The Clifton Suspension Bridge: preservation for utilisation

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Le pont suspendu de Clifton – réparation et exploitation

Die Instandstellung der Clifton Hängebrücke

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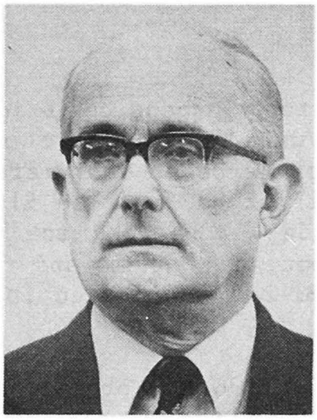

Stuart Cullimore, born in 1920, graduated in Civil Engineering from Bristol University in 1940. Returning after war service with the Royal Engineers engaged in teaching and research in structural engineering, latterly specialising in fatigue, joined Howard Humphreys as a consultant in 1984.

SUMMARY

A brief history of the Clifton Suspension Bridge includes a description of the structure and its operation up to 1952 when the present Trustees assumed responsibility. The extensive survey in 1953 and the subsequent works to make the structure capable of continued service as a part of the busy urban road network are described. Notes on modern operating practice are followed by details of a recent fatigue and fracture appraisal.

RÉSUMÉ

Le bref historique du pont suspendu de Clifton présente une description de la structure et de son utilisation jusqu'à 1952, date à laquelle il y a eu un changement de propriétaire. Une étude détaillée a été réalisée en 1953 et des travaux entrepris afin de poursuivre l'exploitation du pont dans le cadre du réseau routier urbain. Quelques considérations sont faites sur l'exploitation actuelle, et une récente étude sur la fatigue et la rupture est présentée.

ZUSAMMENFASSUNG

Die Geschichte der Clifton Hängebrücke sowie das Tragwerk und seine Funktion werden kurz beschrieben. Im Jahre 1953, in welchem der jetzige Besitzer das Werk übernahm, wurde die Brücke eingehend untersucht und entsprechende Instandstellungsmassnahmen für diese wichtige Verbindung durchgeführt. Das heutige Betriebssystem wird beschrieben wie auch die Ergebnisse einer kürzlich durchgeführten Untersuchung auf Ermüdungsschäden.

1. INTRODUCTION

The Clifton Suspension Bridge is known to engineers the world over as a historic structure and as a memorial to its famous designer I K Brunel. What is less well known is that it still forms an integral part of Avon's urban road network, carrying over three million vehicles annually. The purpose of this paper is to show how the Authority responsible for the bridge operates and maintains the structure so that it fulfils its utilitarian function whilst preserving it as a historical monument and tourist attraction.

2. CONSTRUCTION AND EARLY HISTORY

2.1 History

The history of the Clifton Suspension Bridge, spanning the Avon Gorge at Bristol, begins in 1753 with the bequest of £1000 by William Vick, a wine merchant, to the Society of Merchant Venturers of Bristol. The money was to be allowed to accumulate until it had reached £10,000, which he believed to be the cost of building the bridge. By 1829 the investment was worth £8000 and a company was formed to promote the scheme and raise further capital. One of four designs submitted by I K Brunel was chosen in competition with those of several leading civil engineers.

Construction was started in 1836 and proceeded intermittently, due to financial difficulties, until 1843 when the towers were practically completed. Little further was done. In 1853 the plant and structural ironwork were sold to pay the contractor and the works abandoned. The ironwork was subsequently used by Brunel in the Royal Albert Bridge at Saltash.

After Brunel's death in 1859 a new Clifton Suspension Bridge Company was formed by members of the Institution of Civil Engineers to complete the bridge as a memorial to Brunel. Work was started in November 1862 with Barlow and Hawkshaw as the Engineers [1] and the bridge was opened to traffic in December 1864 (Fig. 1).

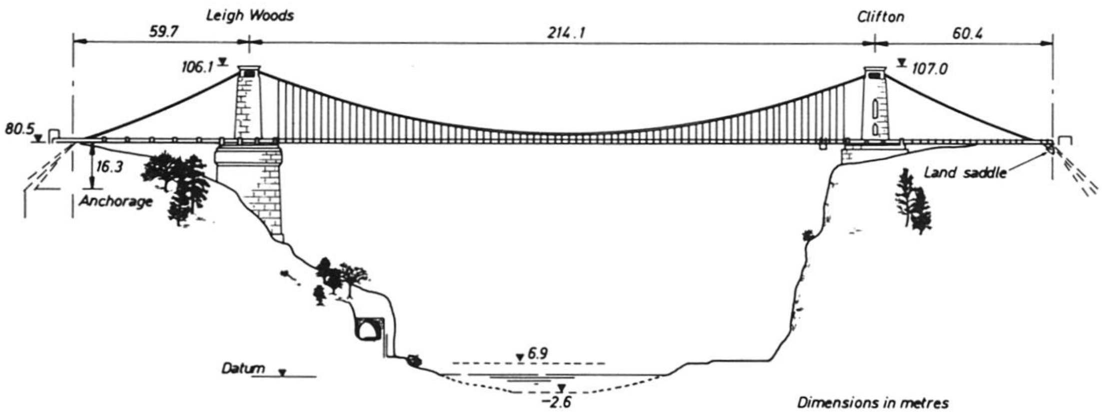

Fig. 1 South elevation of bridge

2.2 Construction

The principal changes made by Barlow & Hawkshaw to Brunel's design were to increase the number of chains in the main catenaries from 2 to 3 and the substitution of iron girders for timber ones to support the roadway. The 1:10 dip ratio of the chains was maintained but the inclination of the land chains was increased, bringing the anchorages nearer the towers.

The alteration of the design of the main catenaries was necessitated by the decision to use the chains from the recently demolished Hungerford Bridge over the Thames in London - also designed by Brunel. These provided sufficient links for only two chains each side and new links were made for the third chain. Both these and the Hungerford links had all been loaded in tension to 154 N/mm^2 stress in the shaft. The purpose of this was to test the hot hammered weld joining the lug ends to the shaft of the link.

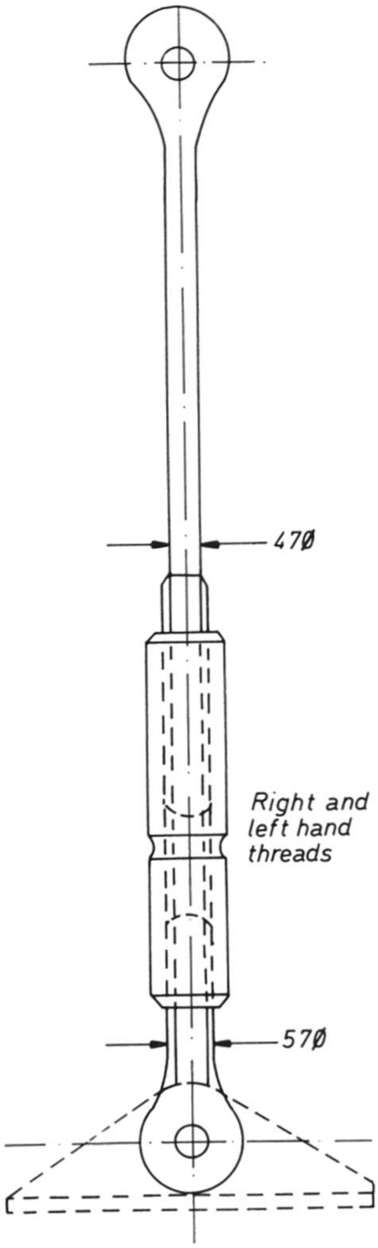

Fig. 2 Suspension rod

The chains are formed from 7315mm long links joined by 114mm diameter pins passing through their lug ends. The links are interleaved on the pin, 10 on one side and 11 on the other with 12 links entering the tower saddles. The three chains are placed one above the other 6100mm apart transversely between their centres and arranged so that the pins are at 2440mm intervals across the span. Straps hang from each pin to which the eye end of a suspension rod is attached; the lower end of the rod is attached to the top flange of the longitudinal stiffening girder (Fig. 2). Each suspension rod is in two parts connected with a turnbuckle (bottle screw) by which its length can be adjusted, enabling the tensions in the rods to be equalised.

The two longitudinal stiffening girders are plate girders 914mm deep and carry a cross-girder from the lower flange at each suspension point (Fig. 3). The booms of the cross-girders are formed by two angles back-to-back with flat bar diagonal bracing members between them. The cross-girders cantilever out from the suspension points to support the footway. At the outward edge of the footway there is a light cross-lattice longitudinal girder with a stout handrail. Diagonal wind bracing is provided between the cross-girders. The deck is made from 127mm deep rectangular timbers placed side by side longitudinally on the cross-girders and covered with 50mm thick transverse planking. The deck is surfaced with 32mm nominal thickness mastic asphalt.

The mass masonry abutments are faced with red sandstone ashlar. The towers, faced in local Pennant stone, have a freestone capping with a cast iron parapet. They carry the saddles and roller beds which also came from the Hungerford bridge and were increased in height to accommodate the third chain. The land chains are attached to the tower saddles and span freely to fixed land saddles just below ground level. From there they dip down at about 45° for 18m separating to pass through separate cast iron anchor plates which are supported by brick arches abutting on the solid rock (Fig. 4).

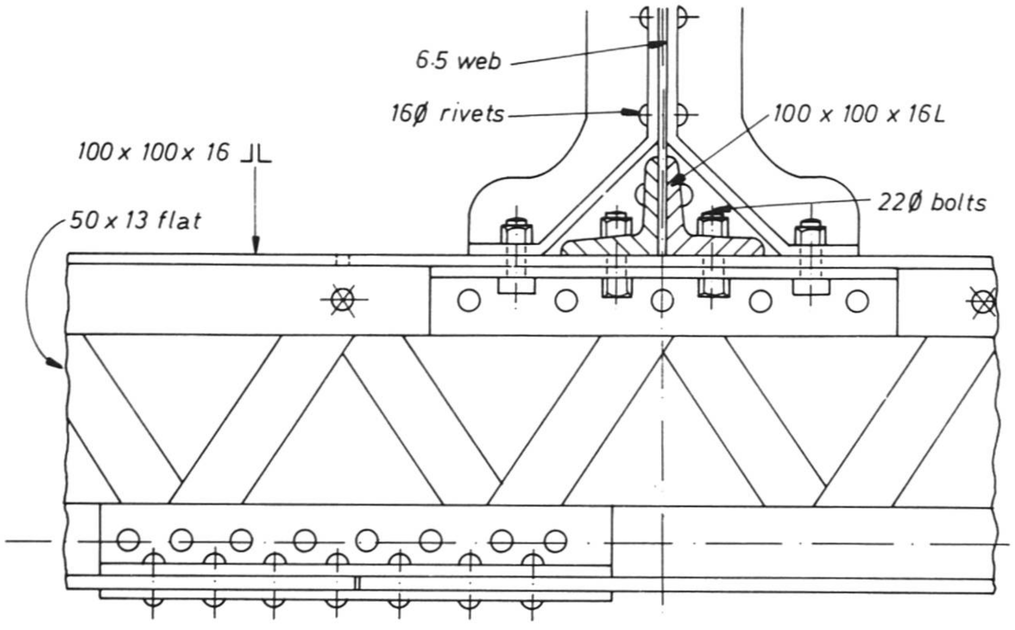

Fig. 3 Longitudinal/cross girder connection

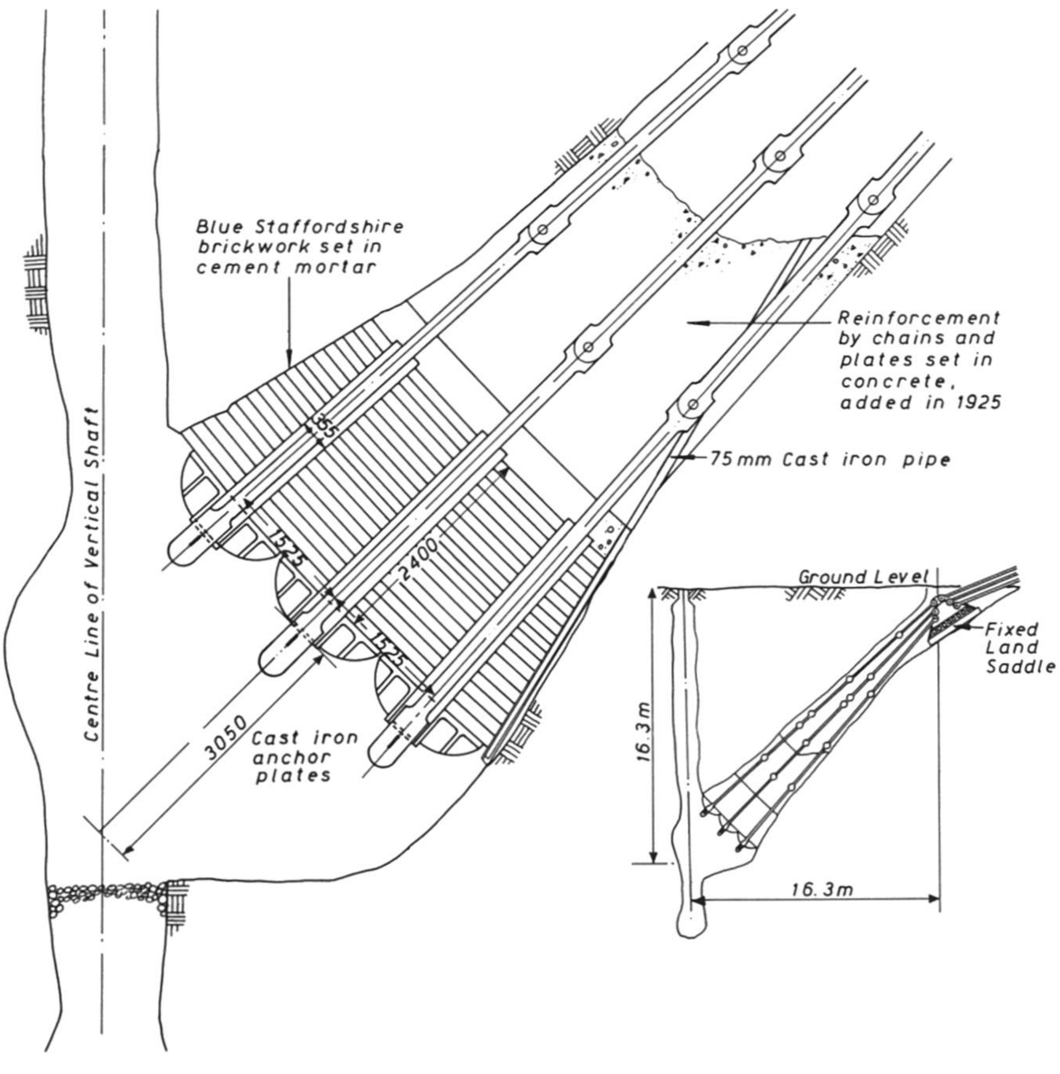

Fig 4 Anchorage

2.3 Operation and Maintenance 1864-1952

The description of the operation and maintenance of the bridge can conveniently be divided into two periods, 1864-1952 when it was in the hands of the Company and since 1952 when it was taken over by the present Trustees, the successors to the trustees of the original legacy. The funds for the upkeep of the bridge and payment of staff are obtained from tolls. The present consulting engineers, Howard Humphreys and Partners were appointed in 1910. Their association with the Bridge for over 75 years thus spans both periods.

The ironwork was originally coated with tar and later with bituminous compounds and paints, although no records are available to indicate how frequently painting was necessary in the increasingly aggressive atmosphere caused by developing local industry. However, by 1924, serious corrosion had occurred in the chains at the bottom of the anchorages, particularly in the Leigh side, caused by drainage from the roadway and inadequate ventilation. The chains were strengthened by adding a new link to each chain where it entered the anchorage arch and reinforcing the sections in the anchorage tunnels by open-link chains from below the land saddles outwards to new anchorages in the tunnel walls. The whole was then embedded in concrete. The Clifton anchorages were similarly treated in 1939.

It is known that the transverse timber planks of the deck were replaced three times: in 1884, in 1897 when the roadway was surfaced with mastic asphalt and again in 1958.

The structure for which the new Trustees had assumed responsibility in 1953 was a part of the national heritage. It was also becoming an integral part of the urban road system linking Bristol with the rapidly developing residential areas to the north west and by 1984 3.1 million vehicles were crossing the bridge annually. It was therefore necessary to confirm that the bridge was adequate for the demands of modern traffic and to decide the work necessary to maintain the structure in a serviceable condition. In the meantime, to prevent damage to the deck, the axle loading was restricted to 2.5 tonnes. Previously a vehicle weight limit of 5 tonnes had been in force.

3. MAINTENANCE AND OPERATION 1953-PRESENT

3.1 The 1953 Inspection and Testing

During 1953 a thorough survey, supported by an extensive programme of testing, was carried out to assist the preparation of plans for the necessary maintenance and replacement projects. Inspection of the superstructure revealed general corrosion of the cross-girders and damage to some of the diagonals. The end cross-girders were badly corroded and it was decided to replace these immediately with steel I-beams. Subsequently one of them was tested to destruction in the laboratory at Bristol University to obtain an assessment of the cross-girder capacity. Fairly extensive attack by brown rot was discovered in the timber decking. The original timber - much of which remained in the longitudinal baulks - had been treated with tar which gives little protection against brown rot.

Loading tests on the bridge were devised to examine the structural behaviour of the deck and so find the appropriate loading and support conditions for the girder tests. Preliminary tests were done in 1953, with a more extensive programme in 1954 [2]. In the latter the loading was provided by trailers with axles at 2440mm centres - the spacing of the cross-girders - each giving a load of 4 tonnes. The tests took place at night with the bridge closed to other traffic. Strains were measured in the suspension rods on a number of elements of the cross-girders and on the longitudinal girder. The results indicated that only about 60% of an axle load was carried by the cross-girder beneath it. In addition to this diffusing action the longitudinal timbers stiffened the top boom of the girder and reduced the compressive stress in it below the expected value. Realising the importance of this action of the decking, close attention is paid to the security of the bolts holding the timbers to the cross-girders. Some bowing of the 'compression' diagonals near the longitudinal girder connection was noted. Apart from this no other distress was observed in the elements adjacent to the connection nor in the U-bolts. The loading of a typical cross-girder under the 4 tonne axle load together with inspection cradle and footpath loading provided a proof test with an adequate margin of safety over the permitted 2.5 tonne axle load. The greatest stresses measured were 19 N/mm^2 tension and 38 N/mm^2 compression. The maximum deflections measured under the 8 tonne trailer and tractor were 56mm upwards and 84mm downwards at the quarter span point.

The influence line for movement of the tower saddle was asymmetrical, indicating that there was appreciable friction in the structure.

For the laboratory tests on the end cross-girder only the length between the longitudinal girders was used. Under 4 tonnes loading through timber simulating the decking the strains measured agreed well with those recorded in the loading tests on the bridge. For the subsequent test to destruction the timber was removed. The failing load was 343kN, departure from overall linearity being noted at 140kN. Final failure was by plastic lateral buckling of the top boom, preceeded by twisting of the lower boom and bowing of the compression diagonals. The mode of failure confirmed the importance of preserving the composite action between the timber decking and the cross-girders.

3.2 Foundations - Site Investigation

The promontory from which the bridge springs on the Clifton side is a stable formation of massive blocks of rock. The Leigh Woods masonry abutment is approximately 23m square, rising some 40m from the side of the cliff (Fig. 1). There has been no evidence of any movement at either abutment although there had recently been 'slips' in other parts of the Avon Gorge. Large solution cavities are known to exist in the limestone in this region together with clay layers within the limestone sequence. In view of this and the presence of water seepage in the vicinity, a site investigation of the Leigh Woods abutment was ordered in 1969.

Four boreholes were sunk - one near each corner of the abutment and a fifth through the abutment from roadway level. The latter showed that the heart of the abutment consisted of plums of sandstone well embedded in sound lime/ash mortar. Cores from the other boreholes showed that the abutment is founded mainly on very hard limestone and partly on well compacted and cemented breccia. Thin mudstone layers below the abutment were also revealed. These were, however, dry and discontinuous, so constituting no risk of sliding along bedding planes. Resistivity measurements between the boreholes indicated the presence of an anomaly - which could possibly have represented a cavity - some 12-30m below the abutment. The collapse of a cavity at such a depth within the lifetime of a man-made structure was ruled out. It was concluded that such very small risks as existed could be ignored and that the Leigh Woods abutment was quite safe.

Rock falls in the Gorge during 1978 caused the Avon County authorities to remove areas of loose and unstable rock from the cliff faces. Blasting operations in the vicinity of the bridge were carefully co-ordinated with the Bridge Manager and were monitored by velocity recorders bolted to the rock beneath the Clifton tower. The maximum resultant particle velocity recorded was 5mm/sec, well below the specified limit of 10mm/sec and there was no detectable damage to the bridge structure.

3.3 Maintenance

3.3.1 Deck Structure

The effects of increased traffic loading were felt mostly in the roadway and suspended structure and it was to these parts of the structure that the maintenance effort was first directed.

The badly corroded end cross-girders were replaced by zinc coated steel I-beams. Transverse chambers were cut in the abutments to improve ventilation and provide access for inspection and painting. Higher capacity roadway drains and watertight movement joints between the suspended deck and the articulated spans, installed recently, have further improved the situation so that the cross-girders keep dry in heavy rain. The diagonals of the cross-girders were strengthened in the areas where the tests had shown weakness and badly corroded bolts and nuts renewed. The whole underbridge was then grit-blasted and zinc sprayed.

3.3.2 The Roadway

The whole of the timber decking was replaced with pressure creosoted Douglas fir. The longitudinal baulks were secured to the cross-girders by Lindaptors on bolts passing through the timbers so that the nuts were accessible from underneath. These could therefore easily be tightened to maintain composite action essential to the effective structural action of the deck. The racking action of the deck causes severe strains in the asphalt surfacing. Cracking and the consequent penetration of water to the timber was reduced by reinforcing the asphalt with expanded metal laid on bituminous felt and nailed to the timbers. The 32mm thick surfacing was put down in two layers the 13mm thick lower one containing no chippings. The surfacing has worn well and apart from minor areas of repairable damage, appears to be serviceable for several more years.

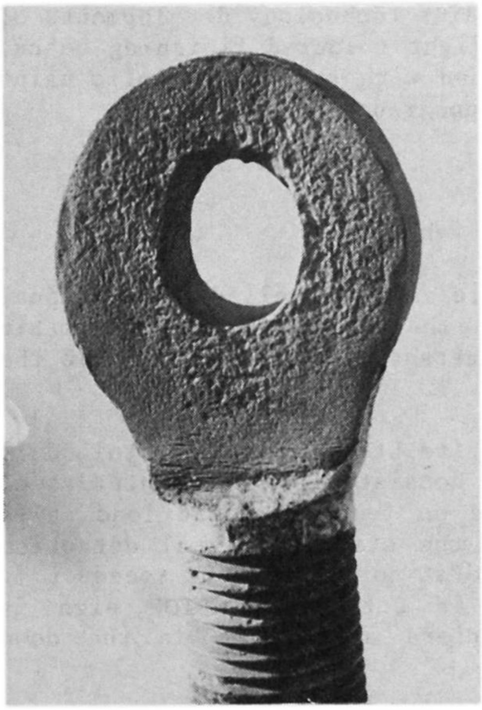

Fig. 5 Wear in suspension rod eye

3.3.3 Suspender Rods

The passage of a load across the span causes relative longitudinal movement between the chain and the stiffening girder. The resulting rotation of the ends of the suspender rods about the pins connecting them to the structure and the consequent wear is greatest at mid-span where the rods are shortest and decreases towards the ends of the span. Bolts in the shorter rods have been replaced at various times since 1887 but by 1970 wear of bolts, eye ends and shackles had become unacceptable in 66 rods around mid-span (Fig. 5).

Rods were removed for repair two or three at a time by transferring the load in each to a 12mm dia. length of pre-stressing strand which was attached to a yoke and tensioned with a pre-stressing jack. The rod eyes were filled with weld metal, bored, grit-blasted and bushed with a steel backed sintered bronze layer impregnated with a PTFE/lead mixture.

The shackles and cleats were repaired in situ by fitting bushes bedded in iron loaded epoxy resin injected into the worn eyes (Fig. 6). Stainless steel studs, 25mm dia. replaced the 38mm dia. wrought iron bolts. The bottle screws in the rods had been freed by injecting penetrating oil under pressure so that when replaced the length of the rods could be adjusted to equalise their tensions.

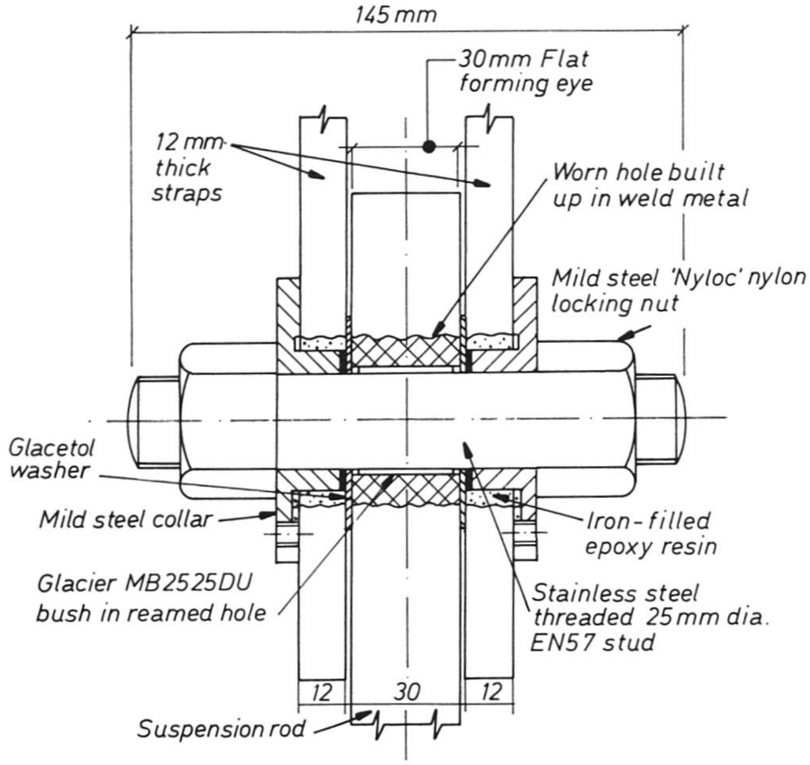

Fig. 6 Suspension rod bolt replacement

3.3.4 Painting

Grit-blasting has now largely replaced manual preparation methods giving improved quality and speed and eliminating the damage caused by hand scraping, particularly in the zinc sprayed areas. Modern paint technology developments of acrylated rubber paints, which are available in light coloured finishing coats, have eliminated the 2 extra 'cosmetic' coats needed with the MIO phenolic paint used previously to give a pleasing 'Flake Grey' appearance.

3.4 Operation

3.4.1 Traffic Control - Prevention of Overweight Vehicles

In the past the detection of overweight vehicles had relied upon visual surveillance by the toll collectors standing in the roadway. When automatic toll collection replaced them in 1975 it was necessary, for the safety of the structure, to instal an automatic detection system.

A modified standard weighbeam, supplied by Weighwrite Ltd, has been installed in the approach roads at each end of the bridge. It consists of a fabricated steel platform, flush with the road surface, supported on four hydraulic load cells with strain gauge transducers. Modification of the standard signal detection and analysis system enables vehicles to be weighed at normal traffic speeds [3]. When an axle exceeding the 2.5 tonne limit is detected a STOP sign is illuminated, an alarm is sounded and the barriers are locked in the down position.

The only significant operating problem has been the failure of transducers caused when the bridge was struck by lightning. Improved earth bonding of the equipment has now practically eliminated this problem and a high level of reliability has been achieved.

4. FATIGUE AND FRACTURE INVESTIGATION

4.1 Introduction

Prompted by the collapse in 1967 of the Point Pleasant Bridge in the USA, caused by a fatigue initiated fast fracture in a chain link eye, the Trustees instructed Howard Humphreys to make an appraisal of the risk of fatigue crack initiation, brittle fracture and propagation of pre-existing cracks.

Areas of load concentration were identified as the suspender rods, the longitudinal/cross-girder connection and the eyes of the river chain links pinned to the tower saddles.

4.2 Fatigue Investigation

4.2.1 Suspender Rods

The lug ends of all the rods and of the chain plates were hammer welded on to the shafts. Tension tests made on a spare rod confirmed the results of tests made in 1923 where breaking strengths of around 340 N/mm^2 were obtained, the strengths of the weld material averaging about 85% of this. Metallurgical examination showed that there was no really sharp demarcation of the weld, demonstrating the soundness and effectiveness with which the welding had been carried out.

A series of axial loading fatigue tests were carried out on round specimens machined from the rod, using a mean stress of 46 N/mm^2 as representative of the dead load stress in the rod. Seven specimens were tested at a stress range of 340 N/mm^2, one failed at 1.23, one at 4.04 and a third at 7.90 million cycles of load. The fourth survived 10 million cycles of load unbroken, as did two specimens tested at a stress range of 333 N/mm^2 and one at 308 N/mm^2. A large slag inclusion was found in the fracture area of the specimen that failed after 1.23 million cycles.

The greatest measured stress range in the rod with an 8 tonne vehicle on the deck was less than 8 N/mm^2 comprising axial and bending stresses. Even allowing for the stress concentration effects due to the rough surface of the rod and changes in section the risk of fatigue failure of the suspender rods could be discounted since the maximum expected stress range is so far below the endurance limit.

4.2.2 The Longitudinal Girder/Cross-Girder Connection

In this area of the structure (Fig. 3) the live load is transmitted from the deck, through the cross-girder to the longitudinal girder and thence to the suspender rod. This load concentration will result in stresses which are higher than in adjacent parts of the structure making it a possible fatigue cracking site.

A 2m long replica of the longitudinal girder and its attachments was fabricated from structural steel and loaded statically. The greatest live load stress measured around the area of the cross-girder connection was 6 N/mm^2 per tonne. The stress range caused by the most severe vehicle loading would be 22 N/mm^2. This maximum loading occurs infrequently - probably not more than 1000 times per year - and the overweight vehicle detection system ensures that it will not be exceeded. The traffic consists mostly of cars and light vans which are estimated currently to cause annally 1.5×10^6 stress ranges of $3N/mm^2$ and 3×10^6 stress ranges of $1.8N/mm^2$.

The fatigue strength obtained from tests on wrought iron of similar age (4), extrapolated to an endurance of 10^8 cycles of fluctuating tension, was $\pm$ $43N/mm^2$ allowing for a corrosive environment and the presence of rivet holes. An investigation to establish a basis for the evaluation of the residual life of wrought iron bridges [5] concluded that a safe design stress range for members with holes, based on a survival stress level of $114/Nmm^2$ at 2×10^6 cycles would be 70 N/mm^2. It has been demonstrated [7] that under variable amplitude loading, for the long endurances relevant to bridge service, stresses below the constant amplitude fatigue limit contribute significantly to the damage. However, the greatest live load stress range is well below these limits and of very low frequency; the magnitude of the high frequency occurences is so small that the risk of fatigue failure must be infinitesimal.

4.2.3 The Chains

Examination of the joints in the chains showed that there was no rotation of the links about the pins joining them so that each chain behaved like a solid flexible bar. Distortion of the chains caused by loads on the bridge is taken up by rotation of the end eyes of the river chains about the pins connecting them to the tower saddles. The largest rotations are those caused by wind and temperature changes.

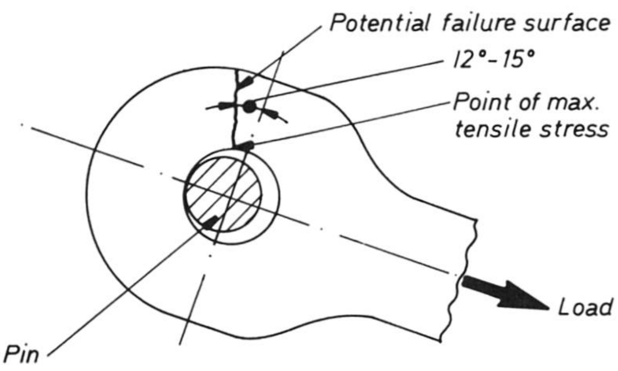

Fig. 7 Chain eye geometry

These rotations cause an effective eccentricity of the line of action of the load from the pin, resulting in variations of stress in the link and possible fatigue damage. A test was carried out in which rotations and strains in the end links were measured for various positions of a heavy load on the deck. The maximum stress at the inside of the eye (Fig. 7) was inferred from the measured strain on the outside of the eye using a finite element analysis, supported by tests on a full scale mild steel replica link. The three sets of results were in good agreement and indicated the maximum equivalent eccentricity of the load was +15mm and the maximum stress fluctuation +33.6 N/mm^2 about a mean stress of 192 N/mm^2. No results were available [4] for mean stresses as high as this but extrapolation indicated that the fatigue limit was near the measured stress fluctuation. It was then realised that not only had each link been subjected to a proof load but that the bridge itself had been proof loaded with 500 tonnes of stone. Either of these loadings would have caused yielding at the inside of the eyes which would give a residual compression stress there. The effect of this is estimated to reduce the mean stress to $150N/mm^2$ at which the existing stress fluctuation would be acceptably below the fatigue limit.

Having demonstrated that there was a satisfactory margin of safety against fatigue crack initiation it was then necessary to examine the possibility of a pre-existing crack growing to the critical length for brittle fracture. It was thought that the dynamic effects of a brittle fracture of one link might cause progressive failure of the remainder. Wrought iron is known to be susceptible to brittle fracture. Although made of steel the catastrophic consequences of brittle fracture were demonstrated by the Point Pleasant Bridge collapse.

The investigation comprised fracture toughness tests using compact tension specimens (CTS) from samples of wrought iron from two contemporary bridges to establish a critical stress intensity factor (K_{IC}) value for comparison with calculated stress intensity (K) factor values for the link eye. The BERSAFE finite element program, developed by the Central Electricity Generating Board [6], was used to calculate K values for cracks of various lengths along the line of maximum stress (Fig. 7). It was found that, for cracks longer than about 13mm, there was little further increase in K.

It proved impossible to obtain stable crack growth in compact tension specimens from the first sample of iron so no fracture toughness tests could be done. In fracture toughness tests at -18°C on the second sample the onset of plasticity prevented a valid K_{IC} value being obtained. However, the value of the stress intensity factor for the onset of rapid crack growth (K_C) was roughly half that obtained by the elastic analysis for a 19mm crack.

In view of this encouraging, but inconclusive, evidence it was decided to extend the analysis to account for yield around the crack tip and to obtain crack opening displacement (COD) data for the material. Because of the great differences in the properties of the two previous samples of iron it was considered essential to use Clifton iron for these tests. A representative link was removed from near mid-span and replaced with an aluminium sprayed mild steel one. CTS samples were cut from the critical areas of one eye for COD testing at -18°C, the lowest expected temperature at the bridge. These tests gave values of the critical crack tip opening displacement, required for unstable crack propagation, exceeding the calculated value for the worst conditions by a factor of 1.7. The risk of failure from a pre-existing crack growing to a critical length and causing a fast fracture was therefore discounted.

Finally, the possibility of the propagation of a pre-existing crack by fatigue loading was considered. Crack growth tests had indicated that a 'safe' value of the threshold stress intensity factor range (ΔK_{TH}), below which a crack would not grow would be 4.22 $MNm^{-3/2}$. The shortest through-thickness crack on the inside surface of the eye at which this value of ΔK would be just exceeded, under oscillations occurring in extreme wind conditions, is 5mm. Cracks of this size or larger are visible and would have been most unlikely to have escaped detection during the proof testing. It is also improbable that a crack in the normal direction would have been caused by the forging process. The maximum load fluctuations are caused by extreme wind and diurnal temperature changes so that their frequency is low and it is estimated that the extension to date of a pre-existing small crack would be of the order of 5mm only.

The links at the tower saddles are now regularly inspected using the fibre-optic endoscope and no cracks have been found. As no other damage of this nature has revealed itself in over 120 years, it is reasonable to conclude that the probability of failure of a chain due to a fatigue of the links is negligibly small and can be discounted. In any case the highly redundant nature of the structure with its many alternative load paths makes it capable of sustaining extensive local damage without causing catastrophic failure.

It is therefore reasonable to assume that given the present very high standard of maintenance, coupled with effective control of traffic loading and backed by rigorous inspection and monitoring, not only can this historic bridge be preserved but that it will continue to perform its structural function well into the next century.

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