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Fatigue Cracking in Two Steel Bridges

Fissures de fatigue dans deux ponts en acier

Ermüdungsrisse in zwei Stahlbrücken

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SUMMARY
A survey of welded details in some revealed fatigue cracking in load bearing stiffeners in two major structures. In both cases analytical investigations were undertaken to identify the causes of the fatigue damage and to determine suitable repair procedures. In-situ static load tests were also carried out and the measured strains compared with the analytical results. The work has highlighted the difficulties in predicting the behaviour of structures and in determining the actual stresses in welded details, and has led the authors to question the current method of fatigue design and assessment.

RESUME
Dans le cadre d'un relevé de l'état de détails soudés de ponts routiers en acier, des fissures de fatigue furent repérées sur des raidisseurs d'appuis dans deux ouvrages importants. Une étude analytique fut entreprise en vue d'identifier les causes des dégâts par fatigue et de déterminer les procédures de réparation appropriées. Des essais de charge statiques in situ furent effectués et les efforts mesurés furent comparés aux valeurs calculées. Ces travaux mettent en évidence la difficulté de prédire le comportement de structures et de déterminer les sollicitations qui s'exercent effectivement sur les détails soudés. Ils amènent les auteurs à mettre en question les méthodes actuelles de calcul et d'évaluation de la fatigue.

ZUSAMMENFASSUNG
Anlässlich einer Untersuchung an geschweissten Teilen einiger Strassenbrücken entdeckte man Ermüdungsrisse in tragenden Versteifungselementen zweier bedeutender Brücken. In beiden Fällen wurden Analysen ausgearbeitet, um die Ursachen der Ermüdungsschäden zu finden, und um geeignete Reparaturmassnahmen zu treffen. An Ort und Stelle wurden statische Belastungsversuche durchgeführt. Die gemessenen Spannungswerte wurden mit den Resultaten der Analysen verglichen. Diese Untersuchung machte wiederum deutlich, dass eine genaue Vorhersage des Ermüdungsverhaltens von Bauwerken schwierig ist. Die Autoren stellen deshalb die derzeitige Methode zur Berechnung des Ermüdungsverhaltens in Frage.

1. INTRODUCTION

1.1 Survey of Welded Steel Bridges

In 1976 the Department of Transport in the UK organised a selective in-situ examination of typical welded joints in a number of steel bridges. The object of the investigation was to see whether cracking or other deterioration was occuring in the welded connections and to get some idea of how welded joints were actually performing in service. Twelve major steel bridges were chosen for the exercise, six being of box-girder and six of plate girder construction, covering different design configurations and severities of traffic loading. The bridges had been in service for from 8 to 20 years at the time of inspection.

For each particular structure critical areas were identified where it was thought that the stresses would be high or that fatigue cracks were likely to occur. The examinations, where possible, concentrated upon typical details within these areas, although the area of inspection depended also on the availability of suitable means of access. The inspections were only intended to cover a representative sample of typical welded joints. The methods used for inspection included careful visual examination and appropriate non-destinctive testing (NDT). The latter methods included magnetic crack detection and ultrasonic testing techniques. In some cases as a result of the initial findings extra inspections were carried out and samples cut from the welds were subjected to metalurgical examination in the laboratory.

1.2 Findings of Survey

Evidence of fatigue cracking was found in four of the bridges but in no case was it considered sufficiently serious to stop traffic on them. In some cases the cracking had been noted previously by the authorities responsible for the maintenance of the structure.

Some examples were also found of cracking which was attributed to defects originated at the time of welding, such as at the stop - start positions on fillet welds. None of these were considered to be of any great significance, but as a precaution it was recommended that they should be monitored from time to time. The fatigue cracking in two of the bridges, and the steps taken to analyse the problems and develop suitable remedial measures, are discussed in this paper.

2. BOSTON MANOR BRIDGE - DESCRIPTION

2.1 History and Description of the Bridge

The bridge forms part of the elevated section of the M4 motorway which runs from the west into London and is very heavily trafficked. It was constructed in 1964 and carries a dual 2 lane carriageway. The bridge consists of 4 eastern and 10 western spans carried by 2 pairs of plate girders and a central section of 3 spans carried by a pair of trusses (Fig 1). Some of the spans are continuous and some are cantilevered with suspended spans. Although sections of the bridge are curved in plan the girders themselves are straight between supports, with the curvature being obtained by laterally displacing the cross-girders relative to the main girder centre lines.

The deck construction is common to all spans and consists of an in-situ reinforced concrete slab cast on precast concrete planks. This is supported on cross-girders at 2.3m centres which bear directly on the top flanges of the main plate girders or on the top chords of the trusses. The deck is connected to the cross-girders by means of stud shear connectors. The cross-girders are fabricated from plates and have intermediate vertical stiffeners at 1.22m crs. Over the main girders (or chords) there are full depth double bearing stiffeners which are of heavier section and which are welded to both flanges (Fig 2). The intermediate cross-girders are fastened to the main girders by means of threaded studs which are welded to the main girder flanges, while the end cross-girders are fastened by means of high strength bolts.

2.2 Cracking in Cross-Girder Bearing Stiffener Welds

An earlier survey had revealed cracks at the top of many of the cross-girder bearing stiffeners at the extremities of the toe of the fillet weld to the top flange. The report on the inspection attributed the cracks to fatigue and noted that their length and number appeared to be increasing and that they were also present at the bottom flange as well. Following this inspection the maintenance authority conducted a visual survey of all the bearing stiffener welds throughout the structure.

Three modes of structural action were considered as possible sources of stress fluctuation and hence fatigue damage in the cracked components, namely:-

- longitudinal racking of the cross-girders under main girder deflections due to there being no longitudinal shear connection between the concrete deck and the main girders

- rotation of the deck slab under wheel loading

- moment restraint to cross-girders provided by each pair of main girders.

From a study of the cracking pattern it was considered that the first mechanism was the most likely source of cracking and it was decided to examine this hypothesis in more detail using both analytical and experimental methods.

3. BOSTON MANOR BRIDGE - FATIGUE ANALYSIS

3.1 Theoretical Stress Analysis

The object of the main theoretical analysis was to see whether a cracking pattern could be predicted which was consistent with what had been observed. The spans were first of all analysed under live load using a continuous beam program, each main beam being assumed to carry a proportion of the load with no composite deck action, in order to find influence lines for the beam rotations. The next step was to establish the correct relationship between the rotation of the main girder and the rotation of the cross-girder, the relationship being dependant not only on geometry but also on the assumptions made about the relative horizontal movement between the deck and the main girder neutral axes. Three different assumptions were considered and it was found that the one which assumed no relative movement best fitted the cracking pattern.

Further analyses were carried out, one of which considered the rotation of the deck slab under the passage of a vehicle. It was found possible to derive an equivalent beam to represent the slab and to use this to calculate the rotations of the slab and cross-girders under load, but it was not possible, because of the number of unknown parameters involved, to model the bearing stiffener to deck slab joint precisely. The flexibility of this joint is important in determining how rotations of the slab are transmitted into rotations of the cross-girder. Analyses were also carried out to determine any possible contribution to the stiffness of the main girders arising from composite action produced by the shear resistance of the cross-girder bearing stiffeners.

3.2 Static Load Tests

Two commercial vehicles were used for a series of static live load tests on the bridge with the resulting deflections and strains at certain locations on the cross-girders and main girders being measured with deflection gauges and electrical resistance gauges. The gauges not only measured strains in stiffeners and girders but also measured the relative displacements of flanges and stiffeners as a result of the racking movements induced by the applied loading. A large vehicle was used to load the main girder and produce the associated global racking movements while a smaller lorry was used to obtain the local effects due to the deflection of the deck slab itself. The measurements were used to produce influence lines of the strains (or stresses) in the stiffeners and cross-girders and the flange-stiffener separations at the fatigue cracks caused by the racking movements.

3.3 Comparison of Theoretical and Measured Results

Whilst there was fairly good matching between the predicted cross-girder rotations and the observed pattern of cracking throughout the structure, the correlation between predicted and observed measurements in other cases was not so good. For example the measured deflections of the main girders and the racking movements were approximately $\frac{2}{3}$ of the predicted values assuming no composite action, although deflections of the deck slab under wheel loading agreed with the results obtained by simple analysis. The greatest discrepancies were found between the predicted and measured stresses in the bearing stiffeners themselves; there were even differences found between the stresses measured in adjacent stiffeners which would have been expected to have similar stresses.

The normal design calculations assuming rigid joints between the cross-girder and main girder, and between the cross-girder and deck slab considerably overestimated the stresses in the bearing stiffeners. However this is not unexpected since the stresses are very much dependant on the fit-up between the various components, sequence of construction, shrinkage and creep of the deck concrete, prestress in the holding-down bolts and studs and other parameters. Without detailed knowledge of these items it proved impossible to achieve any better correlation between the predicted and measured stresses.

3.4 Prediction of Fatigue Life

In addition to the stress analyses already described, some fatigue life calculations were carried out according to the latest UK fatigue design code, BS 5400: Part 10, and using measured stresses. These calculations gave fatigue lives in the bearing stiffeners which were much shorter than those actually observed, and thus conservative. Part of the differences were thought to be due to the fact that traffic flows were lower than assumed in the code and part due to the difficulty in determining dead load stresses and the change of stresses themselves as a result of cracking. The code also predicted fatigue damage on other stiffeners where no cracking has so far been found. The general indication was that lives predicted by BS 5400 Part 10 were significantly shorter than the actual lives.

3.5 Remedial Measures

Two possible approaches to remedying the fatigue damage to the cross-girders and preventing further occurences were considered. These were:

- to make the deck act compositely with the main girders

- to improve the articulation between the deck, cross-girders and main girders.

Both approaches would involve rewelding the damaged welds where appropriate and necessary. The first solution was considered to be too expensive and so a solution based on the second was devised. Two possible ways of providing better articulation were considered; one was to encourage rocking rather than bending at each end of the bearing stiffener, the other was to brace the cross-girders rigidly to the deck so as to prevent all further racking movements and to provide bearings under each cross-girder to allow sliding movements to take place.

In the event it was decided to carry out trials on some of the spans using the former approach with the results being monitored. The existing fillet welds were cut out and replaced by full penetration butt welds. It was envisaged that the heavier weld would not only improve the fatigue life of the joint but would distort the flanges of the cross-girders sufficiently to pull the edges away from the concrete deck slab and the main girders and thus remove the load path from the stiffener tips. The repairs have been carried out and the resulting strains and differential movements are being monitored so that the effectiveness of the repairs can be assessed.

4. MAIDENHEAD BRIDGE - DESCRIPTION

4.1 History and Description of the Bridge

The Bridge is located on the M4 motorway where it crosses the River Thames. It was constructed in 1961 and originally carried dual 2 lane carriageways with cycle tracks and footpaths but in 1971 the cycle track was removed and the bridge converted to dual 3-lane carriageways. The bridge consists of 8 main shaped plate girders which are continuous over the main piers on the river banks and extend over short spans to abutments at each end (Fig 3). The girders are deeply haunched over the piers and the ends of the side spans are held down at the abutments.

The deck consists of an in-situ reinforced concrete slab which acts compositely with the main girders through inverted 'U' bars welded to tee-shaped sections which are in turn welded to the top flange of the main girder (Fig 4). The main girders are inter-connected not only by the deck slab but also by a series of transverse cross-frames. The main bearing stiffeners are constructed from rectangular trough section and extend from the main bearing blocks to the top flange and are welded to each side of the main girder webs.

4.2 Cracking in Bearing Stiffener Welds

The original sample survey covered 8 out of the 16 bearing stiffeners and in 3 cases revealed cracks at the toe of the fillet weld connecting the bearing stiffener to the top main flange. Some cracks were also discovered lower down the stiffeners where temporary erection cleats had been welded to facilitate the erection of the cross-bracing. A subsequent more comprehensive inspection revealed another 4 cases of toe cracks at the top of the bearing stiffeners. A sample was cut from one of the cracked welds and subjected to microscopic examination; the results indicated that the cracking was due to fatigue and it was considered that it was probably due to rotations of the deck slab over each girder under wheel loading.

5. MAIDENHEAD BRIDGE - FATIGUE ANALYSIS

5.1 Theoretical Stress Analysis

Theoretical stress analyses were carried out to find the stresses in the regions of the toe cracks at the top of the bearing stiffeners. In order to model the interaction of the deck slab, the bearing stiffeners and the transverse bracing, four separate 2D plane frame analyses of the cross-section were carried out to determine the stiffener stresses under wheel loads. The four analyses represented the following assumptions about the rigidity of the moment connection between the slab and the top of the bearing stiffener:

- bearing stiffeners rigidly connected to the deck slab
- stiffeners rotate with the slab but there is no significant resistance to horizontal movement
- shear connectors offer transverse shear resistance but no rotational restraint
- shear connectors offer no tensile or shear resistance.

In these analyses the slab was represented as an equivalent continuous transverse beam whose breadth was determined by further analysis.

A grillage model of a local 3 bay section of the deck slab was used to determine a more precise relationship between longitudinal wheel positions and the rotation of the slab at the bearing stiffener location. The opportunity was also taken to examine the effect of the concrete haunching over the main girder flanges and the lateral bending stiffness of the flange itself. In the analyses upper and lower bound values of stiffness were used based on 2 values of modular ratio.

5.2 Static Load Tests

Two commercial vehicles were used for a series of static live load tests on the bridge with the resulting deflections and strains at various locations being measured by transducers and resistance strain gauges. The strain gauges were used to determine the stresses in the bearing stiffeners while the transducers were used to determine the relative displacements of the girder flanges and the concrete deck slab. The large vehicle was used for the majority of the tests, being driven along a constant tracking position relative to the edge of the carriageway. The smaller vehicle was used to examine the effect of different wheel tracking positions on local stresses.

5.3 Comparison of Theoretical and Measured Results

All 4 plane frame analyses using the lower slab stiffness value predicted deflections which were substantially higher than those measured. However using the analysis which assumed a fully rigid moment connection between the slab and the stiffener and using the higher slab stiffness gave results which agreed reasonably well with those measured. This was considered to indicate that the slab-flange connection transmitted the slab rotations very effectively to the top of the bearing stiffeners.

The peak stresses calculated on the bearing stiffener using the above preferred method of analysis and the higher slab stiffness agreed well with the measured stresses. However it should be noted that the assumed slab stiffness was somewhat higher than would normally be used in design and so the agreement was to some extent rather artificial.

5.4 Prediction of Fatigue Life

In addition to the stress analyses some fatigue life assessments were carried out in accordance with the latest UK fatigue design code BS 5400: Part 10, and using the measured bearing stiffener stresses as a basis for the applied stress spectrum. The vehicle flows and weights were based on census point records from a location fairly close to the bridge. The object of the fatigue assessment was to determine whether the theoretical probability of failure of the welds was greater or less than had actually occurred. It was found that agreement could only be achieved if the stresses in the welds were considerably higher than the values calculated from the measured bearing stiffener stresses. This discrepancy was partly attributed to the uncertainty in determining the true weld throat size which is dependent on fit-up and the degree of root penetration. The general indication was that fatigue lives calculated by BS5400: Part 10 were longer than the actual lives.

5.5 Remedial Measures

Although various remedial measures were considered, including repairing the cracks by replacement welds with increased throat thickness, such a method of repair was rejected in favour of some form of deck stiffening aimed at reducing the deck rotations and consequent bearing stiffener weld stresses. The repair method adopted consisted of providing twin lines of universal beams to support the deck on both sides of the pier supports. These UB were fixed to the webs of the main girders by means of bolted cleats. The beams were fixed to the slab at mid-span by means of anchor bolts set in resin which were bolted to plates welded to the top flange of the beams. Additionally a similar connection was made between the top boom of the existing cross-bracing and the underneath of the deck slab (at the line at the pier supports). No repair was done to the cracked welds due to the risks involved in attempting such an operation on a highly stressed tension flange.

6. DISCUSSION

Although the 2 examples of fatigue damage in existing major steel bridges are of interest in themselves, there are a number of observations which can be made regarding fatigue design and assessment in general.

6.1 Fatigue Behaviour of Existing Structures

In both bridges described the fatigue failures occured due to the structures behaving in ways which had not been foreseen or allowed for in the design. In one case it was the racking movement of the cross-girders and in the other it was the rotation of the main girder top flange due to the deflection of the concrete deck. In both cases the welds affected would not be classed as structural welds since their purpose was to fix the stiffeners in place rather than to carry a direct load or shear force.

The examples described show the difficulties in determining the real stresses occuring in structural details, particularly those arising from secondary causes. In both cases different modes of structural behaviour had to be tried and it was also found that the calculated stresses were very dependant upon the fit-up between the different elements. It was only because in-situ strain gauge measurements were taken on the structures that it was possible to select the most appropriate assumptions regarding the structural behaviour and obtain reasonable agreement between experiment and theory.

The examples underline the importance of understanding the causes of any fatigue failure before undertaking any repairs. By identifying the cause of a failure it is possible to see for instance whether the crack should be rewelded or whether other remedial actions, such as modifying the structural behaviour, should be carried out. If such a study is not done then any repair welding may be ineffective in prolonging the fatigue life of the structure.

Although cracking occured in both bridges, and was fairly extensive in one bridge, it was not considered serious enough to prevent the continued use of the bridges concerned. There is thus a difficult decision in such cases as to whether to carry out repairs, which itself may be a costly and risky operation, particularly if it is undertaken under traffic, or whether to just monitor the cracking to see that it does not spread into a more sensitive part of the structure. In one of the cases quoted the cracks were not repaired and the remedial measures were limited to carrying out strengthening to modify the structural behaviour.

6.2 Implications for Fatigue Design

Unless the designer recognises the potential sources of fatigue damage in his structure then the existence of up-to-date codes of practice or design standards based on the latest fatigue testing data will be of little help in achieving satisfactory structures. There is a need for education to help the designer recognise the unlikely areas where fatigue damage can occur and particularly to make him aware of the effects of displacements in producing secondary stresses in any sort of welded connection.

The case studies have also highlighted the difficulties in deriving the real stresses occuring in structural details, particularly those arising as a result of secondary action. Stress levels have the major influence on fatigue damage and yet at present a relatively small proportion of fatigue research is devoted to the prediction of stresses, although there is some being done in connection with off-shore structures. There is thus a need for a better balance in fatigue research with more resources being devoted to the determination of stresses.

In this connection too there is a case for the testing of larger details which are more truly representative of the details found in actual designs. At present the majority of fatigue information is based on laboratory tests on relatively small scale idealised specimens, and so there are added uncertainties introduced in applying these results to real life structures. In other branches of engineering such as mechanical and aeronautical the testing of full scale prototype design details is common and yet this is hardly ever done in general construction although in the off-shore field large specimens are now being tested. Perhaps this trend for larger scale realistic testing will be taken up by the construction industry.

Finally there is a need to appraise the current approach to fatigue design, as for example set out in the UK code BS 5400: Part 10, to see whether a more pragmatic alternative approach might not be just as effective. There are many uncertainties involved in the present fatigue calculations ranging from stress prediction, loading, stress cycle counting, detail classification and fit-up to Miner's rule itself, so that the reliability of any fatigue calculation is very much in doubt. And yet, as the survey revealed, the fatigue problems that have been discovered have been in areas of secondary structural importance and have not been investigated at all for fatigue in the original design. There is a case therefore for developing a series of standard details or connections which would be shown by large scale tests or otherwise to have satisfactory fatigue performance under given load spectra. The approach would be one based on the adoption of good and proven design practices rather than on detailed calculation and assessment.

7. ACKNOWLEDGEMENTS

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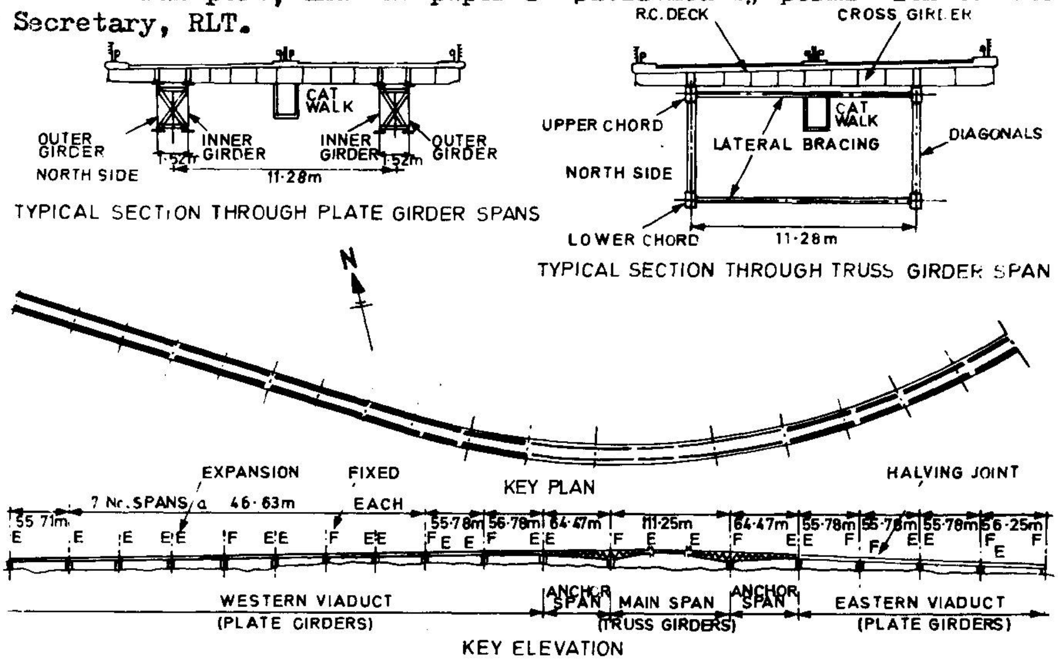
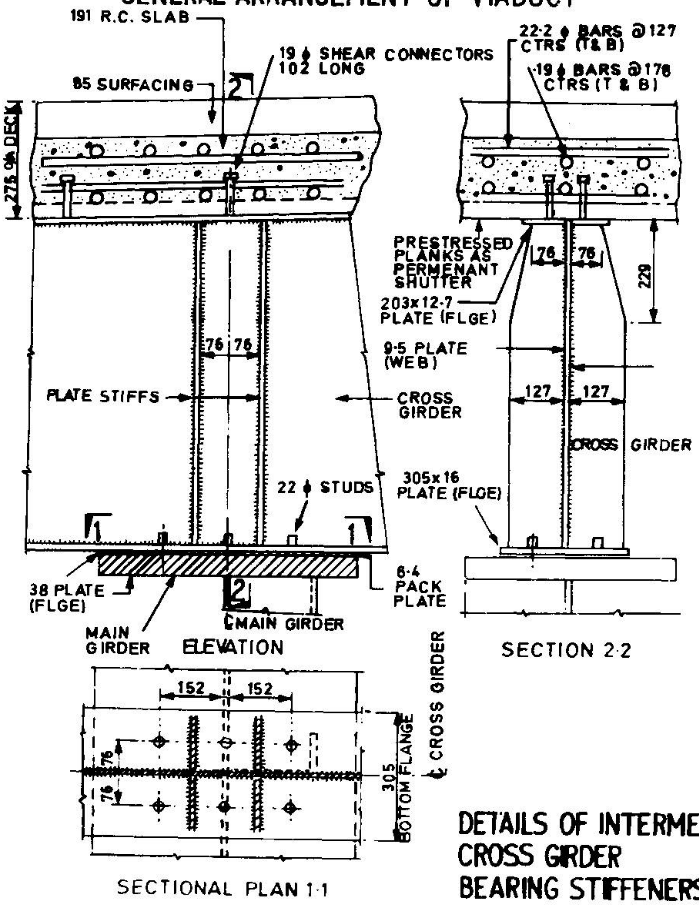


GENERAL ARRANGEMENT OF VIADUCT FIG. I.



DETAILS OF INTERMEDIATE CROSS GIRDER BEARING STIFFENERS FIG. 2

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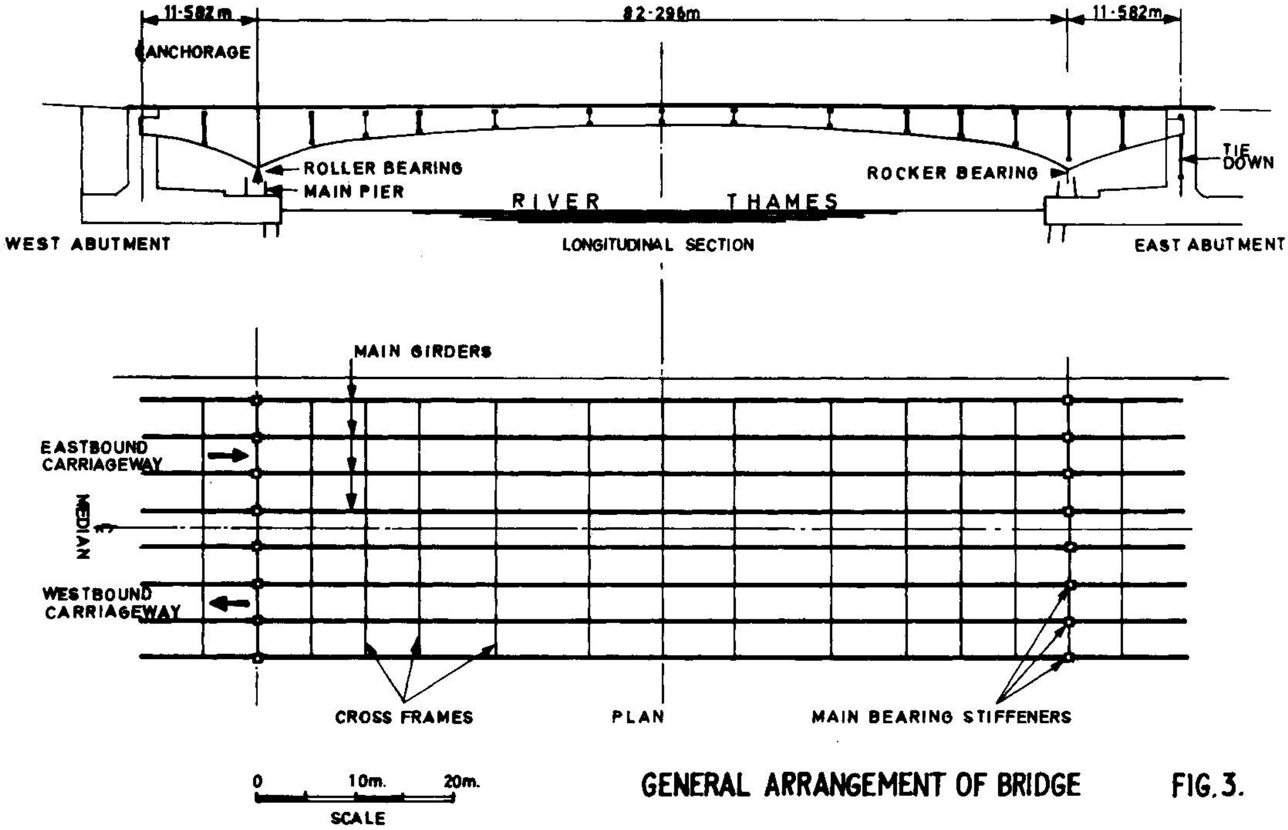

GENERAL ARRANGEMENT OF BRIDGE FIG. 3.

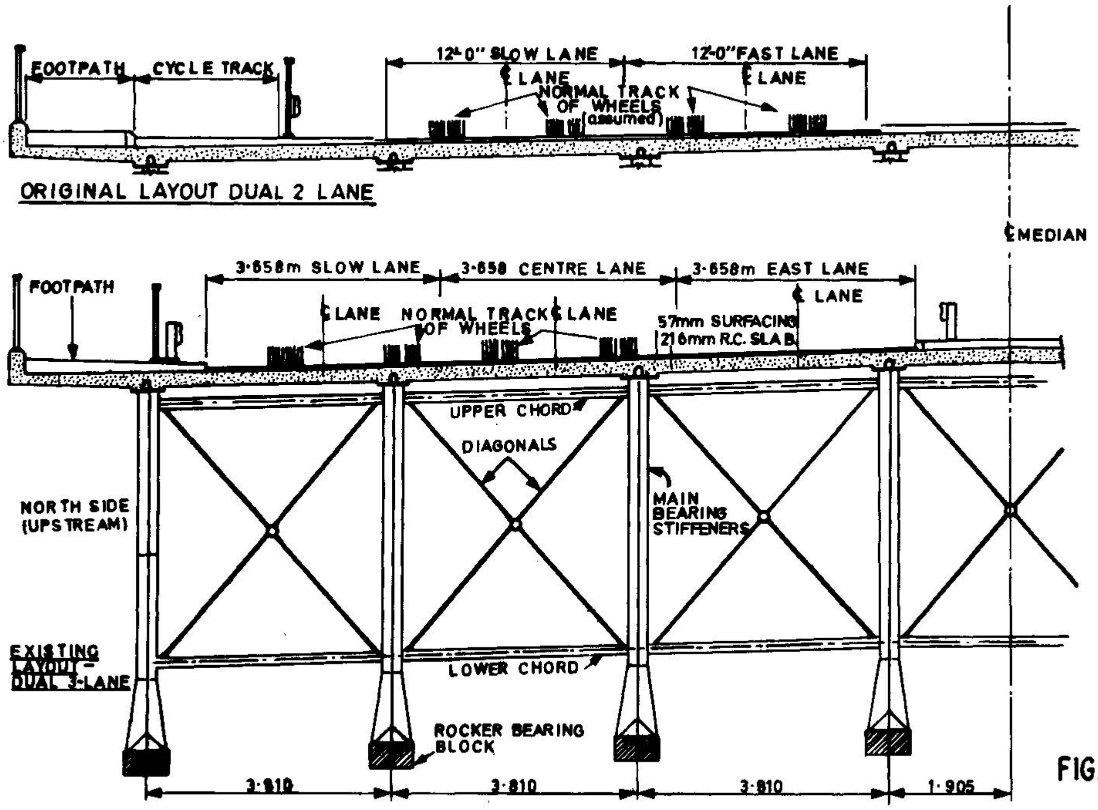

FIG. 4.

CROSS-SECTION THROUGH SUPERSTRUCTURE AT PIER UNDER EASTBOUND CARRIAGEWAY