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Discussion of the Paper "Composite Steel-Deck-Reinforced Concrete Systems Failing in Shear-Bond" by R.M. Schuster

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Diskussion des Beitrages "Composite Steel-Deck-Reinforced Concrete Systems Failing in Shear-Bond" von R.M. Schuster

R.P. JOHNSON
England

Static and fatigue tests on steel-deck-reinforced concrete slabs similar to those reported by R.M. Schuster were carried out at Cambridge University, England, in 1970-71. It was found that ultimate strength of the composite slabs was strongly influenced by two variables not discussed in the present paper.

The first is the detail at the edge of the slab. If the outermost corrugation of the decking can separate from the slab due to transverse bending of the deck, then the longitudinal shear strength is less than when the free edge of the decking is embedded in the slab in some way. For example, the edge detail shown in Fig. 1 of the paper would perform better than that shown in Fig. 3. The effect is of course more significant in narrow laboratory specimens than it is in practice.

The second variable is the roughness of the dimples or corrugations provided in the inclined sides of the sheets. Measurements on sheets provided by one of the same suppliers that R.M. Schuster used showed that the dimple depths ranged from 0.6 mm to 1.1 mm. The mean depths for individual sheets were more uniform (0.79 to 0.97 mm), but it is likely that the variation is sufficient to influence the shear strength of composite plates in which these sheets are used.

IIa

Discussion of the paper "Continuous Composite Beams for Bridges" by J.W. Fisher

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R.P. JOHNSON
England

The information provided by J.W. Fisher and his co-authors on the stiffness of the negative moment region of a continuous beam is most welcome. The existing British design method for continuous composite bridge beams (CP 117 : Part 2) distinguishes clearly between assumptions to be made about stiffness, for the purpose of calculating longitudinal moments, and assumptions to be made about the stress distribution at a cross-section.

It is not clear in the present paper whether the conclusion that C_1 can be taken as 0.6 refers only to equivalent stiffness, from which bending moments can be deduced, or to the calculation of reinforcement-bar stress also. It would seem logical to calculate this stress neglecting concrete in tension altogether, in order to satisfy the equilibrium condition at a cross-section where there is a crack.

The authors show that stiffness decreases as load increases, as one would expect. Has it been established whether stiffness in service depends on the current load level or on the maximum load level previously reached? If the latter, which seems the more likely, then one would presumably base fatigue calculations on the moment distribution found using stiffnesses corresponding to maximum working load. If that load were never reached, the moments and hence the loads on connectors in the negative moment region would be higher than calculated, which could reduce fatigue life. Another problem is shrinkage. Is anything known about its influence on stiffness of negative moment regions? It is possible that predictions of loads on connectors and stresses in reinforcement in service may still be quite inaccurate.

Two Unconventional Bridges in Praha, Czechoslovakia

Deux ponts non conventionnels à Prague, Tchécoslovaquie

Zwei unkonventionelle Brücken in Prag, Tschechoslowakei

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Two unconventional bridges there have been developed on the Technical University of Praha and now are under construction in Praha /Czechoslovakia/.

The first one is the railroad bridge shown on fig.1.

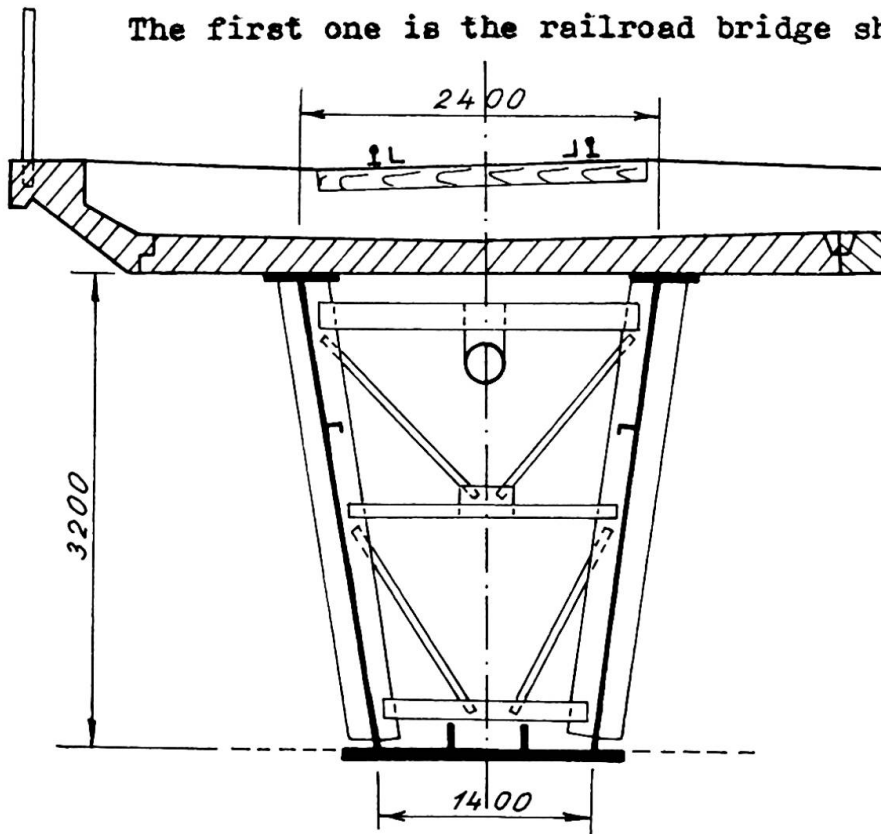


Fig.1.

The superstructure is designed in form of the steel box girder with trapezoidal cross section, connected with precast concrete deck.

The composite action of the deck and steel box girder is achieved by high strength bolts M 24. The composite structure of this type is for the first time used for heavy

railroad loading.

This bridge was developed as a standard structure. Total number of 12 such bridges will be erected in the railroad network of Praha, until now 5 of them are in operation.

The span is 46.5 m, the depth of the steel box is 3200 mm. Used steel is St 52, strength of the concrete is 500 kp/cm^2 . Each box girder is designed for one track loading. This standard section may be used for single and multi-track railroad bridges.

The second interesting bridge is the new steel bridge structure over the railroad station Praha - centre./Fig.2/.

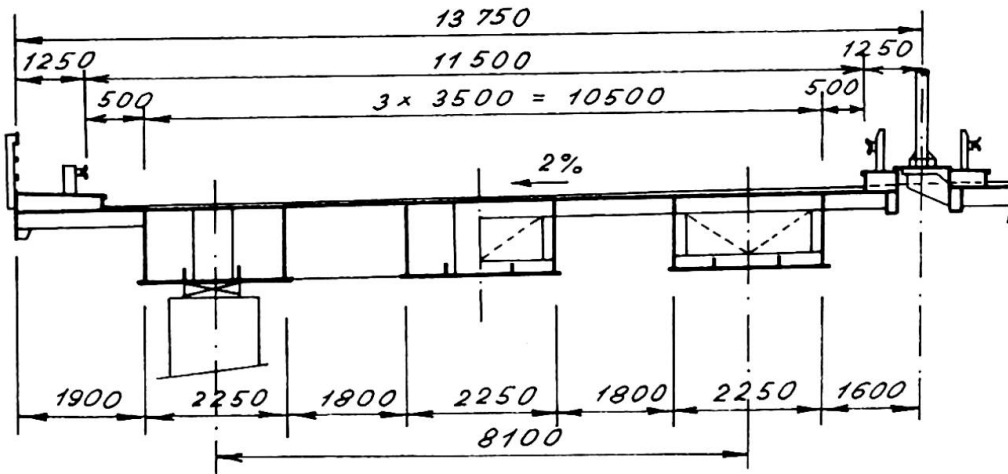


Fig. 2.

dividual bridge structures with total 48 spans and total length of 1600 m. The spans vary from 35 to 45 m.

The carriageway is supported by steel orthotropic plate

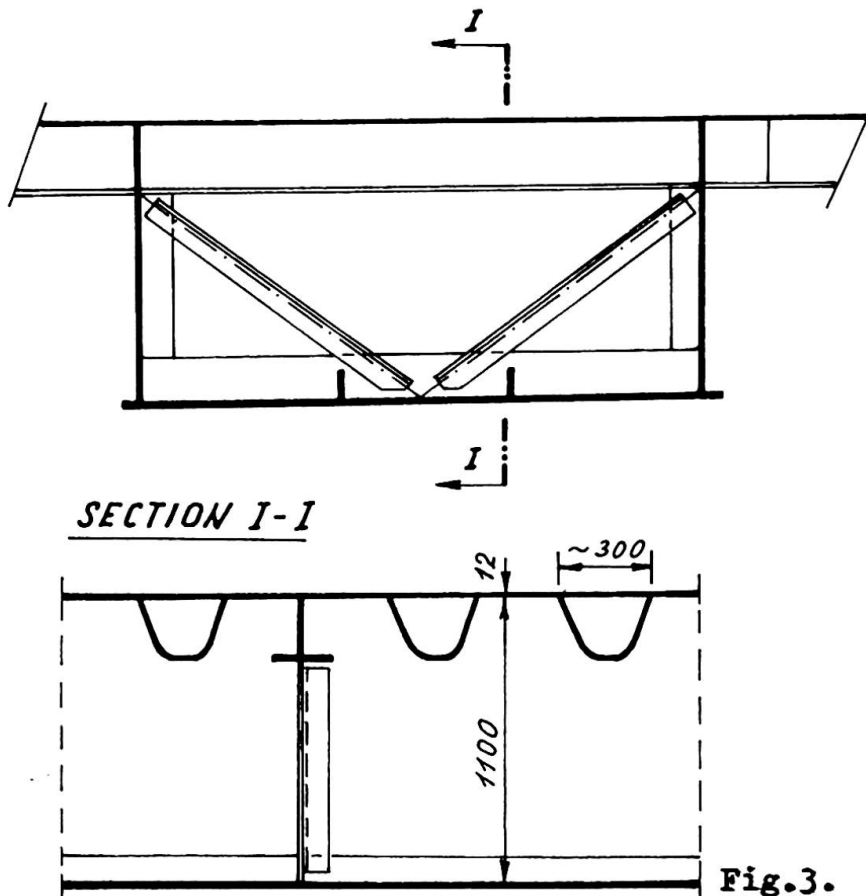


Fig. 3.

With its total length of approx. 700 m it spans also several main streets intersections in the downtown Praha.

The total bridge is composed from 18 in-

dividual bridge structures with total 48 spans and total length of 1600 m. The spans vary from 35 to 45 m. The carriageway is supported by steel orthotropic plate with transverse box stiffeners in trapezoidal shape /fig.3./. Spacing of transverse stiffeners is approximately 600 mm. They rest on webs of main box girders.

Transverse rigidity of the orthotropic deck is sufficient for reliable transverse load distribution. Rigid cross frames are designed at supports only. Additional diaphragms inside of boxes in the quarter and in the middle of each span resist the distortion of the box cross section.

The whole bridge is curved in horizontal plane.