon) did sustain $1,4 \times 10^{6}$ cycles prior to losing bond. After $2,4 \times 10^{6}$ cycles the surfacing had broken loose on one span and the deflections had been increased on both spans up to 0,22 mm = 9 mils and 0,45 mm = 19 mils respectively. It was observed later that the shear failure occured within the mastic asphalt course leaving a thin layer of mastic asphalt over the total area of the Tenaxon bonding compound.

The tests at room temperature will be repeated at lower frequencies; only after that the performance of the surfacings will be studied at extreme temperatures.

We expect that we shall have to deal with flexural fatigue tests for a long time yet. It seems to be of little use to arrange for details of further tests prior to having obtained results from the fatigue tests. We just want to say, that we intend to enlarge our test program by the following test groups:

Rutting tests,
Shoving tests,
Dropping-ball test
Blistering tests
Temperature tests and
Test, as to protection against corrosion.

8. Measurements of Temperatures

It was the purpose of additional tests conducted by the Railroad and Highway Construction Institute of the Munich Institute of Technology to clarify, wether there is any connection between the many cases of rutting and temperature variations, presumably caused within the surfacing by the radiator effect of the longitudinal stiffeners. A test plate has been made 8,8 m = 29'-4" long, 2,7 m = 9' wide; stiffeners on both halves of the deck plate, 12 mm = appr. 1/2" in thickness, were trapezoidals and flat steel bars, respectively. For reading and recording temperatures all the year round, thermocouples were installed at different depths of the surfacing and underneath the plate. See figure 16.

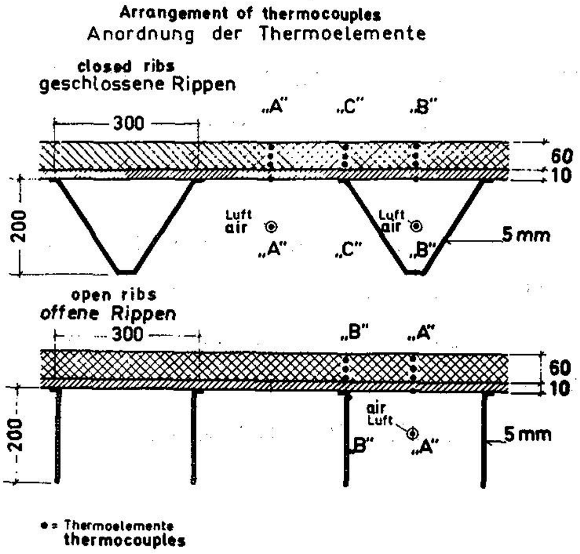


Figure 16:

Measuring temperatures of a test plate

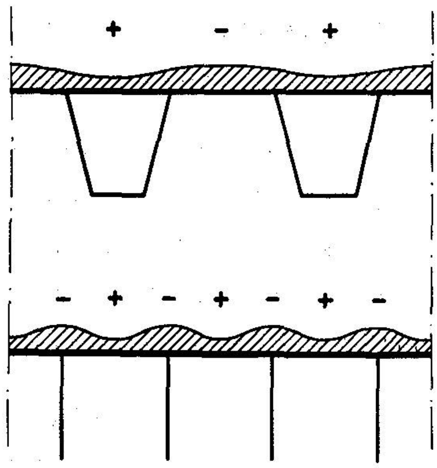


Figure 17:

Waving due to temperature differentials

For the trapeziodal ribs the measuring points were arranged in three vertical cuts, one each between the ribs, at one connection and on the centerline of the trapezoid. For the flat steel bars only two vertical cuts were made, arranging the thermocouples amidst and on top of the ribs.

The results of the first measurements, carried out in the autumn and winter of 1966/67, showed that during some autumn-days with ambient temperatures of more than $+25^{\circ}C = 77^{\circ}F$, the temperatures measured in the lower surfacing course above the rib centre-line on the plate portion with trapezoidal ribs were higher by about $2^{\circ}C = 3{,}6^{\circ}F$ compared with those between the ribs. Temperatures on the plate portion stiffened by flat steel bars were higher amidst than atop of the ribs. In this case the difference of about $3^{\circ}C = 5{,}4^{\circ}F$ was even more distinct.

The a.m. figures were measured on the lower course of the surfacing; readings from the upper course were half that high only.

By this test our opinion has been confirmed, that the magnitude of the supposed temperature differentials can be measured. These differentials were lasting for up to 5 hours, a fact which appears to be very important.

As a result the modulus of elasticity varies significantly. In the warmer zones, the asphalt is prone to become plastic and will be squeezed towards cooler areas by the moving wheel loads. That means: The wave-crests will i n b e t w e e n t h e r i b s in the case of closed rib systems (trapezoidal, V- and Y-shaped ribs), but a t o p o f t h e r i b s for open ribs (flat or bulb steel bars). See figure 17.

The final report, containing the measurements made during the spring and summer of 1967 has not been completed yet. On the basis of some details already known one may conclude that temperature differentials during summer are about twice those measured in the preceding fall.

9. Field Tests

Looking for the wearing surface most suitable on an orthotropic deck plate we have not limited our endeavours to laboratory tests only. They have been paralled by observations in the field and the results have been incorporated in the previous reports, containing a stock-taking of our bridges as well as an attempt, to evaluate the performance, taking into special consideration the period of time elapsed since opening for traffic, and the concentration of traffic. We want to get confirmation by field tests of the results newly obtained at the laboratories. Field tests, therefore, have been started again on some bridges erected during the two preceding years. At first we shall test various bonding compounds, the composition of the surfacing being as closely as possible the same in other respects. A few exemples are given below:

The "Colzumix" bond coat applied in 1965 on the first section of the S t e p h a n i e B r i d g e , B r e m e n , has been compared to the "Okta" bonding compound applied on the second section during the last year.

On the upstream platform of the B r ü c k e ü b e r d i e N o r d e r e l b e (bridge over the northern part of the Elbe River) - located in a new section of Bundes-Autobahn (Federal super-highway) called "Southern Hamburg Bypass" - the first surfacing had been applied in 1962 according to the "Möllerit"-method. A partial replacement became necessary during 1966 and has been made using the "Durastic"-solution of Duncker & Co. for the bond coat. Now, resurfacing of an other section has been done by using Okta-Haftmasse.

Finally, on the new bridge over the Rhine River at R e e s (see figure 18), zig-zag-anchoring bars 20 x 6 mm = 3/4" x 1/4" at 15 cm = 6" centers have been placed on one half of the main span. Four types of pavement using "Isotex" and "Okta" bonding compounds will be compared:

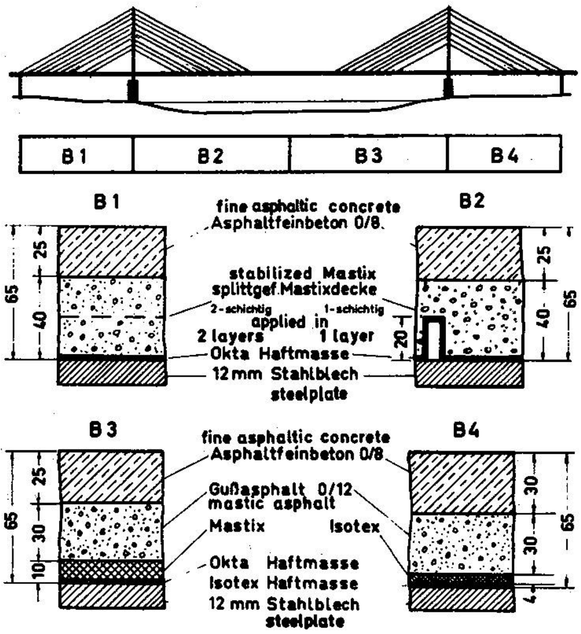


Figure 18: Field testing of various surfacings on the Rees Bridge over the Rhine River

1. "Okta-Haftmasse"; two layers of stone filled mastic surfacing, 40 mm = 1-9/16" thick in total;
2. "Okta-Haftmasse"; stone filled mastic surfacing, 40 mm = 1-9/16", on zig-zag-anchoring bars;
3. "Okta-Haftmasse"; 10 mm = 3/8" mastic; 30 mm = 1-3/16" mastic asphalt (Gußasphalt);
4. "Isotex"-insulating and bonding compound, 4 mm = 5/32" thick; 30 mm = 1-3/16" mastic asphalt (Gußasphalt).

On top of the a.m. surfacings one wearing course of fine aggregate asphalt concrete is applied, 25 mm = 1" thick for the no. 1 to no. 3 surfacings, and 30 mm = 1-3/16" thick for no. 4 surfacing. The total thickness is 65 mm in any case.

However, it is a common drawback of such tests that results normally are obtained but gradually and after quite a number of years. And: there should be at last one summer and winter with extreme temperatures since the completion of the wearing surface. Otherwise criteria for performance are not available.

10. IABSE Inquiry

On the meeting of IABSE's Working Commission II (steel construction) held in Ankara on September 5, 1966, it has been decided to work out a questionary dealing with wearing surfaces on light-weight steel bridge decks, and to send it to all National Groups. Anwers received from 12 countries have been transmitted by the German Group for evaluation to the reporters on the Symposium's three Themata.

Anwers to questions ranging within the scope of Theme I (Asphaltic Wearing Surfaces 1" to 3" thick) have been tabulated below with no comment. The fact that anwers have been received from 12 different countries clearly shows that there is a widespread common interest in solving the problem under review. The other fact that several questions could not be answered underlines the need of starting a world-wide exchange of experiences gained in such an important field within the scope of modern bridge design.

The questions belonging to Theme I are:

1. Which construction methods and "recipes" (for instance, asphalts or rolled asphalts) do you think appropriate for carriageway surfacings?
2. Is there any possibility for a durable marking of those surfacings?

3. What is the behavior of the upper layer with respect to

   a. Roughness

   b. Resistance to wear

   c. Chemical resistance
      (salts, water, oils, gases) ?

4. How is the stability and crack resistance at extreme temperatures ?

5. What is the behaviour of the surfacing with respect to shrinking and creeping ?

6. What is the behaviour of the surfacing with regard to aging and fatiguing ?

7. To what extent is the surfacing sound insulating and heat resistant ?

8. Is the surfacing permanently insensitive to unequal elastic bedding (formation of waves) ?

9. To what extent is the surfacing load distributing

   a. during a short-time loading

   b. during a long-time loading ?

10. How do you judge tne composite action with steel deck plate ?

11. Can the surfacing by faultlessly repaired if local damage may occur ?

12. Does the surfacing offer a sufficient protection against corrosion for the carriageway plate ?

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DENOMINATION						COMPOSITION OF BRIDGE-SURFACING		PROPERTIES												REMARKS
						question 1		question 2	question 3		question 4	question 5	question 6	question 7	question 8	question 9	question 10	question 11	question 12	
crt. no.	country	reporter	no.	structure location	thickn. cm	bonding and insulating layer	surfacing	marking	resistance to skid and wear	resistance to chemicals	stability and resistance to cracking	shrinking and creeping	ageing and fatiguing	sound- and heat-insulating	waving, rutting and shoving	load-distributing, short- and long-time	composite action	ability to be repaired	protection ag. corros.	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
1	CANADA	The Government of the Province of British Columbia; Department of Highways	1.1	Port Mann Bridge near Vancouver	5,5	Sandblasting; red-lead-epoxy primer; coal-tar-epoxy bonding coat sprinkled with chippings	1/2" levelling course: sand asphalt containing 8% air voids max.; bituminous bond coating; 1-1/2" wearing course: asphalt concrete containing 6,5% of bitumen and mineral aggregate up to 1/2"	---	sufficient	sufficient	no difficulties up to now	see col. 12	see col. 12	not known	not sensitive up to now	---	---	yes, repairable	yes, sufficient	steel deck plate: 7/16" to 1/2" thick minimum asphalt concrete according to specifications of Department of Highways
2	DENMARK	Ministry of Public Works, Road Dept.	2.1 2.2	Langebro bascule bridge Guldborgsund bascule bridge Kong Frederik IX's bridge	5,0	cleaning by steel-wire brooms; removing loose rust and shale; bitumen coating, containing 5% rubber additive (250 g/m²)	2 cm soft mastic asphalt containing bitumen 10% 2,5 cm mastic asphalt containing bitumen 8% Trinidad-Epuré 2% limestone filler 45% grit (2/6 mm) 45% bituminized chippings (12/15 mm) additive	yes	at 50 km/h value of friction μ = 0,6 to 0,7; wheel inclined and on wet surface	---	performing very satisfactory; no shoving, no rutting, no waving	---	---	---	---	---	---	yes, not necessary up to now; repairs according to spot samples o.k.	insulation still o.k. on spot-samplings	steel deck plate: 13 mm minimum thickness; stiffeners at 50 cm centers; anchoring device: 2o mm by 10 mm flat steel bars, welded to deck plate
		Port Authority	2.3	2 movable bridges in Copenhague Port					as on normal road pavements		as on normal road pavements	see col. 12	see col. 12	no experience; no measurements	no waving since 13 years	at 45° angle, when loaded instantanously	very satisfactory			
3a	GERMANY, EASTERN	Kammer der Technik F.U.A. Straßenbrücken; Berlin - Ost	3.1		6,0	sprayed zinc coating, sometimes combined with aluminum-coat 50 microns or: Mastix (asphalt mastic) shear forces are poorly transferred by insulating foils; surfacing shoves	2,5 cm binder course of stone-filled Mastix containing bitumen B 200 2,5 - 3,0 cm wearing course of asphalt concrete or: 3,0 cm binder course containing bitumen B 200 2,5 cm wearing course of fine aggr. asphalt concrete	yes, white Mastix 3 - 5 mm th. or: epoxy-trowel-compound	same as normal road pavements; may be improved by indentations or sprinkling with bituminized chippings	---	resistance to cracking more important than stability; cracks are caused by high-frequent vibrations	no experience	see col. 13	no measurements taken	shoving ahead of expansion joints, when improperly applied or using too soft mastic asphalt	not tested	not accounted for, because not existing at high temperatures	yes, when carefully done, using painting materials and heating devices	resistance to cracking not sufficient; protection against corrosion, therefore, necessary	steel deck plate: 10 mm minim. thickness using torsional-rigid longitudinal stiffeners at ≦ 300 mm centers and transverse beams at ≦ 3 m centers; anchoring device recommended on slopes steeper than 3% only.
3b	GERMANY, WESTERN	Bundesverkehrs-Ministerium (Ministry for Traffic), Bonn	3.2		6,5	bonding compound or bond coating; course of Mastix	2 layers of mastic asphalt	Yes inserting thermo-plastic material	same as normal road pavements	sensitive against oils; oil-drippings are not dangerous	even at extreme temperatures good results prevailing, when carefully applied	bituminous surfacings with rubber-additive tend to shrinking; creeping in laboratory test only observed	performing like road pavements	sound - and heat-insulating quality improved	depending on "receipe", composition of surfacing and on structural features of orthotropic plate	not depending on loading time; depending on thickness, composition and temperature of surfacing	depending on loading time and temperature (modulus of elasticity) furthermore on efficiency of bonding layer; good composite action, in the cold and under instantaneous load	yes to be done easily with no objections	surfacing only not sufficient; insulating and protecting layer is necessary (Mastix)	- - - - -
	UNITED KINGDOM	Scottish Development Department; Road Research Laboratories	4.1	Forth-Bridge	6,0	grit blasting to "white" metal finish; sprayed zinc coating, followed or not by a protecting coat of bitumen; "Bostic 1255" bond coating	2 mm course of Mastix containing bitumen 23% Pulvatex 2% limestone filler 75% 4 cm asphaltic Mastix acc. to B.S. 1447 containing bit. chippings up to 18 mm or: rolled asphalt acc. to B.S. 594	yes	performing well when materials are compsed carefully	---	no objections at temperatures between - 10°C and + 50°C	no objections	---	heat-insulation of surfacing 5 cm thick amounts to appr. 10°C	-----	at low temperature load distributing and composite acting; at high temperature negligible	composite action not considered	yes	additional insulating layer highly desirable	B.S. : British Standard
			4.2	Severn-Bridge		o.1" cushion layer of a rubberized mastic	1-1/2" mastic asphalt acc. to B.S. 1447/1; 3; containing chippings up to 3/4".													

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		DENOMINATION				COMPOSITION OF BRIDGE-SURFACING					P R O P E R T I E S									REMARKS
						question 1		question 2	question 3		question 4	question 5	question 6	question 7	question 8	question 9	question 10	question 11	question 12	
crt. no.	country	reporter	no.	structure location	thickn. cm	bonding and in-sulating layer	s u r f a c i n g	marking	resistance to skid and wear	resistance to chemicals	stability and resi-stance to cracking	shrinking and creeping	ageing and fatiguing	sound- and heat-insulating	waving, rutting and shoving	load-distributing, short- and long-time	composite action	ability to be repaired	protection ag. corros.	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
4	UNITED KINGDOM	Husband & Co. Consulting Engin-eers, London	4.3	---	--	removing of all loose dust and dirt; sprayed metallic coat; tack coat of "Bostic C"; 1/8" ins. layer of rubberized filled bitumen. insulating foils render difficult bonding	1-¼" wearing course of mastic asphalt according to B.S. 1447/1 col.3 including 40/45% of coarse aggregate	---	---	---	---	--	---	---	insulating layer must be neither too thick nor too soft	angle of load-distri-bution is assumed to be 45° according to B.S. 153	stiffness of steel deck plates, de-signed to carry B.S. wheel loads, is sufficiently high; composite action no problem	---	---	B.S. : British Standard anchoring devices required on very steep slopes only
5	INDIA	The Central Road Research Institute, New - Delhi	5.1	road and railroad-bridge over Jamuna River near Delhi	--	---	mastic asphalt surfacing containing bitumen, lime-stone filler and chippings	yes, using sterene-based paints; but not durable	good, by spreading and tamping aggregate while the mastic is still hot	good against water and salt; poor against natural oils and gases, due to solubi-lity of asphalt	the surfacing is both stable and crack resistant at extreme tem-peratures	no inconvenien-cies observed over a period of 3 years	not very appreciable so far	reasonable	reasonably insensitive	---	composite action quite good	yes	yes; effective protection against corrosion	3 years of service time
		Government of Maha-rashtra, Bombay; Designs Circle	5.2	---	--	---	asphalt concrete or cement concrete					good resistan-ce to shrinkage	asphalt oxid-izes and is rendered brittle	not tested	asphaltic surfacings form waves	---	not to be considered			- - - - -
6	THE NETHER-LANDS	Rijkswaterstraat Directie Bruggen, Den Haag	6.1	---	5,0	protecting coat: bitumen B 150/250 0.8 - 1,0 kg/m² bonding coat: bitumen 95% Pulvatex 5% mastic asphalt: bitumen 20/30 19,0% filler 39,5% aggregate 41,5% insulating foils are not required	2,0 cm Gußasphalt (mastic asphalt) cont.: bitumen 20/30 4,0% bitumen 50/60 4,0% Trinidad asphalt 8,0% filler 24,0% sand 40,0% fine grit 20,0% 2,2 cm Gußasphalt (mastic asphalt) cont.: bitumen 50/60 5,5% Trinidad asph. 5,5% filler 16,0% sand 33,0% grit 2/5 mm 40,0% sprinkled with bitum-inized chippings 5 - 6 kg/m²	yes	resistance to skidding not high; (μ= 0,45 to 0,50) light tracks after 10 years under traffic	good against water and salt; sensitive to oils;	at extremely high temperature indentations caused by vehicle tires; sensitive to cracking at low temperature, especially on unequally stiffened spots	good, as far as shrinking is concerned	no experience	very well	no; waving on spots of disconti-nuity of the plate e.g. at plate joints	very good during a short-time loading; not existent during a long-time loading	good composite action with surfacings properly designed and applied	yes	yes; insulating layer pro-vides for adequate protection against corrosion, even in cases where there are cracks in the surfacing	anchoring devices to be rejected on account of discontinuity produced
7	AUSTRIA	Tiroler Landes-Regierung, Landes-baudirektion, Innsbruck	7.1	Europabrücke, Innsbruck	6,0	sandblasting; hot sprayed "Okta-Haftmasse" (bonding compound)	3,5 cm stone-filled asphalt mastic (Mastix-Asphalt) 2,5 cm fine aggregate asphalt concrete according to "VABIT"-method	yes; e.g. with "Helabit"	with "VABIT"-sur-facings good experiences	---	no inconvenien-cies observed during 3-½ years	see col. 12	see col. 12	appropriate insulation by 6 cm thickness of surfacing	no waving observed so far	load-distribu-tion to be ex-pected during short-time loading only	composite action due to zig-zag-grids	yes	time of service still too short	steel deck plate using welded-on zig-zag-grid
8	SWEDEN	Secretary of the Swedish Group	8.1	movable bridges: 5 bascule bridg. 1 lifting bridge 1 swinging bridge	5,0	sandblasting; insulating foils render difficult bonding; sprayed zinc coating 150 microns thick	5,0 cm asphaltic surfacing cont.: bitumen B 55 11,0% Trinidad-Epuré 2,5% rubber powder 6,5% limestone filler 16,0% sand 32,0% hard rock chipp. 32,0%	---	experiences do not allow for final conclusions	see col. 10	see col. 10	see col. 10	see col. 10	---	---	---	using round steel bars for anchoring device	no significant repairs necessary so far	---	steel deck plate: 12 mm minimum thickness; longitudinal stiffeners at 300 mm centers; 18 mm ∅ round steel bars, 60 mm long, welded-on at 200 mm centers
9	SWITZER-LAND	Tiefbauamt of Basle Town	9.1	St. Alban's Bridge, Basle	6,0	8 mm asphalt Mastix	2,5 cm levelling course of mastic asphalt (Gußasphalt) 3,0 cm wearing course of hot bituminous mixture	yes; frazed-in marking "Signo-falt"	good	good	good, so far	not known	not known	depending on thickness of surfacing	almost no shovings	not known	seems to be good during short-time loading	yes	yes	- - - - -

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crt. no. 1	country 2	DENOMINATION reporter 3	no. 4	structure location 5	thickn. cm 6	COMPOSITION OF BRIDGE-SURFACING question 1 bonding and insulating layer 7	surfacing 8	question 2 marking 9	question 3 resistance to skid and wear 10	resistance to chemicals 11	PROPERTIES question 4 stability and resistance to cracking 12	question 5 shrinking and creeping 13	question 6 ageing and fatiguing 14	question 7 sound- and heat-insulating 15	question 8 waving, rutting and shoving 16	question 9 load-distributing, short- and long-time 17	question 10 composite action 18	question 11 ability to be repaired 19	question 12 protection ag. corros. 20	REMARKS 21
10	U.S.S.R.	Ministry for Traffic, Central Administration for Bridge Building	10.1	movable bridges and some fixed bridges	--	not concerned with; bitumen coating	as specified by special construction firms	---	no observations	no observations	not required by administration for bridge-building	---	one surfacing performed well during 30 years	not investigated	---	no observations	---	comp. action by anch. bars; stiffnes produced by c.a. unknown; not to be cons'ed	---	steel deck plate and anchoring bars made of stainless steel
11	UNGARY	Secretary of the Ungarian Group	11.1	Elizabeth-Bridge, Budapest	7,5	sandblasting; hot sprayed zinc coating with 2 layers of 50 and 150 microns respectively 1 kg/m² rubberized bitumen-coating 5 - 8 mm rubberized bituminous mortar, containing: bitumen B 144 14,8% rubber powder 1,2% limestone powder 42,0% fine sand 42,0% insulation by aluminum foils difficultly to apply; to be used for mastic asphalt surfacings only.	3 - 4 cm rolled-on asphalt, cont.: bitumen B 144 5,5% limestone powder 6,5% fine sand 4,6% coarse sand 9,6% basaltic chippings 0/5 mm 12,3% 5/12 mm 21,1% 12/20 mm 40,4% 3 cm mastic asphalt, cont.: bitumen B 45 9,0% rubber powder 0,7% limestone powder 22,0% fine sand 9,4% coarse sand 13,6% basaltic chippings 0/2 mm 9,1% 0/5 mm 9,0% 2/5 mm 13,6% 5/10 mm 13,6%	yes; using chlorium-rubber for marking in order to avoid cracking	skid-resistance produced by rolling-in of chippings (10 to 15 mm), mixed with 1,5 to 2,0% of bitumen B 90; about 1 to 2 mm per year worn off under traffic of 20 000 tons/day	good against water, salts and oil-drippings; dissolvable by gasoline and light oils	cracking occured already during the first winter on areas, where the mastic asphalt had been applied in 2 layers; on the other hand the mastic asphalt applied on binder-course is still o.k. after several years of service	service-time of 3 to 4 years is still too short; no experience yet	test results of Research-Institute only have been evaluated; specification for surfacing written accordingly	not investigated	no waving observed so far	tests conducted without results, because composite action and load-distributing quality cannot be separated	see col. 17	no experience	surfacing is not providing for adequate protection against corrosion; top surface of steel deck plate, therefore, to be cleaned by chromium-acid and sprayed with zinc-coating	steel deck plate: 12 to 14 mm minimum thickn.; stiffeners spaced at 300 to 340 mm centers; opinions based on results of Institute of Research; anchoring device, e.g. zig-zag-grid, on slopes steeper than 4% only
12	U.S.A.	State of California, Dept. of Public Works, Division of Bay Toll Crossings, S.Francisco	12.1	San Mateo - Hayward Bridge San Francisco, California	--	epoxy bonding coat asphaltic epoxy	5 cm asphalt concrete, containing coal-tar and resin	yes	not determined; satisfying	---	not investigated	satisfactory so far	see col. 13	see col. 13	yes	distribution of load at non-plastic condition only	composite action determined by flexural fatigue test	not known	yes, provided by zinc-coating	surfacing applied on 37 500 sq.m of orthotropic steel deck plate
		State of California, Dept. of Public Works, Division of Highways, Sacramento	12.2	---	5,5	hot sprayed zinc coating 1/4" coal-tar-epoxy bonding layer, sprinkled with chippings	5 cm asphalt concrete containing coal-tar and resin	yes, using white asphalt	highly depending on mixture and mineral aggregates	sensitive to gasoline	may tend to waving at high temperatures	no experience yet	see col. 13	see col. 13	see col. 12	no experience	existing on surfacings free of crackings	yes, but no experience	surfacing only not sufficient	surfacing applied on 900 sq.m of orthotropic plate of 1 structure only
		State of Illinois, Dept. of Public Works and Buildings, Springfield	12.3	Poplar-Street-Bridge, St. Louis, Miss.	--	sandblasting to metallic white; zinc coating; coal-tar-epoxy bond coating (2 coats), sprinkled with chippings	bond coating of bitumen with fluid latex additive; levelling levelling and wearing course: 5 cm asphalt concrete cont. latex additive	yes, using marking paint	no experience	---	good stability is expected; hair-cracking will be possible	no difficulties expected according to test-results	see col. 13	see col. 13	no problem expected	not to be considered	not to be considered	yes, readily and adequately	no; surfacing does not offer sufficient protection	according to tests and experiences made on Troy Test Bridge during 4 years of service-time
		Bethlehem Steel Corporation, Bethlehem, Pa.	12.4	Humphreys-Creek-Bridge	--	metal-foils and plastic films rejected	5 cm asphalt concrete acc. to specification of the Maryland State Road Commission, Art. 33.12 or: 5 cm asphalt concrete with elastomer type asphalt cement	---	---	---	---	---	---	---	---	---	not to be considered	yes, most of the surfacing systems	special protection is required	ARBA and AASHO committees are working on new specifications for surfacings

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11. Results and Conclusions

Up to now there are no final results available from both laboratory and field tests respectively. We can, therefore draw but provisional conclusions based on observations made by us during nearly two decennia. These conclusions have to be discussed time and again and, if necessary, revised. As far as we know, a suitable pavement, as widely agreed, should be constructed as follows:

The surface of the steel plate, after s a n d - b l a s t i n g to metallic white, has to be protected against moisture and dirt.

The bonding compound (or bond coat) applied on the prepared plate has to provide for a shear-resisting bond of the surfacing with the steel plate.

A special s e a l i n g l a y e r is required, because we have not found, up to now, any bonding compound also providing reliable p r o t e c t i o n of the steel plate against c o r r o s i o n . This sealing layer likewise will have to protect the bonding compound underneath against heat-convection from the wearing surface proper to be applied at high temperatures.

Normally, the sealing or insulating layer consist of asphaltic mastic, 8 to 10 mm = appr. 5/16" to 3/8" thick, containing about 15% by weight of bitumen. Sometimes rifled strip-metal has been used.

In general, on top of the insulating layer a wearing surface of mastic asphalt (Gußasphalt), 5 cm = 2" thick, is used today and applied in two courses. The lower or levelling course compensates for out-of-planenesses of the steel deck, inevitable in steel construction. Being a wearing course the upper or surface course must contain an harder bitumen.

Either one of the Gußasphalt-layers contains about 8 to 9% by weight of bitumen.

In deviation from this type of construction most frequently used in Germany one binder course and one course of fine aggregate asphalt concrete may, instead of Gußasphalt, be used for both the lower and upper course respectively.

The so called stabilized (or stone filled) mastic system, as developed by Teerbau GmbH, Essen, is of a fundamentally different type of construction compared to the afformentioned ones. It is applied in one layer or in two, and Okta-Haftmasse (bonding compound) is used in either cases.

The field tests now under way on the Rees bridge over the Rhine will determine whether or not zig-zag-steel bars are useful.

Searching for the most suitable bonding compound still remains the central point of our considerations. The different bonding compounds/bond coatings tried out by us may be classified into two groups according to physical behavior:

The hard and shear-resistant compounds on the one hand, and

the thermoplastic compounds, transferring limited shear forces only, on the other.

As shown by our investigations, the first group of products, i.e. bituminous varnishes, resin coatings, zinc paints with a binder consisting mainly of resin, and hot-sprayed zinc coatings, cannot, in the long run, cope with the shear stresses produced by braking and accelerating vehicles as well as by the elastic deflections of the deck plate. The stresses are extremely high at low temperatures since then all of the asphalt loses its thermoplastic behavior, and composite action has its full effect. Either the pavement springs loose from the steel plate, or the asphaltic surfacing is separated from the bonding compound. But splitting-up of the zinc layer for instance may occur as well. Then, the deflection of the steel deck is increased, due to lack of composite action, to a degree the superimposed asphaltic surfacing cannot cope with. Destruction then is just a matter of time.

The second group comprises soft bonding compounds having little resistance to shear forces, e.g. the Isotex-Masse containing rubber powder (Pulvatex), and the Okta-Haftmasse. For about 15 years the Okta-Haftmasse has been used in many pavements, and experience has been good in general.

It may well be possible that special, if not highest, importance must be attached to a property of bonding compounds that has not been studied by us so far: The ability to regenerate an effective and reliable bond as soon as, after a period of separation from the superimposed wearing surface, the thermoplastic condition has been regained. However, this ability to regeneration may be expected in bonding compounds of the second group only.

Checkered copper or aluminum foils, frequently used during the fifties, are rarely installled today; plastic films and intermediate layers made up of paper or of bituminized felt are no longer used. It is not even understandable why the risk of later damage should be taken as long as there are other means to positively protect the steel plate against corrosion. Foils and films may be damaged very easily during installation and cannot be used at all for rolled-on asphaltic surfacings. This opinion is shared by other countries, as shown by the inquiry of IABSE.

Furthermore, it is widely agreed that using zig-zag-steel bars will be suitable only for pavements on bridges with a steep longitudinal slope or on bascule bridges.

The problem of the correct composition of che upper layers of a pavement has not been solved yet. This will largely depend on the climate of the location as well as on application methods and mechanical equipment normally used for pavement construction in the respective countries. In case Gußasphalt for instance should finally prove to be the material most suitable for bridge surfacings, nobody will expect the North American road construction industry, exclusively used to rolled asphalt for asphaltic roads, to switch to Gußasphalt.

Although things are still developing, it may be said that orthotropic steel decks are no longer in danger of fading out of the picture due to surfacing. The Syposium on Wearing Surfaces for Steel Bridge Decks will no doubt confirm this opinion.

List of Figures

SUMMARY

In steel bridge engineering the most important innovation in two decennia has been the orthotropic plate. It will not, however, find final acceptance before a surfacing of adequate durability is found.

Today, therefore, it can safely be said only that the wearing surface will be a bituminous one, having a thickness of about 6 cm = appr. 2-1/2 inches. The causes of damages to wearing surfaces could be determined by observations of many different types of pavements installed. The number of competing systems is continually being limited by tests both in laboratories and in the field.

The most important results of the IABSE inquiry consist in that similar experiences have been gathered likewise in countries other than Germany and that solution to the problem cannot be found by modifications of the structural features of the orthotropic plate. Further systematic investigations will reveal one day what a bituminous wearing surface must be like to best meet the requirements to be made in the interest of durability, economy and riding safety.

RESUME

Le tablier métallique léger (dalle orthotrope) constitue l'innovation la plus importante des deux dernières décennies dans le domaine des ponts métalliques. Il ne pourra toutefois être sanctionné définitivement que lorsque l'on disposera d'un revêtement suffisamment durable.

A l'heure actuelle, on peut uniquement indiquer que ce sera vraisemblablement un revêtement bitumineux d'environ 6 cm d'épaisseur. Les observations effectuées sur de nombreux revêtements réalisés de façon différente ont permis de déceler les raisons des dégâts constatés. Les essais entrepris tant au laboratoire que sur des ouvrages existants tendent à réduire constamment le nombre des types de recouvrement entrant en ligne de compte.

L'enquête réalisée par l'AIPC a conduit au résultat principal suivant : dans les autres pays, on a fait les mêmes expériences qu'en Allemagne, et la solution du problème ne consiste pas à modifier la conception structurable du tablier métallique. Des recherches nouvelles et des observations systématiques montreront un jour quelle doit être la composition d'un revêtement bitumineux satisfaisant au mieux les conditions relatives à sa durabilité, à l'économie et à la sûreté de la circulation.

ZUSAMMENFASSUNG

Die orthotrope Platte ist seit zwei Jahrzehnten die wichtigste Neuerung auf dem Gebiete des Stahlbrückenbaues, sie wird jedoch ihre endgültige Bestätigung erst dann erhalten, wenn ein ausreichend dauerhafter Belag gefunden ist.

Mit ziemlicher Sicherheit läßt sich daher bisher nur sagen, daß es ein bituminöser Belag von etwa 6 cm Dicke sein wird. Die Beobachtung einer Vielzahl verschiedenartiger ausgeführter Beläge ermöglichte es, die Ursache der Belagschäden zu erkennen. Durch Laborversuche sowie Großversuche auf Brücken wird laufend eine Einengung der Zahlen der miteinander im Wettbewerb stehenden Belagsysteme erreicht.

Die IVBH-Umfrage zeigt als wichtigstes Ergebnis, daß man auch in anderen Ländern ähnliche Erfahrungen gemacht hat wie in Deutschland, und daß die Lösung des Problems nicht in einer Änderung der konstruktiven Merkmale der orthotropen Platte zu suchen ist. Weitere systematische Untersuchungen und Beobachtungen werden eines Tages erkennen lassen, wie der bituminöse Belag aufgebaut sein muß, damit die im Interesse seiner Haltbarkeit, Wirtschaftlichkeit und Fahrsicherheit zu stellenden Forderungen in optimaler Weise erfüllt sind.

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RECENT EXPERIENCE ON WEARING SURFACES FOR STEEL BRIDGE DECKS OF LIGHTWEIGHT CONSTRUCTION

Expériences récentes sur des revêtements de chaussée pour ponts à tablier extraléger

Die neuesten Erfahrungen mit Fahrbahnbelägen auf Leichtbrückendecken

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Great Britain

This contribution describes some experience, including maintenance difficulties with asphalt surfacing on a series of movable bridges which were completed between 1961 and 1967.

Figure 1 illustrates the cross section of a swingbridge at Regents Canal Dock, London, designed in 1959. The deck plate is 9/16th in. (14. 3 mm.) thick with stiffeners at 12 in. (305 mm.) centres. At that time the British Road Research Laboratories were testing sample panels of steel battledeck to investigate alternative methods of construction and surfacing under heavy traffic.

After considering the interim results from these trials and other available information the surfacing was specified as shown in Figure 2 as follows.

As a matter of interest this bridge was fabricated at a shipyard on the north-east coast of England, launched and towed by tug down the coast to the Thames Estuary. It was therefore delivered unpainted so that shot blasting and metal spraying was carried out on site. The steel deck was primed by applying a rubber bitumen emulsion at the rate of 6 to 8 gallons per sq. yard (32. 6 to 43. 5 litres per sq. metre).

A 3/8th in. (9. 5 mm.) thick layer of mastic asphalt to B. S. 988, Table 1, Column 3, was then laid. This is an insulating and waterproofing layer of fairly flexible characteristics of the type used on the roofs of buildings.

Mastic asphalt to B. S. 1446, Table 1, Column 3, was then laid as a wearing course. This was specified to include 35% - 40% of coarse aggregate with a penetration of 25 - 35. Whilst the asphalt was still warm and plastic, hard $\frac{3}{4}$ in. (19 mm.) stone chippings, pre-coated with asphaltic cement were rolled into the surface.

This swingbridge is located at a dock entrance which is subject to very heavy commercial traffic. Immediately at one end of the bridge a sharp turn is required on one of the carriageways owing to a one-way traffic system on the approaches. As a result the bridge deck surfacing in this area is almost continuously subject to heavy braking forces.

Four years after the bridge was opened a series of transverse waves or corrugations at about 1. ft. 6 in. (0. 457 m.) centres appeared in a section of the deck, coincident with wheel paths in the areas of heavy braking. This type of defect is illustrated in Figure 3. The defective areas were removed without difficulty and re-laid to the original specification. We found that the mastic insulating layer had been laid to a greater thickness than specified in this area and the Road Research Laboratories also made tests on samples. As a result it was recommended that for future work the stone content of the wearing course should be increased and a harder grade of asphalt cement adopted. Measurements of the rate of indentation during a wheel tracking test with a loaded wheel repeatedly running over specimens at 45°C. suggested that deformation was likely if heavy traffic was turning and braking hard, although the surfacing might be satisfactory for normal conditions.

Analysis of samples showed that the stone content was at the lower limit of the specified range, that is, about 35% and the penetration value of the recovered soluble bitumen was found to be rather low.

As a result of this experience and experiments, we modified the specification and deck details for surfacing the Woolwich ferry bridge ramps in London, and the Middlesbrough Dock swingbridge on the north-east coast. These were designed in 1964 and 1966 respectively.

This revised specification is illustrated in Figure 4 and you will notice the following improvements. A tack coat of Bostik C was applied to the zinc sprayed steel deck followed by a 1/8th in. (3. 2 mm.) thick insulating layer of rubberised filled bitumen having a final softening point of 90 to 95°C. This material was obtained by mixing

75 parts of limestone filler with 25 parts of 80 - 100 penetration bitumen to which a sufficient quantity of unvulcanised rubber powder was added to give the required ring and ball softening point. The insulating layer was laid at about 180°C. by means of squeegees to give about 1/8th in. (3.2 mm.) thickness.

The wearing course is $1\frac{1}{4}$ in. (32 mm.) thickness of mastic asphalt to B.S. 1447, Table 1, Column 3, having a penetration of 15/25 including 40 - 45% coarse aggregate. The surface was finished, as before, with $\frac{3}{4}$ in. (19 mm.) pre-coated chippings at the rate of 100 sq. yards per ton (82 sq. metres per tonne).

Thus the specification was improved from that adopted for the earlier swingbridges by replacing the relatively thick and soft insulating layer by the thinner and more resilient rubberised filled bitumen and the stone content and hardness of the wearing course was slightly increased.

The steel deck plates on the bridge ramps at Woolwich Ferry are stiffened by bent flats of trapezoidal section. They operate through a tidal range of 30 ft. (9.1 m.) and because the heavy vehicles using these spans are either embarking or dis-embarking on steep gradients, up to 1 in 12, the road surface is subject to heavy shearing forces due to the inclination and to braking and traction.

At each end of the ramps, reinforcement in the form of steel strips $1\frac{1}{4}$ in. x $\frac{1}{4}$ in. (32 mm. x 6.4 mm.) were welded to the battledeck at 6 in. (152 mm.) pitch in a chevron pattern along the vehicle wheel tracks. This arrangement is shown in Figure 5 and although the ramps have been operating for two years and the traffic has been very heavy, there are no signs of trouble.

For medium span steel decks and heavily trafficked movable bridges of this type I prefer to adopt a relatively stiffer steel deck which not only reduces welding distortions but prolongs the life of the asphalt surfacing. The additional load arising from, say, an extra 1/16th in. (1.6 mm.) thickness is not usually significant in these cases and because fabrication costs will be the same, increased capital costs are usually limited to the price of the extra material. For movable bridges subject to heavy traffic in dock areas we believe this small additional cost is more than offset by savings in maintenance.

Having said that, however, I would agree with the reporters' contention that more surfacing troubles have been caused by faulty specification and quality control than by the flexibility of the deck plates, and, of course, for long span bridges savings in dead weight become much more significant.

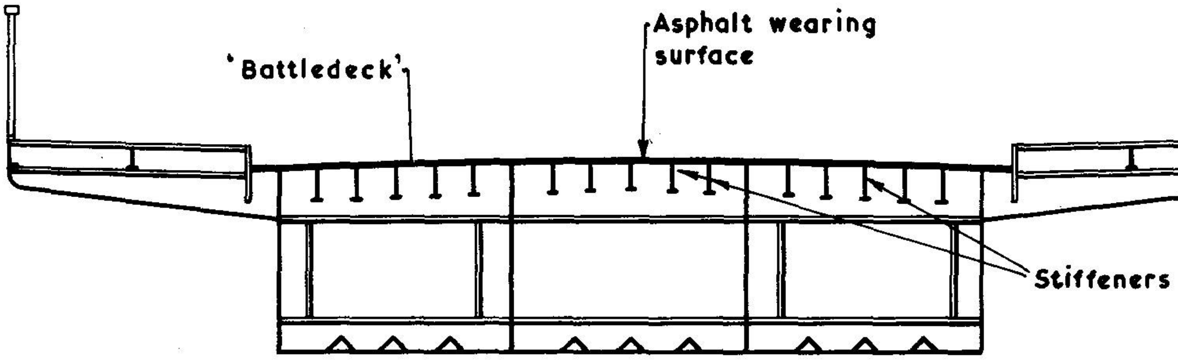

FIG. 1. CROSS SECTION OF SWING BRIDGE AT REGENTS CANAL DOCK. LONDON.

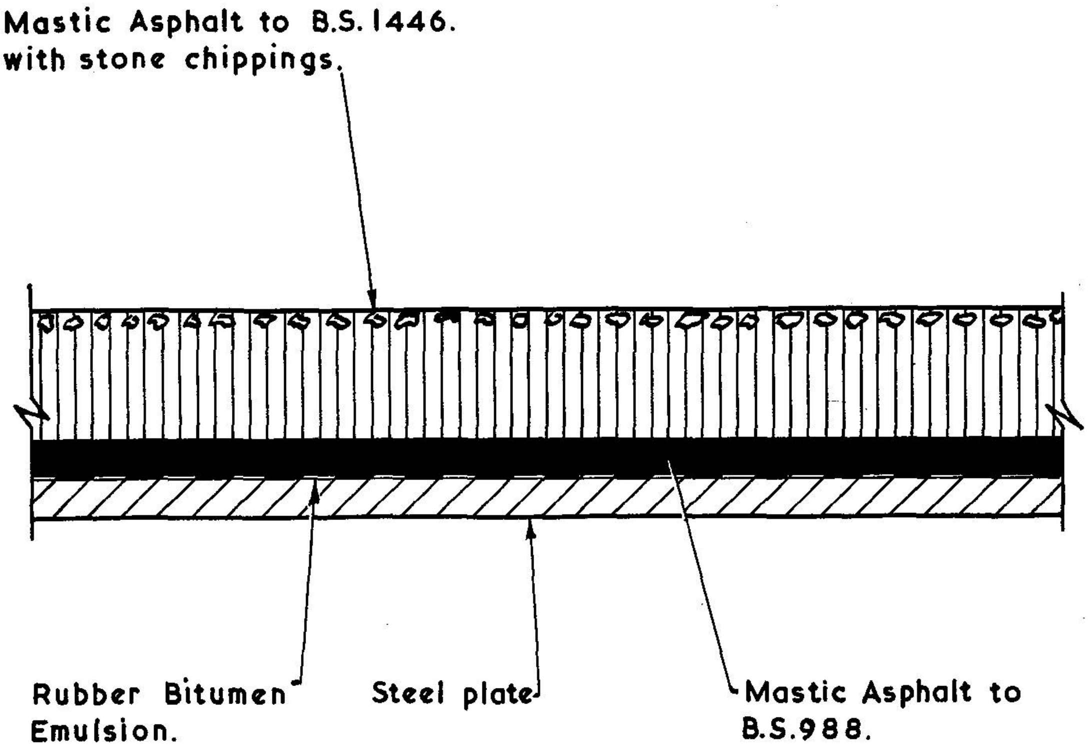

FIG. 2. SURFACING OF SWING BRIDGE AT REGENTS CANAL DOCK, LONDON.

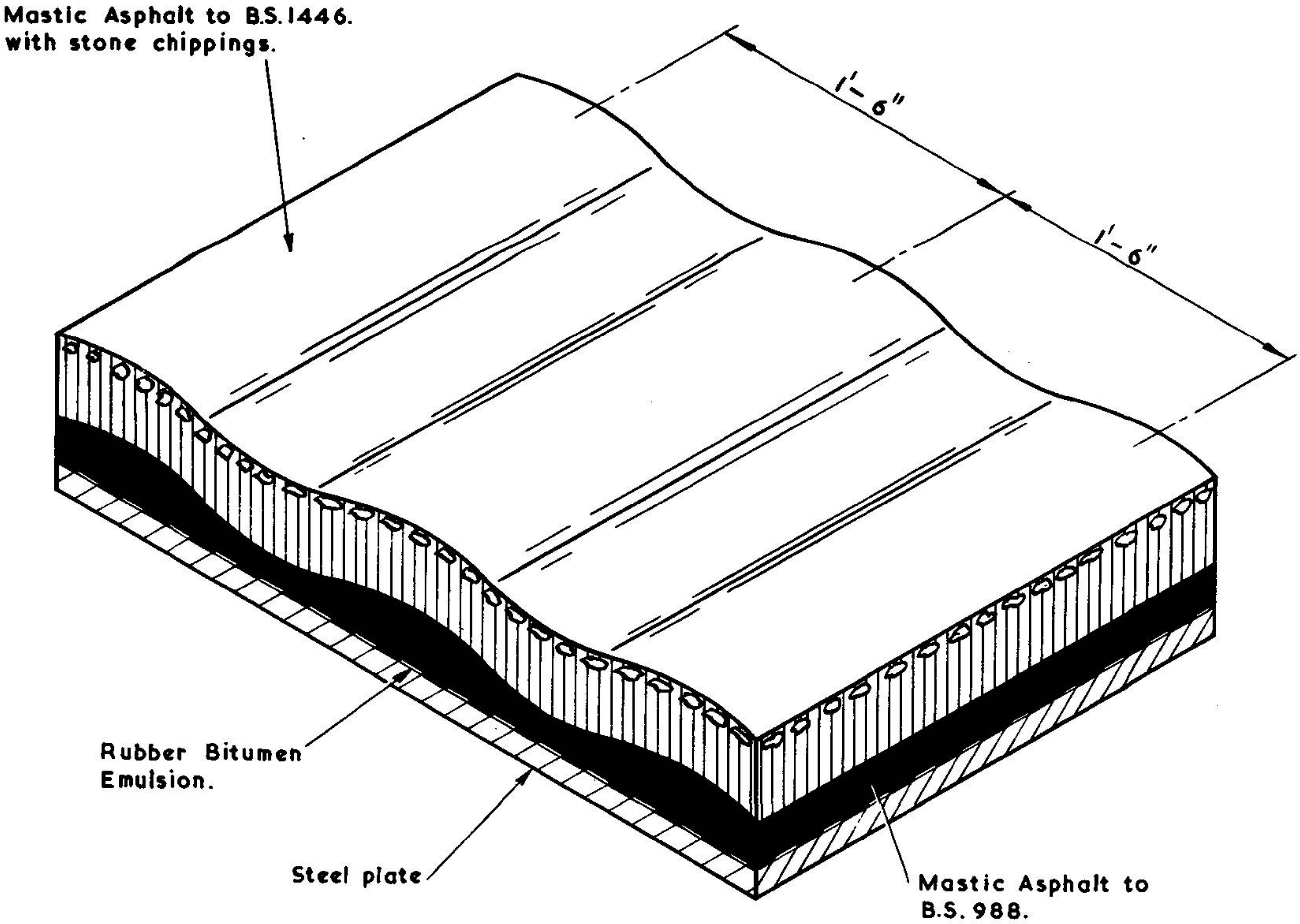

FIG. 3. DEFECT-TRANSVERSE WAVES IN ROAD SURFACE.

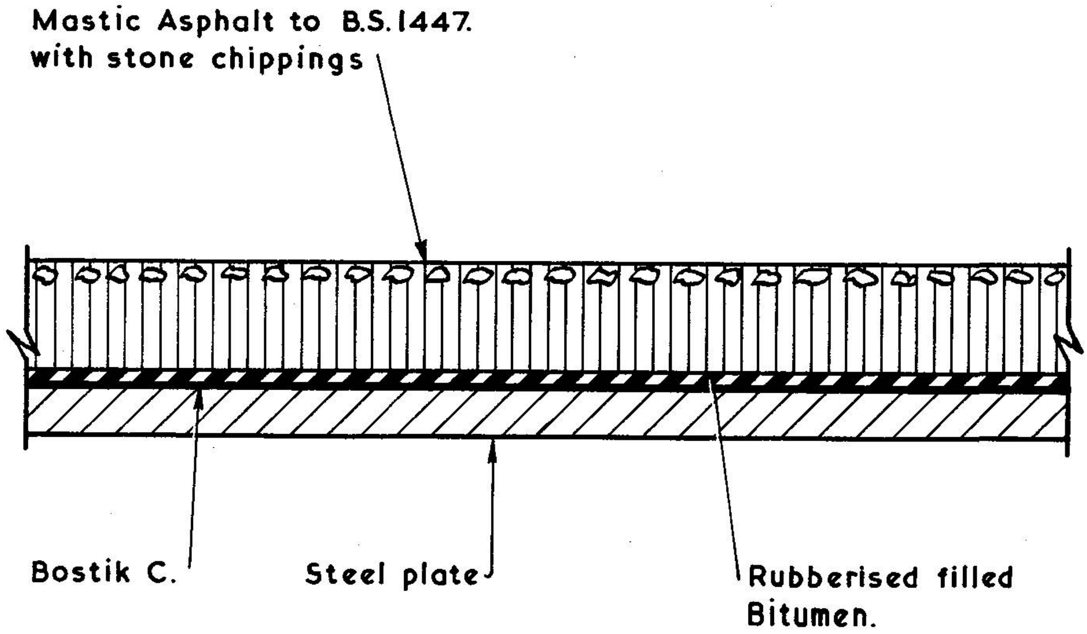

FIG. 4. NEW SPECIFICATION FOR SURFACING AT WOOLWICH AND MIDDLESBROUGH DOCK SWING BRIDGE.

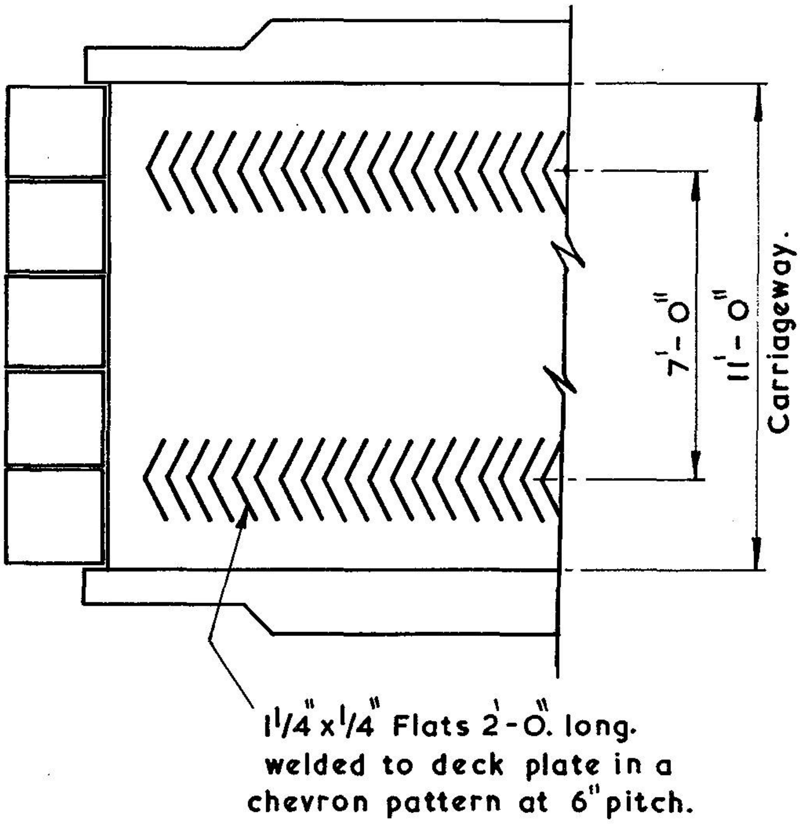

FIG. 5. REINFORCEMENT AT END OF RAMPS.

SUMMARY

Four years after completing a swing bridge in the London Docks a section of surfacing in a limited area subject to braking forces from heavy traffic had to be replaced.

As a result of this experience and tests on samples the specification for recent movable bridges has been modified. The insulating layer is reduced in thickness, the stone content and penetration of the mastic asphalt increased, and steel strips are welded to the deck plate to reinforce the surfacing in heavy duty areas.

RESUME

Déjà quatre années après l'ouverture d'un pont tournant dans les docks de Londres, on a été obligé de remplacer le revêtement dans une zone limitée, soumise aux efforts de freinage du trafic lourd.

A la suite de cette expérience et après des tests sur modèles, on a modifié les spécifications de revêtements pour les ponts mobiles récents. On a réduit l'épaisseur de la couche d'isolation et augmenté la part de gravier et la pénétration de l'asphalte coulé. Dans les sections les plus sollicitées, des bandes d'acier sont soudés sur la tôle pour renforcer le revêtement.

ZUSAMMENFASSUNG

Vier Jahre nach Eröffnung einer Drehbrücke in den Londoner Docks mußte ein Abschnitt, welcher den Bremskräften schweren Verkehrs ausgesetzt war, erneuert werden.

Als Ergebnis dieser Erfahrungen und Prüfungen ist die Spezifikation der Fahrbahnbeläge für neue, bewegliche Brücken abgeändert worden. Die Isolationsschicht ist in der Dicke vermindert, der Splittanteil und die Eindringtiefe des Gußasphaltes erhöht und Stahlsteifen sind zur Verstärkung der Oberfläche in schwerbeanspruchten Abschnitten angeschweißt worden.

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THE CURRENT PAVEMENT ON STEEL DECKS IN JAPAN

Procédés actuels de revêtements de tabliers en acier au Japon

Der gegenwärtige Stand der Straßenbeläge auf Stahldecken in Japan

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Japan Highway Public Corporation

SHINKO KIKEGAWA
Managing Director
Nippon Hodo Co., Ltd.

SABURO MATSUNO
Chief of Pavement Section
Public Works Research Institute
Construction Ministry

It is in 1955 that steel decks were for the first time applied to bridge construction in Japan. After that, scores of bridges with steel decks were constructed in various places up to date. However, the performance of the pavement on the steel decks are not necessarily good and there are still a many problems to be solved.
Public Works Research Institute of Construction Ministry made survey and observation of the bridge pavement in 1962 and 1968 for the purpose of revealing the best type of pavement on steel decks.
This report describes the results of the survey made in 1968.

(1) DESCRIPTION OF SURVEY

The bridge surveyed are 21 bridges with asphalt pavement of 5cm or more in thickness shown in Table-1, which are located on the streets or trunk road under comparatively heavy traffic. The structure and the composition of the asphalt pavement of the bridges differ in each one. But the typical cross section of the pavement and the composition of mixtures are shown in the Fig.-1 and Table-2.
The traffic volume shown in Table-1 indicates the number of traffic of all lanes in 12 hours. In Japan, the percentage of heavy vehicles in the traffic is 10 - 30% and 20% in average.

No.	Bridge	Date in Service	Volume of Traffic: Date Measured	Volume of Traffic: No./12hrs. all lanes	Structural Details: Length	Structural Details: Deck Plate Thickness	Pavement Details: Pavement Thickness	Surface Course	Leveling Course	Insulation Layer	Prime Coat	Cracking Index	Waving Index	Rutting Index	Serviceability Index
1	Shiraga Br.	Feb.'59	Nov. '67	21,300	37.0^{m}	12^{mm}	66^{mm}	60 As.Con.		6 Mastix	Tar Rubber Emulsion	25.8%	0%	0%	2
2	Gasu Br.	Nov.'59	Jul. '68	11,282	174.9	12	80	45 Warbit	28 Guss-asphalt	7 Mastix	Tar Rubber Emulsion	21.3	100	100	2
3	Jogashima Br.	Apr.'60	Jul. '68	10,102	235.0	12-16	84	45 Warbit	29 Guss-asphalt	6 Mastix 6 Foils		0	16	100	2
4	Taisho Br.	'61	Nov. '67	38,400	92.0	9	60	30 Guss-asphalt	30 Guss-asphalt		Tar Rubber Emulsion	10.4	0	80	2
5	Nishiarai Br.	Mar.'61	Oct. '65	10,173	181.2	12	70	30 As.Con.	34 As.Con.	6 Foils		3.1	0	0	4
6	Hinode Br.	Sep.'62	Oct. '65	26,073	54.3	10-14	60	30 As.Con.	30 Guss-asphalt		Tar Rubber Emulsion	3.5	3.4	5.7	3
7	Takamatsu Br.	Dec.'62	Jun. '68	13,514	19.3	12	50	25 Guss-asphalt	25 Guss-asphalt		Tar Rubber Emulsion	26.3	0	0	2
8	Ajigawa Br.	Apr.'63	Mar. '68	34,322	206.5	12	92	35 Guss-asphalt	27 Gussasphalt 30 Gussasphalt		Tar Rubber Emulsion	36.0	3.9	100	2
							92	35 As.Con.	27 Gussasphalt 30 Gussasphalt		Tar Rubber Emulsion	6.1	3.1	30	3
9	Shinjuku Br.	May '64	Oct. '65	43,339	100.7	12	80	50 As.Con.	25 As.Con.	5 Foils		1.2	0	0	4
10	Nishi Br.	May '64	Nov. '67	53,300	30.6	14-16	62	35 As.Con.	25 Guss-asphalt		Tar Rubber Emulsion	0	23	0	2
11	Minato Br.	Aug.'64	Jun. '68	31,960	250.0	12	97	40 As.Con.	57 Guss-asphalt		Tar Rubber Emulsion	11.1	3.4	0	2
12	Yodogawa Br.	Sep.'64	Nov. '67	28,000	230.0	12	65	35 Guss-asphalt	30 Guss-asphalt		Tar Rubber Emulsion	0	0	0	4
13	Biwako Br.	Sep.'64	Jun. '68	1,820	330.0	12	80	40 As.Con.	40 Guss-asphalt		Rubberized Asphalt	0	1.4	0	4
14	Ukita Br.	Nov.'64	Oct. '65	18,755	36.5	10	80	40 As.Con.	40 Guss-asphalt			9.0	0	56	3
15	Hanshin S-Br.	Nov.'64			192.0	12	74	10 Silica Sand As.	25 As.Con. 39 Gussasphalt		Tar Rubber Emulsion	0.7	0.2	0	4
16	Hozumi Br.	Jul.'65			440.0	12	80	40 As.Con.	40 Guss-asphalt		Rubberized Asphalt	0.3	0.5	0	4
17	Shinroku Br.	Jul.'66	Oct. '66	17,000	25.5	10-12	70	30 As.Con.	34 As.Con.	6 Foils		4.5	0	0	4
18	Watanabe Br.	Sep.'66	Nov. '67	49,100	58.9	12	80	40 Guss-asphalt	40 Guss-asphalt		Tar Rubber Emulsion	0	1.7	0	4
19	Higo Br.	Sep.'66	Nov. '67	49,100	29.1	12	80	40 Guss-asphalt	40 Guss-asphalt		Tar Rubber Emulsion	0	4.1	0	4
20	Shinjuso Br.	Oct.'66	Nov. '67	46,800	790.0	12	80	30 Guss-asphalt	30 Guss-asphalt		Tar Rubber Emulsion	0	0.3	0.3	4
21	Akigase Br.	Mar.'68	Jun. '68	12,495	80.0	12	60	30 Guss-asphalt	30 Guss-asphalt		Tar Rubber Emulsion	1.1	0	0	4

The single axle load of most heavy vehicles is 10 - 16t, but some are recorded to weigh as heavy as 20t. This is extraordinary severe condition comparing with those in U. S. or Europe.

The performance of the pavement is, as shown in Table-1, estimated by Cracking Index, Waving Index, Rutting Index and Serviceability Index.

The Cracking Index, Waving Index and Rutting Index are expressed by the formula in next page.

Serviceability Index is the marking by a particular individual on observation of every pavement with marks from 1 to 5 like AASHO Road Test.

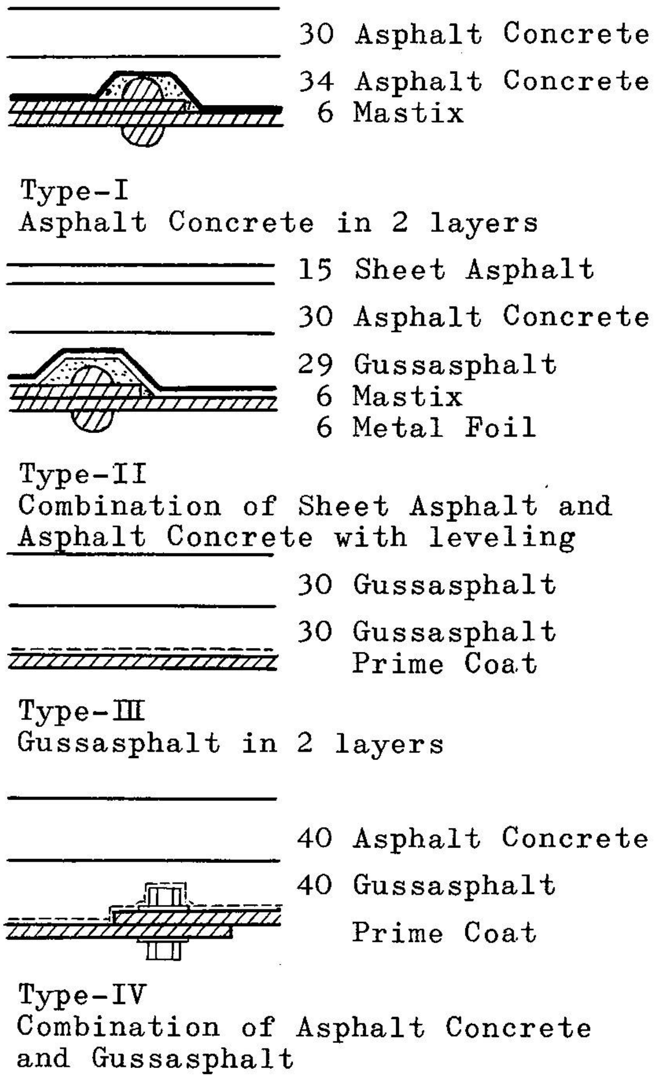


Fig.-1 Typical Cross Section of Pavement (mm)

Materials	Type of Mixtures				
	Asphalt Concrete for Surface	Asphalt Concrete for Leveling	Guss-asphalt	Sheet-Asphalt	Mastix
Asphalt Cement	7.5%	6.0%	8.5%	14.0%	35.0%
Crushed Stone (13-5mm)	24.5	61.0	25.0		
Crushed Stone (5-2.5mm)	15.0		17.0		
Sand	45.0	24.0	21.5	68.0	
Mineral Filler	8.0	9.0	28.0	18.0	55.0
Asbestos					10.0

Table-2 Composition of Mixtures (by wt.)

$$\text{Cracking Index} = \frac{\text{total of cracked area}}{\text{whole area of pavement}} \times 100\%$$

$$\text{Waving Index} = \frac{\text{total of waving area}}{\text{whole area of pavement}} \times 100\%$$

$$\text{Rutting Index} = \frac{\text{total length of rutting}}{\text{total length of traffic lanes}} \times 100\%$$

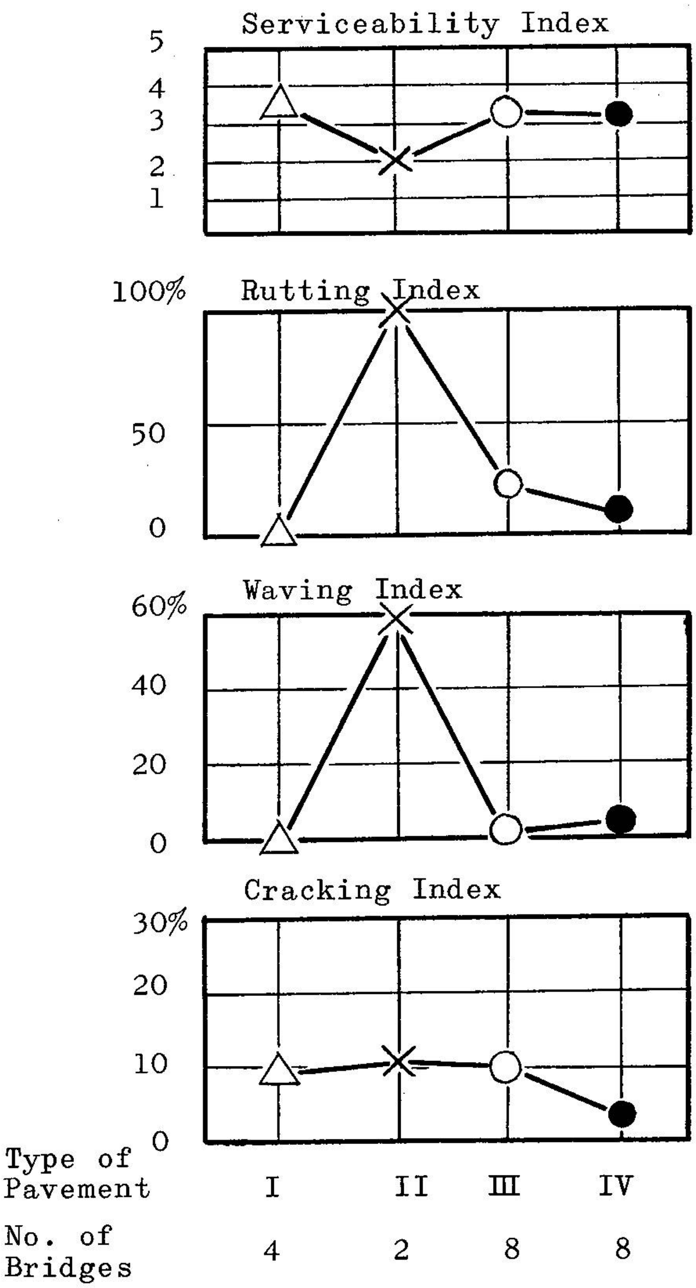
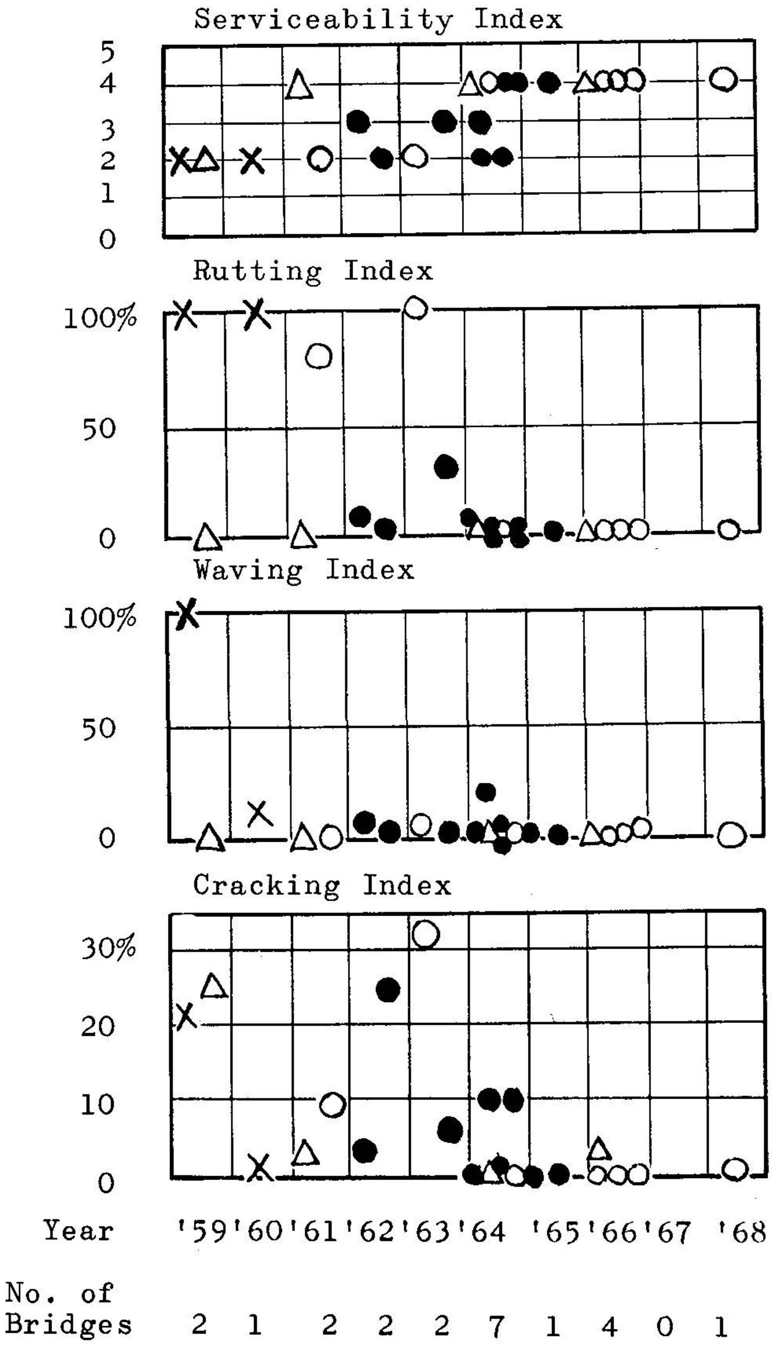


Fig.-2 Performance by Type of Pavement

Serviceability Index 4 denotes comparatively good and needing little repair. 3 denotes needing a little repair and 2 denotes needing considerable repair.

(2) STUDY OF THE RESULTS

As to the factors influencing on the performance of the pavement, period in service, volume of traffic, thickness of steel decks, interval of ribs, type of joint of plates, thickness of pavement, presence of insulation layer, are presumed, and relations between performance and those factors were seeked. The factors which are most related to the performance are period in service, type of the pavement and the volume of traffic. Fig.-2 indicates the relation of the performance and the type of pavement.



note.

△ Type I
✕ Type II
○ Type III
● Type IV

Fig.-3 Performance by Year of Construction and Type of Pavement

The pavement that resulted best in performance was Type I pavement consisting of asphalt concrete mixtures placed in 2 layers and Type III pavement consisting of new type gussasphalt mixture placed in 2 layers.

The worst was Type II pavement consisting of sheet asphalt and asphalt concrete mixtures.

Type IV pavement consisting of gussasphalt and asphalt concrete mixtures appears to cause less cracks than other 3 types.

Fig.-3 indicates the relation of the performance and the period in service in each type of pavement.

Many of the pavements constructed before 1963 are found faulty.

The pavements with asphalt concrete in 2 layers (Type I) are keeping very good condition without relation to the period in service.

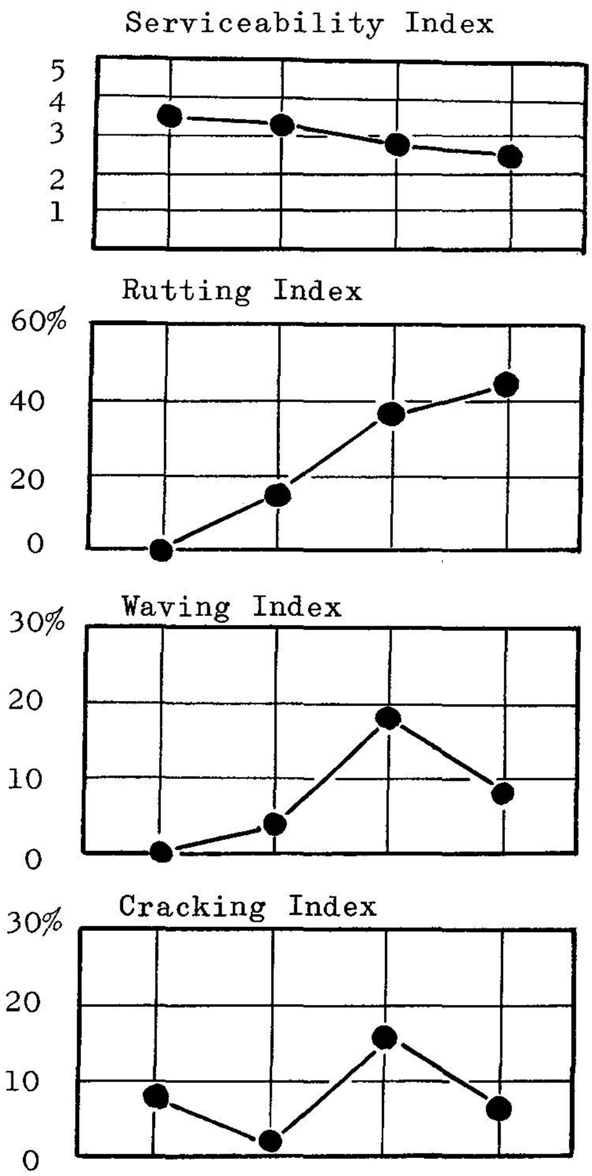
The sole gussasphalt (Type III) constructed after 1964 also keeps good condition. This is supposed due to the fact that some improvement of pavement fitting to Japanese climate were made such as adopting gussasphalt with low penetration asphalt (20 - 40) and the fact that the quality control has been strictly executed since 1964.

Type IV pavement consisting of asphalt concrete and gussasphalt mixtures recorded good tendency from the trial pavement, and those constructed in recent years show a good performance.

This type of pavement may be recommended as the one of less fault.

Fig.-4 indicates the relation of the performance and the total traffic (number of vehicles) of a lane since opening for traffic up to date, showing a considerable correlation in waving, rutting and serviceability index, but correlation with the type of the pavement was not so conspicuous as that with the period in service.

The correlation of the performance and the other factors are as follows:



Vol. of Traffic/lane (million)	10 & less	10-12	12-15	15 & more
No. of Bridges	4	7	6	3

Fig.-4 Performance and Total Traffic/Lane

i) There is no correlation between the performance and thickness of the steel deck nor between performance and rib intervals.

ii) The pavement on the all welded steel deck showed good performance than that on the steel decks with riveted joints. This may be due to the fact that the all welded steel decks were comparatively new and it was easy to pave on them.

iii) Thicker pavement has a tendency to cause easily waving or rutting.

iv) In the insulation layer, those with asphalt mastic are not good because it easily causes flow. However, waterproofing is well attained as the crack of the pavement does not reach to the mastic layer.
The insulation layer with the metal foils under Type I pavement show a very good results as well as the pavement upon it.

Besides above, the following points were obtained by the observation.
i) In many cases, there were cracks near expansion joints. Also, in many cases, cracks were found at the construction joints of asphalt concrete.

ii) The bonding of steel decks and the pavement is passably attained with rubberized asphalt or tar rubber emulsion. But these materials are not so good as resinous bonding materials.
In the cracked parts, water infiltrates between steel deck plate and the pavements. However, as far as observed, there was no rust upon the steel deck plate.

iii) Generally, the pavement of which design and composition was not good, or the quality control was not well executed, cause many faults.
The pavements constructed in the past 3 - 4 years find very less faulty.

(3) CONCLUSION

At present, the final conclusion can not be set forth, as the above report is the results of surveys of only 21 bridges, besides, many of them need continuation of observation after this, however, summing up these surveys, the following may be remarked.

i) The combination of insulation layer with metal foils and asphalt concrete is the best type of pavement, and the well designed and well controled gussasphalt is recommended as well.

ii) Bonding of pavement and the steel deck is well attained with rubberized asphalt or tar rubber emulsion, however, it is advisable to develop a more suitable bonding materials.

iii) The steel deck should be all welded deck as the irregularity by splice plates or rivet heads are not good for the pavement.
The thickness of the pavement is possible to be reduced to approximately 5cm by the use of all welded steel decks, and the thinner pavement is thought to be better than the thicker ones.
It may be necessary to fix the expansion joint firmly, and to reinforce the part of steel deck near joint. Pavement near expansion joint should be finished smooth in particular.

SUMMARY

Selecting typical 21 bridges with steel decks, performance of the pavement on the steel deck were surveyed.

The following were revealed by the results of the survey:

i) Asphalt concrete pavement with insulation layer pasted metal foils with blown asphalt and mastic asphalt pavement of which quality is well controled proved good types of pavement for the steel decks.

ii) The bonding of pavement and the steel decks are passably well kept by means of rubberized asphalt and tar rubber emulsion.

However, for the final conclusion, the continuous survey and further study are necessary on this problem.

RESUME

L'observation de 21 ponts typiques à tablier en acier, concernant leur revêtement, a donné les résultats suivants:

1) Deux types de revêtement ont fait leur preuve: le béton asphaltique avec feuilles métalliques collées contre une couche d'isolation et avec asphalte soufflé, et l'asphalte coulé à qualité strictement contrôlée.

2) L'adhésion du revêtement sur la tôle est bien garantie par de l'asphalte caoutchouté et par des émulsions goudron-caoutchouc.

Cependant, une surveillance continue ainsi que des études complémentaires sur ce problème sont requises pour permettre une conclusion définitive.

ZUSAMMENFASSUNG

An 21 repräsentativen Brücken mit Stahlfahrbahn wurde das Verhalten des Belages auf der Stahldecke erforscht.

Die Forschungsergebnisse offenbaren folgendes:

i) Asphaltbetonbelag mit Isolationsschicht verklebt mit Metallfolie mit geblasenem Asphalt sowie Gußasphalt, dessen Qualität geprüft ist, sind geeignet für Stahldecken.

ii) Die Haftung zwischen Belag und Stahlunterlage vermittels Gummiasphaltes und Teer-Gummi-Emulsion ist verhältnismäßig gut. Wie auch immer, es zeigt sich, daß künftig vermehrt Forschung nötig sein wird, um dieses Problem zu lösen.

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AMERICAN EXPERIENCES WITH THICK PAVEMENTS ON ORTHOTROPIC STEEL DECK BRIDGES

Expériences faites en Amérique avec les revêtements épais sur ponts à tablier orthotrope

Erfahrungen mit dicken Belägen auf orthotropen Brücken in Amerika

F.F. FONDRIEST

INTRODUCTION

The use of orthotropic steel deck bridges on the American Continent did not come until 10 or 15 years after developments in Europe. It was quite natural then that many of the construction methods and materials used would closely follow those used in Germany and elsewhere. The problems with the wearing surface were somewhat unusual, however, because of basic differences in asphalt concrete made in the United States and Canada as compared to the mastic asphalts used in Europe. Experience with paving materials other than asphalt was quite limited and not considered appropriate without a previous performance history. Despite anticipated problems, it was nonetheless the opinion of many American designers that a smooth and durable pavement could best be provided by an asphalt wearing surface with a thickness of 1-1/2 to 3 inches (3.75 to 7.5 cm). In some cases attempts were made to modify the properties of the asphalt with additives. In essentially all cases, special measures were taken to provide a good bond between the pavement and the steel deck.

This report will attempt to briefly describe (1) the paving materials and procedures used on bridges on the American Continent, (2) the performance of these pavements to date and (3) a discussion of limited laboratory studies which relate to the observed performance.

BRIDGES IN SERVICE

There are presently nine orthotropic bridges on the American Continent which are wholly or partially paved with thick wearing surfaces. Table 1 gives a summary of the paving details on these bridges.

Of particular importance to this discussion are the details of the bond coat used to anchor the pavement to the deck plate. In all cases where a bond coat was used, crushed stone chips were broadcast over the surface of the epoxy while still tacky. The purpose was to form a mechanical anchor between the protruding stone chips and the pavement. A variety of quantities and sizes of stone chips have been used for this purpose. Table 2 gives a summary of the bonding details used.

Service Performance

Of the nine bridges listed, three have been in service for less than one year with a fourth subjected to traffic only slightly longer than a year. All of these bridges have been inspected within the last three months and no damage or defects were observed. It is expected that at least another year will be required before any comments may be made about the performance of the pavements on these bridges. The remaining five bridges have been subjected to approximately 3 or more years of service and ample evidence is available to assess their suitability. The following is a brief description of the observed perfomance of each of these bridges.

Port Mann Bridge -- After more than 4 years under heavy traffic, the condition of the pavement must be considered excellent. The only noticeable defect observed was some slight unevenness over a few of the splice plates after about 2 years in service. This condition is shown in Figure 1. A more recent inspection (after four years) indicated that transverse cracks may be forming at a few of these locations although there are none yet visible to the unaided eye. One such location is shown in Figure 2.

TABLE 1. DETAILS OF THICK PAVEMENTS ON ORTHOTROPIC BRIDGES ON THE AMERICAN CONTINENT

Bridge	Date In Service	Surfacing Details				Remarks
		Prime Coat	Bond Coat	Leveling Course	Wearing Course	
Port Mann	June, 1964	Red Lead Epoxy	Coal Tar Epoxy	3/4" SA[1]	1-1/4" AC[2]	
Humphreys Creek	July, 1964	None	Coal Tar Epoxy	1" AC	1" AC	E half
	July, 1964	None	Coal Tar Epoxy	1" AC-L[3]	1" AC-L	W half
Ulatis Creek	Sept.,1965	Inorganic Zinc	None	None	1-1/4" AC	1/5 section
	Sept.,1965	Inorganic Zinc	None	None	1-1/4" EAC[4]	1/5 section
Concordia	Aug., 1965	None	Coal Tar Epoxy	None	2" AC	
Dublin	Dec., 1965	Zinc Metallizing	Coal Tar Epoxy	None	2" AC	1/4 section
	Dec., 1965	None	Coal Tar Epoxy	None	2" AC	1/4 section
Battle Creek	May, 1967	None	Coal Tar Epoxy	None	1-3/4" AC	E half
San Mateo	Nov., 1967	Inorganic Zinc	None	3/4" EAC	3/4" EAC	
Poplar Street	Nov., 1967	Inorganic Zinc	Coal Tar Epoxy	1-1/4" AC-L	1-1/4" AC-L	
Longs Creek	Dec., 1967	Inorganic Zinc	Fiber Glass[5]	None	1-1/2" AC	

(1) SA = sand or sheet asphalt.

(2) AC = asphalt concrete.

(3) AC-L = rubber latex modified asphalt concrete.

(4) EAC = epoxy asphalt concrete.

(5) Fiber glass impregnated with asphalt emulsion and sealed with mastic asphalt.

TABLE 2. SUMMARY OF SYSTEMS USED TO ANCHOR BITUMINOUS PAVEMENTS TO STEEL DECK BRIDGES

Bridge	Primer lbs/yd^2	Chips lbs/yd^2	Gradation - percent passing										Leveling Course
			3/4"	1/2"	3/8"	1/4"	#4	#6	#10	#16	#20	#30	
Troy(1)	1.0	15-18										100	AC & SA
Port Mann	2.8	7.5					100			0			SA
Humphreys Creek	1.85	4						100			10	0	AC & AC-L
Ulatis Creek	None	None											AC
Concordia	2.5	2.5					100	50					AC
Dublin	6.6(2)	7.5(3)	100	0									AC
Battle Creek	2.5(3)	10(3)				100			0				AC
Poplar Street	1.0	5-8			100		90		10	5			AC-L

(1) Small test bridge used to evaluate pavements for the Poplar Street Bridge.

(2) Applied in two equal coats before and after chips were applied.

(3) Estimated.

FIGURE 1. SLIGHT UNEVENESS IN THE SURFACE COURSE OVER THE SPLICE PLATES ON THE PORT MANN BRIDGE AFTER 2-1/2 YEARS IN SERVICE

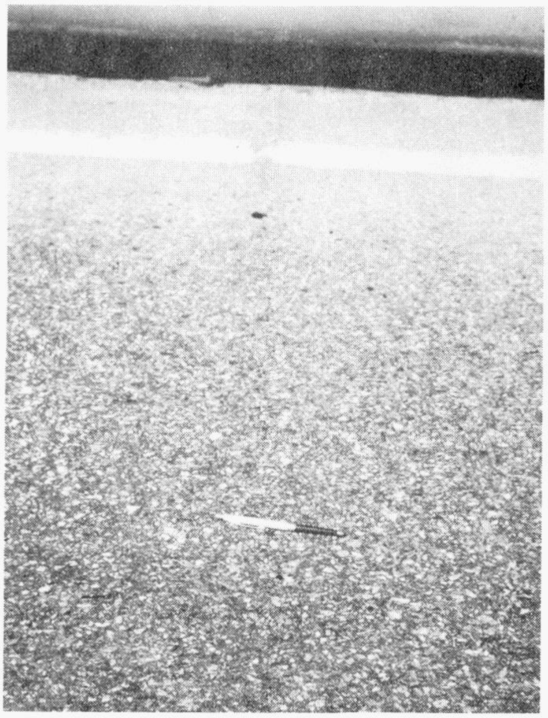

FIGURE 2. EVIDENCE OF A TRANSVERSE CRACK FORMING OVER THE SPLICE PLATES ON THE PORT MANN BRIDGE AFTER FOUR YEARS OF SERVICE

With the exception of these few isolated spots, no other defects such as longitudinal cracking or shoving and rutting were noted. It is expected that any cracks which may form over the splice plates will be noticeable within an additional year.

Humphreys Creek Bridge - Soon after this bridge was opened to traffic, several sections of the pavement were removed to correct minor structural deficiencies in the bridge. These sections could be removed only with great difficulty indicating excellent bond between the seal coat and the leveling course. The appearance of the underside of the pavement sections removed showed that the leveling course conformed well to the protruding stone chips of the bond coat.

The major portion of the pavement which has not been disturbed reamins in excellent condition with no cracking or other defects visible.

Ulatis Creek Bridge - Approximately three years ago, one lane of this bridge was paved with 1-1/4 inch (3 cm) of five experimental surfacing materials. Three of these materials were epoxy mortars (discussed in a separate report) while the two of interest here were asphalt concrete and epoxy asphalt concrete. In the case of the AC, only an emulsion tack coat was used to bond the pavement to the deck plate. For the epoxy asphalt, a tack coat of the epoxy binder was used.

Within a few weeks after being opened to traffic, random cracks appeared in the AC surfacing. Shortly thereafter the pavement began debonding and within three months was literally shoved off the bridge. The mode of failure can be seen in Figure 3. The AC was replaced again using only an emulsion tack coat to bond the pavement to the deck plate. Within nine months the new AC pavement had again failed in the same manner as before. The pavement was replaced a second time in the same manner. This third pavement has remained in place and is performing reasonably well after almost two more years of service.

After three years in service, the epoxy asphalt pavement is in excellent condition with no signs of cracking or other defects. Test cores taken from the pavement indicated a bond strength of over 200 psi (14 kg/cm^2).

Concordia Bridge - This bridge is located within the Exposition grounds in Montreal. Half of the bridge width is used for rail traffic while the other half has little traffic at this time. The bridge was however used for all construction traffic prior to the opening of Expo '67. An inspection of this bridge was made approximately two years after completion. Numerous longitudinal cracks were formed essentially over the entire length of the bridge as shown in Figure 4. The most severe crack (shown in the center of Figure 4) is located over the web of the supporting box girder while the finer cracks are located over the stiffening ribs where they are attached to the deck plate.

About the same time a small breakout occurred in the wearing course over a splice plate as shown in Figure 5. Although the damage is still relatively confined, the pavement adjacent to the breakout could be easily lifted up indicating essentially no bond to the deck plate. It is merely a matter of time before this defect increases in size.

Dublin Bridge - After about one year of service, an inspection of this bridge revealed transverse cracks over a pier and several longitudinal cracks in the pavement. The transverse cracks widened to about 1 in. (2.5 cm) in width after about a year when they were filled with a mastic. The longitudinal cracks were generally very slight with the most pronounced being located over a main support girder. Approximately a year after these cracks were first observed, the major cracks were still visible although most of the fine cracks, located over the ribs, closed up and were no longer visible. No other defects or indications of loss of bond were visible.

Analysis of Performance

Although there is as yet limited field performance, observations have pointed out two problem areas associated with thick asphalt pavements or steel deck bridges. These will be discussed separately below.

Bonding Mechanisms

The structural interaction between the deck plate and the wearing surface necessitates a strong bond. The reliance on a mechanical bond requires a careful match between the size and distribution of the stone chips and the coarseness of the leveling course as it is generally assumed that the bond is achieved by the interaction of the finer of the sand asphalt or AC which penetrates the interstices of the stone chips. If the leveling course mate-

FIGURE 3. MODE OF FAILURE DUE TO DEBONDING OF THE AC PAVEMENT ON THE ULATIS CREEK BRIDGE AFTER ONLY A FEW MONTHS IN SERVICE

FIGURE 4. LONGITUDINAL CRACKS IN THE ACPAVEMENT ON THE CONCORDIA BRIDGE AFTER TWO YEARS IN SERVICE

FIGRE 5. BREAKOUT OF WEARING COURSE OVER SPLICE PLATE ON CONCORDIA BRIDGE AFTER TWO YEARS OF SERVICE

rial lacks sufficient fines and/or the stone chips are too small, there will be no significant interlocking. If the stone chip gradation contains a high percentage of fines and the application rate is heavy, the interstices which would ordinarily be formed by the larger particles will be filled by the excess number of finer particles making any interaction difficult. Such a surface would provide horizontal shear resistance but would undoubtedly be low in tension bond.

As was shown in Table 2 there is a wide variation in the amount and gradation of aggregate chips used. The importance of an effective bond can be determined from the experience of the AC surfacing placed on a portion of the Ulatis Creek Bridge. In this installation, only an emulsion tack coat was used to provide bond between the deck and the pavement. This lack of bond resulted in a complete failure of the pavement. Another instance of a lack of good bond was noted on the Troy Test Bridge where a heavy loading of fine stone chips was used in conjunction with an AC leveling course. After about 2 years in service, moderate cracking had occurred in the surfacing and upon removal of the pavement, little or no bond was found between the seal coat and the asphalt pavement. There was, however, sufficient shear resistance to prevent the pavement from shoving. The only other instance where the bond between the bond coat and the pavement was qualitativly checked was on the Humphreys Creek Bridge. In this instance the bond was found to be excellent.

The relationship of bond strength and shear strength to the performance of the pavement is not known quantitatively, but undoubtedly both are important. No definite limits as to the gradation and rate of application of aggregate chips can be determined on the basis of present information but it appears that a number of systems will provide sufficient bond. The important factor appears to be the need to provide a careful match between the size and quantity of chips used and the consistency of the asphalt used in the leveling course. This intuitively implies that the coarser the asphalt material, the larger the stone chips need to be. The converse would undoubtedly hold true.

It should be pointed out that the epoxy asphalt material requires no stone chips to provide a mechanical bond to the deck plate as a tack coat of the binder provides an excellent bond strength by itself.

Cracking

Cracks have been observed in the wearing course of the Concordia and Dublin Bridges. On both of these bridges the most severe longitudinal cracks occurred over the main girder webs where high negative bending movements are produced. Transverse cracks occurred over a pier on the Dublin Bridge--also an area of high negative movements. Comparable points of high negative movements are not present on the Port Mann and Humphreys Creek Bridges where no surface cracks have been observed.

While the above explanation may account for the cracks over the griders it does not fully explain the cause of cracking over the stiffening ribs. It is known that AC is subject to fatigue cracking under continuous flexing but it is not known to what extent low bond strength may contribute to the cracking.

LABORATORY STUDIES

In conjunction with an extensive laboratory study on this wearing surfaces, occassional experiments were conducted on thick-wearing surfaces of asphalt concrete and epoxy asphalt. The experiments of greatest interest were bond strength and fatigue resistance.

Bond Strength

No real attempt was made to quantitatively determine the bond strength of AC to a steel plate because of the variations possible with differing amounts and size of stone chips. However a 1/2 x 4 x 16-inch (1.3 x 10 x 40 cm) steel plate was paved with a duplicate of the pavement used on the Troy Test Bridge (see Item 1 in Table 2). This plate, with a 1-1/4 inch (3.0 cm) AC surfacing was cooled to 0 F (-18 C), simply supported at the ends and loaded at midspan with the pavement on the tension side. The pavement sheared free of the steel plate at a length/deflection ratio of approximately 900. At 0 F (-18 C) the modulus of elasticity of AC may reach 1×10^6 psi (70,000 kg/cm^2) or more. With this stiffness, high shear stresses will be set up between the pavement and the deck plate. If loading produces deflections sufficient to destroy the bond, failure of the pavement will follow.

Similar experiments were carried out in which the bond was provided by a coal-tar fraction modified synthetic resin. In this case no shear failure occurred at a load/deflection ratio of 300 at temperatures of 0, 77, and 140 F (-18, 25, and 60 C). No failures were encountered in similar specimens using a 1-1/4-inch (3 cm) epoxy asphalt pavement bonded to the steel plate with a tack coat of pure asphalt epoxy binder.

Fatigue Resistance

A series of fatigue specimens were run to determine the maximum deflection a steel plate paved with 1-1/4 inches (3 cm) of AC or epoxy asphalt could withstand for 5×10^6 cycles without cracking. These experiments were conducted using 3/8 and 1/2 inch (10 and 14 mm) steel plates and conducting the experiments at 0 and 77 F (-18 and 25 C). The results of these experiments are given in Table 3.

Table 3. Span/Deflection Ratios Sustained Without Failure for Five Million Cycles

Material	Pavement Thickness inches	3/8 inch Steel Plate		1/2 inch Steel Plate	
		0 F	77 F	0 F	77 F
AC	1-1/4	1340	1240	1400	1150
Epoxy Asphalt	1-1/4	<300	700	<300	770

Concluding Remarks

When considering even this limited laboratory data it becomes apparent that some of the failures observed in the field could have been anticipated. For relatively thin deck plates or where high deflections are expected, it is questionable whether AC as presently applied can ever be expected to withstand failure. The fatigue resistance at low temperatures is quite low and reliance upon mechanical methods of bonding to the steel deck appears to be unrealistic. Undoubtedly the flexibility or fatigue resistance of AC may be improved by changes in the mix design or by using additives but the greatest improvement will likely result by using more efficient bonding agents.

Both laboratory and field studies to date indicate that epoxy asphalt may be an excellent paving material for steel deck bridges having many of the combined advantages of asphalt concrete and epoxy mortars.

SUMMARY

There are nine orthotropic bridges on the American continent paved with thick wearing surfaces. The performance of these pavements after three or more years in service has been mixed. The primary problems encountered are fatigue cracking of asphalt concrete and low bond strength between the pavement and the steel deck. These problems could have been anticipated by a brief laboratory investigation.

RESUME

Actuellement, il y a sur le continent américain neufs ponts à tablier orthotrope couvert d'un revêtement épais. Dans les trois années de service ou plus, ils n'ont pas toujours donné satisfaction. Les défaillances principales étaient la formation de fissures de fatigue dans le béton asphaltique et l'adhésion insuffisante du revêtement. Quelques essais de laboratoire vite faits auraient pu prévoir ces difficultés.

ZUSAMMENFASSUNG

Zur Zeit gibt es auf diesem Kontinent neun orthotrope Brücken mit dicken Fahrbahnbelägen. Nach einer Beanspruchungszeit von drei oder mehr Jahren haben sich diese Decken nur teilweise bewährt. Die Hauptschwierigkeiten lagen in der Bildung von Ermüdungsrissen im Asphaltbeton sowie in der mangelnden Haftfestigkeit zwischen Belag und Stahldecke. Diesen Problemen hätte man durch kurze Laborversuche begegnen können.

EXPERIENCE IN DRY SUB-TROPICAL CONDITIONS

Expérience dans un régime sec et subtropical

Erfahrungen in trockenen subtropischen Verhältnissen

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The River Tigris is crossed at Amara in Lower Iraq by a welded steel, double cantilever bridge which was opened to traffic in December, 1957. It provides one 9 m carriageway and two 3 m footways. The orthotropic deckplate forming the carriageway has a uniform thickness of 19 mm of which 2 mm is an allowance for corrosion. The deckplate which is surfaced by a rolled gravel asphalt carpet is stiffened longitudinally by torsion free stringers spaced at 370 mm centres and serves as the top flange of the stringers and cross girders and with the stringer acts as the main girder top flange. The upper surface of the deck is plain without key ribs. The maximum gradient is 4 per cent.

At the time of construction of the bridge little was known of the behaviour and life of asphalt carpets laid on steel decks; some designers had applied steel mesh to act as a key, while others laid the carpet directly on the plate.

Lower Iraq is an area of low rainfall; annual precipitation hardly exceeds 250 mm. Temperatures range between 0°C at night in winter to above 40°C in summer.

Following recommendations by Dr. R. S. Millard, Deputy Director of the British Road Research Laboratory, which were based on his experience in tropical conditions, a 5 cm thick rolled gravel asphalt carpet was laid on the carriageway. This operation was carried out in October 1957. Particulars are as follows:

The shot blasted and shop painted upper surface of the deck was swept clean and dry and immediately sprayed with cut-back bitumen, which gave a good bond between steel and carpet which was asphalted in a single course by Barber-Greene finisher travelling at 76 cm sec. in accordance with B.S.594 : 1950, the asphaltic cement being in accordance with Table 13 therein. This heated the deck so much that differential expansion caused the cantilever noses to dropp temporarily by 3 cm.

The Local Authority responsible for the bridge has reported on the condition of the carpet after ten years service as follows: "Apart from surface dressing which is quite usual, no repairs or renewals have been undertaken. Surface dressing is usually done in Iraq after two years."

The traffic, although heavy at times of pilgrimage, does not exceed 10,000 units per day as an annual average. The carpet, however, has to resist the penetration of hooves of draught, pack and other domestic animals and occasionally of tracked vehicles. The good performance of this asphalt wearing surface is to the credit of the firm who laid it - Morison Knudsen.

SUMMARY

The Amara Bridge spanning the River Tigris in Iraq has been in service for ten years. The orthotropic steel deck forming the carriageway is surfaced with rolled asphalt according to B.S.594 (1950) laid on the shop painted and primed steel surface, in accordance with recommendations based on experience furnished by the British Road Research Laboratory Tropical Division.

The traffic although heavy at times of pilgrimage does not exceed 5000 units per day per lane as an average. The wearing surface which has also to resist the penetration of hooves of draught, pack and other domestic animals, remains in good condition.

RESUME

Le Pont d'Amara qui franchit le Tigre en Irak est en usage depuis dix ans. Le tablier d'acier orthotropique formant la route carossable est couvert d'asphalte cylindré selon le B.S.594 (1950). La surface de l'acier fut imprégnée et peinte d'avance dans l'atelier, ceci suivant les recommandations basées sur l'expérience acquise par la Division Tropicale du Laboratoire Britannique des Recherches Routières.

La circulation, quoique intense aux temps de pèlerinage, ne dépasse pas, en moyenne, 5000 unités par voie et par jour.

La surface frottante qui doit également résister à la pénétration des sabots des animaux de trait et de bât, se maintient en bon état.

ZUSAMMENFASSUNG

Die Amara-Brücke über den Tigris in Irak befindet sich seit 10 Jahren in Betrieb. Die Fahrbahn, die aus einer orthotropen Platte besteht, ist nach B.S.594 (1950) mit Walzasphalt überzogen, welcher auf die in der Werkstatt gestrichene und grundierte Stahloberfläche aufgebracht ist, gemäß der auf Erfahrung gegründeten Empfehlungen der Tropischen Abteilung der British Road Research Laboratory (Britische Forschungsanstalt für Straßenbau).

Obwohl der Verkehr durch Wallfahrten zeitweilig sehr stark ist, werden im Durchschnitt täglich 5000 Einheiten pro Spur nicht überschritten. Die Verschleißschicht, die ebenso dem Eindringen der Hufe von Zug-, Saum- und anderen Haustieren widerstehen muß, bleibt in gutem Zustand.

VABIT-FAHRBAHNBELAG AUF STÄHLERNEN LEICHTFAHRBAHNEN

VABIT Wearing Surface for Steel Bridge Decks of Lightweight Construction

Revêtements VABIT sur tabliers extra-légers

HELD & FRANCKE
Deutschland

Von dem Fahrbahnbelag auf orthotroper Platte werden folgende Eigenschaften verlangt:

a) Sicherer Korrosionsschutz

b) Gute Haftung des Belags am Stahlblech

c) Gute Standfestigkeit bei anhaltender hoher Temperatur

d) Rißfreiheit bei großer Kälte und bei raschem Temperaturabfall

Außerdem spielen noch eine Rolle:

e) Ebenheit der Belagsoberfläche

f) Griffigkeit

g) Verschleißfestigkeit

h) Lastverteilende Wirkung

i) Gute Möglichkeit des Ausgleichs von Unebenheiten in der Stahlplatte

k) Einfaches mechanisches Einbauverfahren

l) Einfache, wirtschaftliche Reparaturmöglichkeit

In dem Bestreben, einen Brückenbelag für orthotrope Stahlplatten herzustellen, der den oben aufgeführten Bedingungen möglichst weitgehend entspricht, hat sich die Held und Francke Bauaktiengesellschaft München besonders durch die ersten vier Forderungen leiten lassen. Die Versuche und Vorarbeiten begannen in Zusammenarbeit mit der Franz VAGO AG Straßenbauunternehmung, Wigoltingen/Schweiz, im Jahre 1960.

Held und Francke hat in Deutschland seit 1964 bisher auf 10 Stahlbrücken mit orthotropen Fahrbahnplatten insgesamt 61.000 m^2 VABIT-Beläge ausgeführt.

Der Aufbau des Fahrbahnbelages ist auf Abb. 1 dargestellt:

1.) Entrostung der Stahlplatte metallisch blank
2.) Beschichten mit Epoxiharz
3.) Einstreuen von Feinsplitt 1 - 3 mm in das noch weiche Epoxiharz
4.) Oberseite der kunststoffbeschichteten eingestreuten Stahlplatte anspritzen mit ca. 0,5 kg/cm^2 Haftkleber
5.) Aufbringen einer sandasphaltartigen Isolier-Schutzschicht mit ca. 1 cm Stärke
6.) Versiegeln der Isolier-Schutzschicht mit ca. 0,2 kg/qm Lackbitumen
7.) Aufbringen einer 2,5 bis 3,5 cm starken Ausgleichschicht aus Asphaltfeinbeton
8.) Aufbringen einer 2,5 bis 3,5 cm starken VABIT-Deckschicht aus Asphaltfeinbeton

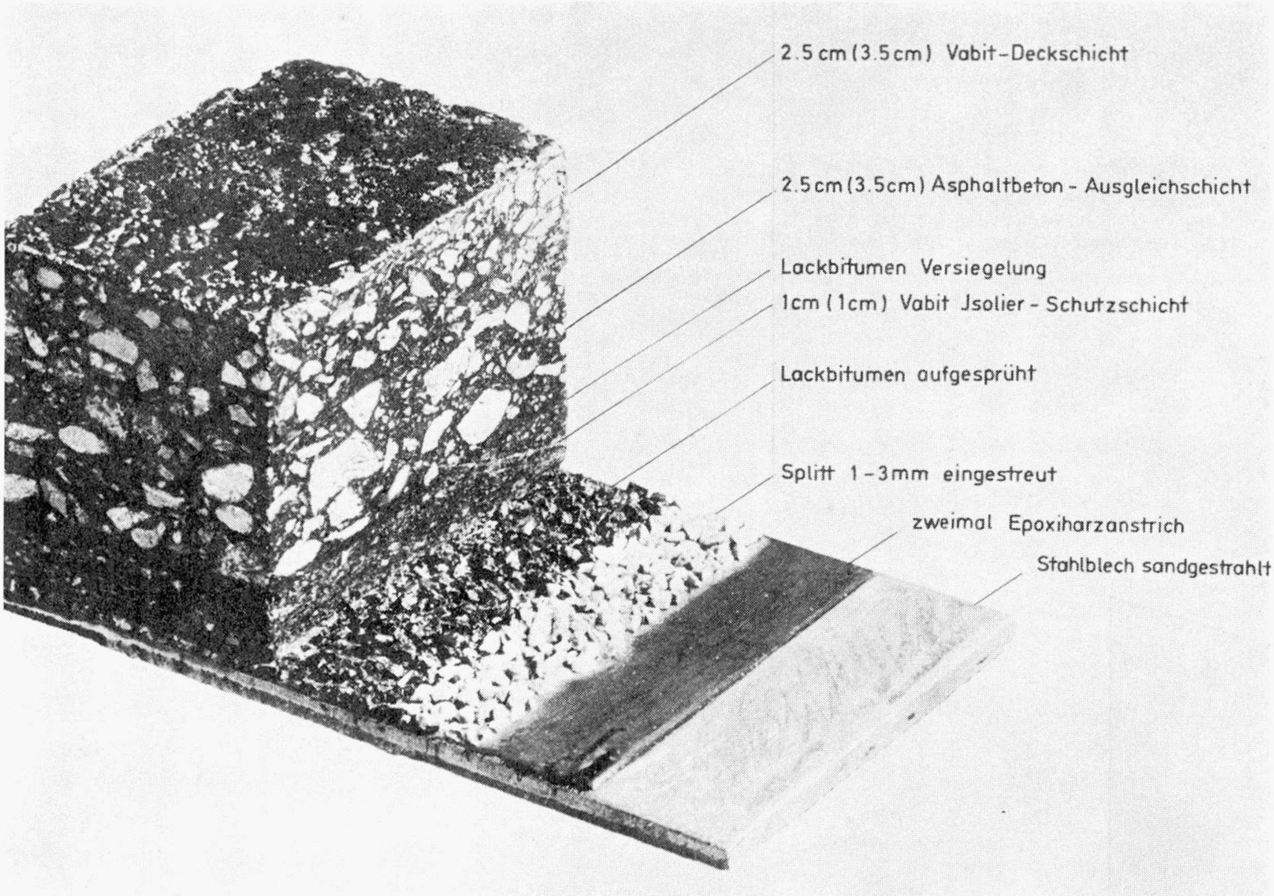


Abb. 1

1.) <u>Sicherer Korrosionsschutz</u>

Dies erscheint durch die Verwendung von lösungsmittelfreiem Epoxiharz auf metallisch blank sandgestrahlter Stahlplatte einwandfrei gegeben zu sein. Der höchste Grad „metallisch blank" ist mit Rücksicht auf die sichere Verkrallung des Epoxiharzes mit der rauhen Stahlfläche wichtig. Das Sandstrahlen muß bei einer relativen Luftfeuchtigkeit von über 80 % unterbrochen werden, weil der Feuchtigkeitsfilm, der sich schon in wenigen Stunden auf der sandgestrahlten Fläche bei dieser Luftfeuchtigkeit bilden würde, eine einwandfreie Haftung des Epoxiharzanstriches auf der Stahlplatte verhindern könnte. Die Temperatur soll bei der Verarbeitung des Epoxiharzes mindestens + 10°C haben. Falls zu Jahreszeiten beschichtet werden muß, während welchen die Außentemperatur niedriger ist, müssen fahrbare Zelte eingesetzt werden, welche mit Infrarotstrahlern ausgerüstet sind, so daß die Stahlplatte auf + 20 bis + 35°C erwärmt werden kann. Um zu verhüten, daß die eingestreuten Splittkörner durch das noch weiche Epoxiharz hindurchsinken bis auf die Stahlfläche, muß der Epoxiharzanstrich in 2 Schichten erfolgen, wobei die 2. Schicht aufzubringen ist, bevor die erste Schicht ausgehärtet ist, damit die beiden Schichten noch eine gute Verbindung miteinander eingehen. Die erste Schicht muß aber andererseits soweit ausgehärtet sein, daß man sie betreten kann ohne sie zu beschädigen. Eine andere Methode besteht darin, daß die Epoxiharzschicht in einer Lage aufgebracht wird und daß in diese Schicht ein Glasfasergewebe mit einer Maschenweite von ca. 1,5 mm eingelegt und eingerollt wird und darüber erst der Feinsplitt mit 1 - 3 mm Korngröße eingestreut wird. Der Splitt sinkt dann nur bis zur Matte durch und erreicht nirgends die Stahlplatte. Dies wurde durch Versuche auf Glasplatten mit anschließender Durchleuchtung nachgewiesen. Für die Wahl des Epoxiharzes ist wichtig, daß es keine Lösungsmittel enthalten darf, weil durch die Verdunstung des Lösungsmittels Poren entstehen würden.

2.) Gute Haftung des Belages auf dem Stahlblech

Die gute Haftung des Epoxiharzes auf dem Stahlblech steht bei richtiger Verarbeitung allgemein außer Zweifel. Die Haftung des Asphaltbelages auf der Epoxiharzschicht muß mindestens so gut sein, wie die Schubfestigkeit des Asphaltbelages selbst. Dies wird - wie oben beschrieben - durch Einstreuen von Feinsplitt der Körnung 1 - 3 mm in den noch weichen Epoxiharzanstrich und durch einen dann aufgesprühten bituminösen Haftkleber erreicht. Außerdem entsteht durch das Aufwalzen der ersten Asphaltbetonschicht (Sandasphalt) eine Verzahnung mit den Splittkörnern. Die Untersuchungen haben gezeigt, daß verschiedene Teerepoxiharze nicht geeignet sind, weil nach dem Aufwalzen der heißen Asphaltschichten (160°C) ölige Aussonderungen beobachtet worden sind, welche die Haftung des Asphaltbelages auf der Beschichtung verhindern und zwar mindestens solange, bis sie verdunstet oder verharzt sind. Die dauernde Haftung zwischen Asphaltbelag und der Stahlplatte wird sowohl durch die Horizontal-Komponente der durch den Verkehr auf die Fahrbahndecke ausgeübten Kräfte als auch durch die Schubspannungen in Anspruch genommen, die sich aus der Verbundwirkung infolge des statischen Zusammenwirkens des Fahrbahnbelages mit der Stahlplatte ergeben (siehe weiter unten Bericht über Pulsatorversuche). Das Epoxiharz muß bereits bei der Herstellung des Asphaltbelages eine harte Forderung erfüllen: Es darf trotz Einwirkung von 160°C durch das heiße Mischgut nicht weich werden, weil sonst während des Walzens, besonders der ersten Asphaltbetonschicht, eine Zerstörung der Beschichtung entstehen würde. Diese Forderung muß bei der Auswahl des Epoxiharzes berücksichtigt werden. Die Prüfmethode, nach der vorgegangen wurde, ist folgende: Fertig beschichtete Blechstreifen von 3 cm Breite wurden in einem Paraffinbad auf 190°C erhitzt und dann unter eine 20 kg schwere Stahlwalze gelegt, wobei zwischen Blechstreifen und Walze ein Hartgummilappen gelegt wurde. Dieser Hartgummilappen wurde sogleich mit Gewalt unter der Walzenlast herausgezogen und beansprucht dabei die Beschichtung auf Abschiebung (Abb. 2 u. Abb. 3). Beschichtungen von mangelhafter Qualität werden dabei glatzig, d.h., sie gehen z. Teil vom Blech ab (Abb. 4). Gute Beschichtungen halten diese Beanspruchungen aus ohne daß die Splittkörnchen von der Epoxiharzbeschichtung herausgerissen werden (Abb. 5).

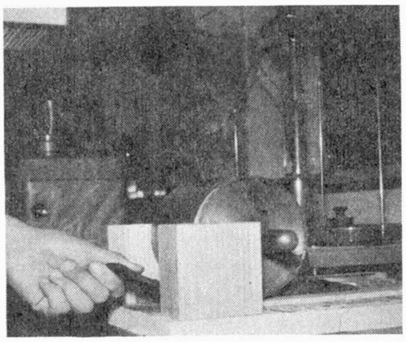


Abb. 2

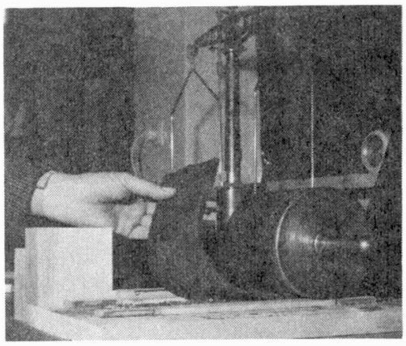


Abb. 3

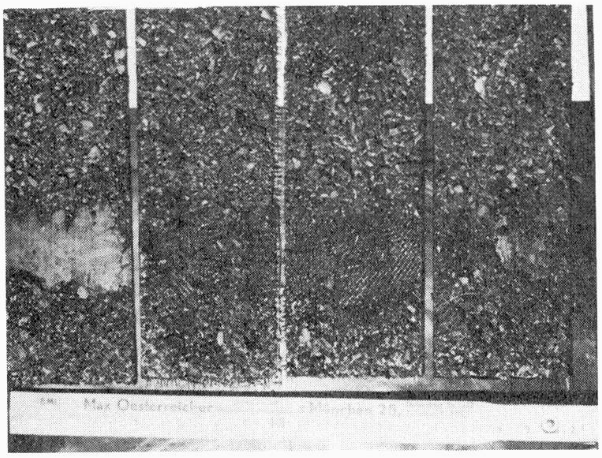


Abb. 4

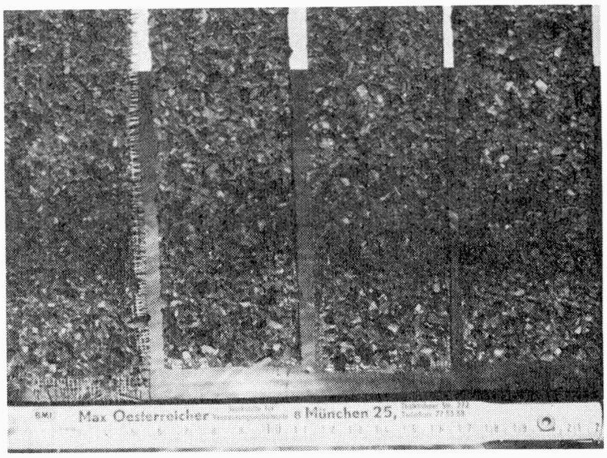


Abb. 5

3.) Gute Standfestigkeit und Rißfreiheit

Diese beiden Forderungen können mit den in den vergangenen Jahrzehnten hauptsächlich verwendeten Gußasphaltdecken oft nur sehr unvollkommen erfüllt werden, weil die beiden Forderungen gegensätzliche Eigenschaften des Gußasphaltes voraussetzen. Ein standfester Gußasphalt müßte ein sehr hartes Bitumen haben, während ein rißfreier Gußasphalt ein weicheres Bitumen verlangt. Die Folgen dieser Schwierigkeiten sind häufig Verwalkungen, Spurrillenbildungen, Querwellen und Längswellen und Risse in den aus Gußasphalt hergestellten Fahrbahnbelägen auf orthotropen Stahlplatten. Mit der Verwendung von Asphaltbeton anstelle von Gußasphalt können die oben beschriebenen Schwierigkeiten vermieden werden, weil der Asphaltbeton infolge seines, wenn auch nur geringen Porengehaltes, und wegen seines Mineralgerüstes, und wegen des bei seiner Herstellung verwendeten weicheren Bitumens (bei geringerem Bitumengehalt) bei allen auftretenden Temperaturen standfest und rissefrei bleibt.

Die erste Asphaltbetonschicht ist 1 cm stark und besteht aus einer Mineralmasse der Körnung 0 - 3 mm (Sandasphalt aus Brechsand) und hat einen Bitumengehalt von ca. 7,5 Gew.% und dient als Schutzschicht für den Epoxiharzanstrich und als Verbindungsglied zwischen der Haftbrücke und dem eigentlichen Asphaltbelag.

Die zweite Schicht dient zum Ausgleich der Unebenheiten in der Stahlplatte und ist ein Asphaltbeton der Körnung 0 - 8 mm bzw. 0 - 12 mm, je nach der gewünschten Stärke zwischen 2,5 und 3,5 cm. Der Hohlraumgehalt liegt zwischen 3 und 4 Vol.-%.

Die dritte Schicht, die Fahrbahndeckschicht, ist ebenfalls ein Asphaltbeton der Körnung 0 - 8 oder 0 - 12 mm, je nach der gewünschten Stärke von 2,5 bis 3,5 cm. Diese soll nicht nur standfest sein, sondern auch auf die Dauer gut griffig bleiben. Sie soll außerdem eine möglichst große Verschleißfestigkeit haben. Diese Forderungen verlangen einander etwas widersprechende Eigenschaften, welche unserer Meinung nach durch den VABIT-Belag gut erfüllt werden können. Wir haben Grund zu der Annahme, daß ein Asphaltbeton-Mischgut eine besondere Güte bekommt, wenn zu seiner Herstellung ein mit bituminösem Bindemittel imprägnierter Füller verwendet wird. Daher setzen wir hierfür unseren bituminösen präparierten VABIT-Füller ein, wovon der ganze Brückenbelag seinen Namen „VABIT-Belag" hat. Die VABIT-Deckschicht wird möglichst dicht aufgebaut, so daß sie nach der Verdichtung nur noch einen Porengehalt von ca. 2 Vol.-% hat. Damit wird erreicht, daß sie eine sehr gute Verschleißfestigkeit hat und doch noch die erforderliche Standfestigkeit behält. Es ist natürlich damit zu rechnen, daß trotz des geringen Porengehaltes etwas Tagwasser in den Belag eindringen kann, Dieses Wasser kann zwar nach unten nicht wegsickern, es besteht jedoch kein Anzeichen dafür, daß diese geringen Wassermengen im Fahrbahnbelag zu Schäden Anlaß geben können; im Gegenteil, es gibt viele Beispiele dafür, daß sie unschädlich sind.

4.) Die unter e) mit l) auf Seite 1 aufgezählten Forderungen können mit dem VABIT-Brückenbelag ebenfalls erfüllt werden.

Die Ebenflächigkeit (Pkt. e) und i) der Belagsoberfläche kann infolge des Fertigereinsatzes immer gewährleistet werden, wobei der besondere Vorteil zu verzeichnen ist, daß Unebenheiten, die in der Stahlplatte immer vorhanden sind, in den beiden untersten Schichten, vornehmlich in der 2. Schicht, ausgeglichen werden können.

Die Griffigkeit (Pkt. f) der VABIT-Beläge ist nach den bisher vorliegenden Erfahrungen höher als von den meisten normalen Asphaltfeinbetondecken. Sie ist der Griffigkeit einer glatt gefahrenen Gußasphaltdecke überlegen. Die Griffigkeit nimmt erfahrungsgemäß im Lauf der Jahre zu, während sie beim Gußasphalt abnimmt.

Zur Erzielung einer möglichst großen Verschleißfestigkeit (Pkt. g) sind im Vergleich zur Griffigkeit entgegengesetzte Belagseigenschaften erforderlich. In erster Linie ein höherer Bitumengehalt und ein geringerer Porengehalt. Je mehr man diesen beiden Forderungen nachgibt, umso mehr nähert man sich den Verhältnissen wie sie bei Gußasphaltdecken gegeben sind, insbes. kann Blasenbildung auftreten, wenn die Fahrbahndeckschicht infolge von Ungenauigkeiten der Praxis statt des angestrebten Porengehaltes von 2 Vol-% nach der Verdichtung z.B. nur noch 1 Vol.-% oder weniger hat.

Der Einbau aller 3 Schichten (Pkt. k) erfolgt mit einem gummibereiften Schwarzdeckenfertiger und das Verdichten wird mit einer 4 bis 6 to Tandem-Glattmantelwalze, einer 12 to Dreiradwalze und einer 15 to Gummiradwalze vorgenommen, so daß der jeweils erforderliche Verdichtungsgrad mit Sicherheit erreicht werden kann.

Eine wirtschaftliche Reparaturmöglichkeit besteht darin, daß nach einem eingetretenen starken Verschleiß der Deckschicht ohne besondere Schwierigkeiten eine neue, etwa 2 cm starke Deckschicht aufgebracht werden kann. Schließlich kann auch ein Abschälen bzw. Abfräsen der oberen Schicht mit einem Spezialgerät und Erneuerung der beiden oberen Lagen in Betracht gezogen werden. Auf keinen Fall braucht die Stahlplatte bloßgelegt und erneut gesandstrahlt und beschichtet werden.

5.) Pulsatorversuche

Zur Klärung des Zusammenwirkens von Stahlfahrbahnplatte, Epoxiharz-Haftbrücke und Asphaltbeton-Belag sind im März und April 1965 Pulsatorversuche an 14 Versuchsplatten von 80 cm x 80 cm Größe auf 12 mm Stahlblechplatten durchgeführt worden. Der zu prüfende VABIT-Belag war wie folgt aufgebaut:

1. Entrostung der Stahlplatten durch Sandstrahlen metallisch blank
2. Beschichtung mit Epoxiharz in 2 Schichten
3. Einstreuen von Feinsplitt 1 - 3 mm in die noch weiche 2. Schicht
4. Entfernen der nicht fest eingebundenen Splittkörner mit Hilfe von Preßluft
5. Oberseite der kunststoffbeschichteten eingestreuten Platten anspritzen mit 0,5 kg/cm^2 Lackbitumen aus Bitumen 80 und Testbenzin, im Mischungsverhältnis 1 : 1.
6. Aufbringen der sandasphaltartigen Isolier-Schutzschicht mit ca. 1,5 cm Stärke (Gesamtbindemittelgehalt einschließlich Bitumen des VABIT-Füllers betrug 7,8 bis 8 Gew.%).
7. Versiegeln der Isolier-Schutzschicht mit ca. 0,2 kg/qm Lackbitumen
8. Aufbringen der ca. 2,5 cm starken Ausgleichschicht aus Asphaltfeinbeton mit ca. 5 Gew.% Bitumen 80.
9. Aufbringen der ca. 2,0 cm starken VABIT-Deckschicht aus Asphaltfeinbeton mit 13,5 Gew.% VABIT-Füller und 6,0 bis 6,2 Gew.% Bitumen 80.

Für den dauernden Bestand eines Brückenbelages auf einer orthotropen Platte ist eine wesentliche Voraussetzung die vollkommene Erhaltung des Verbundes zwischen der orthotropen Platte und dem Asphaltbelag. Die orthotropen Stahlfahrbahnbleche erfahren durch die Verkehrsbelastung erhebliche Durchbiegungen, welche der Asphaltbelag mitmachen muß, ohne zerstört zu werden. Er darf in der Zugzone keine Risse bekommen und er darf vom Stahlblech nicht abplatzen. Der Elastizitätsmodul des Asphaltbelages hat bei kurzfristigen Belastungen, wie sie der Verkehr mit sich bringt, und besonders bei tiefen Temperaturen eine beachtliche Größe. Wie die Untersuchungen bestätigt haben, kann er ohne weiteres bis auf 300 000 kp/cm^2 steigen. Daraus ist ersichtlich, daß sich der Asphaltbelag in Verbundwirkung mit dem Stahlblech in erheblichem Maß an der Lastaufnahme beteiligt. Dies wiederum verursacht in dem Asphaltbelag sowohl Zugspannungen als auch Schubspannungen, die er aufzunehmen imstande sein muß.
Die Aufgabe bestand also darin, durch pulsierende Lastaufbringung auf die Versuchsplatten die Verkehrsbelastung, und durch Variation der Temperatur den Wechsel der Jahreszeiten nachzuahmen und festzustellen, ob durch die dabei auftretenden Spannungen zu einer Zerstörung des Belages durch Auftreten von Rissen bzw. durch Überwinden der Schubfestigkeit zwischen Stahl und Asphaltbelag und damit Loslösen des Belages von der Stahlplatte führen. Die Anzahl der Lastwechsel sollte so gewählt werden, daß die in der Praxis im Verlauf von

ca. 10 Wintern bei starker Verkehrsbelastung auftretenden Frostperioden erfaßt würden.

Die Festlegung der Prüfbedingungen erfolgte aufgrund der oben angegebenen Aufgabenstellung in Zusammenarbeit mit dem Institut für Eisenbahnbau und Straßenbau der Technischen Hochschule München (Dir. Prof. Dr. Ing. Maier), vertreten durch Privatdozent Dr. Ing. J Eisenmann, dem Institut für bituminöse Baustoffe der Technischen Hochschule München, Herrn Dipl.-Ing. S c h u l z e und Herrn Dipl.-Ing. Hans G r a s s l , Ing.Büro Düsseldorf.

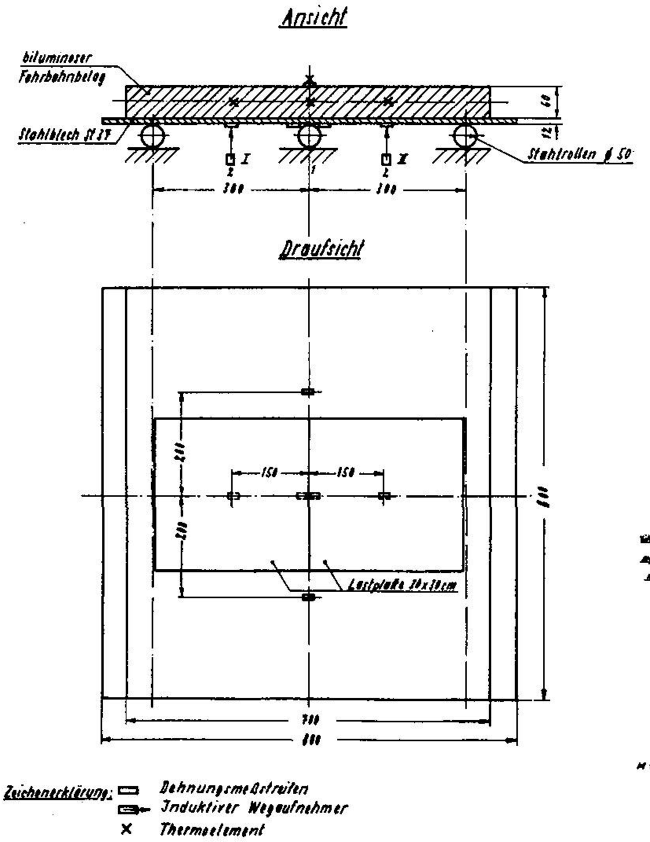


Abb. 6

Die ganze Versuchsanordnung wurde in einem Isolierkasten untergebracht, in dessen Innerem die Versuchstemperatur genau reguliert werden konnte.

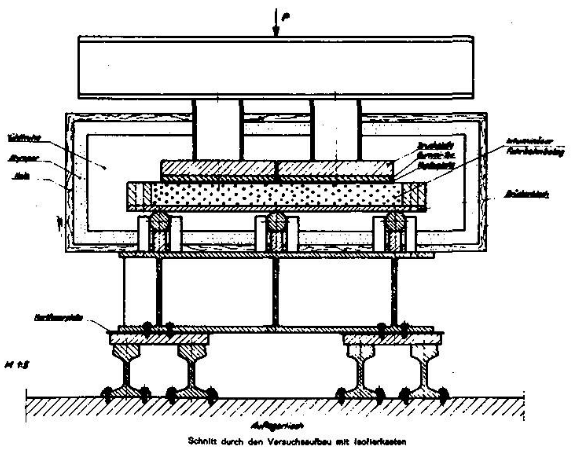


Abb. 7

Auflagerung der Versuchsplatten

Die Wahl der Lagerung auf 3 Stützen erfolgte deshalb, weil damit erreicht wird, daß auch konvexe Durchbiegungen auftreten, da der Asphalt sowohl Druckspannungen als auch Zugspannungen bekommt, wie es in der Praxis auch vorkommt.

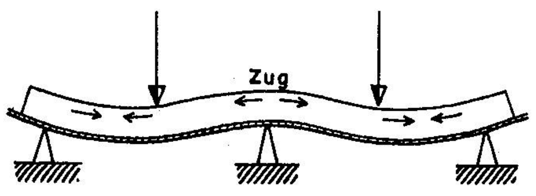


Abb. 8: Zug- und Druckspannungen im Asphaltbelag

Die Auflagerung der Brückenplatte auf stählernen Rollen anstelle der in der Praxis vorhandenen Stegbleche ergibt die günstigste Nachahmung der in der Praxis vorhandenen statischen Bedingungen. Für die Lasteintragung sind quadratische gelochte Gummiplatten von 30 x 30 cm gewählt worden, die ähnlich der Aufstandsfläche eines Gummireifens einen ungefähr gleichmäßigen Flächendruck auf die Oberseite des Asphaltbelages ausüben sollten. Die Versuchsplatten sind im Dauerversuch mindestens ebenso stark auf Biegung beansprucht worden wie das in der Praxis unter ungünstigsten Bedingungen auch der Fall sein kann. Es wurden im Anschluß daran auch noch Laststeigerungen vorgenommen, die eine Größenordnung erreichten, wie sie in der Praxis nicht mehr auftritt. Damit sollten die Platten zum Bruch gezwungen werden.

Versuchstemperatur: Der Belag kann in der Praxis von + 50°C bis - 30°C haben. Die Pulsatorversuche sind jedoch nur zwischen + 20°C und - 30°C durchgeführt worden. Es wurde davon ausgegangen, daß für die Asphaltbeläge bei hohen Sommertemperaturen unter der Verkehrseinwirkung hauptsächlich die Gefahr eines Schiebens und Gleitens bzw. die Gefahr von nachteiligen Formänderungen in Gestalt von Wulst- und Wellenbildungen besteht. Bei tiefen Temperaturen dagegen überwiegt die Gefahr einer Rißbildung oder des Abplatzens von der Stahlplatte.

Die durch die Pulsatorversuche zu prüfende Belagskonstruktion hat sich auf Betonfahrbahnplatten von großen Straßenbrücken bereits seit längerer Zeit gut bewährt. Nachteilige Formänderungen und ein Schieben bzw. Gleiten des Asphaltbelages oder eine Wulst- bzw. Wellenbildung wurde dabei nicht festgestellt. Daraus kann geschlossen werden, daß die hohen Sommertemperaturen für die gewählte Belagskonstruktion auch auf Stahlbrücken ungefährlich ist. Auf die Anwendung hoher Sommertemperaturen wurde deshalb bei den Pulsatorversuchen verzichtet. Es wurde hauptsächlich Wert auf die tiefen Temperaturen gelegt, wobei allerdings auch eine kontinuierliche Veränderung der Temperaturen von + 20°C bis - 30°C der Prüfung zugrunde gelegt wurde, weil diese besonders ungünstig ist, da bei der allmählichen Abkühlung des Belages auch noch die Abkühlung - Zugspannung im Belag hinzukommt. Diese tritt auf, weil der Wärmeausdehnungskoeffizient der bituminösen Massen wesentlich größer ist als der vom Stahl.

Anzahl der Lastwechsel und Frequenz

Die Frequenz wurde mit etwa 3 Hz gewählt und entspricht annähernd der Lasteinwirkung unter einem mit 50 km/Std. fahrenden Rad. Die Anzahl der insgesamt aufzubringenden Lastwechsel bei Temperaturen tiefer als - 25°C mit 100.000 soll einer Lebensdauer von 10 Jahren entsprechen. Die Anzahl dieser Lastwechsel wurde zum Teil bis 230.000 gesteigert.

Herstellung der Versuchsbeläge

Die 14 Versuchs-Stahlplatten von 12 mm Blechstärke wurden auf einem Unterbeton verlegt, so daß nach dem Sandstrahlen und der Beschichtung mit Epoxiharz als Korrosions- und Haftschicht die 3 Asphaltschichten in einer 3 m breiten Bahn naturgetreu, maschinell mit Fertiger (Vögele, Super 100) eingebaut und jede Schicht mit einer Stahlmantelwalze von 10 to Gewicht verdichtet werden konnte. Am darauffolgenden Tag wurden die einzelnen Probeplatten von 80 x 80 cm Größe samt dem Asphaltbelag mit einem Trockenschneidgerät herausgeschnitten.

Durchgeführte Versuche:

1.) Statische Belastung bei + 35 ^{o}C bei Doppellast an 2 Probeplatten. Lastaufbringung allmählich bis 7 Mp, Mittelwert der Durchbiegung der Probeplatten in Feldmitte, zwischen Unterlast von 1,5 Mp und Oberlast von 7 Mp betrug 0,47 mm. Bei dieser Temperatur und allmählicher Laststeigerung ist eine mittragende Wirkung des Asphaltbelages nicht vorhanden. Der Elastizitätsmodul betrug entsprechend der theoretischen Untersuchungen ca. 100 kp/cm^2

2a) Dauerversuch bei - 30 ^{o}C abwechselnd mit Doppellast und Einzellast an 4 Probeplatten. 80000 Lastwechsel abwechselnd alle 20 000 Lastwechsel (später fortgesetzt bis 230 000 Lastwechsel) Frequenz 170 Lastwechsel/Min.
Doppellast: Oberlast 7 Mp - Unterlast 1,5 Mp
Einzellast: Oberlast 4,5 Mp - Unterlast 1,5 Mp
Starke mittragende Wirkung des Asphaltbelages entsprechend der theoretischen Untersuchungen mit einem dynamischen Elastizitätsmodul von ca. 300 000 kp/cm^2 Sowohl Durchbiegung als auch Dehnung blieben während des Dauerversuches nahezu konstant. Dies läßt darauf schließen, daß während des Dauerversuches keine Lösung des Verbundes zwischen bituminösem Belag und der Fahrbahnplatte auftrat.

2b) Dauerversuch mit fallender Temperatur von + 20 ^{o}C bis - 30 ^{o}C (2 Probeplatten aus Versuch 1) 100 000 Lastwechsel mit Doppellast Frequenz 170 Lastwechsel/Min.
Oberlast 7 Mp - Unterlast 1,5 Mp.
Die Versuche zeigen, daß mit fallender Temperatur die Dehnungen und Durchbiegungen abnehmen. Im Bereich von $\pm$ 0 ^{o}C bis - 10 ^{o}C stellt sich die größte Steifigkeit ein. Bei weiter abfallenden Temperaturen nimmt die Dehnung immer mehr ab bis zu Versuchsende bei - 30 ^{o}C. Dies läßt darauf schließen, daß keine Störung des Verbundes aufgetreten ist. Das Abnehmen der Durchbiegung und Dehnung bei fallender Temperatur ist auf das temperaturabhängige Anwachsen des Elastizitätsmoduls des Fahrbahnbelages zurückzuführen. Die Folge ist ein Anwachsen der mittragenden Wirkung des Fahrbahnbelages. Durchbiegung und Dehnung sowie die mittragende Wirkung des Belages sind außerdem von der Dauer der Lasteinwirkung abhängig. Die statischen Eichungen ergaben stets wesentlich höhere Werte für Dehnung und Durchbiegung als die unter der pulsierenden Last gemessenen. Eine Verdoppelung der Lastwechselzahl beim pulsierenden Dauerversuch von 170 LW/Min. auf 340 LW/Min. erbrachte indes keine Änderung. Eine theoretische Untersuchung, bei der die gemessenen Dehnungen mit der unter der Annahme eines Verbundsystems gerechneten Spannungen verglichen wurde (siehe Anhang) zeigt, daß von einer Temperatur von - 20 ^{o}C abwärts, der dynamische E-Modul des Belages etwa 300 000 kp/cm^2 beträgt. Die theoretische Untersuchung zeigt weiter daß die Schubspannung zwischen Stahlblech und bituminösem Belag bei einer Temperatur von + 20 ^{o}C in der Größenordnung von 8 kp/cm^2 bis 9 kp/cm^2 liegt. Der Größtwert von 10,6 kp/cm^2 stellt sich bei einem Elastizitätsmodul von etwa 84 000 kp/cm^2 ein. Bei einer Temperatur von - 10 ^{o}C bis - 20 ^{o}C. Die mittragende Wirkung des bituminösen Belages ist demnach bei pulsierender Last (Frequenz 170 LW/Min.) bereits in dem Temperaturbereich von + 20 ^{o}C in starkem Maße vorhanden.

3.) Erster Zusatzversuch von + 20°C bis -30°C nach einer vorausgegangenen einwöchigen Flüssigkeitslagerung der Versuchsplatte (Wasser + Spiritus 70/30) Dabei konnte kein nachteiliger Einfluß festgestellt werden.

4.) Zweiter Zusatzversuch. Im Anschluß an 2 b) weiterer Dauerversuch mit verkleinerter Lasteinwirkungsfläche von 20 x 20 cm bei - 30°C, 20 000 LW, ergibt über die Mittelstütze doppelt so große Dehnungen wie bei Versuch 2 b). Es traten trotzdem kein Riß und keine Lösung des Verbundes auf.

Schlußbetrachtung von Herrn Dipl.-Ing. Schulze, Institut für bituminöse Baustoffe der Technischen Hochschule München.

Die Ergebnisse der Dauerversuche mit pulsierender Belastung haben sowohl bei konstanter Temperatur von - 30°C als auch bei laufender Temperaturabnahme von + 20°C bis auf - 30°C gezeigt, daß die geprüften Asphaltbeläge unter den Prüfbedingungen keine nachteiligen Änderungen erleiden.

Vorstehend beschriebene Dauerversuche an Asphaltbetonbelägen mit VABIT-Aufbau auf orthotropen Stahlplatten unter pulsierender Belastung haben gezeigt, daß die gewählte Belagskonstruktion sehr hohen Beanspruchungen standhält, ohne abzuplatzen oder Risse zu bekommen. Die bei den Versuchen aufgewendete mechanische Beanspruchung der Belagskonstruktion bei Temperaturen von - 30°C ist erheblich größer gewesenals sie es in der Praxis innerhalb von 10 Jahren sein kann. Wenn die Belagskonstruktion diese Belastung ausgehalten hat, ist daraus zu folgern, daß der Belag diejenigen Voraussetzungen für seine Bewährung in der Praxis mitbringt, die man in einem Laboratoriumsversuch überprüfen kann. Es kann deshalb angenommen werden, daß sich ein bituminöser Belag mit dem hier zur Debatte stehenden Aufbau als Fahrbahnbelag auf einer orthotropen Stahlplatte auch in der Praxis bewähren wird.

Einschränkungen:

a) Die Aufrechterhaltung einer Belastung von 1,5 Mp als Unterlast hat selbstverständlich zu einem dauernden Andrücken des Asphaltbelages an die Stahlplatte geführt, was etwas günstigere Bedingungen ergibt als in der Praxis, wo nach Überrollen eines LKW-Rades wieder eine völlige Entlastung eintritt. Da aber die für die Einbringung der Last gewählten Gummiplatten nur 30 x 30 cm groß waren, ist neben den Gummiplatten auf beiden Seiten noch ein Feld von 25 cm Breite unbelastet geblieben, so daß hier Schubspannungen zu einem Abplatzen führen konnten, ohne daß der Belag von oben her an die Stahlplatte gedrückt worden ist.

b) Die evtl. zu erwartenden Alterungserscheinungen der Epoxiharzschicht, die als Haftschicht zwischen orthotroper Stahlplatte und dem Asphaltbelag wirkt, kann natürlich im Laboratoriumsversuch nicht berücksichtigt werden. Erfahrungsgemäß braucht aber eine nennenswerte Alterung nicht befürchtet zu werden, weil man ja voraussetzen kann, daß die Kunststoffschicht durch den aufgebrachten Asphaltbetonbelag vor Wasser und Sauerstoff geschützt bleibt.

c) Einfluß der Witterung und Einfluß der Zeit können natürlich im Laboratoriumsversuch auch nicht erfaßt werden.

Es bestehen jedoch aufgrund der durchgeführten Dauerversuche mit pulsierender Belastung bei tiefen Temperaturen gute Aussichten für eine Dauerbewährung in der Praxis.

Anhang x)

Erläuterung und Ergebnis der theoretischen Untersuchung

Bei der zu untersuchenden Platte handelt es sich um eine Verbundkonstruktion. Der Asphaltbelag wird sich je nach Größe seines Elastizitätsmoduls mehr oder weniger an der Tragwirkung beteiligen. Dabei werden zwischen der Fahrbahnplatte und dem Asphaltbelag Schubspannungen aktiviert.

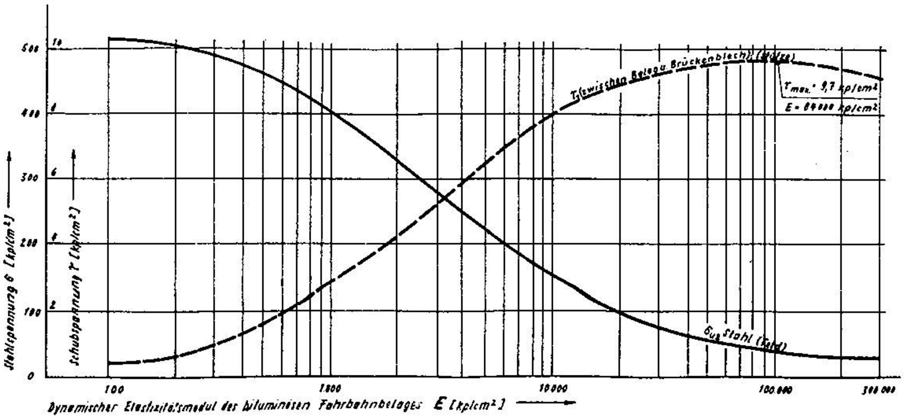
Für die Berechnung der Stahlspannung auf der Blechunterseite in Feldmitte und Schubspannung in der Verbundfuge über dem mittleren Auflager sowie der Spannungen an der Oberseite des bituminösen Belages in Feldmitte und über der Stütze wurden eine lineare Spannungsverteilung, ein Ebenbleiben der Querschnitte, ein starrer Verbund und ein konstanter Elastizitätsmodul des mehrschichtigen Belages zugrunde gelegt. Die Berechnung wurde für eine Radlast von 5 Mp und eine Belastung entsprechend den Versuchsbedingungen durchgeführt. Bei einem Vergleich der theoretischen Werte mit den Versuchswerten ist entsprechend der höheren pulsierenden Last von 5, 5 Mp eine Umrechnung erforderlich.

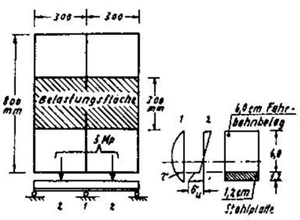
Zur Ermittlung der Momente für die Berechnung der Biegespannungen dienten die Einflußfelder elastischer Platten von A. P u c h e r (Wien, Springer-Verlag, 1958); die Querkraft über der Mittelstütze für die Berechnung der Schubspannung wurde nach den Tafeln von E. B i t t n e r (Wien, Springer-Verlag, 1938) bestimmt. Ein Vergleich mit den Werten nach der Balkentheorie zeigt, daß das Feldmoment bei einem Balken um 24 %, die Querkraft um 12 % größer wird als bei der Platte.

Da der Elastizitätsmodul des Belages stark von der Temperatur und der Belastungsgeschwindigkeit abhängt, wurden die Spannungen für einen Bereich von 100 kp/cm^2 bis 300 000 kp/cm^2 berechnet und in den Abb. 4 und 5 aufgetragen. Der Verlauf der Nullinie ist in Abb. 6 aufgeführt.

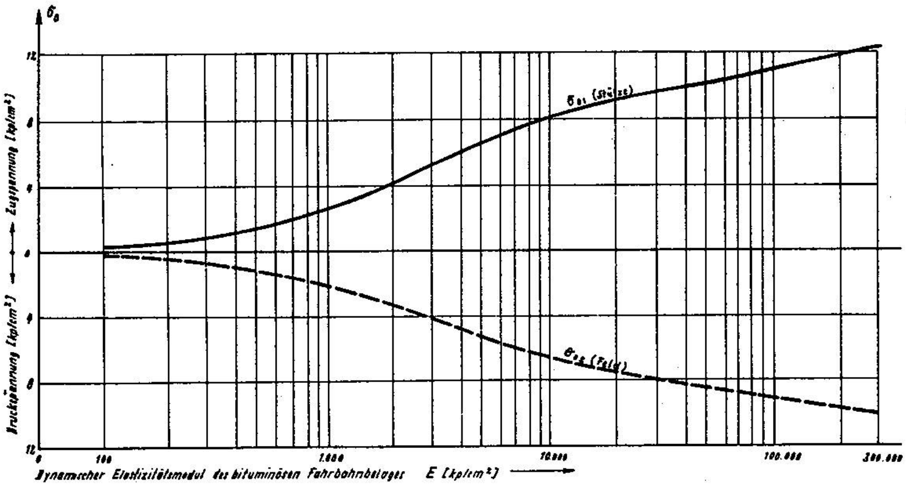
Aus den Abb. 4 und 5 ist gut ersichtlich, wie bei wachsendem Elastizitätsmodul des Belages die Stahlspannung abnimmt, während die Schubspannung entsprechend anwächst. Die größte Schubspannung von 9, 7 kp/cm^2 tritt bei einem Elastizitätsmodul von 84 000 kp/cm^2 auf.

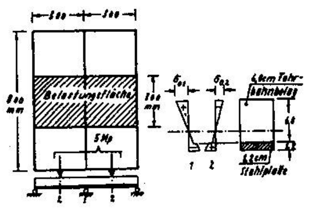
x) Die theoretische Untersuchung wurde im Benehmen mit Herrn Dipl. -Ing. G r a s s l, Düsseldorf, durchgeführt.



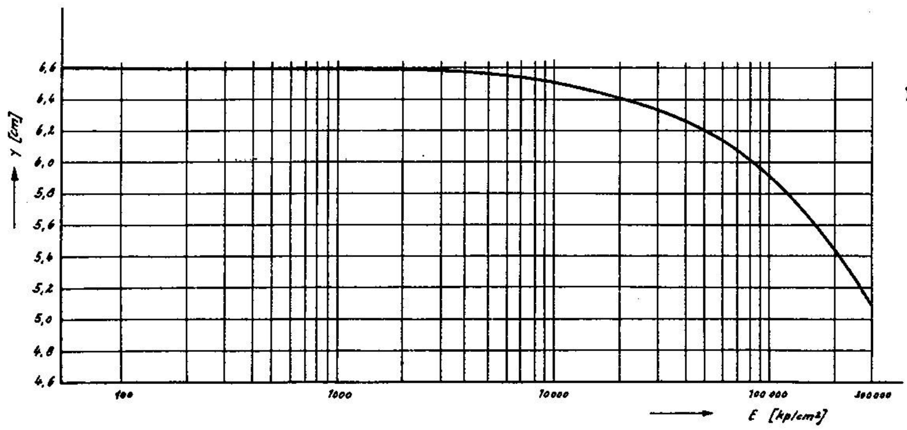


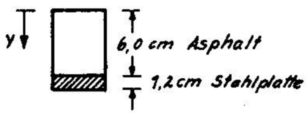
Stahlspannung auf der Fahrbahnplattenunterseite in Feldmitte und Schubspannung in der Verbundfuge über dem mittleren Auflager in Abhängigkeit vom dynamischen Elastizitätsmodul des bituminösen Fahrbahnbelages





Spannung auf der Oberseite des bituminösen Belages in Feldmitte und über der Stütze in Abhängigkeit vom dynamischen Elastizitätsmodul des bituminösen Fahrbahnbelages





Verlauf der Nullinie in Abhängigkeit vom dynamischen Elastizitätsmodul des bituminösen Fahrbahnbelages

ZUSAMMENFASSUNG

Es wird ein bituminöser Fahrbahnbelag (VABIT-Belag) für stählerne Leichtfahrbahnen gezeigt, welcher eine Kombination darstellt, bestehend aus Epoxiharzanstrich als Rostschutz und Haftbrücke und aus 3 Schichten Asphaltbeton. Es wird erläutert, auf welche Art und Weise er die gestellte Aufgabe erfüllt. Ferner wird über die eigenen Laborversuche berichtet, welche zur Auswahl des Epoxiharzes erforderlich waren und über Pulsatorversuche, welche den labormäßigen Nachweis erbrachten, daß auch bei —30° C der Verbund zwischen Stahlblech und Fahrbahnbelag nicht zerstört wird.

SUMMARY

A bituminous wearing surface (VABIT) for steel bridge lightweight constructions has been described, which is a combination consisting of epoxy resin coating for protection against rust as well as bonding agent and of three layers of asphaltic concrete. The manner and the ways in which it serves its purpose have been explained. Furthermore, an account has been given of a series of experiments in our laboratories, necessary for the selection of the suitable epoxy resin and of pulsating tests, which have brought laboratory proof that even at —30° C the bond between steel deck and wearing surface will not be destroyed.

RESUME

On présente ici un revêtement de chaussée bitumineux pour chaussées légères en acier (VABIT) qui se compose d'une couche d'apprêt en résine epoxide faisant fonction de couche anti-rouille et de couche adhérente, et de trois couches de béton bitumineux. On précise également sous quelle forme et de quelle manière ce revêtement remplit la tâche qui lui est assignée. On y traite en outre des essais de laboratoire exécutés par l'Entreprise et nécessaires au choix de la résine epoxide utilisée, mais aussi des essais vibratoires et qui ont apporté la preuve expérimentale que, même par des températures de —30° C, l'adhésion du revêtement routier sur la tôle d'acier subsiste.

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SURFACES ON SEVERN AND WYE BRIDGES

Revêtements sur les ponts Severn et Wye

Belâge auf der Severn- und Wye-Brücke

A.D. HOLLAND
Ministry of Transport
London

1. Principles

The principles involved in the surfacing of these bridge decks were that

(a) The steel was metal sprayed and painted to provide an immediate protection

(b) A rubber bitumen waterproofing membrane was applied between the immediate protection to the steelwork and the running surface

(c) The running surface would be flexible enough to accommodate flexing of the orthotropic plate, but sufficiently rigid to resist deformation due to traffic and hot weather.

The supporting deck in both cases consisted of a 7/16"(11 mm) hightensile steel plate carried on 9" (229 mm) deep troughs 12" (305 mm) wide at 2' (610 mm) centres spanning between diaphragms 15' (4.57 m) apart.

2. Steel Protection

The deck sections were shot blasted, zinc sprayed to a minimum thickness of .003" (.076 mm) and given a coat of etch primer. Edges were masked for a width of 2" (51 mm) for subsequent welding. After all the seams had been welded they were shot blasted, metal sprayed and given a coat of etch primer so that the protection of the steel plate to be surfaced was uniform throughout.

3. Surfacing

To ensure maximum adhesion of the membrane, the etch primed surface was mechanically scrubbed with an industrial detergent and washed clean with water. As soon as the surface was dry, it was primed with a proprietary reclaimed rubber adhesive, Bostik 1255, which as soon as the solvents had evaporated, was covered by the 1/8" (3.2 mm) thick waterproof membrane spread over by means of squeegees. The mastic asphalt was immediately laid by hand to a thickness of 1 3/8" (35 mm) so that the process for a given area of deck was virtually continvous between the commencement of cleaning and the completion of the asphalting.

4. Facts and Figures

	Severn	Wye including Beachley Viaduct
Length of bridge	5240 ft (1597 m)	3783 ft (1153 m)
Area Area surfaced	37000 yd^{2*} (30936 m^2)	29700 yd^{2*} (24800 m^2)
Plant and Labour employed	One 2 1/2 ton (2.5 tonnes) mixer for rubber bitumen. Five 8 ton (8 tonnes) mixers for asphalt. Twenty eight men of whom 12 were engaged in laying the asphalt.	similar
Duration of work	Thirteen weeks	Eleven weeks
Cost (including priming and waterproofing membrane	37/- per yd^2 (44/- per m^2)	37/- per yd^2 (44/- per m^2)

The cost of waterproofing a concrete deck and machine laying base course and wearing course totalling 3" (76 mm) is approximately 32/- per sq. yd. (38/- per m^2).

* These figures are not proportional to the bridge lengths because there is a variation in width from 64' (19.5 m) on Severn to 69' (21 m) on Wye.

5. Specification

Extracts from the specification for workmanship and materials for the membrane, and asphalt surfacing are given in Appendix A.

6. Points to Note

The asphalt was laid at a temperature of over 200°C and in planning the programme for laying material at this temperature on steel decks, consideration should always be given to the thermal effects induced, as if too great an area of deck plate is subjected to a temperature of this order very large forces will be induced in the immediate locality.

In Wye Bridge the main webs join the deck plate near the centre of the slow lanes and it was expected that the surfacing at this point would be more susceptible to cracking. Successful control was achieved by cutting a groove in the surfacing 3/8" (9.5 mm) wide and 5/8" (16 mm) deep and filling it with a proprietary sealing compound. The groove was omitted from a limited length of deck to see how the surfacing would behave, but cracks soon formed and the groove was provided throughout.

7. Performance

The deck has now been in use for two years and its performance has generally been satisfactory. The volume and weight of commercial traffic on one lane has been recorded at intervals, and the number of axles related to their weights is given in the following Table.

Traffic Data

Total number of vehicles through tollgates since bridge opening on 8th September 1966 to 1st August 1968 = 11 022 282

Number of axles on Eastbound slow lane :-

Axle weight lb. x 10^{-3}	Axle weight kg. x 10^{-3}	Weekly average recorded between 8th July 1967 and 3rd August 1967
0 - 4	0 - 1.81	85695
4 - 8	1.81 - 3.63	19083
8 - 12	3.63 - 5.44	7843
12 - 16	5.44 - 7.26	3733
16 - 20	7.26 - 9.07	2309
20 - 24	9.07 - 10.89	815
24 - 28	10.89 - 12.70	205
28 - 32	12.70 - 14.51	86
32 - 36	14.51 - 16.33	74
> 36	> 16.33	35

The troubles which have been experienced can be summarised as :-

(a) In July 1968 an exceptional spell of hot weather led to the sudden appearance of a few blisters in the surfacing and these were generally pierced, heated and rolled flat though in some cases they were cut out and replaced with fresh material.

(b) Some flow of the mastic asphalt down the camber has occurred in one or two places and the maximum movement recorded is about 3" (76 mm). Consideration is being given to the provision of a 1 1/2" x 1 1/2" (38 mm x 38 mm) steel angle attached to the deck in order to restrain the tendency to flow.

(c) In one case a daywork joint in the mastic has opened into a crack which penetrates to the waterproofing membrane, this has been opened out and sealed with a proprietary sealer, Pliastic 164.

8. Conclusions

The problem of surfacing the orthotropic plate deck is difficult and a considerable amount of development work will be required before a completely satisfactory result can be achieved. It would be desirable to save some of the dead weight of the surfacing in long span bridges, in Severn it amounted to about 2, 600 tons (2640 tonnes), but the thin resin carpets have not yet proved sufficiently durable. The hand laid mastic is expensive in labour and needs to be done in good weather, but apart from the penalty of weight, it appears to give the most dependable performance of materials presently available though the correct balance between flexibility and lack of deformation is difficult to achieve.

APPENDIX A

RUBBER BITUMEN COMPOUND (waterproofing membrane)

1. Materials

The compound shall be prepared by blending 90/100 Pen. bitumen with limestone powder, of which not less than 85% passes a 200 mesh sieve. The proportions shall be approximately :-

Bitumen	25 - 30%
Limestone	70 - 75%

The blend shall be adjusted to give a Softening Point (R. & B.) of 80 - 85°C.

Pulvatex rubber crumb shall be added to the extent of 1.3 - 1.5% of the whole by weight and the compound re-adjusted to a Softening Point of 90 - 100°C.

2. Application

The compound shall be re-melted on site and shall be spread onto the prepared base as speedily as possible, using squeegees or trowels, at a temperature not exceeding 205°C.

MASTIC ASPHALT

1. Materials

Gradings for high grade limestone containing not less than 85% of calcium carbonate :-

Passing 200 mesh sieve	45 - 55%
Passing 72 mesh sieve retained on 200 mesh	10 - 30%
Passing 25 mesh sieve retained on 72 mesh	10 - 30%
Passing 7 mesh sieve retained on 25 mesh	5 - 20%
Retained on 7 mesh sieve	Nil

(Sieve sizes are British Standard)

Trinidad Lake Asphaltic Cement :-

Specific Gravity at 25°C	1.17 - 1.25
Penetration at 25°C	20 - 30
Softening Point (R. & B.)	60 - 70°C
Mineral Matter (Ash)	17 - 19%
Loss on heating at 163°C for 5 hours	Max. 2%

As manufactured, the mastic asphalt shall have a soluble bitumen content of 14 - 15% exclusive of any coarse aggregate as hereinafter specified.

2. Preparation on Site

The mastic asphalt shall be delivered to the site in blocks of approximately 1/2 cwt. (25 kg) each. These shall be broken up and re-melted in mechanically agitated mixers and laid at a temperature not exceeding 240°C without the addition (save as hereinafter specified) of limestone dust, sand or any filler whatsoever.

3. Coarse Aggregate

During the process of re-melting, there shall be added to the mastic asphalt in the mixer 3/8" - 1/4" (9.5 mm - 6.4 mm) clean cubical granite chippings. The chippings shall be added in such proportions that it shall represent 40 - 45% by weight of the mixture as laid.

4. Hardness Number

The Hardness Number of the mastic asphalt as laid shall be 25 - 30 at 35°C.

5. Application

The mastic asphalt shall be laid to a thickness of 1 3/8 inch (35 mm) on to the steel plates wich have been cleaned, dried and primed with a special adhesive (Bostik 1255). On the adhesive shall be laid a special rubber / bitumen compound to approximately 1/8 inch (3.2 mm) thickness. The mode of spreading and finishing the mastic asphalt shall be in accordance with the requirements of British Standard 1447 : 1962, Clause 6.

6. Rubber Bitumen Seal to Vertical Steel Surfaces

At the junction of the asphalt surfacing with vertical steel surfaces, a 1/2 inch (12 mm) gap isto be left in the asphalt which is later to be filled by pouring in the hot rubber bitumen sealing compound equivalent to Dulastic "R".

7. Surfacing to the Asphalt

Except for a 9 inch (229 mm) wide strip at the edges and under the white marginal strips, pre-coated approved angular chippings, 1/2 inch (12 mm) size, conforming to B.S. 63 "Single-sized roadstone and chippings", shall be rolled into the asphalt whilst it is still warm, and in a plastic condition. The chippings shall be uniformly distributed at 100 to 150 sq. yds. per ton (85 - 127 m^2 per tonne).

8. Tolerances

The finished surface shall be such that when tested with a 10 ft. (3 m) straight edge the maximum departure from a true surface shall not exceed 1/4 inch (6.4 mm).

The maximum tolerance on the specified thickness shall be $\pm$ 1/8 inch ($\pm$ 3.2 mm).

Where these figures are exceeded, the surfacing shall be cut out and re-laid at the Contractor's expense.

SURFACING ON FORTH ROAD BRIDGE

Revêtement du tablier du pont routier de Forth

Fahrbahnbelag der Forth-Road-Brücke

W. HENDERSON
Scotland

Construction

A considerable amount of testing of various types of surface was carried out by Road Research Laboratories under working conditions prior to the final adoption of the material laid on this bridge.

The deck consists of panels of orthotropic steel construction, some 60' (18.5m) long and 24' (7m.) wide in each carriageway. The panels were prefabricated in 8' (2.4m.) widths, site welded together to complete the full carriageway width. The deck plate is $\frac{1}{2}$" (1.25cm) thick, stiffened longitudinally with steel troughs which provide stiffening webs at approximately 13$\frac{1}{2}$" (34cm) centres. These stiffeners span between cross members at approximately 10' centres (3m) which in turn are supported by stringers at 8' (2.4m) centres resting on the main truss cross girders at 30' (9m) centres. The stringers are in general continuous over two cross girders, joints being provided 5' (1.5m) from the main cross girders where the short cantilever of one stringer supports the 25' (7.5m) projection of the next. All of the steel in the deck is mild steel to B.S.15 since it was considered desirable to limit local deflection by using lower stresses in the interest of the road surface.

All of the steelwork on the bridge was grit blasted and zinc sprayed to a minimum thickness of .003 inches (0.76mm) in shop conditions under cover, the utmost care being taken to ensure that one process followed another without undue delay. The steelwork was then painted under factory conditions. On the steel plate surface to be covered with asphalt, the surface was painted with one coat of etch primer followed by a priming coat of bitumen paint, partly to protect the surface until such time as it was surfaced and partly as a key for the future surface. During the period of upwards of a year from the time of application of this bitumen paint until surfacing commenced, it was found that chemical changes had taken place which left it in a condition that nothing would adhere to it and it was necessary to remove the paint by light grit blasting.

Following closely on this cleaning process and on a thoroughly dust free surface, the steel plate was primed with a rubber/bitumen solvent primer

(Bostik 1255) at a coverage of 25 sq. yds. per gallon (4.5s.m/litre).

On this priming was laid a $\frac{1}{8}$" (.32cm) thick hot rubber bitumen compound having the following specification;

25% to 30% Bitumen (Penetration 90-110)
70% to 75% Limestone Filler (85% passing 200 mesh)
1.3% to 1.5% Pulvatex rubber powder to bring the softening point to approximately 95°C.

The final surface consisted of $1\frac{3}{8}$" (3.5cm) thickness of Lithocrete mastic asphalt filled with 40% to 45% of $\frac{3}{8}$" (1cm) crushed granite, to make the total thickness $1\frac{1}{2}$" (3.8cm).

The specification for this mastic asphalt which was, of course, hand laid was;

Penetration at 25°C 10-15
Loss on heating at 163°C for 5 hours 1% max.
Melting Point 65-70°C
Mineral Content 20-25%
Ductility at 25°C 0.5cm min.
Specific Gravity at 25°C 1.2 to 1.3

The limestone aggregate to be ground so that the following grading is met

Passes 7 mesh sieve	and retained on	25 mesh sieve	15-25%
" 25 " "	" "	" 72 " "	10-30%
" 72 " "	" "	" 200 " "	10-25%
" 200 " "	------------------------------		40-55%

The soluble bitumen content of the asphalt before filling with crushed granite to be 14-15%.

At the ends of each deck panel $1\frac{1}{2}$" (3.8cm) high steel asphalt stops are provided while at the sides steel kerbs perform the same function. A $\frac{1}{2}$" (1.25cm) wide rubber bitumen compound was used to seal the asphalt against the vertical steel surface.

Behaviour in use

The finished running surface has provided excellent riding quality and a high standard of skid resistance during the four years it has been in use. There is a marked tendency for vehicles to follow very closely the same tracks (no doubt the inclined protective railing on each side of each carriageway tends to discipline drivers to follow a more uniform position in lane than usual), but up to date there is no perceptible wear, polishing, or depression of the surface in these tracks.

Total annual traffic using the bridge has been;

1st year	4,665,000
2nd year	4,838,000
3rd year	5,378,000
4th year (9 months)	4,500,000

This has been made up of a fairly normal distribution of private cars, public service vehicles and heavy trucks.

In addition there has been a fairly large number of abnormal loads ranging from above 30 tons gross on the vehicle up to some 280 tons gross. The wheel loads on these vehicles range from 6 to approximately 9 tons.

About eight months after the bridge was opened to traffic it was observed that fine longitudal cracks were beginning to appear in the slow lane, some 8 ft (2.4m) from the outer kerb. These fine cracks extended rapidly to form a more or less continuous line along the full length of the main span decked with orthotropic plates. The location of this crack coincided with the position of high negative bending at a stringer. Further fine longitudinal cracks appeared close to the kerb of the slow lane at points where the corresponding stringer passed over the main cross girder. These cracks were generally some 9" (23cm) to 18" (46cm) long. At these points the stringer webs are stiffened with bearing stiffeners and so stiffer than elsewhere.

Some three to four months later similar fine cracks appeared some 8' (2.4m) from the kerb of the fast, less heavily used lane, again over a stringer, and close to the kerb at the point of support of the outer stringer over the cross girder.

In the case of each of the main longitudinal cracks, the tracks of vehicles straddle the stringers almost symmetrically and the location of these members is such that actual traffic takes up the position of causing maximum negative movement over this support. On these cracks, too, there was a tendency, where the stringer passes over the main cross girder and is reinforced with bearing stiffeners, for two short cracks to form on either side of the main longitudinal crack for a distance of 9" (23cm) to 18" (46cm) and then to return into it at an angle (Fig.1). In other cases the central crack terminated between the apices of these hexagons (Fig.2).

Careful examination showed that the bond between the asphalt, and the steel was not impaired. An area of asphalt was removed for inspection and it may be of some interest to note that over small patches the zinc spray applied to the steel surface pulled off with the asphalt.

Remedial measures were put in hand quickly; these consisted of making a "saw" cut ~~in~~ ½" (1.25cm) wide along the line of each crack to a depth of 1" (2.5cms). This cut was freed from dust by blowing out with compressed air and then filled by pouring in a sealant conforming with the current British Standard 2499(55), and with the American Federal Specification SS/S 164. The actual material used is a proprietory known as Expandite Pliastic 55.

This treatment has been eminently successful. The carriageway surfacing has been kept under regular surveillance and in general the sealant is serving its dual purpose of allowing flexure and at the same time preventing ingress of water. The two cuts 8' (2.4m) in from the sides of the deck are traversed from time to time by wheels which undoubtedly have had the effect of packing and consolidating the sealant into the "saw" cuts in the most effective way.

At a recent inspection of the deck in July of this year, following a spell of warm weather, it was found that in some places the sealant in the short cracks at the sides of the carriageway was being squeezed out, or had become detached from the sides of the "saw" cut. In these locations it is very unusual for wheels to travel and it has been concluded that this is the main reason for this deterioration.

Elsewhere, in a comparatively few places, fresh hair cracks have been found developing from the ends of these original short cracks, or from the corners of some of the hexagonal patterns for a few inches, or again, where the full hexagonal pattern had not completed itself it is now tending to do so. None of this is either extensive or serious. Measures are in hand to make fresh "saw" cuts along the length of the new cracks and to clean out those where the sealant has become detached from the sides of the cuts. In these areas where traffic will only infrequently roll on the filler it is proposed to attempt to pack the material firmly against the sides of the saw cut by tamping it with a hard wood tool after it has to some degree solidified.

In the foregoing a good deal more has been written about what could be regarded as comparatively minor defects than may appear to be justified. It is considered, however, that the wisest approach to ensuring long term economy and continuing good quality on carriageway surfaces supported on this type of deck is to maintain a vigilant inspection for early signs of defects and to take remedial action promptly. The general impression to be gained at this stage in the behaviour of the surfacing on Forth Bridge is that the amount of necessary remedial work of the sort described is now rapidly diminishing and the surfacing has a considerable life ahead of it. It is possible that the development of these cracks could have been anticipated at the outset and suitable provision made. This would have been no different to what has now been done and there is much to be said for allowing the cracks to develop in the more brittle material, thus defining the precise location required for the more ductile sealant insertion. The amount of labour involved is not very different whichever approach is adopted.

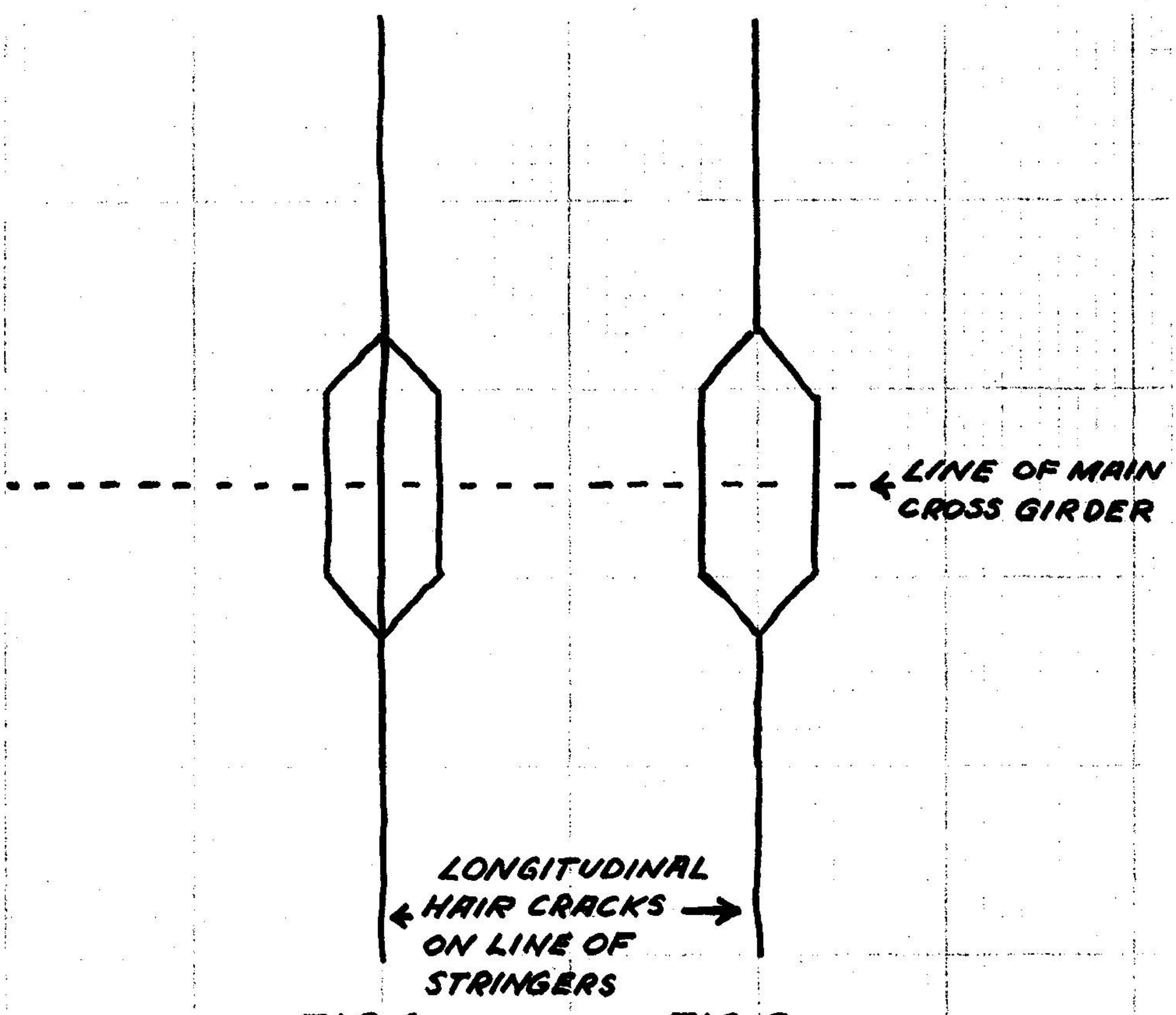

FIG.1. FIG.2.

SUMMARY

The contributor describes the nature and conditions of the mastic asphalt surface on the orthotropic steel deck of Forth Road Bridge. Specifications are given together with details of minor defects which have arisen in four years of use and the remedial measures taken.

RESUME

L'auteur décrit la composition et les conditions d'usage du revêtement en asphalte coulé sur le tablier orthotrope du pont routier de Forth. Il donne des spécifications ainsi que les détails de dégâts mineurs survenus après quatre années de service, et les mesures prises pour y remédier.

ZUSAMMENFASSUNG

Der Verfasser beschreibt das Verhalten und die Bedingungen des Gußasphaltbelages auf der orthotropen Stahlplatte der Forth-Road-Brücke. Erläuterungen zusammen mit Details kleiner Schäden, welche in den vier Jahren Gebrauch entstanden sind, sowie die getroffenen Verbesserungen werden angegeben.

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SURFACINGS ON AN EXPERIMENTAL DECK PANEL IN A HIGHWAY

Essais de revêtements sur un modèle de tablier de pont routier

Beläge auf einer Prüfdecke einer Straßenbrücke

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1. INTRODUCTION

For several years the Road Research Laboratory has been conducting experiments in the United Kingdom to determine the most suitable material for thin surfacings on steel bridge decks, in order that adequate protection for corrosion and good resistance to skidding shall be provided for many years with minimum maintenance and at low initial cost.

The experiments have sometimes been carried out on actual bridge decks, when suitable opportunities arose, and also on panels laid over pits in the highway. The techniques and previous results have been reported in papers to the 6th and 7th Congresses of I.A.B.S.E. (1) (2) A greater latitude in the range of the experiments is possible when the trials are carried out on panels; failures can be corrected quickly, with no risk to a major structure, and experimental sites with a wider range of traffic conditions can be considered. Moreover, better access can be obtained to the panels for the purposes of making measurements, than is usually the case on bridge decks. These were the main factors which influenced the form of the trial which is described in this paper, and which was carried out primarily to make a final selection of the most suitable surfacing material for the Severn Bridge.

2. THE EXPERIMENTAL SITE

An experimental site was chosen on a busy dual carriageway road (A40) on the western outskirts of London. Two deck panels, each 52 ft (15.8 m) long x 13 ft.(4.0 m) wide, were set, end to end, in the slow lane of the eastbound carriageway over a concrete structure (Fig.1). This has the dual function of supporting the deck plate in a manner similar to that in a bridge and of allowing access underneath it for measurements of deflections and strains. A weighbridge and vehicle detector are located in the road on the approach to the panels to record the loads and frequency of vehicle axles in the normal traffic stream over them.

In the $4\frac{1}{2}$ years that have elapsed since the panels were installed, 20×10^6 axles have passed over them. This is approximately equivalent to 9×10^6 vehicles. 13 per cent of the axle loads have been in excess of 4 tonf. (40 kN).

3. THE DECK PANELS

The two panels are of the design shown in Fig. 2. The deck plate is 7/16 in. (11mm) thick and has trough shaped stiffners in the longitudinal direction with welded connections at (12 in.) 300 mm. pitch transversely. The aim was to use a test panel of such a size that local stresses and deformations under wheel loading would be comparable with those in the larger panels forming the deck of a bridge. It was estimated that the tracks of vehicle wheels nearest the kerb would be concentrated in a strip 4 ft (1.2 m) wide and that a margin of at least 4 ft.(1.2 m) should be allowed on either side of this strip to reduce edge effects. A further reduction is obtained from the edge stiffening provided by the channel. In the longitudinal direction, the spacing between transverse diaphragms is shorter at the end of each panel than in the centre, to compensate for the loss of continuity at the ends.

4. THE ROAD SURFACINGS

The road surfacing on each panel has been divided into four sections. Three of the sections are 15 ft (4.6 m) long; they are so located that a transverse stiffener occurs under each one. The fourth section is 7 ft. (2.1 m) long, without a transverse stiffener.

One panel is surfaced with two sections of rolled asphalt and two sections of mastic asphalt, all of them $1\frac{1}{2}$ in.(38 mm) thick. The other panel is surfaced with resin-based materials, $\frac{3}{8}$ in. (9.5 mm) thick; three made with epoxy resins and one with a polyester resin. These are described in a paper under Theme III and they are only referred to briefly here since the comparison of their performance with the asphalts, under the same conditions of traffic, weather and support is of interest. The materials and composition of the surfacings were selected from those which had given the more favourable results in previous trials. The layout of the sections is given in Fig. 3.

The sequence of laying the asphalt sections on Panel 2, after it had been installed in the roadway, were as follows:-

(i). The plate, which was delivered with a zinc sprayed surface and coated with etch-primer, was brushed to remove any dirt present.

(ii). Tack coats and/or waterproofing layers were applied. On Section 2A, a fluid slurry consisting of equal parts of epoxy resin and fine sand was spread at 3 lb/yd^2 (1.6 kg/m^2): before it hardened this was sprinkled with $\frac{1}{4}$ in. (6.4 mm) dry chipping to enable the asphalt to key mechanically to it. On Sections 2B and 2C a thin priming coat of "Bostik 1255" was applied at about 25 to 40 sq. yd/gal (5 to 8 m^2/litre) and allowed to dry: over this a layer of filled rubber bitumen was applied with squeegees to a thickness of about 0.1 in (2.5 mm). This layer is sometimes referred to as the "insulating" layer or "cushion" layer. It consists of approximately 3 parts of limestone filler to 1 part of bitumen, with approximately 1½% of Pulvatex unvulcanised rubber powder added. The exact compositions are adjusted to give a final softening point (ring and ball) of 90 to 95°C. On Section 2D (the short section) a bitumen paint priming coat was applied thinly.

(iii). The asphalt wearing courses were laid. The rolled asphalt (hot process) complied with B.S. 594: 1961, Table 7, Schedule 1, with 30% of coarse aggregate; the asphaltic cement complied with Table 1, Column 3. This mixture is normally laid on the road by machine, but because of the limited size of the panels this was not possible, so that it was spread on Sections 2A and 2B by hand rakes. It was compacted in the usual manner with an 8 ton diesel roller. The mastic asphalt was spread on Sections 2C and 2D by hand with wooden floats and rolled with a hand roller. Its composition complied with B.S. 1447: 1962 and it contained 40% of coarse aggregate; the asphaltic cement conformed with Table 1, column 3.

(iv). Coated chippings of $\frac{3}{4}$ in (19 mm) size were rolled into all the asphalt sections on Panel 2 at about 100 sq. yd/ton (80 m^2/Mg).

The asphalt surfacings were laid in October, 1963 and opened to traffic in November, 1963.

5. PERFORMANCE OF SURFACINGS

The surfacings were inspected periodically for signs of cracking, deformation and wear. The observations may be summarised as follows:-

Section 2A.

A very fine crack, a few inches long, appeared during the Spring of 1964 in the nearside wheel track. It remained largely unchanged until the Winter of 1964/5 when the crack lengthened to extend almost over the length of the section by the Spring of 1965. By that time two intermittent cracks had developed in the outer wheel tracks, near the edge of the panel. During 1966, parallel cracks developed in the near side wheel track over the webs of the underlying stiffener, that is 6 in. (15 mm) on either side of the existing crack. By November, 1967, when the photograph in Fig. 4 was taken, a crazed pattern of cracks was established over a 12 in (300 mm) wide strip, with short crack appearing over the line of the next stiffener towards the centre of the panel. When last observed in July, 1968 (Fig. 5) a depression about ¼ in (6 mm) deep had formed in the centre of the strip. Cracks along the off-side wheel tracks had become more continuous, but these are not such good indicators of performance, because of the proximity of the edge of the panel. It is significant that cracks have not run over the transverse disphragm of the deck panel, but have terminated some 9 in (225 mm) from it.

Section 2B.

In February, 1965, short intermittent cracks were noticed in the off-side wheel tracks near the edge of the panel. These cracks occur mid-way between lines of stiffeners. They have continued to develop, until in November, 1967 there were two fairly definite lines of cracks about 12 in. (300 mm) apart with some interconnecting cracks (Fig. 6). Again the cracks disappear within about 9 in (225 mm) of the position of the transverse stiffener.

Section 2C.

No cracking has so far occurred in this section. In the Spring of 1964, however, a blister about 9 in. to 12 in (225 to 300 mm) dia. appeared near the centre of the section. When it was cut out and repaired in September, 1964, it was observed that separation had occurred between the layer of filled rubber bitumen and the mastic asphalt wearing course (Fig. 7).

Section 2D.

No cracking or other defects have so far occurred in this section.

Panel 1.

All the sections with epoxy and polyester-resin-based surfacings have cracked to a considerable degree during the same $4\frac{1}{2}$ year period.

6. DISCUSSION OF RESULTS.

In terms of freedom from cracking and deformation, this trial confirms that mastic asphalt gives a better performance than rolled asphalt or resin-based systems. The cracking in Section 2B (rolled asphalt on an insulating layer of filled rubber bitumen) could be attributed to proximity to the edge of the panel and so is not to be taken as typical of such a surfacing on orthotropic decks in general. However, the mastic asphalt was subjected to similar conditions of stress and deformation and it has withstood them successfully hitherto.

The value of the insulating layer cannot be determined from a comparison of Sections 2C and 2D. Firstly the panel behaviour on the two sections is not identical, stresses in the deck plate tend to be lower on 2D. Secondly, both surfacings are still in good condition. Nevertheless, the filled rubber bitumen layer will usually be used because it provides additional waterproofing and better adhesion. It is known, however, from experience elsewhere that the risk of sliding of the surfacing is increased when insulating layers are too thick or too soft. It would now be the tendency to reduce the thickness of the insulating layer to .04 in(1 mm) to reduce this risk. No sliding was observed on the panel, but its slope was only that of the road camber (1 in 48) and there was no significant amount of stopping or accelerating traffic over it.

One alternative to the filled rubber bitumen, as a means of providing the necessary adhesion between asphalt and steel, is the use of an epoxy resin instead of a bituminous primer. The asphalt is laid over the epoxy before it has hardened. Such a system was laid in 1960 on Cross Keys Bridge, Lincolnshire[2]. as part of a trial to compare the performance of various types of mastic asphalt, $1\frac{1}{2}$ in. (380 mm) thick . It has given the best performance in terms of resistance to cracking on a very flexible deck. There was difficulty in laying the asphalt by hand on the wet epoxy-resin, because the latter tended to flow under the asphalt. This could now be overcome by the use of resins which harden sufficiently to permit working traffic, but are only fully cured when they receive the hot asphalt. Another conclusion drawn from the Cross Keys Bridge trial was that the addition of natural rubber did not lead to improvement in the performance of $1\frac{1}{2}$ in. (380 mm) of mastic asphalt.

The filled rubber bitumen under the rolled asphalt on Section 2B has probably contributed to the better condition of this Section compared with Section 2A. On both Sections the rolled asphalt has cracked most extensively between lines of stiffeners, which is contrary to the experience with mastic asphalt in earlier trials on deck panels reported by Trott and Wilson [1] where the cracking occurred over the stiffeners. Hennecke[3]. has advanced the explanation that the inter-stiffener cracking is caused by tensile stresses in the base of the asphalt at the interface with the steel and that it is preceded by flow of the asphalt from the region over the stiffener to the region in between, causing a longitudinal rut in the surfacing over the stiffener. This is not in accord with the behaviour of Section A, because in this case a rut of about $\frac{1}{4}$ in. (6 mm) deep developed between the webs of the trapezoidal stiffener under the near-side wheel track. The temperature differential effect referred to by Thul [4] would account for the position of the trough, but there seems no entirely satisfactory explanation of the crack development prior to the appearance of the rut.

One blister formed in the mastic asphalt on Section 2C, about 6 months after laying. Blister formation is sometimes an unfortunate feature of mastic asphalt surfacing and it seems to be associated with one or several of the following factors: the presence of water or of a volatile liquid, or of trapped air, relatively high temperatures and high temperature gradients. Blisters have not been widespread on steel decks in Britain hitherto, but where they hage occurred, they have been satisfactorily treated either by puncturing at the time of formation, resealing the puncture with sealing compound, or by cutting out and replacing the surfacing system.

Work is in progress to measure the strains over the steel decks caused by moving traffic. Some preliminary results show that the presence of $1\frac{1}{2}$ in (380 mm) of mastic asphalt leads to a reduction of at least 20% in the peak transverse stresses at about 20°C. The thermoplastic characteristics of the asphalt cause an increase in its structural effect as the temperature drops. This effect may be resolved into two parts: the one being a load spreading effect through the thickness of the asphalt, the other being the composite behaviour of the asphalt with the steel, with various degrees of interaction depending on the adhesion between deck and surfacing.

7. CONCLUSIONS

(1). Of the surfacing systems tested on the experimental deck panels laid in the highway, mastic asphalt has shown the greatest resistance to cracking and is generally in the best condition after almost five years of service under heavy traffic.

(2). The value of the insulating layer of filled-rubber bitumen under mastic asphalt cannot yet be fully assessed from this trial, but its adhesive and waterproofing qualities are generally beneficial. It probably contributed to a significant reduction in cracking in the rolled asphalt. To avoid problems with flow and sliding at high ambient temperatures, the thickness of the insulating layer should be kept to about 0.4 in(1 mm). An epoxy primer which cures in two stages may offer an alternative system in which the risk of flow is eliminated.

(3). The range of composition for asphalts to give satisfactory performance under various conditions of climate, traffic and deck design is very limited. It seems likely that brittleness and cracking at low temperatures and flow and deformation at high temperatures can be avoided in British conditions provided that the composition of the mastic asphalt complies with B.S. 1447, that the percentage and penetration of the asphaltic cement is near the centre of the permitted range and that bridge decks are not designed to be too flexible.

(4). A rolled asphalt could not be recommended for the type of deck used in this trial. But the performance of this material, laid on a filled rubber bitumen insulating layer, has been sufficiently encouraging to expect that a slightly thicker layer or improved adhesion may prove successful on a deck as flexible as the one used in the trials. There would obviously be a considerable advantage in being able to use a material such as rolled asphalt, which is normally laid by machine.

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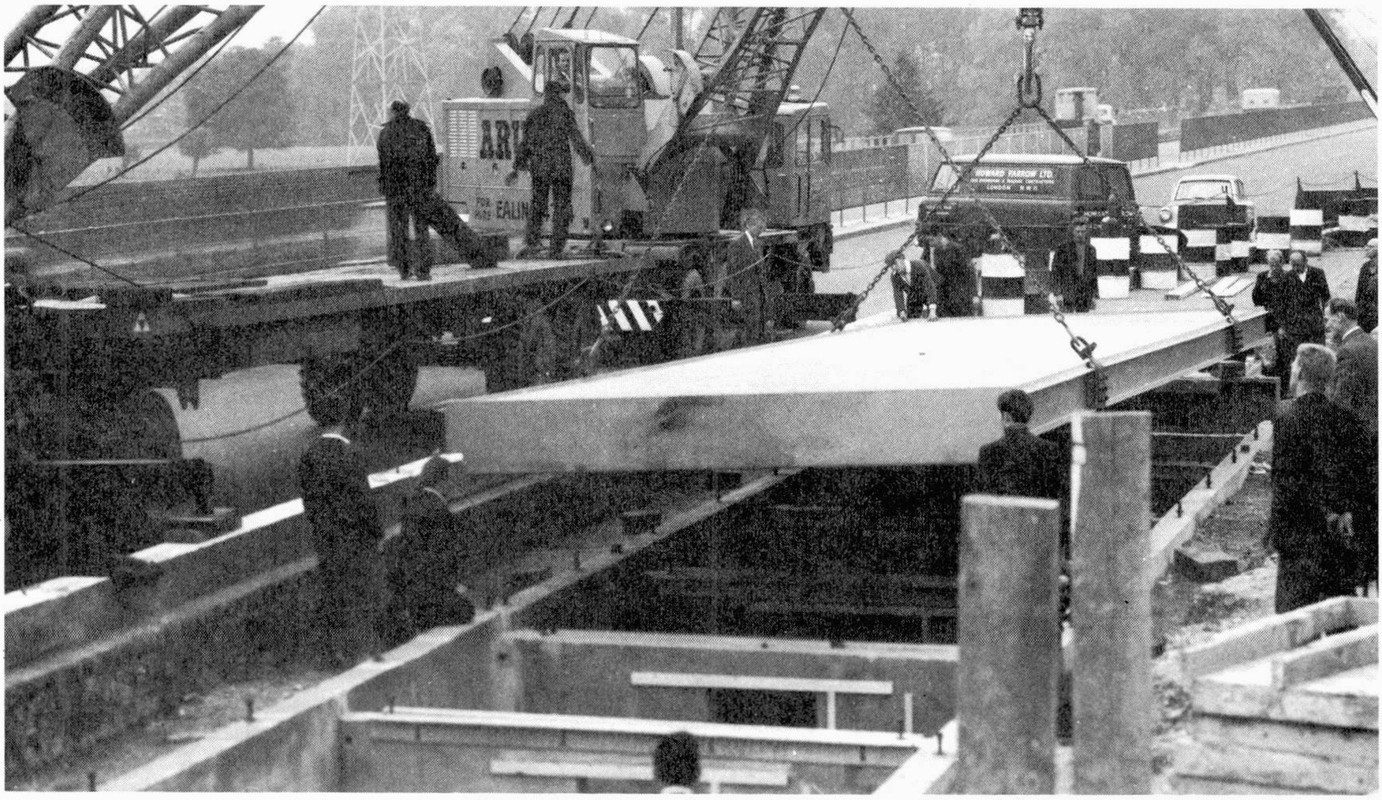


FIG. 1 TEST SITE UNDER CONSTRUCTION

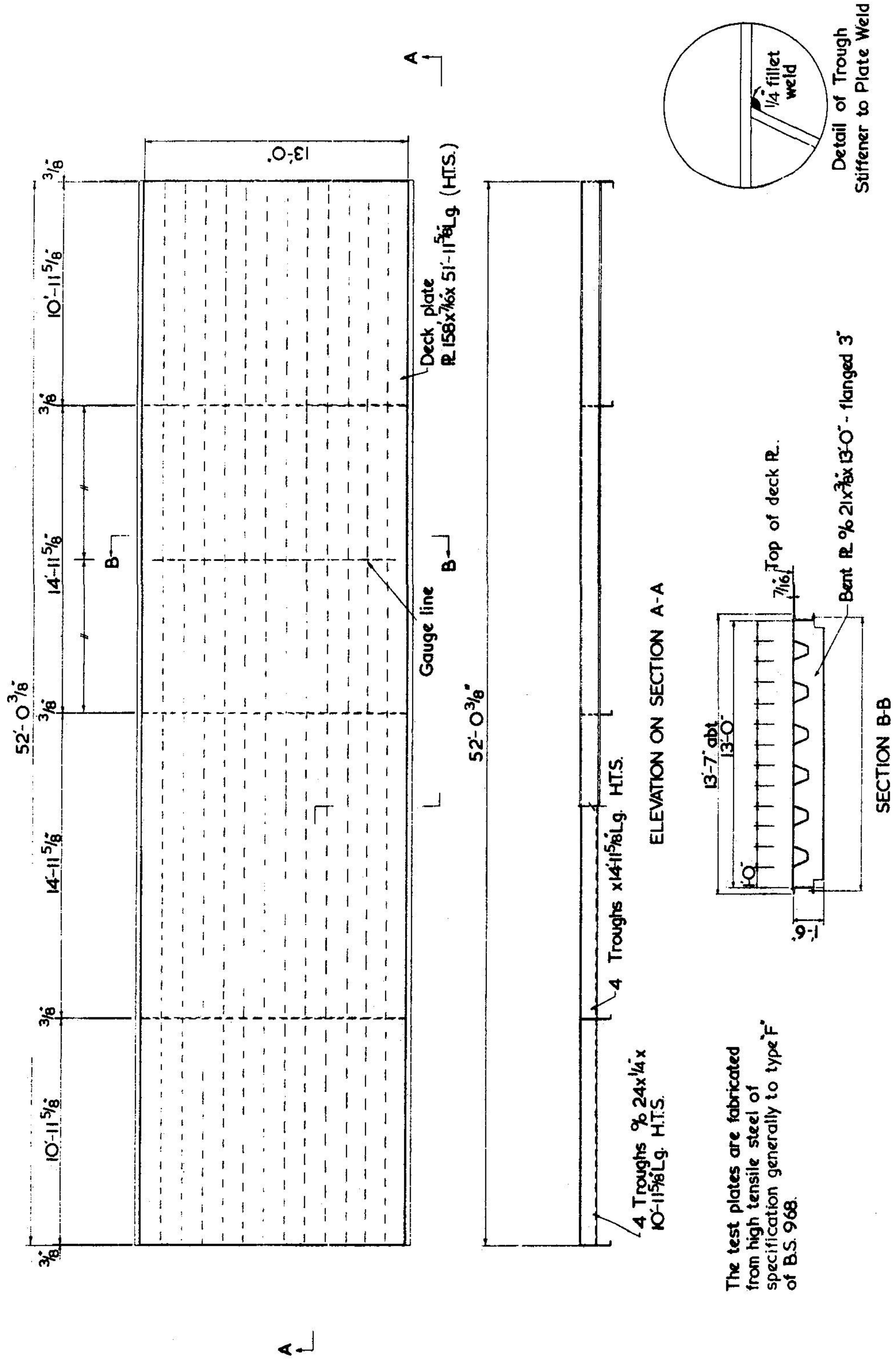
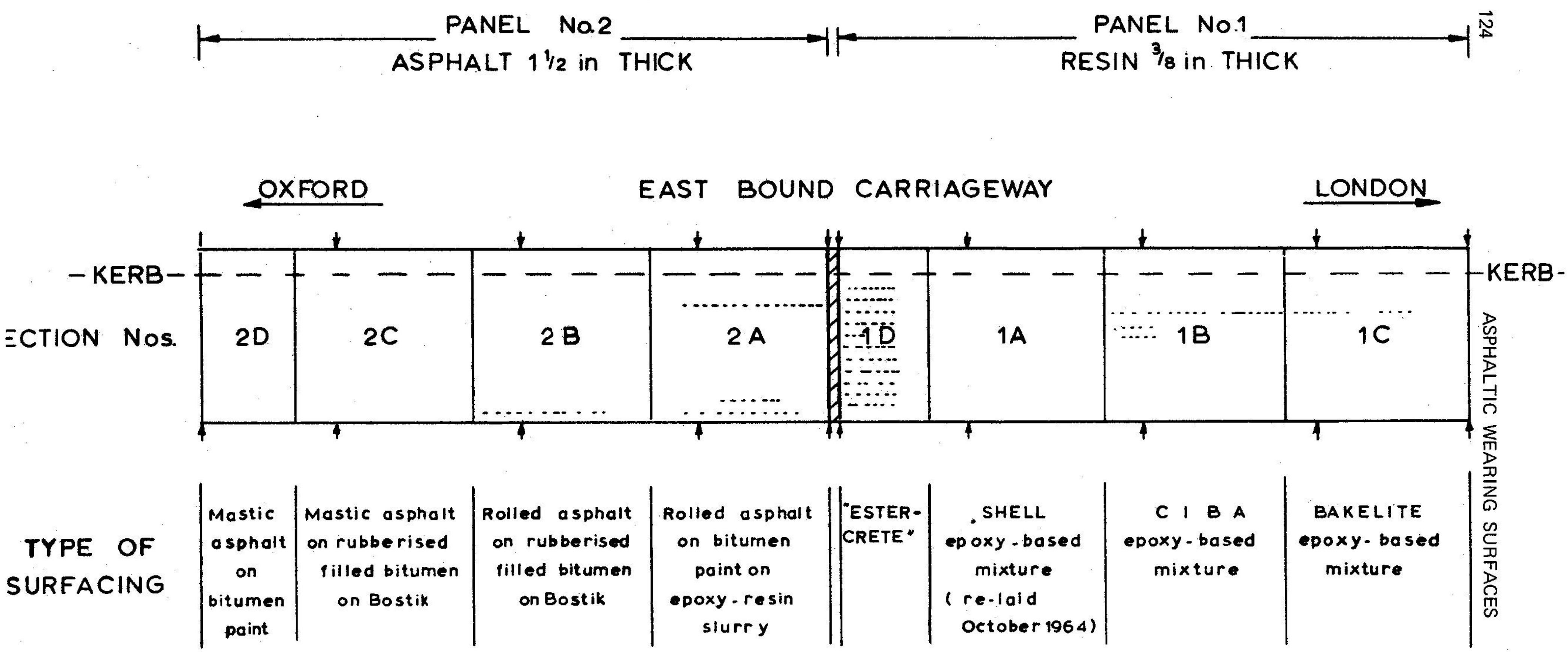


Fig 2 DETAILS OF DECK PANELS INSTALLED ON TEST SITE



NOTE: The small arrows show position of the transverse stiffener.
The cracks visible in the surfacings in April 1965 are shown thus.........

3 LAY-OUT DIAGRAM. SEVERN BRIDGE TEST PANELS ON TRUNK ROAD A40 (WESTERN AVENUE), DENHAM, BUCKS., NOVEMBER 1963.

FIG. 4 CRACKING ON ROLLED ASPHALT OVER GRITTED EPOXY RESIN, NOVEMBER, 1967

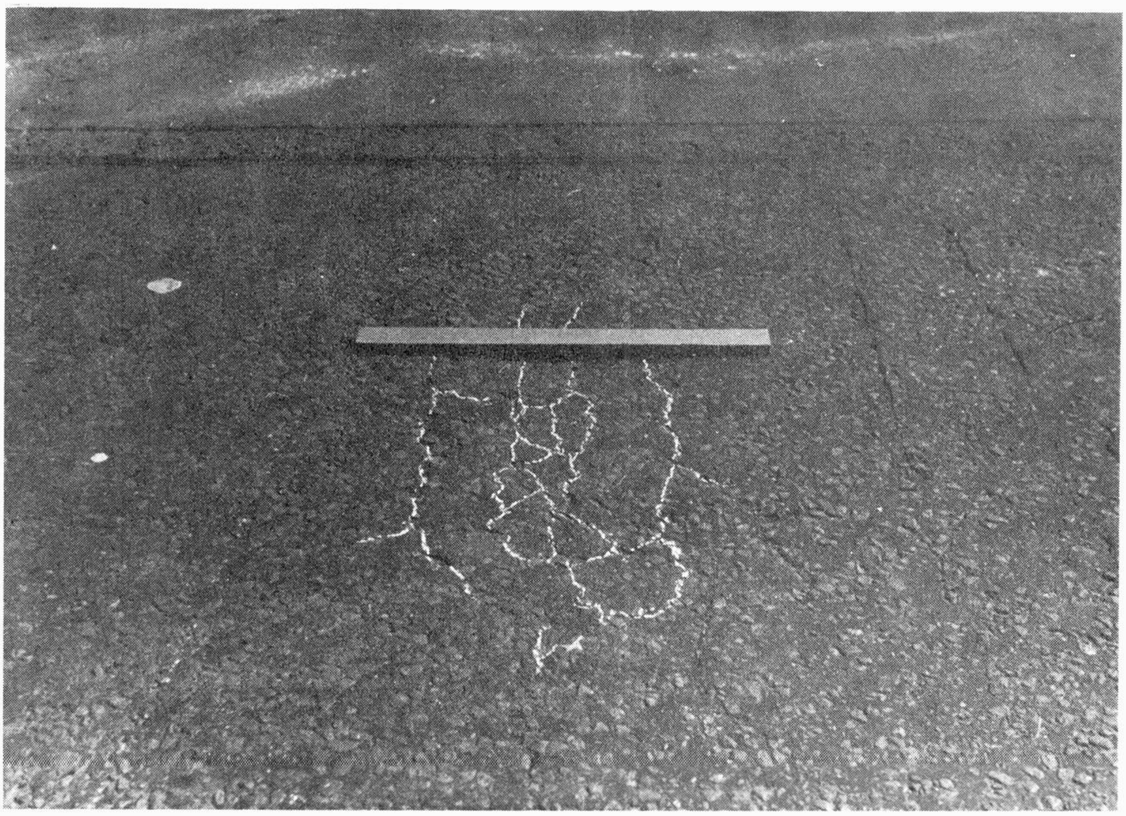


FIG. 5 DEFORMATION AND CRACKING IN ROLLED ASPHALT OVER GRITTED EPOXY RESIN, (SECTION 2A) JULY, 1968

FIG. 6 CRACKING IN ROLLED ASPHALT ON FILLED RUBBER BITUMEN, NOVEMBER, 1967

FIG. 7 MASTIC ASPHALT ON FILLED RUBBER BITUMEN, NOVEMBER, 1967

SUMMARY

Trials with asphaltic and resin-based surfacing materials on bridge deck panels laid into the highway are described. After almost five years of heavy traffic, a mastic asphalt, 1 1/2 in. (38 mm) thick, has given the best performance in terms of freedom from cracking and deformation. The value of an insulating layer of filled rubber bitumen, 0.1 in. (2.5 mm) thick, under the asphalt has not yet been proven, but a similar layer under a rolled asphalt wearing surface, 1 1/2 in. (38 mm) thick, has reduced the amount of cracking and deformation of this asphalt compared with a gritted epoxy underlayer.

RESUME

Des essais de différents matériaux de revêtement à base d'asphaltes et de résines pour tabliers de ponts routiers ont été faits. Après presque cinq années de trafic lourd, on a obtenu les meilleurs résultats concernant le fissurage et les déformations, avec un asphalte coulé de 38 mm d'épaisseur. L'amélioration due à une couche d'isolation de bitume caoutchouté renforcée par du gravier, d'une épaisseur de 2,5 mm, n'a pas encore été démontrée avec l'asphalte, mais une couche semblable sous une couche d'asphalte cylindré de 38 mm a réduit le fissurage et les déformations par rapport à une base d'époxy avec gravier.

ZUSAMMENFASSUNG

Es werden Versuche mit Asphaltbelägen und solchen auf Harzgrundlage, welche auf einer Prüfdecke in eine Straßenbrücke eingelegt worden sind, beschrieben. Nach beinahe fünf Jahren schweren Verkehrs, erwies eine 38 mm dicke Gußasphaltdecke das beste Verhalten in bezug auf die Freiheitsgrade durch Risse und Verformungen. Der Wert einer Isolationsschicht aus splittverfestigtem Gummibitumen von 2,5 mm Dicke unter dem Asphalt konnte bis jetzt noch nicht geprüft werden, hingegen hat eine ähnliche Lage unter einem Walzasphaltbelag von 38 mm Dicke das Auftreten der Risse und Verformungen dieses Asphalts verringert im Vergleich mit einer aufgerauhten Epoxydunterlage.

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EXPERIENCES WITH HOT-LIQUID ADHESIVE "OKTA-HAFTMASSE"

Expériences faites avec l'adhésif "OKTA", liquide à chaud

Erfahrungen mit der heißflüssigen "OKTA-Haftmasse"

Dr.-Ing. R. DETERS
Germany

Further to the detailed report of Mr. Thul I would like report shortly from the view of practice.
It has already been reported by Mr. Thul, that since 17 years good experiences have been made with a hot liquid agent for corrosion protection and binder. This material is applied by flame-spraying and is well known under the market-name OKTA-HAFTMASSE. On the OKTA-HAFTMASSE we prefer to apply a special pavement consisting of one or two mastic layers which is stabilized with crushed stone. Instead of this special pavement there can also be applied one mastic layer of 8 mm, a Gußasphalt-protective layer of 25 mm and a surface course of Asphalt concrete or Gußasphalt.

In the construction of bituminous surfacing the cleaning of the steel plate (for ex. by sandblasting) and application of the corrosion protective primer take a substantial time and effort. Under unfavourable weather conditions it is sometimes necessary to work on short sections at a time and to provide an additional protection by tent or anything else. There it seems very important to simplify this construction procedure. Therefore such binders are to be prefered, that became insensitive to weather immediately after application. This is the case with hot liquid binder, which cool down and became abrasion resistant within a few minutes after application.

In same cases in Germany they used cold liquid materials. The application of such makes necessary the protection against weather until evaporation of solvent is almost complete. The same is true for resinous materials, which need a longer time - up to 24 hours in some cases - till full curing.

The mentioned flame spraying procedure offers from the practical point of view the following advantages:

1.) Any moisture is evaporated immediately before material gets in contact with the steel.

2.) Material is heated by the flame at the nossel during the whole time of spraying.

3.) After application this heat leads to a uniform thickness of binder.

First time on the American continent in summer 1968 works of this type of pavement were carried out on the Saint John Harbour Bridge, New Brunswick. Under quite unfavourable climatic conditions prevailing there, this type of pavement construction procedure has not presented any problems. According to long experiences in Germany the Saint John Harbour Bridge Authority decided to have a stone stabilized mastic layer of 3o mm with zig-zag-bars on the deck plate. There was applied a layer of Asphalt Concrete as a surface course. A bituminous construction of such a stone stabilized mastic layer is under service on the Rhine-Bridge , Düsseldorf-Neuss since 1951 without any failure. In 1966 there was laid a thin additional asphalt concrete of 15 mm thickness - especially for better roadibility.

In addition to this I should mention that most recent constructions of a total thickness of 1o to 25 mm have been used, existing of OKTA-HAFTMASSE and stone stabilized mastic.

These light constructions have been under traffic without problems on temporary mobile bridges, on which a low weight of pavement is very important. In these cases the stone stabilized mastic is both: protecting <u>and</u> surface layer.

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FREE DISCUSSION

Discussion libre

Freie Diskussion

QUESTION by Roman Wolchuk (USA):
What is VABIT (what does it consist of, is it a patented composition) ?

ANSWER:

a) "Vabit" is the trade-name for an asphalt concrete which is made up of a bituminous filler instead of a raw filler.

b) We use the term "Vabit filler" meaning a bituminously prepared stone dust, grade 0 to 0,09 mm, produced according to a procedure which has been patented in several countries. Bituminously prepared means that every single particle of the raw filler (stone dust, grade 0 to 0,09 mm) is thinly coated with bitumen in a special procedure.

c) We speak of "Vabit insulation" when referring to a sand asphalt concrete that consists of above-mentioned "Vabit filler" and crushed stone sand (screenings), grade 0 - 3 mm.

QUESTION by Herbert M. Mandel (USA):
Mr. Bürger mentioned that in one of his tests the number of cycles to which the sample was subjected was far in excess of the anticipated repetitive loading to be anticipated in 10 years of service on an actual bridge. Is he indicating that 10 years is the proposed objective before replacement of the wearing surface becomes necessary ? What do other speakers envision as an idealized useful life of these wearing surfaces ?

ANSWER:
Mr. Bürger's statement only referred to the extent of the bending-tensile and bending-compressive stresses to which the wearing surface was exposed at -30° C and was meant to give an idea of the q u i c k - m o t i o n - e f f e c t achieved during the tests. This statement has therefore no direct relation to the expected duration of life. We expect a longer useful life of this wearing surface.
Other speakers held the opinion that, compared with the experiences made with several other surfaces up to now, a useful life of 10 years without repairs would be a favourable result. 15 years would certainly be regarded as very advantageous.

QUESTION by Dr. Peter Klement (Austria):
Are there any experiences with coldly applied asphaltic surfaces on steel decks ?

ANSWER:
Cold asphaltic bitumen emulsions were occasionally used in the past for road pavements, not, however, for surfacings on bridges. Application of cold asphaltic bitumen emulsions requires the use of solvents. In order that the solvent could volatilize it was necessary to keep the bituminous surfacing as long in a porous state until the solvent had entirely escaped. This method required traffic to compact the surfacing. As a result, shovings occurred very frequently.
This additional cause of shoving should be avoided. We therefore hold the view that the best suitable method of constructing surfacings on bridges is hot laying and, if necessary, subsequent compaction by rolling.

QUESTION by Prof.Dr. K. Széchy (Hungary):
Could the use of zig-zag stiffening ribs welded on the steel plate be recommended in general as to promote better bond between steel plate and surfacing and to reduce deflection movements of the deck plate or ought the use to be restricted to higher grades, only because it could give rise to a greater degree of surface waving ?

ANSWER:
Zig-zag bars cannot be recommended generally, because experience made so far with zig-zag bars varied from case to case. When constructing the Rhine Bridge of Rees we equipped one section of the steel plate with zig-zag bars with a view to furnishing proof - on the basis of a large-scale test - that it is perfectly possible to do without them.
We are of the opinion that there is a variety of high-quality surfacings. Their quality depends primarily on their composition and structure, on the method of application and on the care with which the surfacing is both produced and laid.
The use of zig-zag bars involves the following disadvantages: Sandblasting becomes difficult and thus more costly; repairs are rendered more difficult; negative influence on the competitiveness of steel bridges, specially those with longer spans (heavier weight).

CONCLUSIONS

Conclusions

Schlußfolgerungen

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Due to its economy, the orthotropic plate is being used to a steadily increasing extent, as shown by the numerous contributions received from the various countries and by the survey given by Mr. Thul and me as well. The problems involved by the pavement have been correctly recognized by everybody and there are efforts to solve the problems unsolved as yet. The approaches made and still being made do not differ much. Results of examinations, investigations, laboratory and full-scale tests therefore are quite similar, more or less.

The most important supposition for the durability of a perfect pavement is a bonding material or bond coating providing for a shear resistant connection to the steel deck plate as well as to the wearing surface. This opinion of ours has been shared by the gentlemen contributing to the discussion. If it is possible to make a bonding layer of such composition that it will protect the steel plate, against corrosion and against high temperatures occuring during application of poured asphalt, then a special protecting or insulating course, for instance a Mastix layer, may be omitted. In case, however, such a layer is necessary, it should not have a thickness of more than 5 mm = 3/16" nor a bituminous content of more than 15% by weight. In fact, many sorts of damage have been caused by a Mastix course either too thick or too soft. In general, it should be pointed out that most of the damage to wearing surfaces has been caused by inappropriate composition and by faultly application of the pavement.

Metal foils do have a good sealing effect no doubt. It is our opinion, however, that special care must be taken during application. Furthermore, we think it is necessary to have a bonding compound applied underneath as well as atop of the foil in order to obtain a perfect bond.

Because experience gathered with both mastic (or poured) asphalt and asphalt concrete has been quite the same, it cannot yet be finally decided which one will perform better, eventually. But it is very important to have the wearing surface as dense as possible. Asphalt concrete therefore has to be compacted by means of adequately heavy tyre rollers, as done with the VABIT or the stone filled Mastix surfacings.

Surely, part of the damage has been caused by the deflections of the plates, in particular cracking. Contrary to the opinion of our British colleagues and in accordance to that of our Japanese ones we do believe that the deflections represent a cause of secondary nature only and that it will be possible, indeed, to eliminate those sorts of damage by an appropriate composition and by a respective specification of the wearing surface. This opinion might be confirmed by the performance of many pavements which have done excellent service for several years.

In summary, I may point out that, right now, we have some pavements at hand which are nearly in close accordance with the requirements of traffic. It can be said that these wearing surfaces will have approximately the same life as road pavements. We should not neglect, however, that quite a number of important problems are still to be clarified. We are convinced, however, that all of these problems will be solved already within a short time, as soon as the many tests and experiments, undertaken extensively in different countries, have come to an end. We shall be in a position, then, to place in the engineer's hand together with the respective recommendations a tool which will enable him to provide for the adequate wearing surfaces called for by modern bridge design.

The exchange of ideas, which has been started among so many experts present here and today, should be continued in the future. Those who will gain from this exchange are all of us.