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## **II**

**Interactions dans les structures  
Wechselwirkung in Tragwerken  
Interaction Problems in Structures**

### **II a**

**Interaction de matériaux différents  
Wechselwirkung zwischen verschiedenen Materialien  
Interaction of different Materials**

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## DISCUSSION PRÉPARÉE • VORBEREITETE DISKUSSION • PREPARED DISCUSSION

**Materialoptimierung für einfach symmetrische Biegeträger**

Optimization of the Material for simply symmetrical Bending Girders

Optimalisation du matériau pour poutres à flexion simplement symétriques

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Bei der Optimierung von Biegeträgern durch Veränderung der Stahlmarke gibt es zwei Wege:

1. Wahl eines Stahles höherer Festigkeit für den gesamten Querschnitt
2. Wahl eines Stahles höherer Festigkeit für Teile des Querschnittes.

Bei dem zweiten Weg ist es möglich,

- 2 a) den Querschnitt symmetrisch auszubilden, d.h., Obergurt und Untergurt aus einem höherfesten Stahl als das Stegblech auszubilden. Diese Möglichkeit wurde von Daddi - Mailand, beschrieben.

Interessante Ergebnisse liefert auch der Weg

- 2 b) unsymmetrische Ausbildung des Querschnittes durch Verwendung des höherfesten Stahles nur für den Untergurt.

Durch die Verwendung eines Stahles geringerer Festigkeit im Obergurt ergeben sich Vorteile im Stabilitätsverhalten sowohl hinsichtlich des örtlichen Ausbeulens des Druckgurtes, als auch des Kippverhaltens des gesamten Trägers. Dadurch lassen sich wirtschaftliche Lösungen erreichen, wenn eine Überschreitung der zulässigen Spannung im unteren Stegblechbereich in Kauf genommen wird, wie es im Bild 1 dargestellt ist.

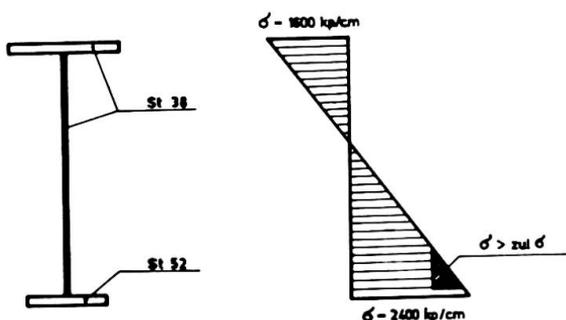


BILD 1

Zur Klärung des Tragverhaltens, insbesondere auch der Schubübertragung in dem bei diesem Träger schon im Gebrauchszustand teilweise plastifiziertem Stegblech sind Versuche an zwei geometrisch gleichen Trägern mit Obergurten aus St 38, Untergurten aus St 52 und Stegblechen bei einem Trä-

ger aus St 38, bei einem anderen aus St 52 durchgeführt worden. Die 6 m langen Versuchsträger waren gelenkig gelagert und durch zwei symmetrisch zur Mitte angeordnete hydraulische 20 Mp Prüfzylinder in 1,20 m Abstand belastet. Durch seitliche Führung an den Lasteintragungsstellen wurde das Kippen der Träger verhindert, so daß sie bis zur vollen Plastizierung belastet werden konnten.

Die Belastung erfolgte in mehreren Stufen mit vollständiger Zwischenentlastung. Mittels eines Theodoliten und aufgesetzter Maßstäbe an den

Auflagern, an den Lasteintragungsstellen und in Trägermitte sind die Durchbiegungen auf etwa 0,2 mm genau gemessen worden.

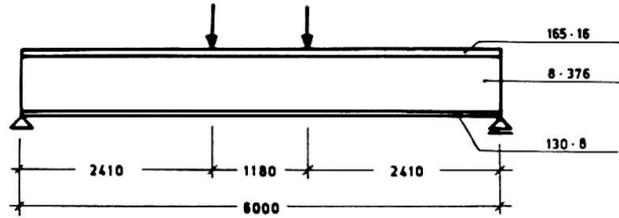


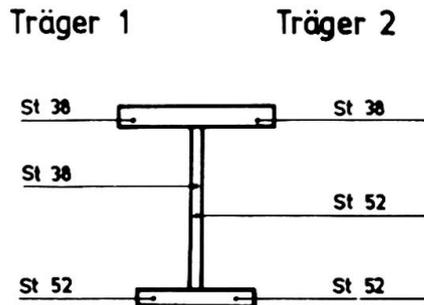
BILD 2

Die Belastung erfolgte in mehreren Stufen mit vollständiger Zwischenentlastung. Mittels eines Theodoliten und aufgesetzter Maßstäbe an den

Dehnungsmessungen mit einem mechanischen Setzdehnungsmesser mit 100 mm Basis erfolgten in einem Querschnitt in Trägermitte an je 3 Stellen beider Gurte und je 5 Stellen jeder Stegseite.

Beim Vergleich der Versuchsergebnisse kann man sich bekanntlich nicht auf die Normwerte der Festigkeiten verlassen, deshalb ist nach den Versuchen die Streckgrenze und Bruchfestigkeit an Proben aus den Gurten und Stegen beider Träger bestimmt worden. Die Proben sind aus Bereichen entnommen worden, die nicht plastisch verformt waren. Die Streckgrenze des St 38 lag über dem Normwert, die des Stegbleches St 52 knapp darunter.

Das elastische Verhalten der Träger, festgestellt aus der Durchbiegungsänderung bei der Entlastung, stimmte gut mit der Theorie überein. Bei Belastung war schon bei niedrigeren Laststufen die wirkliche Durchbiegung größer als



	Träger 1	Träger 2
$M_{PLAST}$ [Mp]	27,7	29,2
$M_{TRAG}$ [Mp]	32,3	34,1
Durchbiegung [mm]		
P 10,0 [Mp]	36,7	37,4
P 12,2 [Mp]	67,6	39,3

BILD 3

die rechnerische, was auf örtliche Plastizierung infolge von Eigenspannungen zurückzuführen ist. Die Dehnungsmessungen bestätigten die Hypothese vom Ebenbleiben der Querschnitte auch bei plastischer Verformung.

Die ertragenen Biegemomente lagen mit 32,3 bzw. 34,7 Mp um 17 bzw. 19 % über den theoretischen vollplastischen Momenten, die mit den gemessenen Werten der Streckgrenze berechnet wurden. Die Überschreitung der vollplastischen Momente kann durch Wiederverfestigung erreicht worden sein, die bei der aufgetretenen Dehnung von etwa 10 % in den Gurten erfolgt sein kann. Außerdem ist es möglich, daß die Fließgrenze etwas höher lag als die 0,2 % - Dehnungsgrenze, die bei der Materialuntersuchung bestimmt worden ist. Die vollplastischen Momente bei Normwerten der Fließgrenze sind 25,6 bzw. 29,2 Mp m bei den vorgegebenen Querschnittsabmessungen.

Bei den Versuchen hat sich gezeigt, daß die Plastizierung der Stege im Zugbereich keinen nachteiligen Einfluß hat und daß die Träger, deren einzelne Teile aus Baustahl mit unterschiedlicher Fließgrenze bestehen, mindestens die Tragfähigkeit erreicht haben, die nach der Plastizitätstheorie berechnet werden kann. Wenn auch die Tragfähigkeit bei Ausführung des Steges aus St 52 etwas größer ist, ist doch insgesamt die Wirtschaftlichkeit besser, wenn der Steg aus St 38 besteht und eine Plastizierung im Zugbereich zugelassen wird.

Diese Ausführung ist seit 1965 in den DDR-Stahlbauvorschriften TGL 13 500 zugelassen. Dabei wird gefordert, daß die rechnerischen Spannungen im Steg unter Gebrauchslast nicht die Fließgrenze überschreiten. Das wird bei Stegen aus St 38 und Gurten aus St 52 stets eingehalten, wenn die zulässige Spannung der höherfesten Gurte nicht überschritten wird. Außerdem dürfen im überbeanspruchten Stegblechteil keine Querlasten angreifen. Zur Übertragung der Querkraft darf dieser Stegteil nicht herangezogen werden. Das zulässige Biegemoment ist nach diesem Berechnungsverfahren etwa ebenso groß wie bei der Bemessung nach dem Traglastverfahren mit den um 10 % erhöhten Sicherheitszahlen, die dort vorgeschrieben sind.

Mit Hilfe eines Rechenprogramms wurde eine optimierte Reihe von Schweiß-

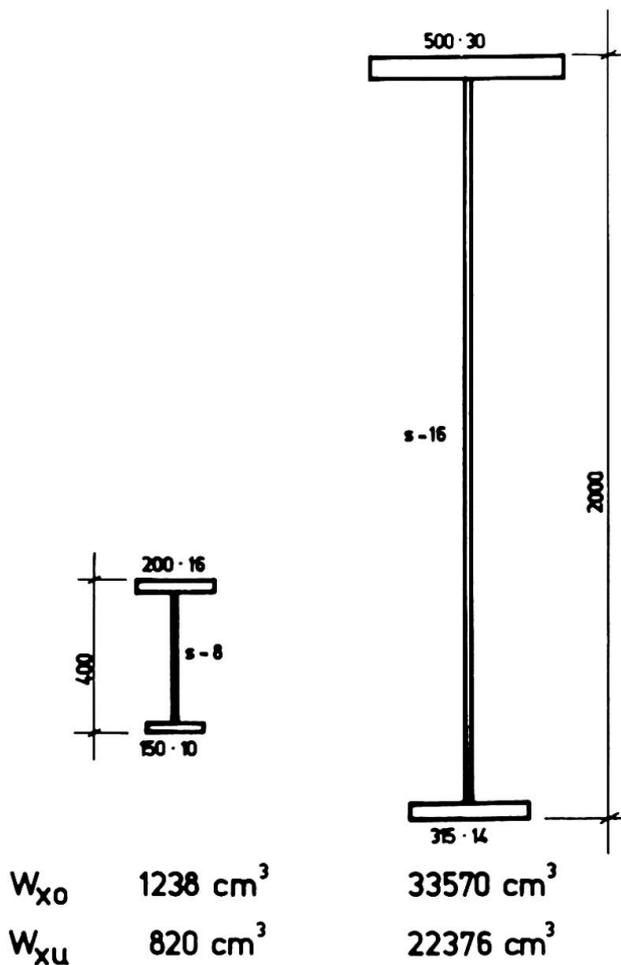


BILD 4

trägern ermittelt, bei denen die Stege und Obergurte aus St 38 und die Untergurte aus St 52 hergestellt werden. Proportional zu den Streckgrenzen verhalten sich dabei die Spannungen im Obergurt zu den Spannungen im Untergurt wie 2 : 3. Bild 4 zeigt den kleinsten und den größten Vertreter dieser Auswahlreihe und die dazugehörigen maßgebenden Querschnittswerte. Bei diesen Trägern ergeben sich Stahleinsparungen von durchschnittlich 12 % und in einem weiten Anwendungsbereich günstigere Effekte, als beim ausschließlichen Einsatz der höherfesten Stahlmarke.

## ZUSAMMENFASSUNG

Wirtschaftliche Biegeträger können hergestellt werden, bei denen der Zuggurt gegenüber dem Druckgurt und dem Stegblech aus einer höherfesten Stahlmarke ausgebildet wird. Bei Ausnutzung der zulässigen Spannung des Zuggurtes wird dabei der Randbereich der Stegbleche bis zur Plastizierung beansprucht. Der plastizierte Stegblechteil darf nicht zur Schubübertragung herangezogen werden und muss bei der Eintragung von Querlasten gemieden werden. Unter Einhaltung dieser Bedingungen wurde mit Hilfe eines Rechenprogrammes eine optimierte Trägerreihe aufgestellt.

## SUMMARY

It is possible to manufacture economical beams submitted to bending in choosing for the tensioned booms a high resistant steel, whereas for the booms in compression and the web plate a standard steel quality is employed. In utilizing the admissible tensions for the tensioned boom the external fibres of the web attain the plastic range. The plastized range of the web does not permit the transmission of the shearing efforts and this state has to be avoided in zones where the loads are transmitted. In taking into account these conditions a serie of economical beams has been developed with the help of a computer.

## RESUME

On peut construire des poutres économiques soumises à la flexion en choisissant pour les membrures tendues un acier à haute résistance, alors que pour les membrures comprimées et l'âme on choisit un acier normal. En utilisant les tensions admissibles pour la membrure tendue, les fibres extrêmes de l'âme atteignent le domaine plastique. Le domaine plastifié de l'âme ne permet pas la transmission des efforts de cisaillement et cet état doit être évité dans les régions où sont transmises les charges. En tenant compte de ces conditions on a développé, à l'aide d'un ordinateur, une série de poutres économiques.

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

Discussion de la contribution "Composite Steel-Deck-Reinforced Concrete Systems Failing in Shear-Bond" par R.M. Schuster

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**

Discussion de la contribution "Continuous Composite Beams for Bridges" par J.W. Fisher

Diskussion des Beitrages "Continuous Composite Beams for Bridges" von J.W. Fisher

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

JIRÍ PECHAR  
 Assoc. Professor  
 Technical University of Praha  
 CSSR

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.

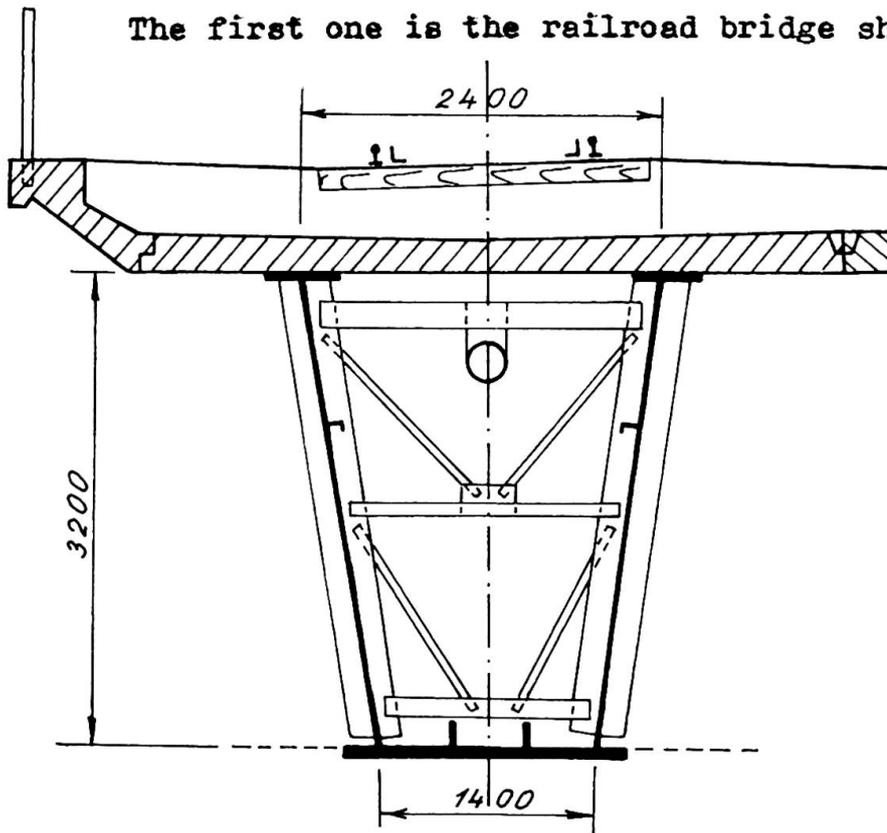


Fig.1.

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

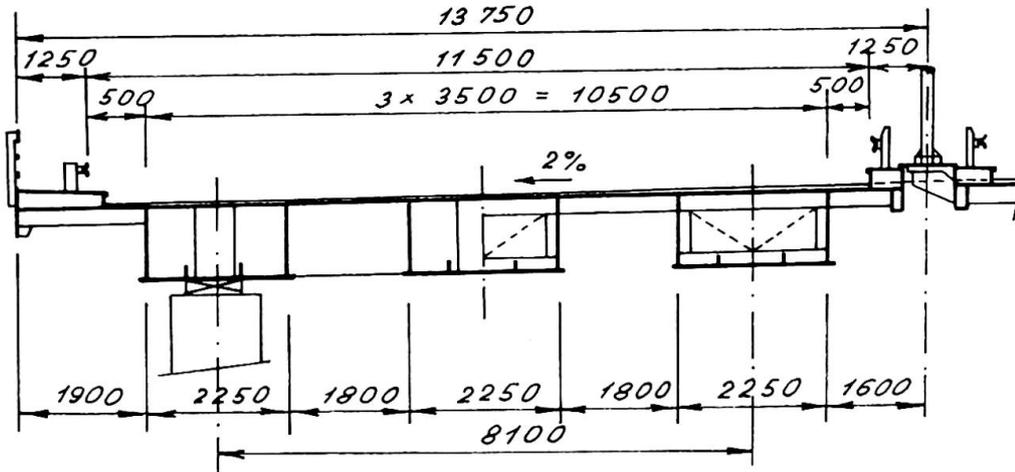
The composite action of the deck and steel box girder is achieved by high strength bolts M 24. The composite structure of this type is for the first time used for heavy

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  $500 \text{ kp/cm}^2$ . 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/.

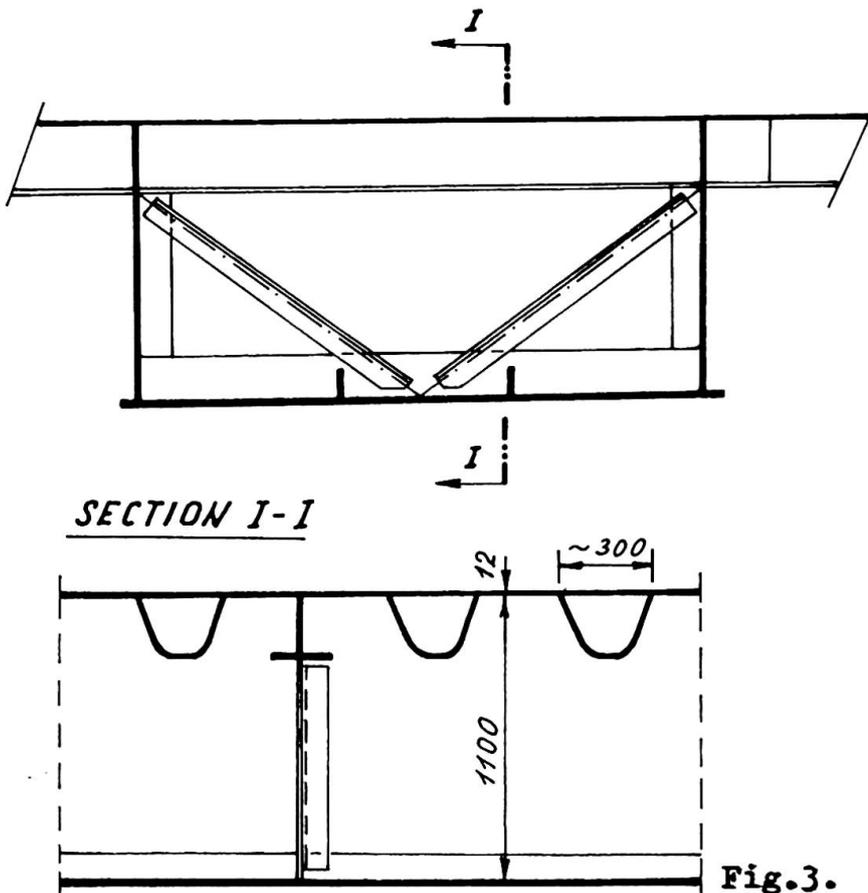


With its total length of approx. 700 m it spans also several main streets intersections in the downtown Praha.

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 s

span resist the distortion of the box cross section.

The whole bridge is curved in horizontal plane.

Fig.3.

## IIa

### REMARQUES • BEMERKUNGEN • COMMENTS

#### A Survey of Using Steel in Combination with Other Materials

Relevé sur l'utilisation d'acier en combinaison avec d'autres matériaux

Übersicht über die Verwendung von Stahl in Kombination mit anderen Materialien

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Bethlehem Steel Corporation

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In the introductory report (164), we expressed the hope that the prepared and free discussions contributed toward Theme IIa at the Ninth Congress IABSE would combine with the introductory report to form an authoritative world-wide survey of the theme subtopics as of 1972. Indeed, after the introductory report was published in 1970, we received a number of private communications referring to important earlier studies and applications not included in the report. Many of these omissions have either been discussed, or referred to, by the authors of the prepared discussions presented at the Ninth Congress. Other contributions, including more recent material published elsewhere, will be discussed in this final report.

The response to the request for prepared discussions on the INTERACTION OF DIFFERENT MATERIALS and the numerous new articles which have been published on this subject since 1970 make it evident that the art of combining steel with other materials is being actively explored and extensively utilized throughout the world. However, it is clear that combining steel with concrete to form composite beams, columns and floor systems has been receiving the greatest attention. Composite steel-concrete construction represents one of the older and more significant exploitations of the interaction of different materials. Other forms of construction that utilize interaction and are treated in this final report include sandwich construction, hybrid steel construction, prestressed steel structures and cable-stayed steel bridges.

#### COMPOSITE STEEL-CONCRETE CONSTRUCTION

The papers presented at the Ninth Congress by Messrs. Daniels (165), Hope-Gill (166), Maeda (167), Roderick (168), and Naka (169) made it clear that problems of continuity in composite structures are of major concern in current research. The remaining contributions dealing with the topic of composite

construction (170) (171) (172) (173) (174) were addressed to problems encountered in simple elements.

### Composite Steel-Concrete Beams

Continuity - The use of composite beams in bridges and multi-story buildings brought forth a number of questions that became the subject of systematic research only during the last decade. The early static tests of continuous beams (54) (55) (56) (57) were followed by investigations of beam-to-column connections (166) (168) and by detailed investigations of the negative moment region of continuous beams (168) (175) (176) (177). Tests have shown that properly reinforced continuous composite beams can develop negative moments equal to the ultimate bending strength of their positive moment region. Furthermore, plastic hinges with adequate moment-rotation capacity can be developed under combined negative moment and shear. These studies furnished the basic tools for investigations of composite beams as part of building frames.

Continuity in frames requires reliable connections. This subject earlier had not received as much attention as it deserves. Dobruszkes, Janss and Massonnet (178) reported on tests of connections between composite beams and concrete columns, and between fully encased steel beams and columns. Roderick (168) briefly discussed tests on exterior and interior connections of composite beams to fully encased columns carried out by Ansourian at the University of Sydney. Naka, Wakabayashi and Murata (169) listed several references to tests of beam-to-column connections encased in reinforced concrete. All found that concrete encasement substantially increases the strength of a connection. An interesting type of a semi-rigid connection of a composite beam to a steel column was developed by Johnson and Hope-Gill (166). It offers many of the advantages of a rigid connection, but lower in-place cost. The connection can develop appreciable strain hardening before the maximum load of the continuous beam is attained.

Concurrent fatigue tests of continuous beams were aimed at bridge applications. Maeda and Kajikawa (167) investigated the fatigue strength of a tension flange with stud shear connectors. The tests, including both direct tension specimens and beams subjected to negative bending, indicated a substantial reduction of the tension flange fatigue strength in the presence of shear connectors. This reduction tended to be smaller in bending tests than in direct tension tests. These results were in good agreement with earlier tests (39) (179) (180). Roderick (168) tested a continuous composite beam in fatigue and concluded that the fatigue life of stud connectors compared favorably with the values given by the British Standard Code of Practice CP117: Part 2 (36). Fatigue strength of shear connectors in continuous composite beams had been investigated in an earlier study at Imperial College (35) and in extensive investigations at Lehigh University (37) (165).

To avoid the deleterious effect of connectors on the fatigue strength of tension flanges, it is a common United States practice

to omit shear connectors from a portion of, or throughout, the negative moment region of a continuous beam. The Lehigh investigations have shown that in such cases additional shear connectors must be provided near the points of contraflexure. The results of those studies have been incorporated in the AASHTO Specifications (118) (181). Further studies of the negative moment region carried out at Lehigh University (165) have shown that the longitudinal reinforcement in the slab over the negative moment region should be at least 1% of the cross-sectional area of the slab; that most of this reinforcement should be placed near the top surface of the slab; and that it is reasonable to assume that the effective width of the concrete slab is the same throughout the beam length. Similar conclusions have been reported by Tachibana (182).

Lightweight Concrete - The results of systematic tests of composite beams with lightweight concrete slabs at Lehigh University and at the University of Missouri, referred to in the introductory paper (164), were published in 1971 (25) (26) and incorporated in the AISC Specification (183). In addition to determining the strength of shear connectors, these studies have shown that the strength of a composite beam with a slab of lightweight concrete is the same as for a beam with a slab of ordinary concrete of the same compressive strength. On the other hand, deflections are larger in beams with lightweight concrete. Accordingly, the design requirements based on beam strength should depend only on the strength of concrete while the deflection requirements should be also a function of concrete density.

Static and fatigue tests of stud connectors embedded in lightweight concrete were made in Great Britain by Menzies (184), who recommended design values for concrete with densities from 87 to 143 lb/cu. ft. Further research on lightweight concrete composite beams was reported by Roderick (168), Janss (170) and Ypeij (185). These studies generally substantiated the results of the above-mentioned American investigations.

Creep and Shrinkage - Long-term static tests of composite beams with lightweight aggregates were conducted at the University of Missouri (26) and at the University of Liege (170). Both studies demonstrated substantial deflection increases from creep and shrinkage. The Missouri investigators recommended that, in design, time-dependent deflection be taken as equal to instantaneous deflection.

Other studies of creep effects have been made recently in Germany by Hasse (186) and by Mainz and Wolff (187). In Great Britain, Menzies (188) analyzed data from the Moat Street Flyover taken during construction and for about two years thereafter. He found that after construction, effects of temperature changes obscured the small effects caused by shrinkage and creep. In the United States, Roll (189) analyzed the stresses and deflections due to differential shrinkage and creep in both shored and unshored beams in a New York State office building. He obtained excellent correlation between measured and theoretical deflections.

Precast Slabs - Composite beams with precast slabs connected to the steel members with high-strength bolts can be advantageous, particularly when on-site construction must be kept to a minimum or when traffic must be maintained during construction. In addition to Sattler (43), this type of construction was pioneered by Faltus (190) who used it for elevated railway bridges built in Czechoslovakia in the early sixties.

Marshall, Nelson and Banerjee (191) have continued Sattler's work on the use of high-strength bolts by making a series of push-out and beam tests at the University of Glasgow. They concluded that the load at first slip can be estimated using a friction coefficient of 0.45 for precast slabs on steel. A higher coefficient can be used for cast-in-place slabs. Beck and Heunisch (192) also made push-out and beam tests and concluded that design should be based either on an allowable coefficient of friction or on forces redistributed by slip. Research at Purdue University (45) (193) demonstrated that it is feasible to replace deteriorated highway bridge decks with precast, prestressed concrete slabs.

Beck and Heunisch (192) used bolted precast slabs in the construction of a 7-story building at Johann Wolfgang Goethe University in Frankfurt-on-Main. Other structures utilizing this type of composite construction included a few demountable overpasses and parking garages. Holloway (194) (195) described the Rosslare Harbor Viaduct and the Tivoli Bridge in Ireland as examples of structures reconstructed economically with precast composite units.

Miscellaneous - Among numerous additional studies, the load factor design for highway bridges developed on both sides of the Atlantic (196) (181) (197) (198) includes provisions for design of composite beams. Reddy and Hendry (199) made a thorough review of the results of British studies of simply-supported composite beams and derived equations for the ultimate bending strength of such beams. Repeated load tests of simple composite beams by Roderick (168) demonstrated that properly designed composite beams will not fail catastrophically, but rather by gradual deterioration. R. P. Johnson proposed design methods for consideration of longitudinal shear (200), transverse slab reinforcement (201) and deep haunches (202) in composite beams; the last proposal was based on experiments conducted at the University of Manchester (203). New studies of torsional properties of composite beams were reported by Heins and Kuo (204) (205). Janss joined the researchers interested in the use of checker plate (170) and L'Hermite joined those interested in the use of epoxy resins (173) for shear connection. Naraharirao (206) and Cran (207) reported on further studies of composite open-web steel joists.

#### Concrete Encased Steel Beams

Interest in concrete encased steel beams appears to be limited. Only one reference (169) to such beams was made in the contributions to the Ninth Congress, and only four other new references were found (194) (195) (208) (209).

According to Naka, Wakabayashi and Murata (169), steel frames encased in reinforced concrete are used extensively in Japan because of their excellent seismic resistance. Encased steel beams are an integral part of such frames. Their design is governed by the specifications of the Architectural Institute of Japan (210) and is based on extensive studies listed in the bibliography of Reference 169. The allowable bending moment for an encased steel beam is specified as the sum of the allowable bending moments of the steel beam and the reinforced concrete beam. Holloway reported on the use of precast concrete-encased steel beams in a footbridge at Balbriggan (195) and as parapet units for the Tivoli Bridge (194). The remaining two references were concerned with tests of concrete encased steel sections. Matty and Narasimhan (208) tested 24 simply-supported and 5 continuous beams to determine their moment-curvature relationships. Babb (209) reported on full-scale and model tests on two new, patented types of concrete encased steel sections.

### Steel-Concrete Columns

The discussion of concrete encased steel columns in the introductory report (164) was concluded with the following statement: "It would seem...that the data and the tools necessary for the development of improved design methods for concrete encased steel columns are available and that improvements of code provisions are in order." The 1971 edition of the ACI Building Code (211) is available and a new British Standard CP117: Part 3 (212) is in draft form. Both include new design methods for steel-concrete columns.

Based on experimental and theoretical research by Furlong (107) (108) and by Knowles and Park (213) (214), the 1971 edition of the ACI Building Code redefined all types of composite columns as composite compression members reinforced longitudinally by either structural steel shapes, pipe or tubing with or without longitudinal reinforcing bars. Composite compression members now can be designed as beam-columns; prior to the 1971 Code rules were available only for the design of concentrically loaded columns. Subject to limitations on maximum stiffness and maximum radius of gyration of the composite cross-section, and to a few detailing rules, the strength of composite compression members is computed using the ordinary procedures for reinforced concrete members.

Following the decision in 1963 to use concrete-filled tubular columns in the multilevel interchange between the M4 and M5 Motorways at Almondsbury (215), exploratory tests were conducted at the Building Research Station and Imperial College. As has been reported in the introductory report, further investigations followed at Imperial College. These included analytical work that was checked against tests of concrete-filled circular tubes conducted at Imperial College (216), in Japan (217) and at Liege (218). A second series of tests at Liege (219) permitted a check of the analytical work for eccentrically loaded square and rectangular concrete-filled tubes. A computer program based on uniaxial strength of concrete was found to predict accurately the strength of the latter columns and the strength of slender circular columns with length-to-diameter ratios over 15 (220). More

recent studies at Imperial College have been concerned with triaxial stress effects that augment the strength of short circular concrete-filled tubes (221). Another such attempt was reported by Janss (170).

The practical utilization of the research at Imperial College took three paths. The uniaxial computer program was used both to generate ultimate load tables for concrete-filled tubes (222) and to develop an empirical method for calculating the ultimate strength of concrete-filled square and rectangular columns (223). Finally, a draft was prepared of a design specification for composite columns in buildings and bridges (212).

In Japan, steel columns encased in reinforced concrete are used in frames (169). The allowable axial load and bending moment are computed either (1) as the allowable axial load on the concrete section and the sum of the allowable moments on the steel and concrete sections or (2) as the allowable moment on the steel section and the sum of the allowable axial loads on the steel and concrete sections.

Roderick (168) reported the results of tests of concrete encased steel columns bent about both principal axes. In nearly all tests the theoretical column instability was reached shortly after the steel had begun to yield. The theoretical collapse loads were lower than the actual collapse loads for eccentricities of up to 0.8 in. about any centroidal axis. Good agreement with the theoretical solution was found also in tests of concrete-filled square steel tubes eccentrically loaded about their diagonal axis.

The development of two additional computer programs for concrete-filled tubular beam-columns was reported recently from Canada (224) and the United States (225). Another paper discussed prestressing of tubular columns through the use of expansive cement (226).

Two earlier studies of concentrically loaded concrete-filled tubes (227) (228) were discussed in the introductory report, but the corresponding references were omitted from the bibliography.

### Steel-Concrete Slabs

The developmental work leading to the use of composite steel-concrete plates as blast resistant hatch covers, mentioned in the introductory report, is described in Reference 229. Composite steel-concrete plates are also well suited for mine shaft and tunnel liners (230) (231). While ordinarily the concrete and steel connection is mechanical, L'Hermite (173) proposed to bond the two with epoxy resin applied to blast-cleaned steel plates just before placing the concrete. In these applications, the steel plate serves both as a tension or shear carrying member and a concrete form.

A brief, interesting history of the development and current status of cellular steel floors was presented by Dallaire (232). The interaction of composite sheet steel flooring with concrete

has been under investigation at the University of Iowa (134) (233). The behavior of these systems was described by Schuster (172) for simple one-way slabs and by Porter and Ekberg (171) for simply supported two-way slabs.

### Composite Steel-Concrete Structures

Since the individual elements, i.e. beams, columns, slabs and walls of a structure may be composite or noncomposite and, furthermore, of several different types, the choice of possible structural systems is substantial.

In bridge construction, the most common structural system is a steel beam and girder framing acting compositely with a concrete slab.

In building construction, the older fully encased steel framing and the newer unencased steel framing acting compositely with the floors are probably the two best known systems. As we have seen earlier, a modification of the fully encased framing is used extensively in Japan (169) because of its excellent seismic resistance. Unencased steel framing acting compositely with the floors is used extensively in the United States. Roderick's studies of connections (168) suggest the use of fully encased columns in combination with ordinary unencased composite beams. A similar system was developed recently by F. R. Khan for medium-rise tall buildings (234). In this system, the gravity loads are supported by steel columns, both exterior and interior, while the wind loads are resisted by exterior concrete walls reinforced with the exterior steel columns and acting as a vertical box cantilever. The exterior columns are encased in concrete walls. In the three buildings discussed in Reference 234, the floors were of ordinary steel-concrete composite construction. The buildings, located in Houston, Chicago and New Orleans, are 24, 36 and 50 stories high.

### SANDWICH CONSTRUCTION

None of the contributions to Theme IIa dealt with sandwich panels and only one (235) among the numerous other contributions to the Ninth Congress dealt with sandwich construction. It would seem then that there is little interest in the use of sandwich plates as structural construction elements. However, some recent references suggest that the interest is not entirely absent, either in Europe (235) or in the United States (236).

Several recent articles by Jungbluth (235) (237) (238) described the properties, manufacture and applications of steel-polyurethane sandwich panels that consist of two steel sheet face plates and a comparatively thick core of foamed polyurethane. The principal use of the panels is for walls but they have been used or proposed for use as a self-supporting roof and for cable-supported roofs. Reichard (236) discussed paper honeycomb-core panels used by the building industry for walls of

single-story residences. The panels are faced with steel, aluminum, fiber-reinforced plastics or other materials. The article concluded with the statement that the construction industry in the U.S. is now using several different sandwich panel systems and is contemplating the use of others.

### HYBRID STEEL CONSTRUCTION

Two contributions to the discussion were concerned with hybrid sections. In one, Bo and Daddi (174) pointed out that unsymmetrical hybrid sections with bottom flanges made of 100 ksi yield steel would improve Preflex beams. The other paper presented an analysis and four tests of hybrid columns (239) with 100 or 50 ksi steel flanges and 50 or 36 ksi steel webs.

The results of the hybrid girder fatigue tests at the University of Texas, referred to in the introductory report, have been published (240) (241) (242). The reports contain information on fatigue tests of 71 plate girders having stiffened webs 3 or 4 feet deep of 36 ksi steel and flanges of 100 or 50 ksi steels.

The use of hybrid girders in the United States has grown rapidly since the 1969 adoption of specifications for their design (84) (118). Today, hybrid girders with 50 ksi flanges and 36 ksi webs are used extensively for highway bridges. The discussion provided no information on the use, if any, of hybrid girders in other countries. On the other hand, hybrid construction, i.e. the use of different steels in various parts of the same structure, has been common for decades. Structures of this type can be found throughout the world. The recent development of new steels has increased the importance of hybrid construction and widened its use.

### PRESTRESSED STEEL STRUCTURES

For this discussion, it seems useful to distinguish between prestressing of individual components (e.g. a girder) and prestressing of the whole structure (e.g. a cable-supported roof).

Two contributions were directed to the first subject. Bo and Daddi (174) proposed to build Preflex beams using hybrid girders with bottom flanges of 100 ksi steel. In the second contribution, Tochacek, Rosenkranz and Ferjencik (243) outlined an extensive research and development of prestressed steel that has been in progress since 1960 in Czechoslovakia. One recent news item (244) dealing with prestressed elements is pertinent here. It reports the first -- and thus far the only -- use of Preflex beams in the United States; this type of construction was selected primarily to reduce construction depth.

The scarcity of Western literature on this subject suggests that prestressed steel components have been used there only infrequently, probably because they do not offer an economic advantage. On the other hand, the literature cited in the introductory report and the contribution by Tochacek, Rosenkranz and Ferjencik suggest that prestressed steel components are used in Eastern Europe.

One contribution to Theme IIa dealt with prestressing of steel structures (245). Describing examples of prestressed steel structures built in Czechoslovakia, it included, among others, cable-supported roofs, strengthening of existing bridges by prestressing, cable-supported pipe crossings and guyed towers. Further information on this general subject may be found in Reference 246. It should be mentioned that Theme IIIa of the Ninth Congress was devoted specifically to the subject of cable-supported roofs and the contributions to that subject covered more than 100 pages of the Preliminary Report (247). Five other contributions to various themes in the same Report dealt with cable-stayed bridges. Recent papers covering these and other types of prestressed steel structures are too numerous to be included in this final report.

Prestressing of steel structures is a widely used art that includes many varied forms. It is noteworthy, however, that East European countries treat it as a separate, distinct discipline.

### CABLE-STAYED STEEL BRIDGES

The subject of cable-stayed bridges was introduced in Theme IIa by reference to six earlier papers and in Theme IIIa by Professor Leonhardt's introductory paper (248). Outlining recent trends and further developments, Leonhardt pointed out several advantages of cable-stayed bridges as compared to suspension bridges. He concluded that cable-stayed bridges are particularly suitable for spans in excess of 1,000 meters and may even be used for spans of 1,500 meters in the future. Leonhardt suggested also that there is a trend toward a large number of cables in one plane; that the use of aerodynamically stable deck cross-sections is particularly desirable for long spans; that parallel wire cables increase stiffness and reduce weight; and that a newly developed process of socketing with plastics instead of zinc will increase the fatigue strength of cables and lead to higher working stresses. In his discussion (249), Thul expressed general agreement with Leonhardt's views. However, Thul expects suspension bridges to remain dominant in very long spans. Thul's discussion of cable properties, socketing and delayed cracking of wires was a particularly useful contribution. Other contributions to the Ninth Congress included discussions of: a cable-stayed bridge with a three-level deck (250); the construction of the bridge over the railroad station in Ludwigshafen (251); the investigation of the aerodynamic behavior of the Toyosato Ohhashi bridge in

Japan (252); and another method of socketing and protecting parallel wire cables (253). Additional information on cable properties can be found in Reference 254.

It is interesting that, with one exception, all contributions pertaining to cable-stayed bridges were authored by German engineers. On the other hand, a review of the recently published reports indicates that cable-stayed bridges are receiving considerable attention throughout the world. General surveys have been published in India (255), the United States (256) and elsewhere (257). Numerous articles have described new bridges in Great Britain (258), Canada (259), Australia (260), Czechoslovakia (261), the Soviet Union (262), the United States (263), France (133), Germany (264), Austria (265), Argentina (266) and elsewhere; and plans for new bridges (267) (268) (269) (270). Contributions to the theory of design are also multinational (271) (272) (273) (274) (275) (276) (277) (278) (279) (280) (281). It also should be pointed out that some of the articles on cable-stayed bridges deal with problem areas (249) and a catastrophic failure (282) (283) (284) (285). Report 282 made it clear, however, that this failure was unrelated to the cable-stayed feature of the bridge.

This brief review of the recent contributions to the field of cable-stayed steel bridges has highlighted one of the most significant structural developments of recent years: the spread of this construction throughout the world. Counting bridges that have been completed and those now under construction, there are presently more cable-stayed steel bridges outside than inside the German Federal Republic. This development alone is bound to lead to further innovations that will make the cable-stayed steel bridge an even more effective tool of the bridge builder.

#### CONCLUDING REMARKS

This survey would appear incomplete unless it included at least a thought or two on what the past suggests for the future.

In building construction we can expect more complete integration of the components into systems. In low-rise structures, walls and floors will be integrated into room modules before they are brought to the site. In high-rise structures, the development of submodules can be predicted from recent experience. Frames will probably be integrated with curtain walls to serve both as a structure and a temperature and sound barrier. This subsystem may also include electrical and mechanical services. Floors, ceilings, wiring and ducts are candidates for another subsystem.

In bridge construction we can expect to see extensive use of single-span preassembled bridges for short spans. In medium spans, one or two girders with an appropriate portion of the slab are likely to be subassembled before lifting the unit into

place. Transverse and longitudinal continuity can be provided through prestressing or some other means. In the long span range, we have already seen full-width modules lifted in place, joined longitudinally and supported by cable stays or suspended from cables.

The development of improved materials and better design tools will lead not only to more economy, but also to more useful structures. As in the past, the structures of the future will play an important part in improving the quality of life.

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ACI	American Concrete Institute
AISC	American Institute of Steel Construction
AISI	American Iron and Steel Institute
AASHO	American Association of State Highway Officials
ASCE	American Society of Civil Engineers
IABSE	International Association for Bridge and Structural Engineering

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## SUMMARY

This final report, combined with the introductory report on interaction of steel with other materials, constitutes a survey of the state-of-the-art in composite steel concrete construction, sandwich construction, hybrid steel construction, prestressed steel structures and cable-stayed bridges as of the middle of 1972.

## RESUME

Ce rapport final forme, ensemble avec le rapport introductif sur l'interaction de l'acier avec d'autres matériaux, un aperçu sur l'état actuel de la construction en acier-béton, de la construction sandwich, la construction hybride, les structures en acier précontraint ainsi que des ponts à haubans, vers le milieu de l'année 1972.

## ZUSAMMENFASSUNG

Dieser Schlussbericht bildet zusammen mit dem Einführungsbericht über die Wechselwirkung von Stahl mit anderen Materialien eine Uebersicht über den Stand des Stahlbetonbaues, der Sandwichkonstruktion, des hybriden und vorgespannten Stahlbaues sowie der Schrägkabel-Brücken um die Mitte des Jahres 1972.

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