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Studies on the Resistant Behaviour of Steel Deck Plates for Bridges

Recherches sur le comportement à la ruine des platelages métalliques pour ponts

Studien über das Tragverhalten von Stahlblechfahrbahnen für Brücken

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1. Introduction

Papers on experimental and theoretical studies on steel deck plates for bridge structures have been presented by many scientists and engineers [1, 2, 3, 4, 5]. These papers mainly discuss the methods of analysis within the elastic limit of the material and the design methods for steel deck plates. Steel deck plates, just as other plate structures, show the considerable reserve of load-carrying capacity which exists when these structures have been loaded beyond the elastic limit. In recent years, advantage of the reserve of strength in the steel deck plate of bridges was taken by Prof. KLÖPPEL [6]. There are also some investigations on ship plates loaded beyond the elastic limit of the material [7, 8].

This paper reports the results of experimental studies on the resistant behaviour of steel deck plates for bridges and also discusses these results.

A model bridge, 4,920 mm span length and 2,000 mm in width (as shown in Fig. 1), made of ordinary structural steel, was used for the experimental studies, and the experiments were conducted on the Kyoto University Structural Testing Machine.

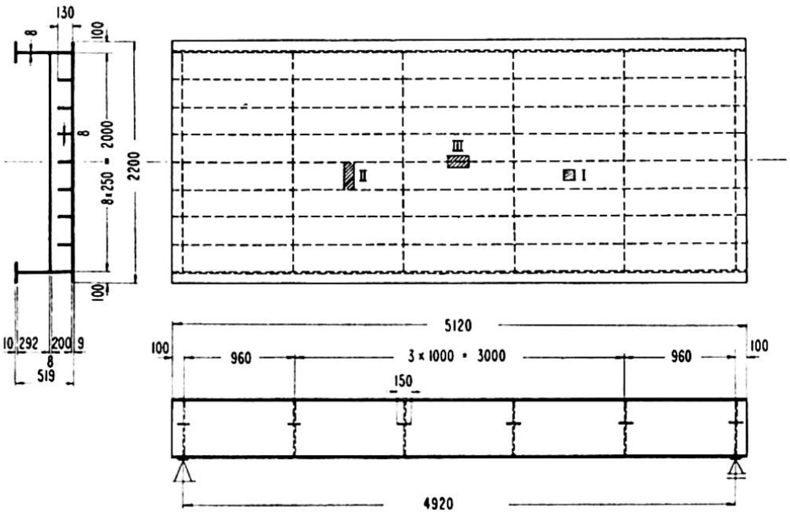

Fig. 1. Model bridge.

Prior to the experiments beyond the elastic limit, experimental studies within the elastic limit had been carried out. Detailed descriptions of the experiments within the elastic limit will be omitted in this paper, and the main results obtained are summarised as follows.

1.1. When a distributed load is applied at the center of a stringer, the resulting stresses extend over three spans of stringers in the direction of the bridge axis and over four stringers in the perpendicular direction.

1.2. When a load is applied directly on a cover plate, stresses and deformations of the plate are restricted to the span under the load in the direction of the bridge axis and to three spans of the plate supported by the stringers.

1.3. Cover plate and stringer stresses due to beam and plate actions may be approximately superposed.

1.4. Load distribution by means of the asphalt pavement is not so effective as the value given in the design specifications issued in Japan.

The resistant behaviour beyond the elastic limit of steel deck plates will be discussed using experimental results obtained with a model bridge as follows.

2. Experimental Method

The load was applied by rectangular loading plates of three different sizes. For the sake of convenience the loading plates are designated as Loading Plate *A*, *B*, and *C*. A wood block and a rubber plate were used in layers as a loading plate, and the rubber plate serves for load distribution. Dial gauges were used for the measurement of the deflections, and the strains were measured by SR-4 type strain gauges.

Three experiments were performed for the different loading positions and loading plates, and the loading position for each experiment is given by the hatched area in Fig. 1. The sizes of the loading plates used are:

For the experiment for Case I Plate *A* (100×100 mm^2),
Case II Plate *B* (250×100 mm^2),
Case III Plate *C* (200×125 mm^2).

For Cases I and II, the load was applied directly on a cover plate, and for Case III the load was applied on a rib. The resistant behaviour of the plate and the rib were mainly investigated by these loadings.

3. Experimental Results and Their Interpretation

3.1. Results and Discussion of the Tests (Case I and Case II)

The results of the tests (Case I) are summarised in Figs. 2 and 3. Fig. 2 shows the load-deflection relation obtained under the applied load, and these deflections were measured from the rib lines. Fig. 3 shows the load-strain relation obtained by SR-4 type strain gauges under the applied load in both the bridge axis and the perpendicular direction. The results of Case II are not given in this paper owing to lack of space.

The three different stages in the deflection of the loaded plate are given in Fig. 2. The plate is considered to behave elastically until a plastic line develops at a section of the plate. Yielding of the plate is considered to occur when

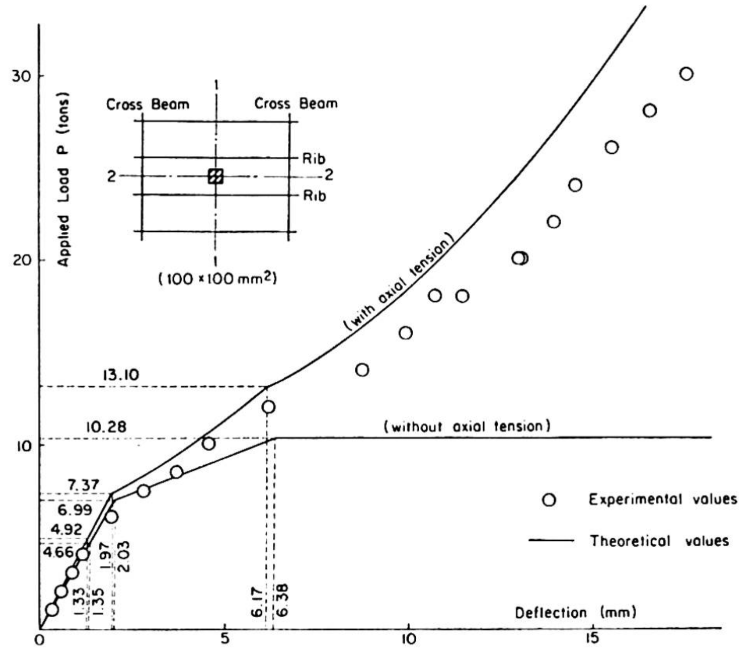

Fig. 2. Deflections of cover plate.

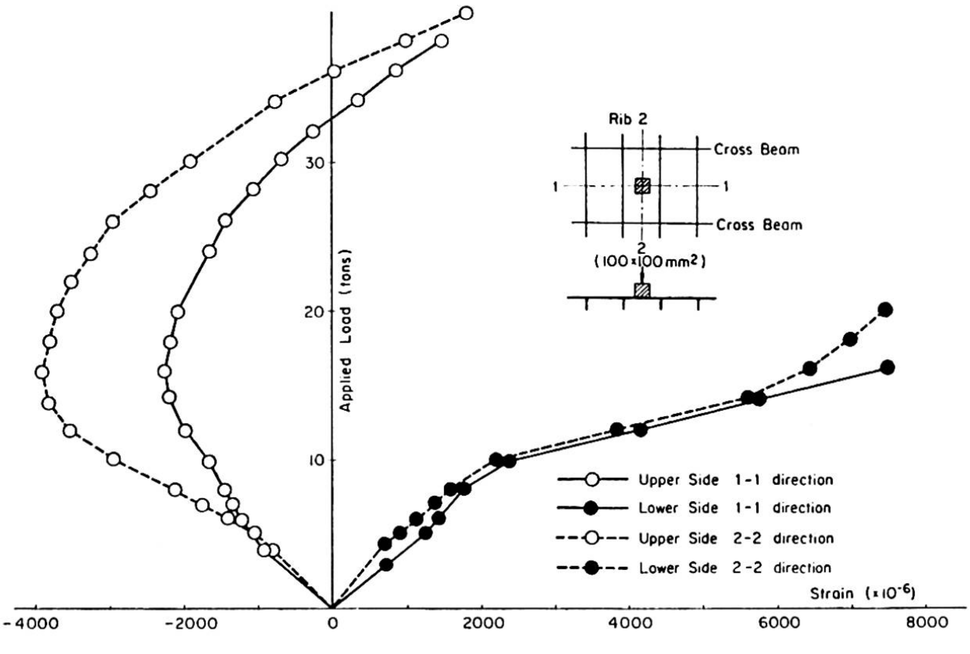

Fig. 3. Strains of cover plate.

the applied load amounts to about 4 tons. The 2nd stage is the yielding stage, and the 3rd stage is the hardening stage. In the 2nd stage, the effects of yielding, due to bending of the plate, are predominant, and large plastic deformations take place, and in the 3rd stage, membrane tension exerts predominant effects.

The first residual deflection occured for the applied load $P=4$ tons, and the corresponding deflection was 1.15 mm. This corresponds to 1/218 the rib distance ($b=250$ mm) and to 0.24 times the thickness of the plate ($h=9$ mm). The theory of elastic plates with medium thickness may be applied within this limit.

The effects of axial tension can also be explained by the load-strain relation shown in Fig. 3. Due to stretching of the middle plane the strain of the upper side of the cover plate changes its directions at a certain amount of loading, and thereafter the strains of both surfaces increase almost in parallel. The strain gauges on the upper surface were attached by making small holes in the loading plates.

No cracks in the steel plate were found after the maximum load of 40 tons was applied. For Case II the load was increased to 52 tons without any crack.

3.2. Results and Considerations on the Experiment (Case III)

Figs. 4 and 5 show the results obtained in the experiment for Case III. In this case the load was applied directly on the rib. Fig. 4 shows the load-deflection relation of the rib under the load, and Fig. 5 the corresponding load-strain relation. The positions of the strain gauges are given in the figure.

The first residual deformation occurs for $P=13$ tons, and the corresponding deflection is 2.30 mm at the centre of the rib. Stresses in the rib increase rapidly after the yielding of the rib. The rib itself shows no reserve of strength after the stresses in a section of the rib reach the yield stress as shown in Fig. 5.

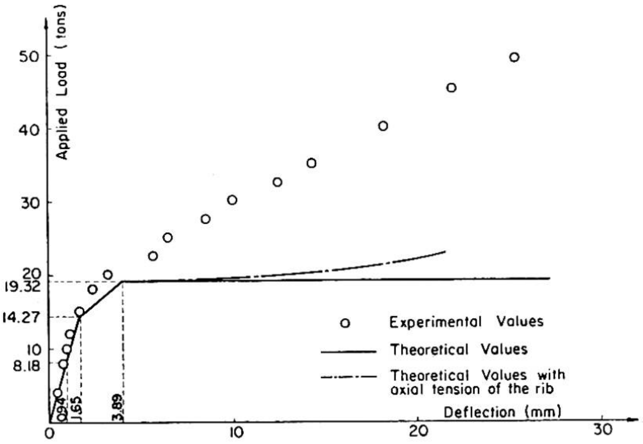

Fig. 4. Deflection of the rib.

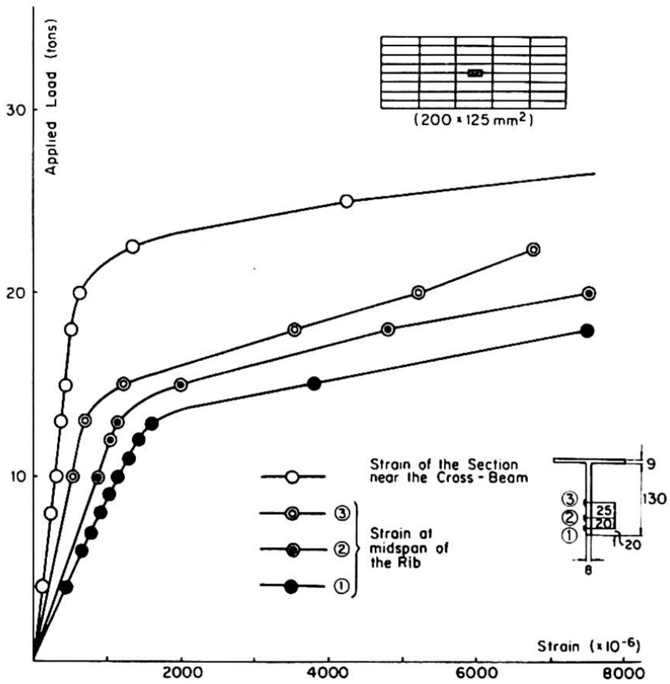

Fig. 5. Load-strain relation of the rib.

No noticeable damage was observed until the applied load reached $P = 60$ tons which is five times as great as the yield load.

Residual deformations of adjacent ribs started at the load $P = 20$ tons, but a considerable increase of deflection, such as that in the centre rib, was not measured. After the yielding of the centre rib, the ribs of adjacent sides play an important part in the carrying capacity of the deck plate.

Strain distributions at the section of the rib due to the applied load are shown in Fig. 6. The position of the neutral axis rose with the plastification and increase in the axial tension of the rib.

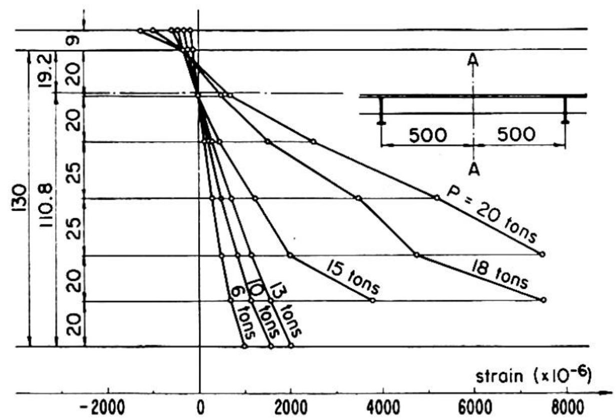

Fig. 6. Strain distribution of the rib.

At $P = 20$ tons the rib under the applied load buckled at the section near the cross beam subjected to negative bending moment, and the corresponding stress at the buckled section is 1,030 kg/cm².

4. Theoretical Investigations

4.1. Assumptions Made

In the theoretical investigation, the behaviour of the structural part within the broken lines in Fig. 7 is taken into consideration in order to investigate the resistant behaviour of the steel deck plate.

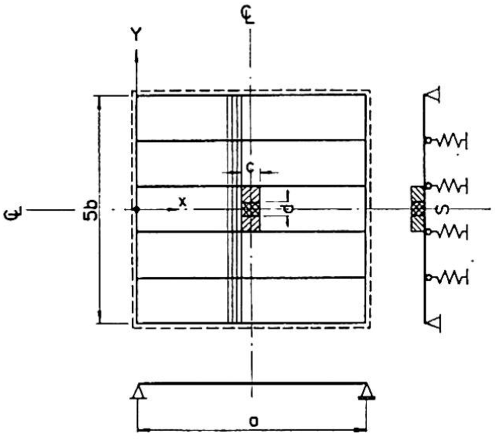

Fig. 7.

The cover plate within this area is assumed to be divided into narrow strips in the direction perpendicular to the bridge axis, and the elasto-plastic analysis of beams is applied to each of these divided strips.

The uniformly distributed load on the plate is converted into an equivalent load supported by each of these strips, and it is determined by equating the deflections of the strips and the plate. The strips so defined are assumed to be elastically supported by four intermediate ribs and to be simply supported by the edge ribs as shown in Fig. 7, and the original plate is assumed to be a five-span continuous plate with six rigid supports. The distribution of intensity of the equivalent load is considered to be the same in all stages.

The theoretical investigations are limited to one of the strips, the centre strip, subjected to the equivalent applied load. Referring to the experimental results, the behaviour of the beam as defined is considered in three stages. In the first stage, the beam is considered to behave as wholly elastic until a yield stress develops at the centre of the beam. The stage of plastification until a plastic hinge develops at the centre of the beam is also considered in the first stage.

In the second stage, the plastic hinge is formed at the centre and it continues until additional hinges develop at the supporting point of the center span of the beam. If the rigidity of the ribs is comparatively high, and the uniform load is distributed over a wide range of the beam, plastic hinges at the supporting points will develop in the first stage, but this is not a case to be taken into consideration in this investigation.

In the third stage, the plastic hinges are formed at three sections, and it corresponds to a mechanism condition of plastic analysis free from the axial force.

Two kinds of investigation are made, one is an analysis in which the effects of axial tension are disregarded, and the other is an analysis in which the effects of axial tension are taken into consideration.

The effects of shearing deformation and of Poisson's ratio are disregarded in this analysis. The plate was assumed to behave as an elastic structure, except the sections of the plastic hinges. The effects of beam action as a whole structure are omitted in this analysis.

In what follows, the results of the experiment for Case I are mainly discussed.

4.2. Behaviour in the First Stage

In this stage, axial tension has little effect on the deflections of the plate, and the effects of the applied load are considered to extend over five spans as shown in Fig. 7.

The first yield stress at the centre of the plate occurs at the computed load of 4.66 tons, and the corresponding deflection is computed to be 1.35 mm. If the axial tension is considered in these cases, a slightly larger value for the load and almost the same value for the deflection are obtained as shown in Fig. 2.

The first plastic hinge is formed at the applied load of 6.99 tons and the corresponding computed deflection is 2.03 mm. If axial tension is taken into consideration in the analysis the applied load which develops the plastic hinge is 5% greater than in the case free from the axial force.

In the analysis of the first stage, the load-deflection relations are considered as straight lines, except the effects of the axial tension.

4.3. Behaviour in the Second Stage

In this stage the axial tension has greater effects than in the first stage, and there are considerable differences between the computed value obtained by the analysis in the absence of axial tension and that obtained when the axial tension is taken into consideration.

At the end of the second stage, the computed applied load when the axial tension is taken into consideration is almost 27% greater than that free from the axial tension. The experimental results are almost in the middle of the space between these lines shown in Fig. 2.

4.4. Behaviour in the Third Stage

After the plastic hinges are formed at the centre and the supporting points of the centre span where the load is applied, the analysis can be limited to the centre span of the beam.

In this stage the axial tension is very important. If the axial tension is disregarded in this stage, since a mechanism has been formed by the development of plastic hinges no further reserve of strength in the structure is to be expected. If the axial tension is considered in the analysis, it shows a remarkable increase in strength which continues until the axial tension amounts to pure yielding. When the pure plastic axial tension acts in the middle plane, the plastic moments become zero. The structure will still show an increase in strength after this point, due to the geometric change of the middle plane. Although the effect of stress hardening is omitted from the analysis, it also has considerable influence on the increase in strength after the plastic axial tension occurs.

The computed plastic axial tension occurs at 46.3 tons. Although the experiment was not carried up to this point, the structure still had a considerable reserve of strength when the test was finished.

In the second and third stages, the experimental results are a little below the theoretical results when the axial tension is taken into consideration. This shows that we obtain with the theoretical results a greater strength than with the experimental results. The experimental results and the theoretical results for Case II also show the same tendency, although detailed explanation of these results are omitted from this paper. In proper assumptions regarding the equivalent load and the stiffness of the edge ribs in the second and third stages are considered to be the main reasons for these discrepancies. Consequently the equivalent loads are to be regarded as more intense than is assumed in this investigation.

4.5. Investigation of the Behaviour of the Rib

The theoretical results obtained by considering the rib as a beam are given in Fig. 4. Depending upon the stiffness in the perpendicular direction to the ribs, experimental results show considerably greater strength than do the theoretical results. Even if the axial tension in the rib is taken into consideration, there is a slight increase in strength, as shown in Fig. 4.

5. Conclusion

Steel deck plate subjected to increasing loads undergoes three different types of deformation. These are, (1) elastic deformation in conformity with ordinary plate theory, (2) plastic deformation due to yielding of the material, and (3) membrane deformation due to elastic and plastic stretching of the middle plane.

The behaviour of steel deck plates under these categories was investigated experimentally by a model bridge, and some theoretical investigations were undertaken on the experimental results.

The large load-carrying capacity of the deck plate, which was several times greater than the capacity assuming elastic behaviour, was confirmed by experimental and theoretical studies.

In the analysis of the deck plate, the theoretical results obtained in this study indicate a tendency towards good agreement with the experimental results obtained. The values obtained from the theoretical investigations, however, showed a slightly greater strength than the experimental results. The axial tension is very important for the analysis of ultimate carrying capacity of the cover plate, and it must be taken into consideration in the theoretical investigation.

In the analysis of the ribs, the effects of the adjacent ribs and of the axial tension of the plate must be taken into consideration.

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Summary

This paper deals with experimental investigations and also some theoretical considerations regarding the resistant behaviour of steel deck plates for bridge structures. The experiments were carried out beyond the elastic limit of the material, and the ultimate behaviour of the steel deck plates was investigated. These studies show that the axial tension in the middle plane of the cover plate participates substantially in the load carrying capacity of the steel deck plates, and therefore the high ultimate strength must be given proper consideration in the design of steel deck plates for bridge structures.

Résumé

Les auteurs présentent des recherches théoriques et des essais concernant le comportement des platelages métalliques de ponts. Les essais ont été poursuivis au delà de la limite élastique de l'acier et on a étudié le comportement à la ruine de ces platelages. Les recherches montrent que les contraintes longitudinales agissant dans le plan moyen de la tôle de couverture contribuent largement à la résistance du platelage et que, en dimensionnant les platelages métalliques, il faut tenir compte de leur résistance à la ruine élevée.

Zusammenfassung

Die Arbeit berichtet über Versuche und theoretische Betrachtungen zur Ermittlung des Traglastverhaltens von orthotropen Platten aus Stahl für Brückenfahrbahnen. Die Untersuchungen wurden über die Elastizitätsgrenze des Baustahls hinaus erstreckt, um die tatsächliche Tragfähigkeit zu ermitteln. Die Ergebnisse zeigen, daß die Normalspannung in der Plattenmittelfläche einen großen Einfluß auf die Tragkraft der orthotropen Platte besitzt und daher bei der Berechnung berücksichtigt werden muß.