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de trafic les plus sévères. Elles sont si faibles que pour établir d'une manière précise la distribution des moments dans ces systèmes, il a fallu employer des instruments de mesure particulièrement sensibles et recourir à des charges d'essai très élevées. On peut toutefois d'ores et déjà indiquer que la connaissance plus approfondie du comportement de ces tabliers sous les charges roulantes permettra de réaliser de substantielles économies de matériaux.

Les essais de laboratoire concernant les dispositions a) et b) ont été faits sur des modèles à l'échelle de 1:3 ou de 1:4; pour les grilles c), on a adopté l'échelle 1:2. La portée de tous les modèles était de 2,70 m. Un dispositif spécial comportant un cadre en acier permettait d'appliquer les charges individuelles en tous points voulus. Dans chaque essai, on a utilisé plus de cent extensomètres électriques pour la détermination des contraintes dans le béton et dans les poutres en acier.

Ces mesures ont mis en évidence une très large distribution transversale, même en cas de fissuration de la dalle de béton. On constate encore un comportement combiné très net entre la dalle et les poutres en acier du fait de frottement, même lorsque la dalle n'est pas rendue solidaire des poutres par un goujonnage. Il y a néanmoins lieu de prévoir un goujonnage lorsque l'on veut réaliser la coopération intégrale des deux éléments béton et acier.

c) Problems of Impact and Fatigue and their Effect on Permissible Stresses in Cast Iron Girder Bridges

Stoß- und Ermüdungsprobleme und ihre Auswirkungen auf die zulässigen Spannungen in Balkenbrücken aus Gußeisen

Les questions de choc et de fatigue et leurs répercussions sur les contraintes admissibles dans les ponts à poutres en fonte

G. R. Mitchell B. Sc.

Introduction

The purpose of the following notes is to amplify certain aspects of the work on cast-iron bridges already referred to by Dr. Davey and to indicate recent trends in the method of approach to problems of impact, fatigue and permissible stresses in bridge girders.

Dynamic Stress Measurements

The Ministry of Transport Standard Loading for Highway Bridges includes an allowance for impact equivalent to a 50% increase in the nominal static load. A similar impact allowance has in the past been used in this country when estimating the load-carrying capacity of existing bridges of less than 75 ft. (22,8 m.) span. In other words the impact factor, defined as the ratio of the peak value of the tensile strain produced by a moving vehicle in the bottom flange of a girder at mid-span to the peak value produced at the same point by the same vehicle when stationary, has in both cases been assumed to be 1.5.

In order to test the validity of this assumption in the particular case of cast-iron girder bridges a series of tests was made on a number of bridges using a variety of loading vehicles of both tracked and wheeled types. A vibrating wire type of gauge,[1]) firmly clamped to the underside of one of the girders at the mid-span position, was used for the measurement of dynamic strains. The electrical oscillations produced electro-magnetically in the strain gauge circuit by the mechanical vibrations of the wire stretched between the knife edges of the gauge itself were combined with similar oscillations produced by a "standard" wire of a fixed, lower, frequency to give electrical beats with a frequency equal to the difference between the frequencies of the two wires. Thus any change in the frequency of the wire in the "test" gauge due to changing strain in the girder caused a numerically equal change in the frequency of the beats. A continuous record of the changing beat frequency as a vehicle crossed the bridge was made by applying the nett voltage in the strain gauge circuit to the plates of a cathode ray oscilloscope and photographing the resulting movement of the spot on a strip of sensitised paper in a moving film camera. The record so obtained was in the form of a series of waves, each wave being the envelope of the traces produced by the high frequency oscillations, the distance between successive waves being inversely proportional to the beat frequency and therefore inversely proportional to the strain at the time.

For timing purposes arrangements were made to superimpose on the record a second trace which was controlled at a constant frequency by an alternator. The amplitude of this second trace could be reduced to zero when the loading vehicle passed any one of a number of points on the bridge; this was done by arranging for the vehicle to interrupt the light falling on a photo-electric cell at each such point as the vehicle passed that point, thus operating a relay and disconnecting the timing voltage. In this way the actual beat frequency and the position and speed of the vehicle at any time could be determined.

Measurements of the static strains when the loading vehicle was stationary, or nearly so, at the position for maximum strain at the point of observation were made by a similar method and the impact factors were calculated from the ratio of the dynamic to the static strains.

The results showed considerable variation for tests which were apparently similar in all respects. However, when a frequency distribution diagram for

[1]) R. S. Jerrett: "The Acoustic Strain Gauge", Journal of Scientific Instruments Vol. 22 No. 2 February 1945.

the impact factor was plotted (see Fig. 1) using all the observed values (114) it was found that the impact factors were approximately normally distributed with a mode of about 1.0 and with a range of about 0.6 i.e. the most frequently occurring value was about 1.0 and the maximum and minimum observed values were about 1.3 and 0.7.

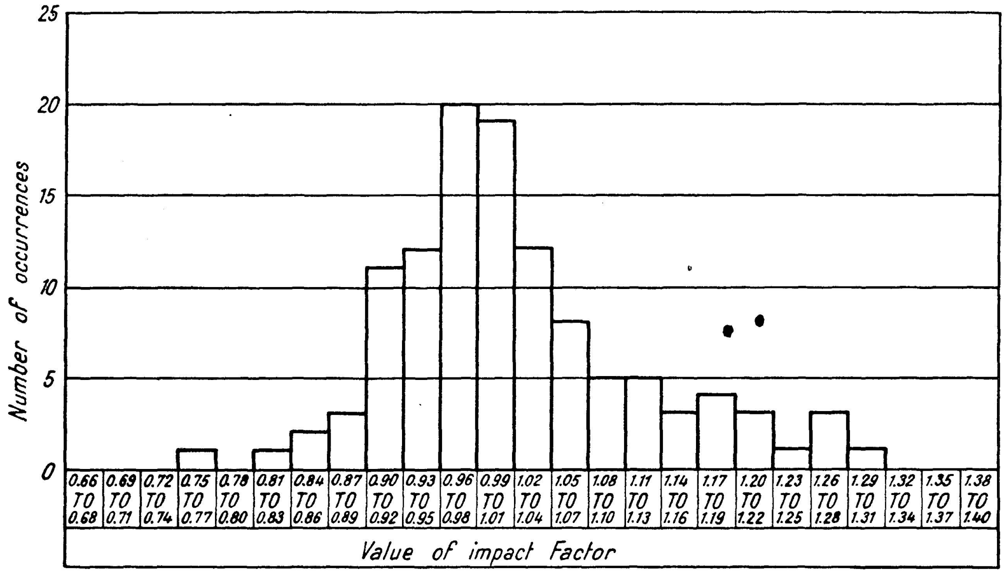

Fig. 1.
Distribution of Impact Factor.

Whilst it is true that no attempt was made during the above tests to provide obstructions for the wheels or tracks of the vehicles to over-ride and thus cause impact due to shock on hitting, or dropping from, the obstruction, nevertheless there were a number of tests made with a solid-tyred vehicle on one bridge where there was a sunken trench near the mid-span line giving rise to a one inch (2.5 cm) depression in the road surface and the results of these tests are included in the figures given above.

It is interesting to note that in no case was an impact factor as high as 1.5 observed.

The existence of fractional values of the impact factor (which had been indicated by deflection measurements during earlier tests) is of considerable significance. It is believed that the approximately normal distribution about a mean of about 1.0 is probably the result of the vehicles lurching or bouncing on their springs, which would, of course, produce the effects observed. If this is the case it would be expected that the impact factors would show greater variance about a constant mean of 1.0 as the speed of the vehicle is increased. There may, however, be other effects which play a part in any given case. For instance, in the case of bridges with longitudinal cambor there might be a progressive shift of the mean towards lower values as the speed increases, this being due to the vertical curvature effect which tends to reduce the apparent weight of the vehicle. On the other hand there may be actual shock effects due to pot-holes or surface irregularities on certain bridges and these will be superimposed on the effect due to lurching and cause an increase in the value and frequency of occurence of the higher values of the impact factor. (Indeed this may account for a slight skewness which is noticeable in the distribution of the impact factors already observed).

In order to provide further information about all these points it is intended to carry out a further series of tests, much greater in number, using resistance-type strain guages with an oscilloscope and moving film camera or other means of obtaining a continuous record of strain during dynamic loading. Opportunity will probably be taken to observe the effect of obstructions and also to compare the factors obtained on a poorly surfaced bridge with those obtained after re-surfacing with a smooth road carpet.

There is reason to believe that the results of this work on impact factors will have a general application to modern bridges of about the same spans and deck thicknesses in steel and concrete construction.

Permissible Stresses

In the past the permisssible tensile stress in cast-iron bridge beams has in this country been 2.5 tons/sq. in. (394 kg/sq. cm). Recently, however, there has been a growing realisation of the unsatisfactory nature of a permissible stress which, in effect, applies the same factor of safety to dead load stresses as to live load stresses. This has been a particularly important point in the case of cast-iron girder bridges where the dead load stress forms a higher proportion of the total stress in a girder. For example, a recent survey of 31 such bridges showed an average dead load stress of 1.84 tons/sq. in. (290 kg/sq. cm) which is nearly three-quarters of the total permissible stress of 2.5 tons/sq. in. (394 kg/sq. cm). It was therefore natural that a demand should arise for the permissible stress to be based on different factors of safety for the two types of stress, depending on the possible variations in the loadings causing them. At the same time it has been felt that some attention should be paid to the question of fatigue failure in view of the increasing frequency of traffic and the heavier loads now permitted to use these old cast-iron girder bridges.

These problems have received some attention at the Building Research Station and by a fundamental approach from the point of view of fatigue it has been found possible to suggest a formula for the permissible live load stress in terms of the dead load stress and the modulus of rupture of the beams.

The condition of stress in the lower flange of a bridge beam is that of a steady dead load stress together with a live load stress which alternates between zero and a maximum a large number of times during the life of the bridge. This is, of course, a condition in which fatigue is an important consideration. Whilst it is true that many of the loadings applied to the bridge will be lower, and some will be higher, than the design loadings or, in the case of existing bridges, the loading for which it is intended to restrict the bridge, nevertheless it appears desirable to assume that the live load will in fact be equal to the design load and will be applied an infinite number of times. In these circumstances the stress due to dead load can be considered as the minimum stress of the fatigue cycle and the stress due to dead plus design live load as the maximum stress of that cycle.

The modified Johnson-Goodman formula proposed by Moore and Kommors[2]) for use in such cases of alternating stress is

$$F \text{ max.} = \frac{3}{2-r} \text{ x the endurance limit for complete reversals of stress.}$$

Where F max. is the maximum stress of the cycle and r is the ratio of minimum to maximum stress. The endurance limit, which is, of course, the maximum stress which can be applied and then reversed in sign an infinite number of times (say $5 \cdot 10^6$) without fatigue failure occurring, would normally be found by experiment. However, consideration of the many tests by other investigators leads to the conclusion that for a cast ferrous metal the endurance limit is not less than one third (and normally 0.4 to 0.6) of the ultimate tensile strength. Moreover, comparative tests have shown that for cast-iron beams of the broad flange I or $\perp$ section which are the normal types found in this country the ultimate tensile strength of the material can with safety be replaced by the modulus of rupture of the beam itself. The average modulus of rupture found from full scale tests on a number of cast-iron beams taken from bridges was 8 tons/sq. in. (1260 kg/sq. cm).

Making use of these facts the above relationship can be reduced to the form

$$2\,F_l + F_d = \text{modulus of rupture} = 8$$

where F_l is the live load stress and F_d the dead load stress in tons/sq. in.

In this formula the stresses are those which actually exist in the girder and the modulus of rupture is that for the actual girder for which the calculation is being made. In practice, of course, factors of safety must be used to allow for (a) errors in the estimation of the dead load, (*b*) errors in the estimation of the live load (including impact effects), and (*c*) the presence in the girder of flaws and material of inferior quality. Except in the case of impact there is little factual information to hand as to variations in these quantities and it becomes necessary to use a certain amount of judgement based on experience. It is not proposed to give detailed reasons here for the adoption of the particular factors which have in fact been used in preparing the proposed formula. It may however be stated that the impact allowance adopted

[2]) H. F. MOORE and J. B. KOMMERS: "The Fatigue of Metals", Mc Graw Hill 1927.

was 10%. From the point of view of fatigue, of course, this implies that all the assumed infinite number of applied loadings will have an impact factor of 1.1. In practice the experimental work referred to in the earlier part of this paper showed that about 80% of the impact factors were less than this.

The formula ultimately adopted was

$$5.5\, f_l + 2.2\, f_d = 8$$

where f_l and f_d are the live and dead load stresses in tons/sq. in. calculated by the ,,$\frac{D}{d}$-method'' referred to, in the first paper of this series, by Dr. Davey.

It should be noted that there are several reasons why the formula given may be considered to be conservative. In the first place it has been assumed that the beams will have to withstand an infinite number of loadings whereas the number of alternations of the permissible stress will often be less than

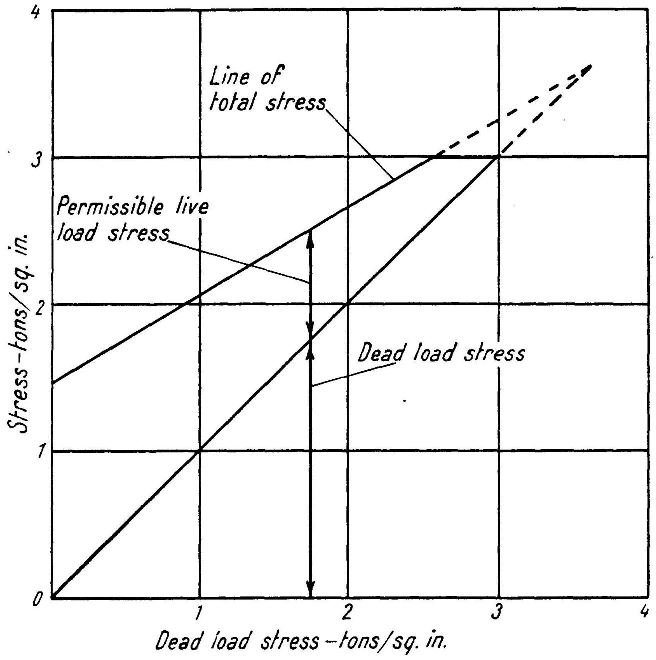

Fig. 2.
Relationship between Permissible Live Load Stress and Dead Load Stress
$5{,}5\, f_l$ — $2{,}2\, f_d$ — 8 Tons Sq. In.

5 x 10^6 with a large consequent increase in the maximum stress which could have been permitted. In the second place the Moore and Kommers formula and the additional assumptions which have been made regarding the endurance limit and the ultimate tensile strength of the material are all safe limiting relationships and give in general conservative estimates of fatigue strength. In the third place it has been assumed that all the errors requiring stress factors to cover them will occur simultaneously, which is improbable. Finally,

the „$\frac{D}{d}$-method'' of calculation of live load stresses is known to be conservative; in fact recent tests with 18 to 70 ton loadings showed an average value of $2\frac{1}{2}$ for the ratio of calculated to observed stress.

Arrangements are in hand to make fatigue tests on a number of full sized cast-iron girders taken from old bridges in order to obtain additional information about their behaviour under fatigue conditions. Meanwhile the above formula has been accepted by the Ministry of Transport and the Railway Executive for use in assessing the load bearing capacity of bridges of cast-iron girder and brick jack arch conctruction provided that the total permissible stress does not exceed 3 tons per sq. in. Figure 2 is a diagrammatic representation of the formula.

Summary

The first part of the paper describes experiments to measure dynamic stresses in the girders of old cast-iron bridges using both tracked and wheeled vehicles for loading purposes. The results of these experiments indicate that there is a range of values of the impact factor, defined as the ratio of dynamic to static stress for the same vehicle, from about 0.7 to about 1.3 with a most frequently occurring value of about 1.0. The implications of these results are discussed and an indication of further work to be carried out is given.

The second part of the paper deals with the problem of fatigue in bridges of this type and also, briefly, with the question of different factors of safety for dead and live load stresses. The stress in a bridge girder is considered as a steady dead load stress together with a live load stress which varies from zero to a maximum a very large number of times during the life of the bridge. It is shown that, by looking at these stresses from the point of view of fatigue, the permissible total stress should vary according to the value of the dead load stress. Making use of suitable factors of safety, a formula for the permissible live load stress in cast-iron bridge girders is given.

Zusammenfassung

Der erste Teil des Beitrages beschreibt Untersuchungen über die Spannungen in alten Balkenbrücken aus Gußeisen, mit gezogenen und Selbstfahrzeugen als Belastungen. Die Ergebnisse zeigen, daß der Stoßfaktor, der als das Verhältnis von dynamischer zu statischer Beanspruchung definiert wird, für das gleiche Fahrzeug von 0,7 bis 1,3 variiert, mit dem Mittelwert 1,0. Die Folgerungen aus diesen Resultaten werden diskutiert und einige Hinweise für weitere Forschungen gegeben.

Der zweite Teil des Beitrages befaßt sich mit Ermüdungserscheinungen in Brücken dieser Art und ganz kurz noch mit der Frage des Sicherheitsgrades für statische und dynamische Lasten. Die Beanspruchung in einem Brückenträger wird hervorgerufen durch eine stetige Belastung, der eine Beanspruchung überlagert ist, die von Null bis zu einem bestimmten Maximalwert während seiner Lebensdauer viele Male variiert. Es wird gezeigt, daß vom Standpunkt der Ermüdung betrachtet, die zulässigen Hauptspannungen variieren sollten, gemäß der Größe der statischen Beanspruchung. Eine Formel für die zulässigen Spannungen in Trägern aus Gußeisen wird gegeben, wobei angemessene Sicherheitsfaktoren berücksichtigt wurden.

Résumé

La première partie de ce mémoire expose des investigations portant sur les contraintes qui se manifestent dans les vieux ponts à poutres en fonte, avec charges remorquées et charges automotrices. Les résultats obtenus montrent que le coefficient de choc, qui est défini comme le rapport entre la contrainte dynamique et la contrainte statique, varie pour un même véhicule de 0,7 à 1,3 avec valeur moyenne de 1,0. L'auteur discute les conséquences qui en résultent et donne certaines indications pouvant guider les recherches ultérieures.

La deuxième partie du mémoire porte sur les phénomènes de fatigue qui se manifestent dans les ponts de ce type et, d'une manière très brève, sur la question des coefficients de sécurité pour les charges statiques et dynamiques. Dans une poutre de pont, les contraintes sont dues à la superposition d'une charge fixe et d'une charge qui varie en passant à plusieurs reprises par une valeur maximum, durant l'existence du pont. L'auteur montre que, du point de vue de la fatigue, les contraintes principales admissibles doivent varier suivant la grandeur de la contrainte statique.

Il donne enfin une formule concernant les contraintes admissibles dans les poutres en fonte, formule établie en tenant compte de coefficients de sécurité appropriés.