# 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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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.

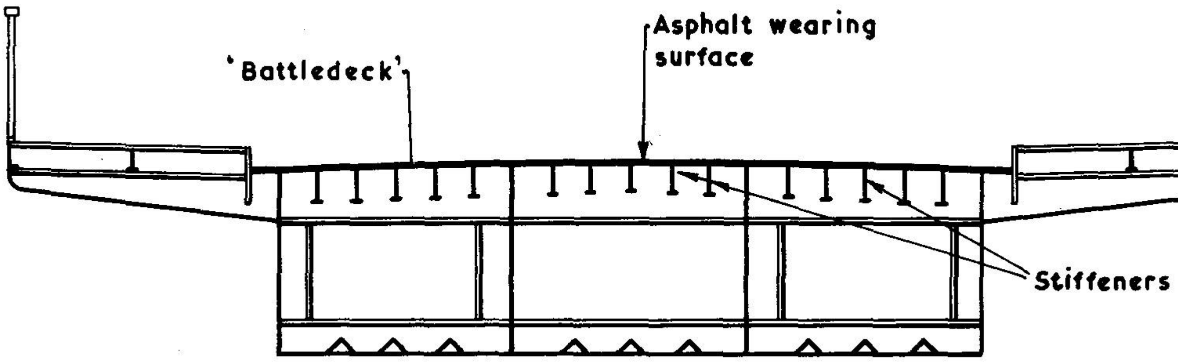


FIG. I. CROSS SECTION OF SWING BRIDGE AT REGENTS CANAL DOCK. LONDON.

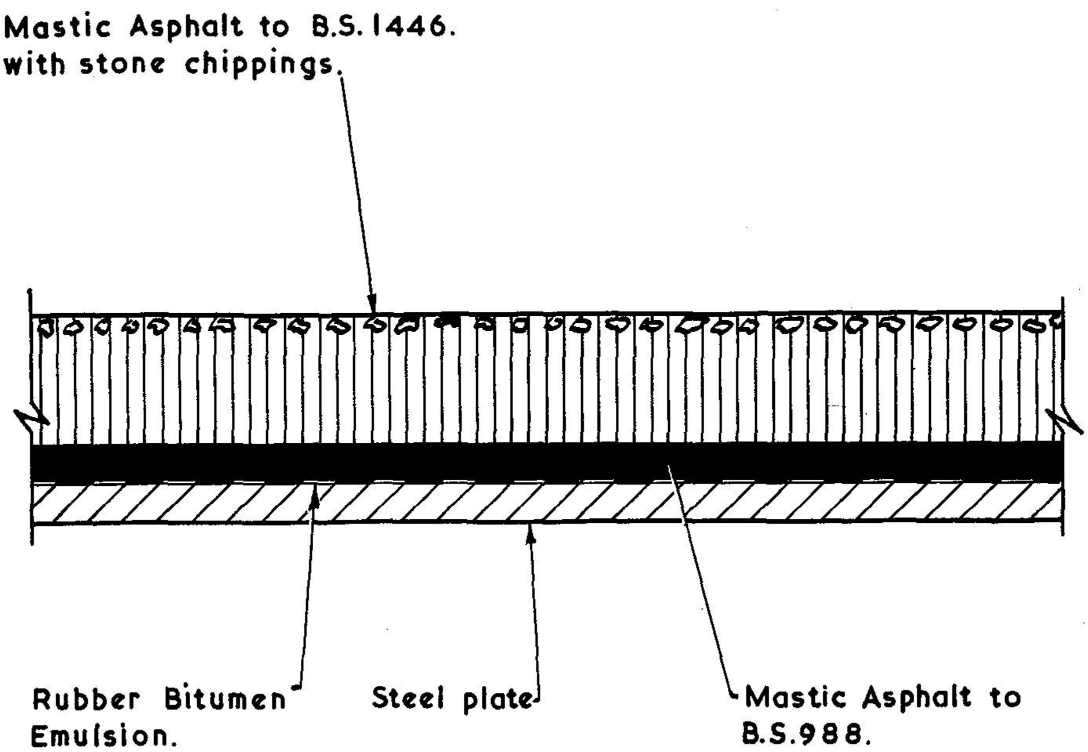


FIG. 2. SURFACING OF SWING BRIDGE AT REGENTS CANAL DOCK, LONDON.

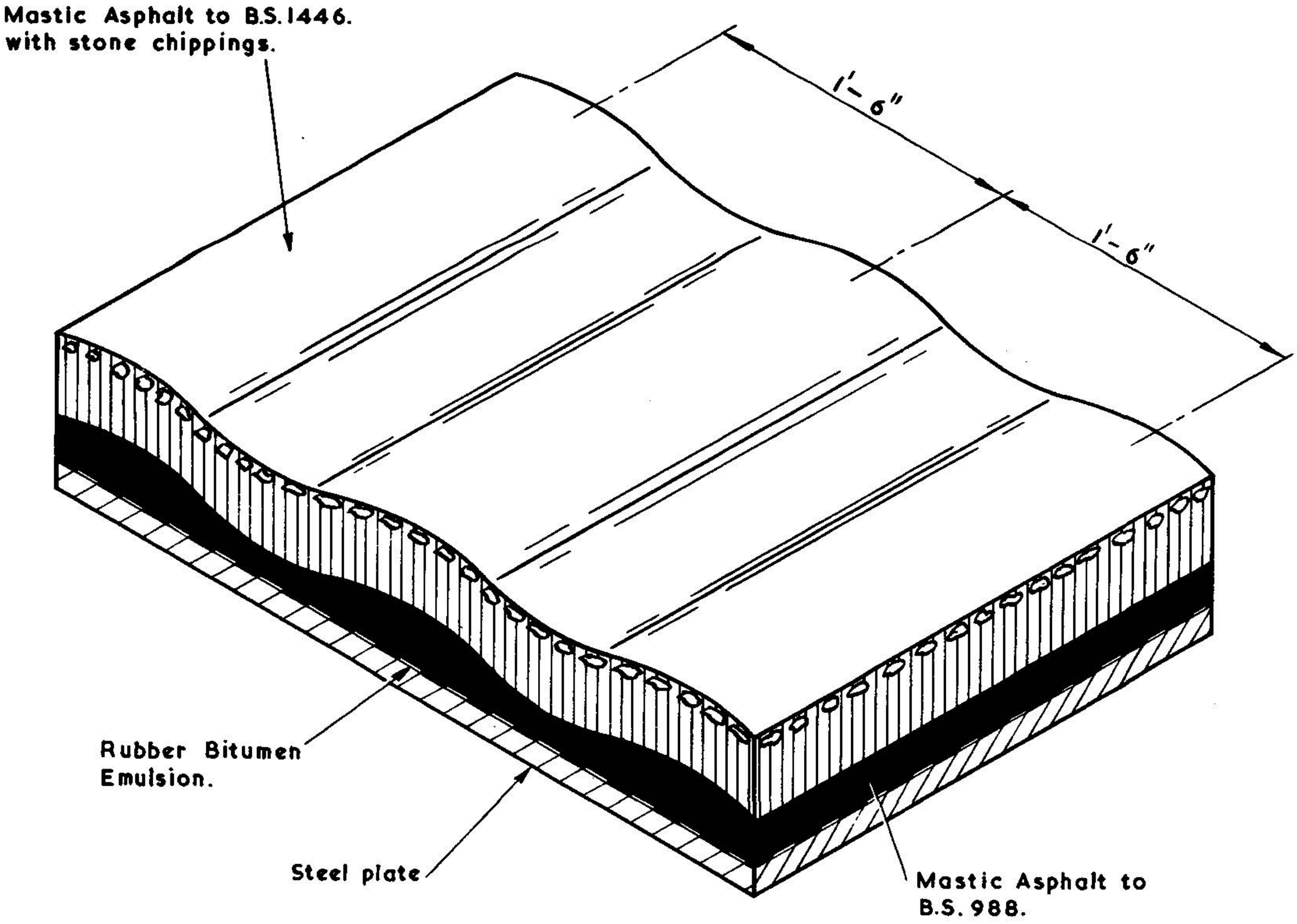


FIG. 3. DEFECT–TRANSVERSE WAVES IN ROAD SURFACE.

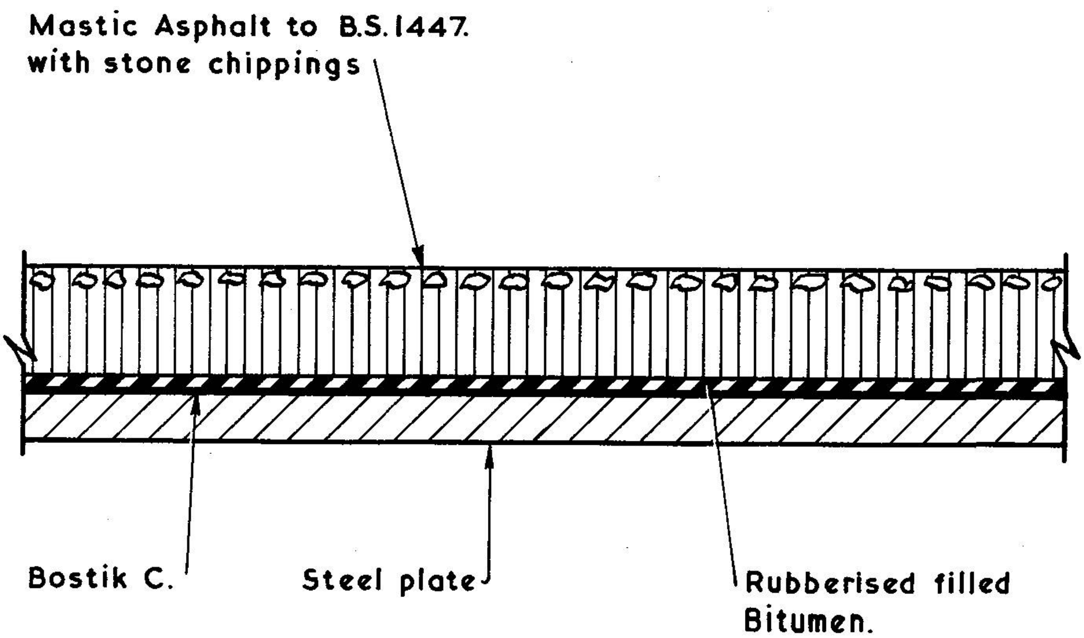


FIG. 4. NEW SPECIFICATION FOR SURFACING AT WOOLWICH AND MIDDLESBROUGH DOCK SWING BRIDGE.

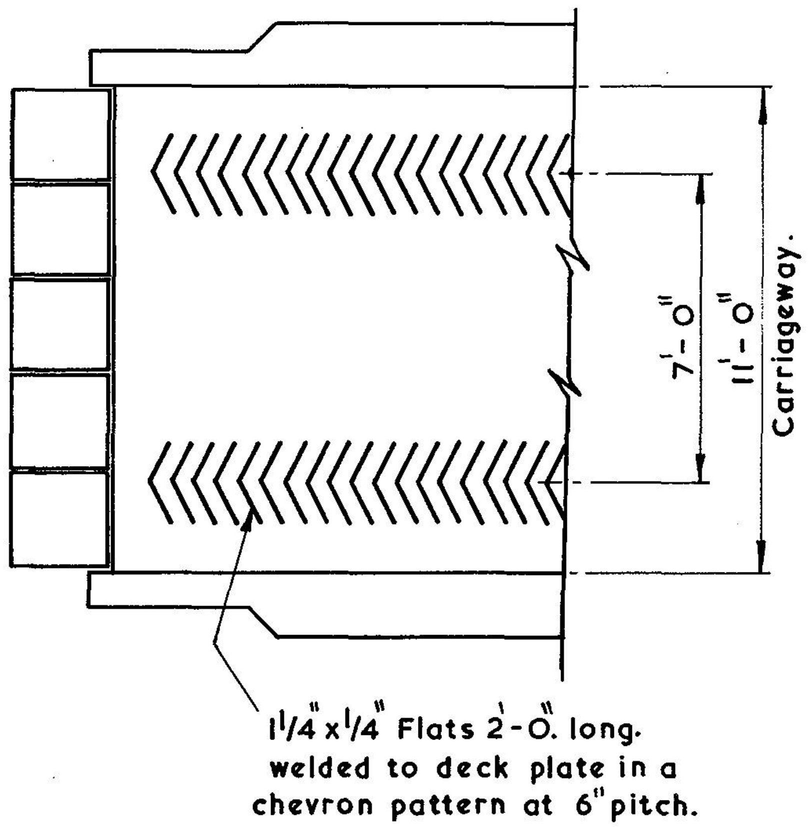


FIG. 5. REINFORCEMENT AT END OF RAMPS.

## 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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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<br>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<br>30 Gussasphalt | | Tar Rubber Emulsion | 36.0 | 3.9 | 100 | 2 |
| | | | | | | | 92 | 35 As.Con. | 27 Gussasphalt<br>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.<br>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.

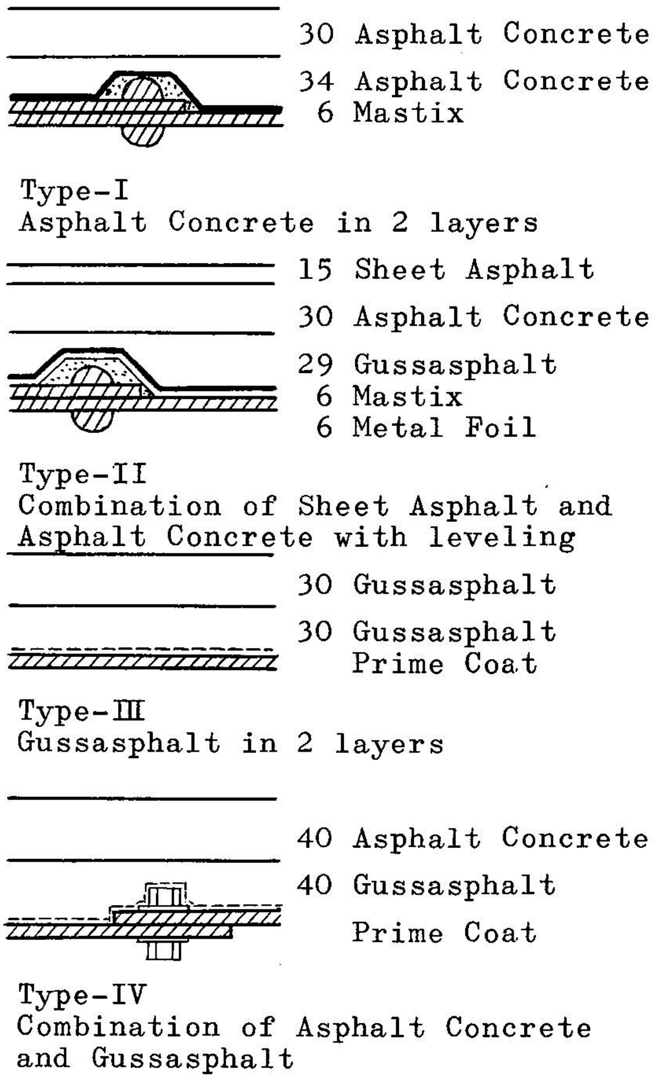


Fig.-1 Typical Cross Section of Pavement (mm)

| 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.)

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

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

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

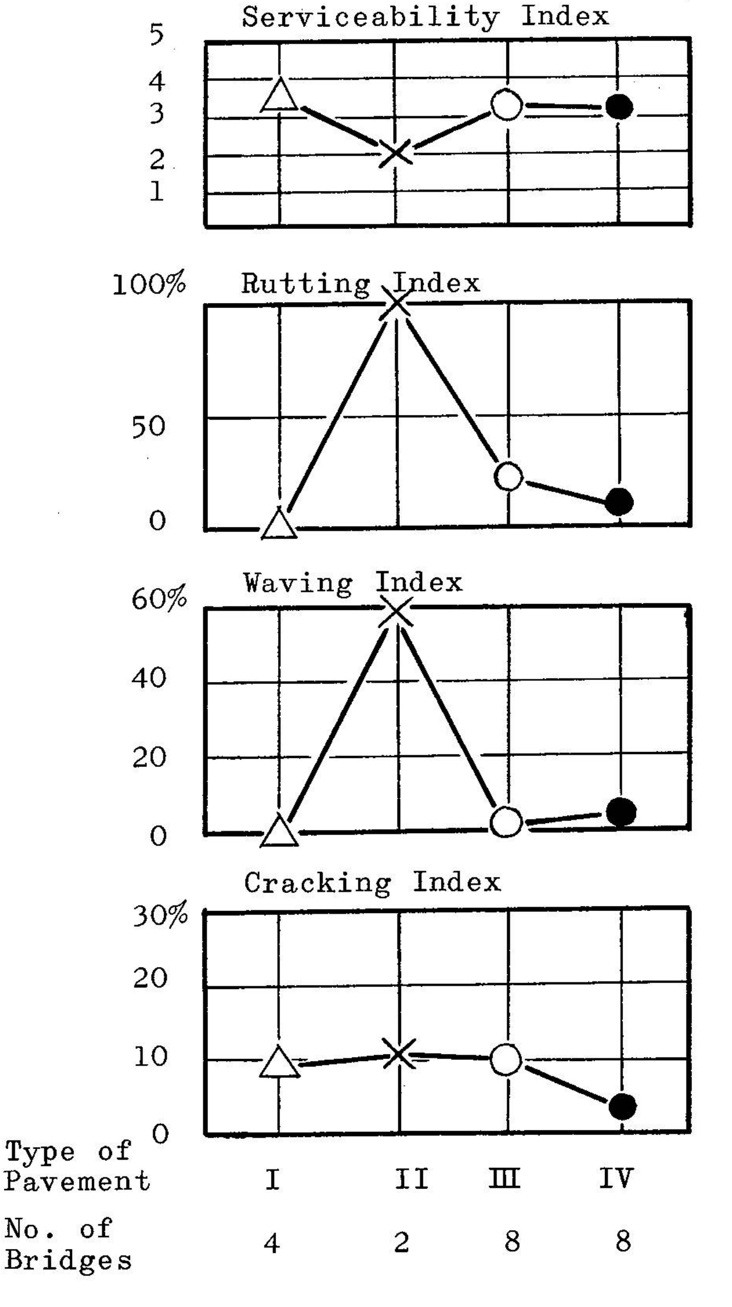


Fig.-2 Performance by Type of Pavement

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.

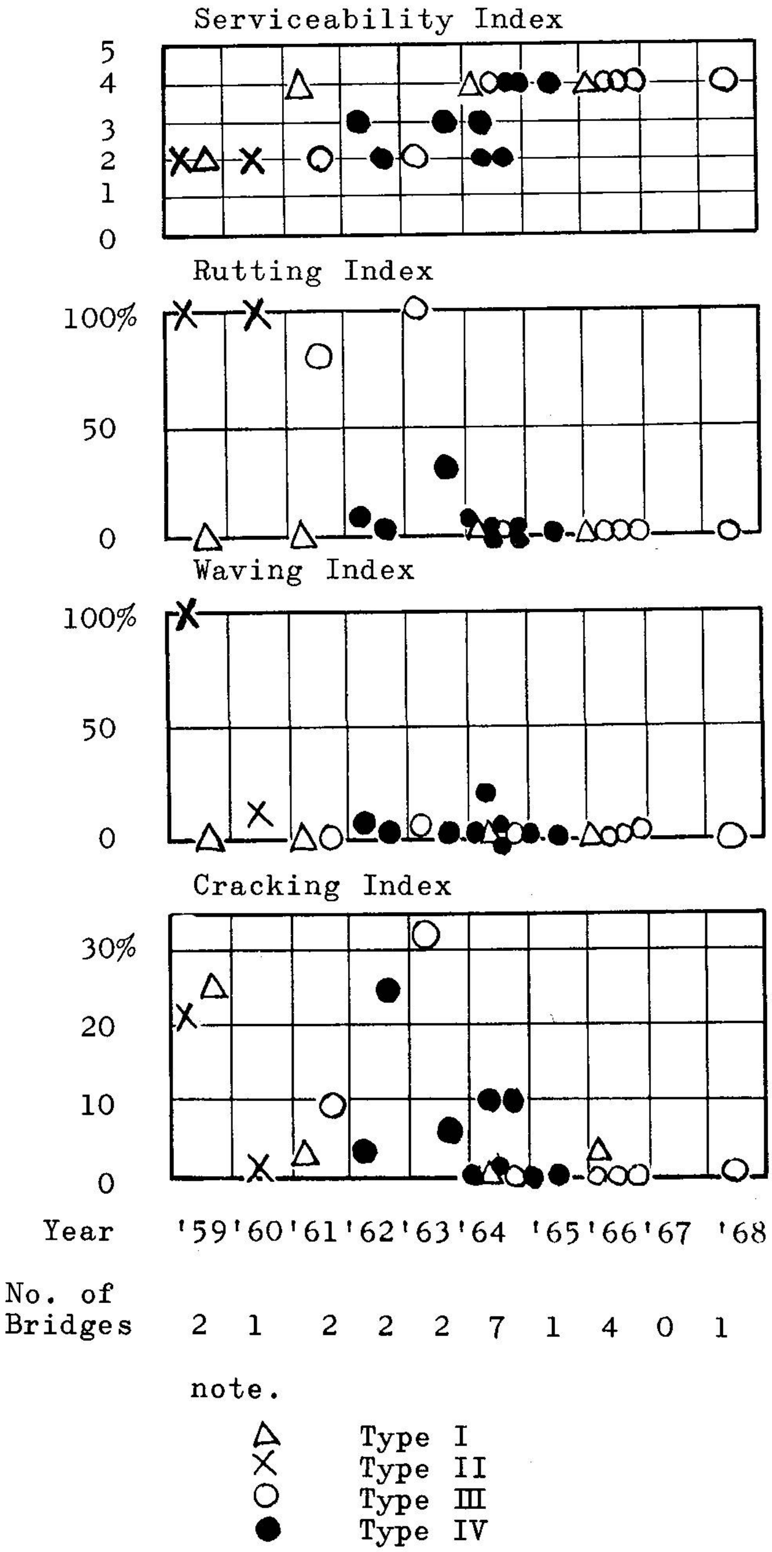


Fig.-3 Performance by Year of Construction and Type of Pavement

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:

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

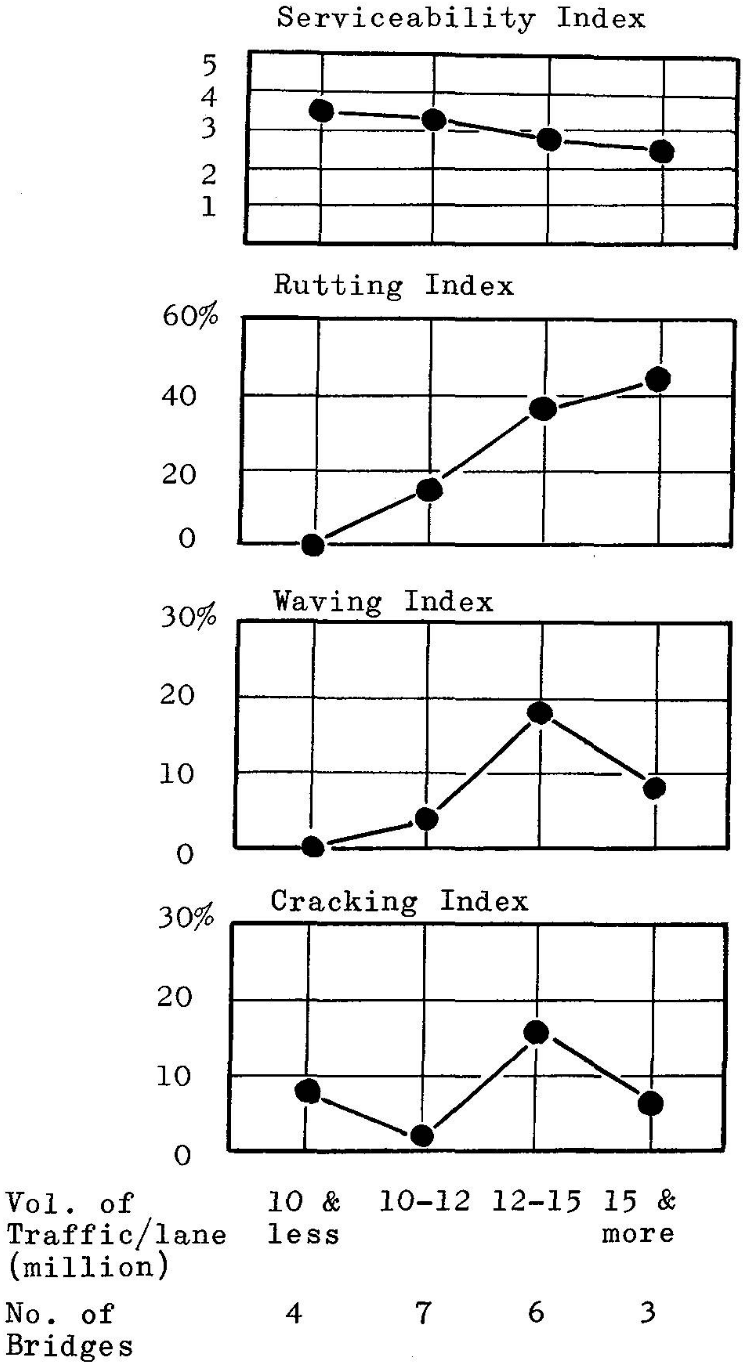


Fig.-4 Performance and Total Traffic/Lane and rib intervals.

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 3/4" | 1/2" | 3/8" | 1/4" | #4 | #6 | #10 | #16 | #20 | #30 | Leveling Course |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 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

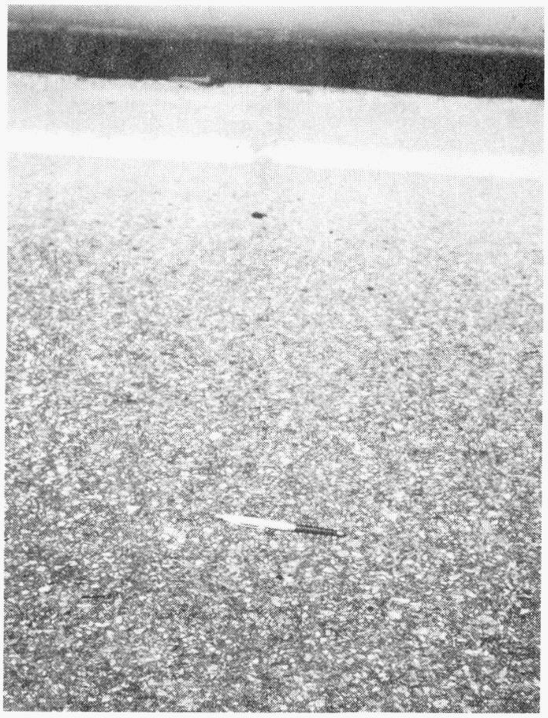

FIGURE 2. EVIDENCE OF A TRANSVERSE CRACK FORMING OVER THE SPLICE PLATES ON THE PORT MANN BRIDGE AFTER FOUR YEARS OF SERVICE

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 $1 \times 10^6$ 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 $5 \times 10^6$ 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

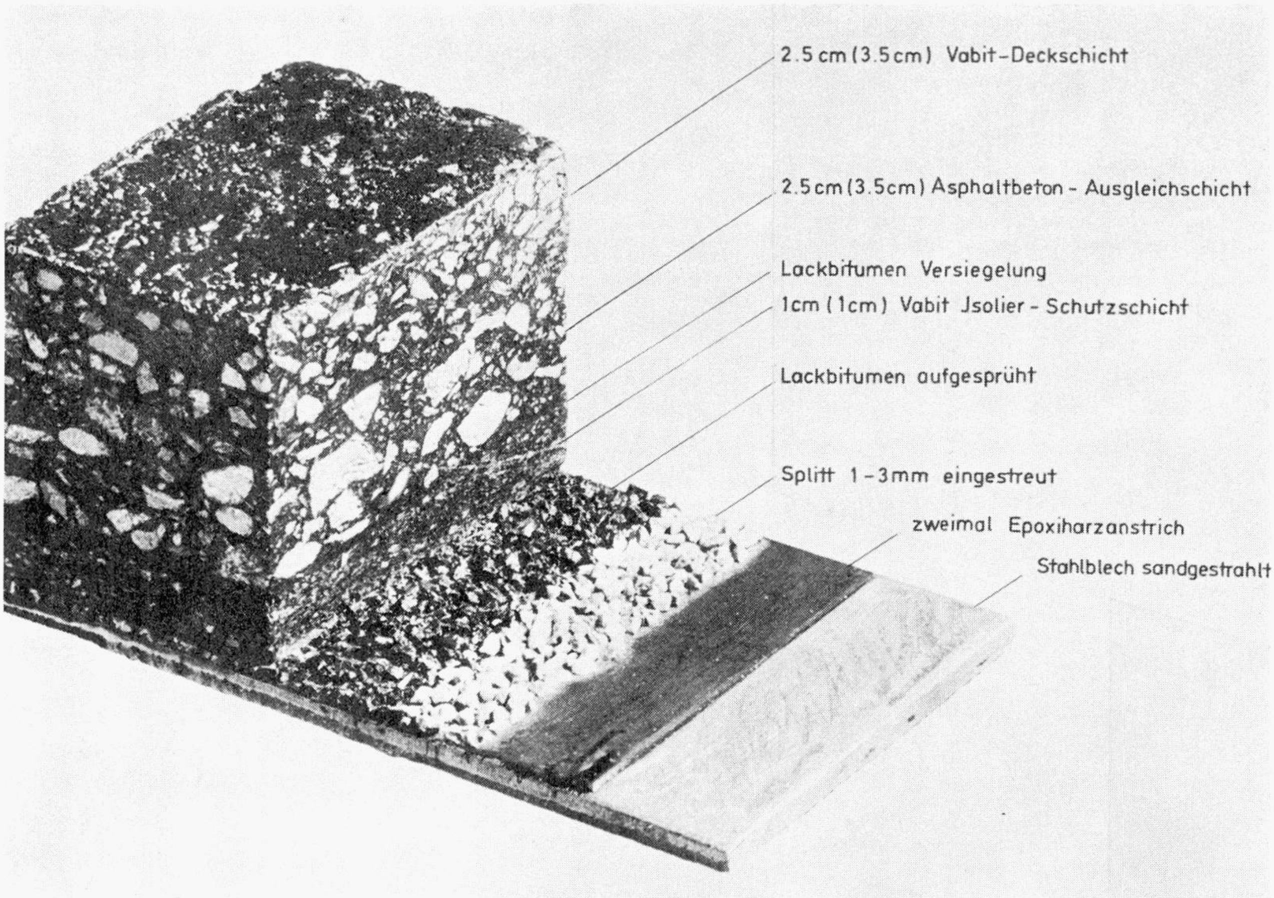



Abb. 1

1.) Sicherer Korrosionsschutz

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).

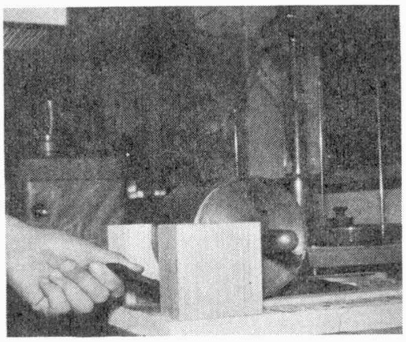


Abb. 2

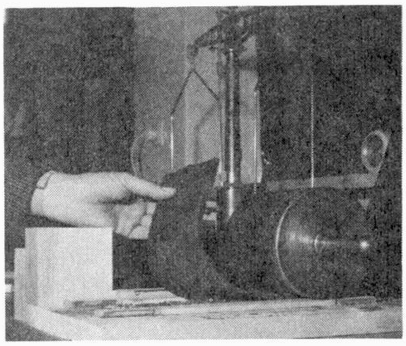


Abb. 3

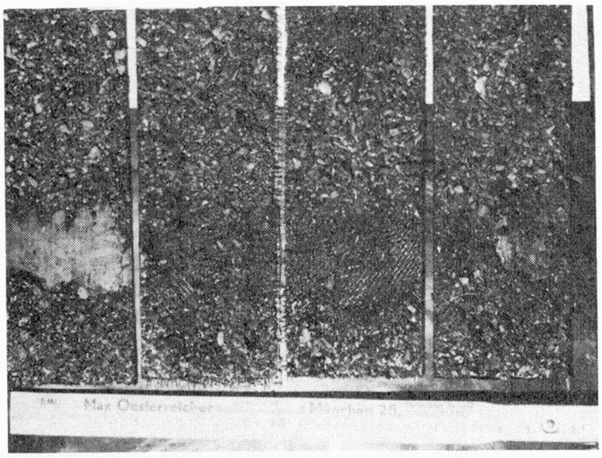


Abb. 4

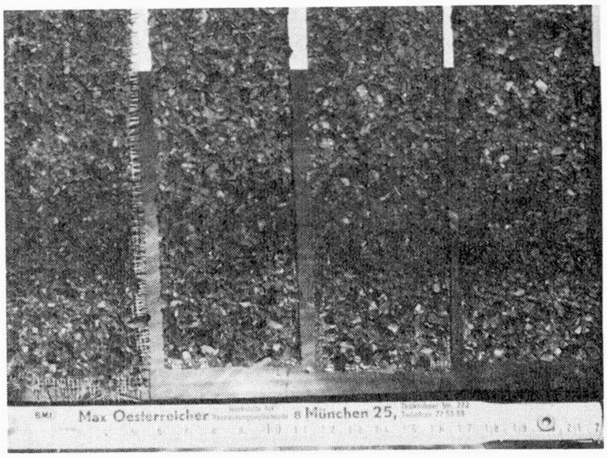



Abb. 5

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.

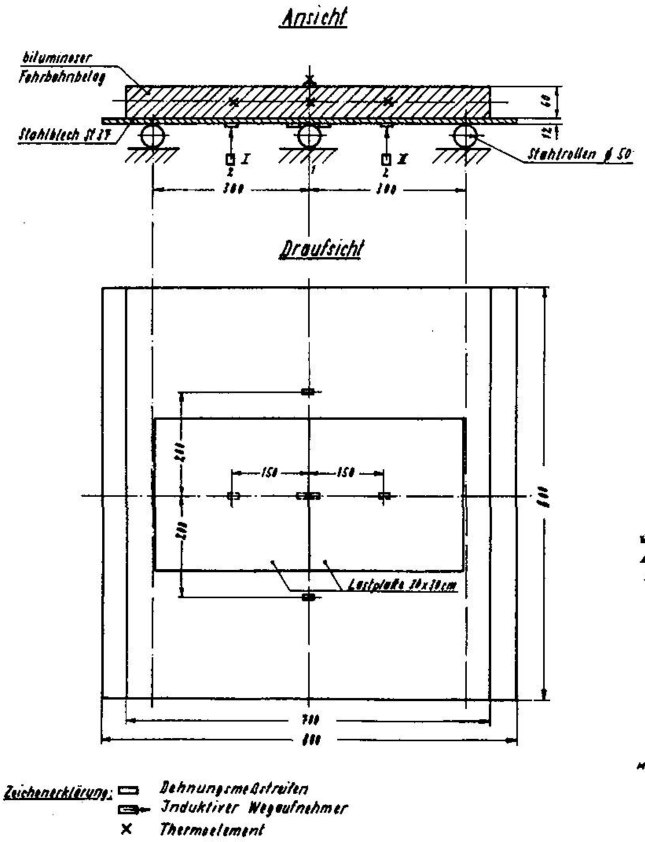


Abb. 6

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

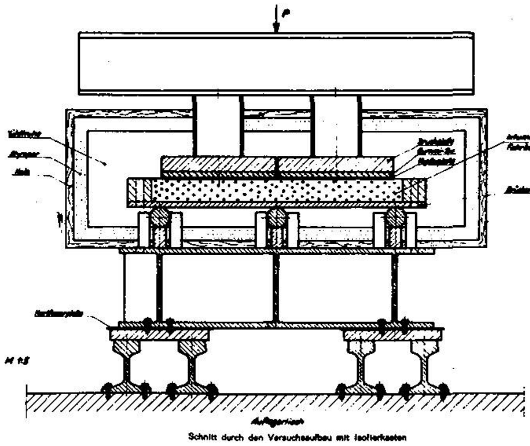


Abb. 7

## 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.

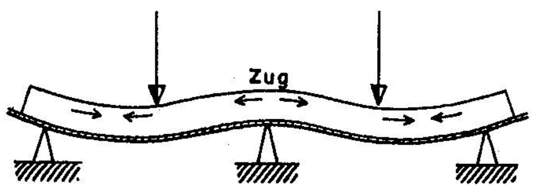


Abb. 8 : Zug- und Druckspannungen im Asphaltbelag

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. S c h u l z e , 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.

E i n s c h r ä n k u n g e n :

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.

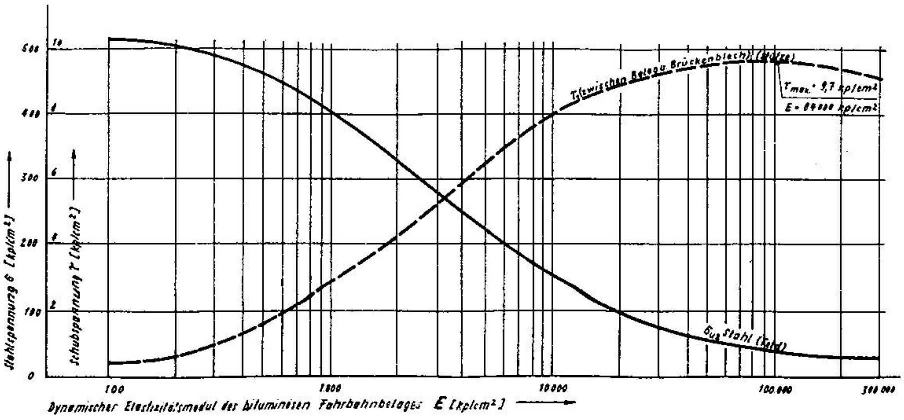
Für die Berechnung der Stahlspannung auf der Blechunterseite in Feldmitte und Schubspannung in der Verbundfuge über dem mittleren Auflager sowie der Spannungen an der Oberseite des bituminösen Belages in Feldmitte und über der Stütze wurden eine lineare Spannungsverteilung, ein Ebenbleiben der Querschnitte, 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.

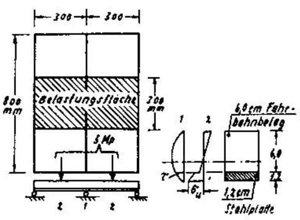
Zur Ermittlung der Momente für die Berechnung der Biegespannungen dienten die Einflußfelder elastischer Platten von A. P u c h e r (Wien, Springer-Verlag, 1958); die Querkraft über der Mittelstütze für die Berechnung der Schubspannung wurde nach den Tafeln von E. B i t t n e r (Wien, Springer-Verlag, 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.

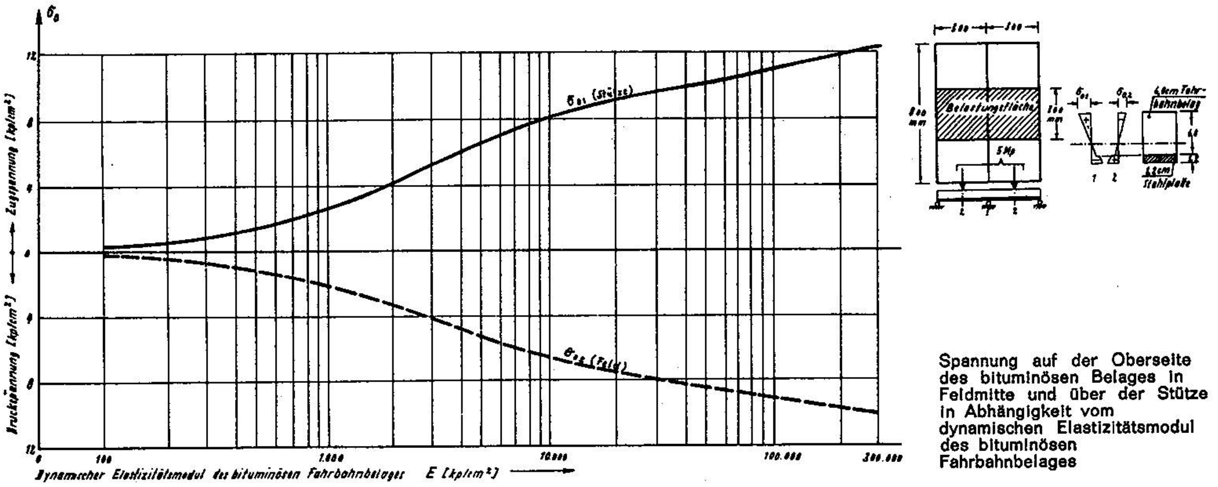
Aus den Abb. 4 und 5 ist gut ersichtlich, wie bei wachsendem Elastizitätsmodul des Belages die Stahlspannung abnimmt, während die Schubspannung entsprechend anwächst. Die größte Schubspannung von 9,7 kp/cm$^2$ tritt bei einem Elastizitätsmodul von 84 000 kp/cm$^2$ auf.

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

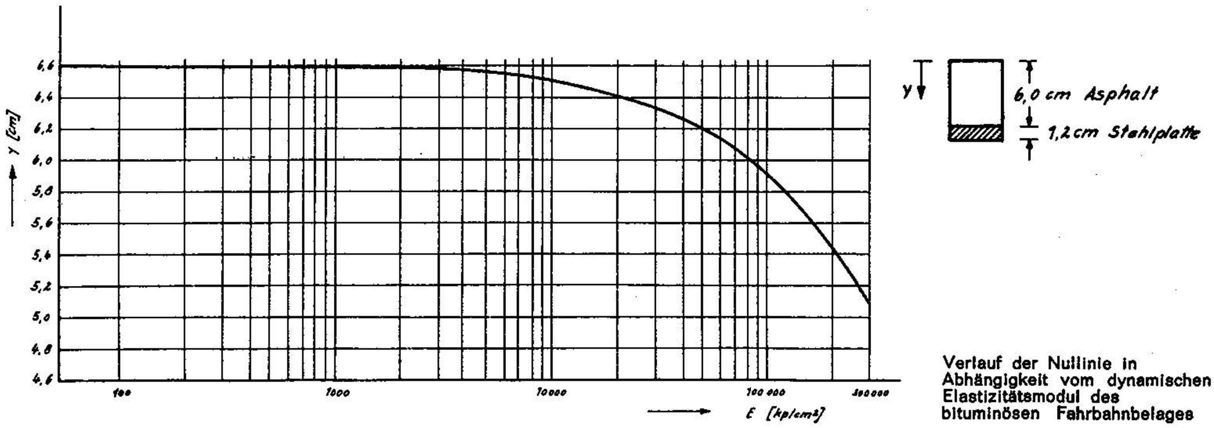



Stahlspannung auf der Fahrbahnplattenunterseite in Feldmitte und Schubspannung in der Verbundfuge über dem mittleren Auflager in Abhängigkeit vom dynamischen Elastizitätsmodul des bituminösen Fahrbahnbelages



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

Beläge auf der Severn- und Wye-Brücke

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## 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<br>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.

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# 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 ½" (1.25cm) thick, stiffened longitudinally with steel troughs which provide stiffening webs at approximately 13½" (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~~ $\frac{1}{2}$" (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.

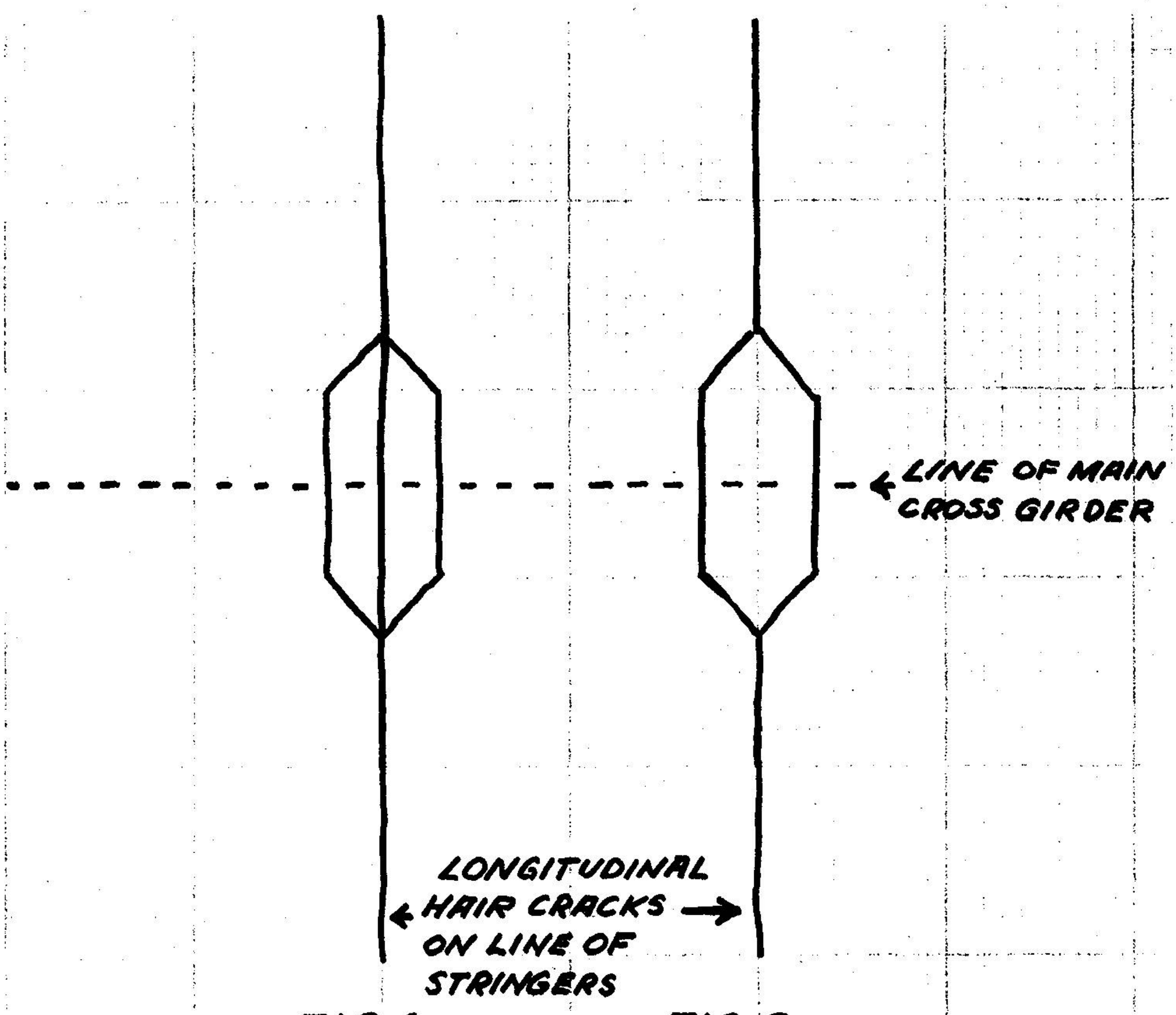


FIG.1. FIG.2.

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.

# 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, $20 \times 10^6$ axles have passed over them. This is approximately equivalent to $9 \times 10^6$ 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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(3). Henneke H., Gedanken über Gussasphaltbeläge auf Brücken mit Orthotropen Platen,Bitumen, 1964, 26 (5), 114 - 120

(4). Thul H. and F.K. Reinitzhuber, Asphaltic Wearing Surfaces, I.A.B.S.E., Symposium on Wearing Surfaces, Sept. 7th, 1968, New York.

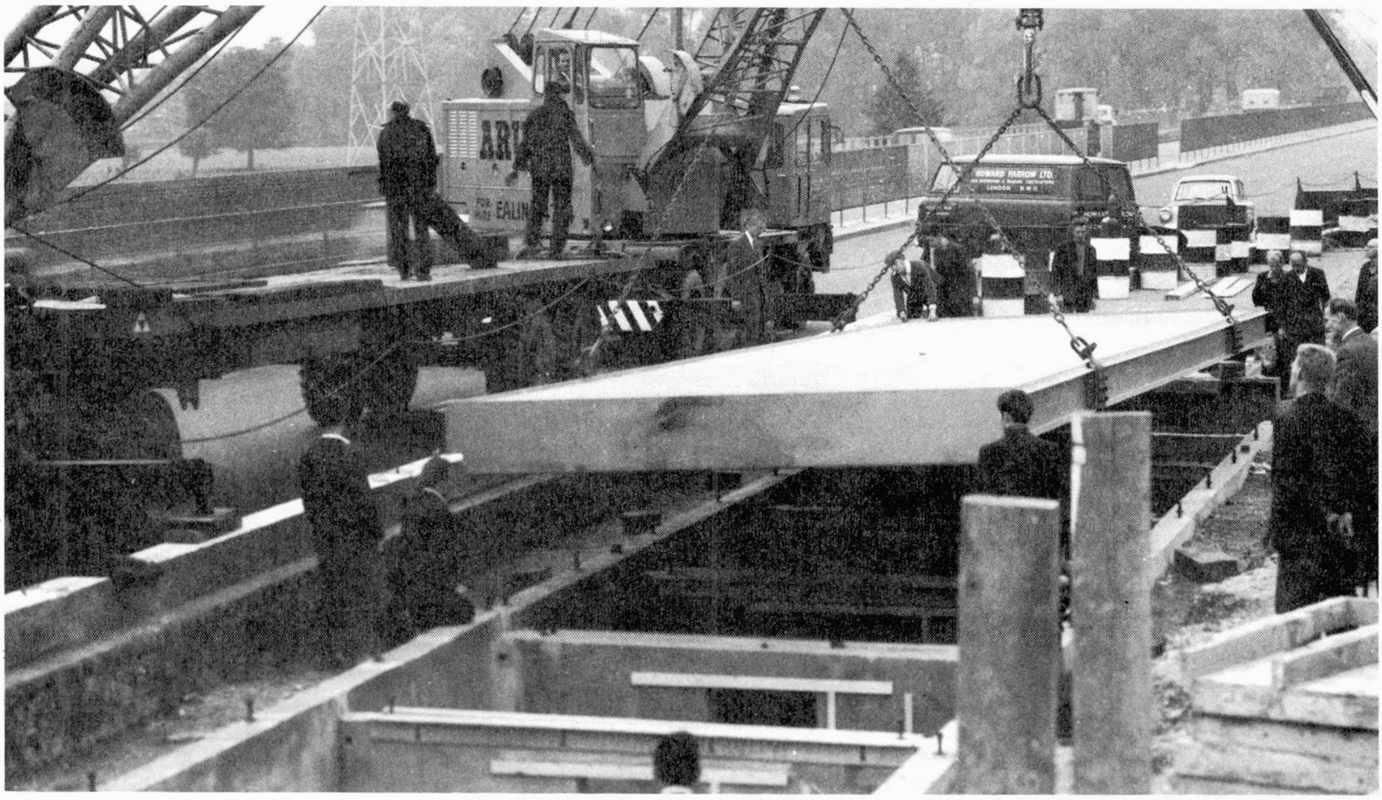


FIG. 1 TEST SITE UNDER CONSTRUCTION

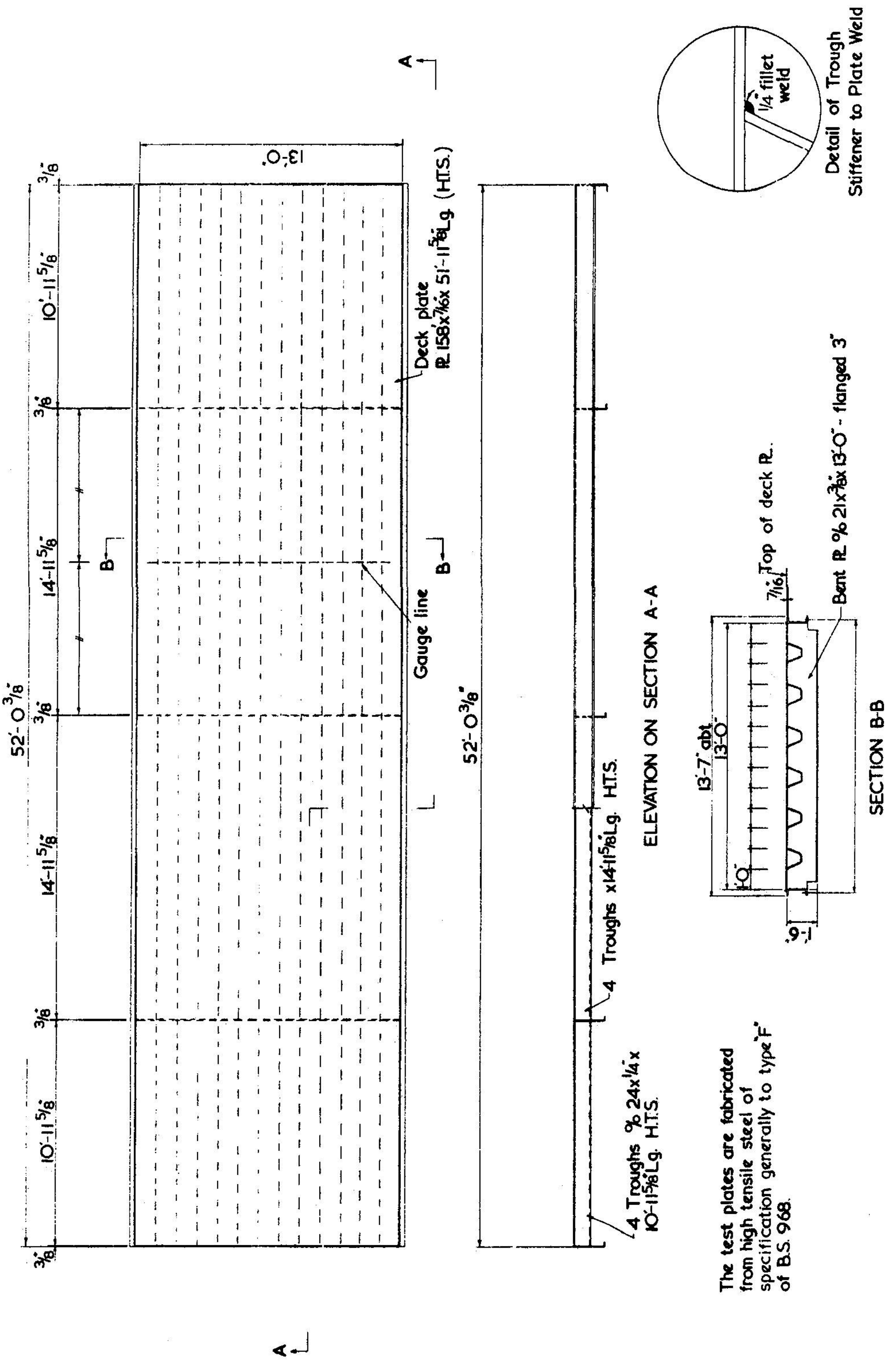
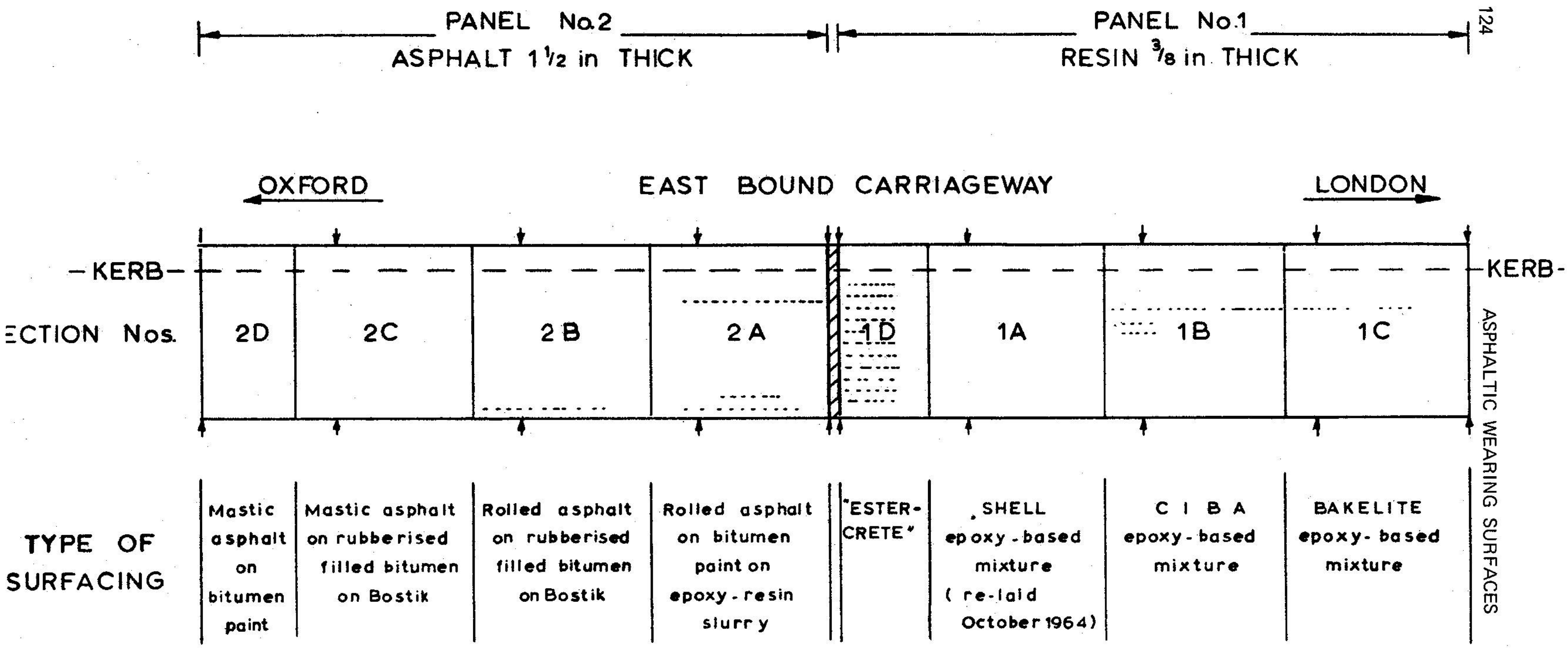



Fig 2 DETAILS OF DECK PANELS INSTALLED ON TEST SITE




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

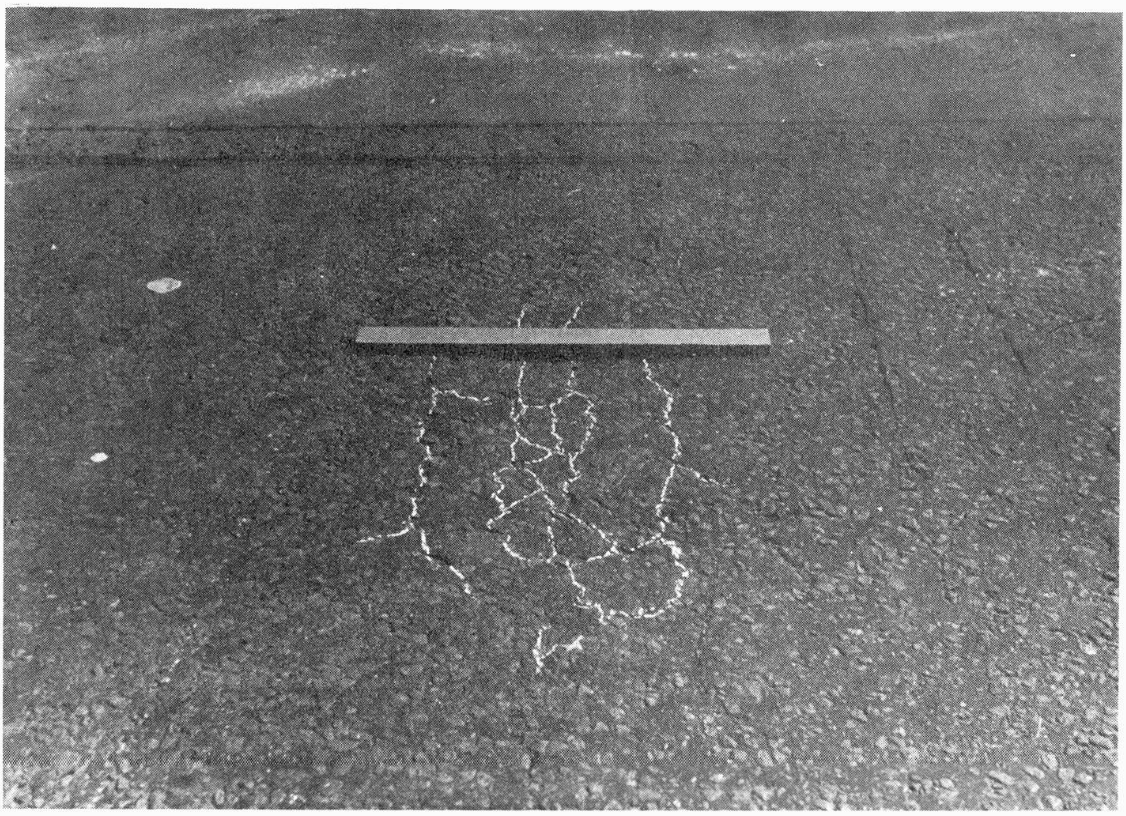


FIG. 5 DEFORMATION AND CRACKING IN ROLLED ASPHALT OVER GRITTED EPOXY RESIN, (SECTION 2A) JULY, 1968


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"

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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.