Zeitschrift: IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht

Band: 6 (1960)

Rubrik: V. Prefabricated structures

Béton armé et béton précontraint
Stahlbeton und Spannbeton
Reinforced and Prestressed Concrete

V

Structures composées préfabriquées
Bauweise aus Fertigteilen
Prefabricated Structures

V a

Moyens d'assemblage
Verbindungsmittel
Connection Methods

V b

Redistribution due au fluage des efforts intérieurs
Kräfteumlagerung durch Kriechen
Redistribution of Stresses Due to Creep

General Report

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V a. Connection Methods

The subject under consideration has been elucidated considerably by the contributions made in connection with the Stockholm congress. The main points of the corresponding papers will be reviewed in what follows.

D. McHenry and A. H. Mattock give in their paper a most valuable report on very extensive investigations of precast prestressed constructions made at the PCA Research and Development Laboratories in USA. The tests dealt with individual girder-slab members, and included studies of continuity performance, horizontal shear, diagonal tension, flexural strength, creep and shrinkage effects, and reverse bending. Then a complete two-span, two-lane bridge was constructed on a half scale in the laboratory and tested to failure.

The test results were in every respect favourable to the pre-cast, prestressed construction system under consideration. Continuity from span to span of the precast girders was obtained by the use of diaphragms at the girder ends, together with a situ-cast deck slab in composite action with the girders. The deck contained deformed bar reinforcement to carry the negative moments at intermediate supports. This simple connection of the precast girders gave a degree of continuity of about 90% for live load.

The authors state at the beginning of the paper that continuity between adjacent spans of bridges leads to well-recognized advantages. And finally they state that application of this type of connection to building construction as well as to bridge construction results in sounder and more economic structures than those made from precast concrete members which do not utilize continuity.

The interested reader is referred to a series of PCA Development Department Bulletins containing detailed reports on the investigations and aspects on design criteria.

D. H. NEW gives in his paper some comments on the paper by CASADO and GOÑI. He emphasizes that careful attention must be focused on the necessity of developing joints that give adequate strength and rigidity combined with economy and speed erection. He explains his points of view in discussing three cases of detailed connection joints shown in the author's paper. NEW also emphasizes a statement in the General Report that the Designer should design any unit for handling in all its stages of manufacture and erection and not merely in its final position.

C. F. CASADO gives a short answer to Mr. NEW, with some supplementary information.

H. ZEIDLER describes in his paper a system which has successfully been used in the erection of a 10-storey hospital building. In each storey, pairs of reinforced concrete members were set up as half frames opposite each other, and were supported in the middle of the structure by a latticed steel column. The steel column in the first place served as an auxiliary mounting scaffold but was later enclosed in a situ-cast concrete column then serving as reinforcement. Simultaneously the half frames became monolithically connected with the column. The author of this paper wishes to draw the attention to the possibility of combination of steel constructions with precast reinforced concrete structural units.

E. LEWICKI announces in his contribution some trends in the practice of the last four years while commenting a survey of different connection joints that the author had published at the Lisbon congress 1956. The author considers that additional arrangements to make connection joints rigid for full continuity should be restricted only to cases where this is absolutely necessary. The reason is saving in cost and in time for erection. It is seen that this attitude is somewhat contradictory to that of the authors above.

Conclusions

1. Careful attention must be focused on the necessity of arranging supports for, and connection joints of, precast concrete members that give adequate strength and rigidity combined with economy and speed of erection.

2. The Designer of precast concrete units must co-operate with the Fabricator and the Erector, and must consider fully manufacture, handling off the casting beds, transport to site, safety during lifting, and erection.
3. Continuity between adjacent spans leads to well-recognized advantages, e.g. less field moments, less deflections, less sensitivity to secondary influences, such as eccentric loadings or eccentric supports.
4. Expansion joints or shrinkage joints are in some cases necessary.
5. Supports and connection joints without continuity, but with adequate fixing and stability, are often quite satisfactory.
6. Many examples of details from precast building constructions are given and discussed from practical points of view.
7. The possibility of a combination of steel constructions (used as auxiliary mounting scaffolds later enclosed in concrete) and precast reinforced concrete structural units should be considered for special cases.
8. In a single paper, by McHenry and Mattock, very extensive experimental investigations on the properties of connection joints are reported. Further studies of that kind are highly desirable.

Vb. Redistribution of Stresses due to Creep

In the paper by J. N. Distefano the author treats in principle the problem of calculating the deflections of a concrete beam under loading, resting on a continuous bed of visco-elastic material. In this case there will be visco-elastic deformations both in the beam and in the bed. In particular, the author makes different assumptions regarding the influence of age of concrete on the creep function. The author is of the opinion that this influence is considerable for situ-cast constructions but that it is less pronounced for precast units long-time-stored before use. For the latter case the author has made explicit calculations and confirmed the correctness of the classical method of using a reduced elastic modulus.

It has been advisable not to formulate any conclusions regarding theme Vb but only to refer to the General Report.

Note. In accordance with the proposal made in the General Report, a subcommittee of the IABSE Working Commission III was created during the Stockholm congress in order to study more systematically problems connected with prefabricated structures.

Rapport général

Va. Moyens d'assemblage

Les communications présentées au congrès de Stockholm ont largement contribué à éclaircir le problème des moyens d'assemblage. Nous allons passer en revue les points principaux des mémoires y relatifs.

MM. D. McHenry et A. H. Mattock décrivent dans leur intéressant mémoire des recherches très poussées concernant des ouvrages préfabriqués et précontraints; ces recherches ont été effectuées aux Etats-Unis, dans les Laboratoires de Recherche et de Développement de l'Association des Ciments Portland. Ces essais ont porté d'abord sur des poutres-dalles isolées; ils comprenaient des recherches relatives à la continuité, aux efforts rasants, aux contraintes principales, à la résistance à la flexion, aux effets du fluage et du retrait, ainsi qu'à un moment de flexion de signe inversé. Finalement, on réalisa en laboratoire, à l'échelle 1 : 2, un pont complet à deux voies, continu sur deux travées, et on l'essaya jusqu'à la rupture.

Les résultats des essais se sont montrés favorables sous tous les rapports pour le système préfabriqué et précontraint étudié. De travée en travée, la continuité des poutres préfabriquées était assurée par des diaphragmes disposés aux extrémités des poutres et par une dalle de tablier coulée sur place, participant à la résistance des poutres. Cette dalle était munie de fers d'armature profilés, destinés à reprendre les moments négatifs au droit des appuis intermédiaires. Ce procédé simple d'assemblage des poutres préfabriquées a permis d'obtenir un degré de continuité s'élevant à environ 90% pour les surcharges.

Au début de leur article, les auteurs constatent que la continuité entre les travées adjacentes d'un pont présente des avantages évidents. Pour terminer, ils indiquent que l'utilisation de ce type d'assemblage, aussi bien dans la construction des immeubles que dans celle des ponts, permet de réaliser des ouvrages en béton mieux adaptés et plus économiques que ceux comportant des éléments préfabriqués sans continuité.

Si l'on désire une documentation plus complète, on pourra consulter les Bulletins de la Section de Développement de l'Association des Ciments Portland, bulletins qui contiennent les rapports détaillés des recherches effectuées et quelques aspects des critères relatifs aux études.

M. D. H. New commente dans sa contribution le mémoire de MM. Casado et Goñi. Il relève particulièrement qu'il est nécessaire de développer des assemblages présentant une résistance et une rigidité suffisantes, tout en étant économiques et rapides à réaliser, et que ce point doit retenir spécialement l'attention. Il expose son point de vue en discutant les détails de trois assemblages présentés par les auteurs précités. Il souligne également une remarque émise dans le Rapport général: l'ingénieur chargé de l'étude doit tenir compte de tous les états de fabrication et de montage et ne peut se limiter à considérer les sollicitations de l'ouvrage terminé.

M. C. F. Casado répond brièvement à M. New et donne quelques indications complémentaires.

M. H. Zeidler décrit un système qui a été utilisé avec succès lors de la construction d'un hôpital comportant dix étages. A chaque étage, des éléments en béton, formant deux demi-cadres, ont été érigés l'un en face de l'autre et soutenus au milieu de l'ossature par un poteau métallique à treillis. Ce poteau métallique servait d'abord d'échafaudage auxiliaire de montage; il venait ensuite s'incorporer dans la colonne en béton coulée sur place et y tenait lieu d'armature. En même temps, les demi-cadres étaient assemblés monolithiquement à la colonne. L'auteur veut montrer dans sa communication comment on peut combiner des parties métalliques avec des éléments préfabriqués en béton armé.

M. E. LEWICKI indique dans son mémoire quelques tendances constatées dans la pratique de ces quatre dernières années et il passe en revue à cet effet les différents types d'assemblage qu'il a présentés lors du congrès de Lisbonne en 1956. Il pense qu'il faut limiter au strict minimum les dispositifs additionnels destinés à réaliser des liaisons parfaitement continues, ceci afin de réduire les dépenses et la durée du montage. Ce point de vue est quelque peu en contradiction avec celui des auteurs précédents.

Conclusions

1. Pour les éléments préfabriqués en béton, il est nécessaire de prévoir des appuis et des assemblages présentant une résistance et une rigidité suffisantes, tout en étant économiques et rapides à réaliser; ce point doit retenir spécialement l'attention.
2. L'ingénieur chargé d'étudier des éléments préfabriqués en béton doit travailler en collaboration avec le fabricant et l'entrepreneur chargé du montage; il doit considérer à la fois la fabrication, le démoulage, le transport à pied d'œuvre, la sécurité pendant le levage et le montage.
3. La continuité entre travées adjacentes présente des avantages évidents, par exemple une diminution des moments en travée, des flèches plus petites, une sensibilité moins grande aux influences secondaires, comme celles de charges ou de réactions excentriques.
4. Dans certains cas, il est nécessaire de prévoir des joints de dilatation ou de retrait.
5. Des appuis ou des assemblages sans continuité sont souvent parfaitement satisfaisants, pourvu qu'ils présentent une stabilité et un degré de fixation suffisants.
6. De nombreux exemples de détails relatifs à des ouvrages préfabriqués sont présentés et discutés du point de vue pratique.
7. Dans des cas spéciaux, on peut envisager d'utiliser des parties métalliques (servant d'échafaudages auxiliaires de montage et s'incorporant par la suite dans le béton) combinées avec des éléments préfabriqués en béton armé.
8. Un seul mémoire, celui de MM. MCHENRY et MATTOCK, décrit des recherches expérimentales très poussées relatives aux propriétés des assemblages. Il serait hautement souhaitable que l'on dispose d'autres études de ce genre.

Vb. Redistribution due au fluage des efforts intérieurs

Dans son mémoire, M. J. N. DISTEFANO traite en principe le problème de la détermination des flèches d'une poutre en béton chargée, reposant sur une fondation continue visco-élastique. On obtiendra donc des déformations visco-élastiques à la fois dans la poutre et dans sa fondation. L'auteur fait en particulier différentes hypothèses concernant l'influence de l'âge du béton sur la fonction de fluage. Il estime que cette influence est considérable pour les ouvrages coulés sur place mais qu'elle est moins marquée pour les éléments préfabriqués, qui demeurent longtemps en dépôt avant d'être utilisés. Pour ce dernier cas, l'auteur développe des calculs explicites et confirme l'exactitude de la méthode classique dans laquelle on utilise un module d'élasticité réduit.

En ce qui concerne le thème Vb, il paraît judicieux de ne pas formuler de conclusions et de se contenter de se référer au Rapport général.

Remarque: Comme nous l'avions proposé dans le Rapport général, il a été créé lors du congrès de Stockholm, au sein de la Commission de travail III de l'AIPC, une sous-commission chargée d'étudier plus systématiquement les problèmes relatifs aux ouvrages préfabriqués.

Generalbericht

Va. Verbindungsmethoden

Dieses Thema ist durch die am Stockholmer Kongreß vorgelegten Beiträge weitgehend erläutert worden. Die wichtigsten Punkte dieser Veröffentlichungen sollen nachstehend nochmals erwogen werden.

D. McHENRY und A. H. MATTOCK geben in ihrem Beitrag einen äußerst wertvollen Bericht über sehr weitführende Untersuchungen an vorfabrizierten, vorgespannten Bauteilen, ausgeführt an den PCA Research and Development Laboratories in den USA. Die Versuche befaßten sich mit einzelnen Träger-Platten-Elementen, einschließlich Studien über Herstellung der Kontinuität, horizontale Schubspannungen, Hauptzugspannungen, Biegefestigkeit, Kriech- und Schwindeffekte sowie Vorzeichenwechsel der Momente. Im weiteren wurde im Laboratorium eine vollständige zweispurige Zweifeldbrücke im Maßstab 1 : 2 errichtet und auf Bruch untersucht.

Die Versuchsergebnisse fielen für die betrachtete, vorfabrizierte, vorgespannte Bauweise in jeder Beziehung günstig aus. Kontinuität zwischen den Feldern wurde durch Auflagerquerträger und einer Verbundplatte aus Ortsbeton erstellt. Zur Aufnahme der negativen Momente über dem Auflager wurden profilierte Stahleinlagen in der Platte verwendet. Diese einfache Verbindung mit den vorfabrizierten Trägern erzielte einen Kontinuitätsgrad für Nutzlast von 90 Prozent.

Einleitend weisen die Autoren darauf hin, daß Kontinuität zwischen den Spannweiten zu offensichtlichen Vorteilen führt. Schließlich halten sie auch fest, daß durch Herstellung der Kontinuität sowohl im Hoch- wie im Brückenbau bessere und wirtschaftlichere Konstruktionen erstellt werden können, als wenn auf solche Verbindungen verzichtet wird.

Die besonders interessierten Leser werden auf eine Reihe Veröffentlichungen des PCA Development Department aufmerksam gemacht, welche detaillierte Rapporte über die Untersuchungen und Aspekte der Konstruktions-Kriterien enthalten.

D. H. New kommentiert in seinem Aufsatz die Veröffentlichung von Casado und Goñi. Er betont, daß die Entwicklung von Verbindungen, die genügend fest und steif, andererseits aber auch wirtschaftlich und schnell erstellbar sind, besonders wichtig ist. Er erläutert seine Ansicht am Beispiel dreier Verbindungen aus der Veröffentlichung der beiden Autoren. New weist erneut auf die Bemerkung im Generalbericht hin, daß der Konstrukteur jedes Element in der Weise entwerfen soll, daß alle Phasen der Herstellung und Montage berücksichtigt werden müssen und nicht allein die Endstufe.

C. F. Casado gibt eine kurze Replik zu den Ausführungen News sowie einige zusätzliche Erläuterungen.

H. Zeidler beschreibt in seinem Aufsatz ein System, das mit Erfolg beim Bau eines 10stöckigen Spitalgebäudes zur Anwendung kam. In jedem Stockwerk wurden paarweise armierte Betonelemente als Halbrahmen aneinandergesetzt und in der Mitte auf eine vergitterte Stahlstütze abgestellt. Die Stahlstütze diente in erster Linie der Montage der Halbrahmen; sie wurde jedoch später mit Ortsbeton ausbetoniert und so monolithisch mit den Rahmen verbunden. Der Autor dieser Arbeit möchte hier auf die Möglichkeit der Kombination von Stahlkonstruktionen mit vorfabrizierten Betonelementen hinweisen.

E. Lewicki verweist in seinem Beitrag auf die Entwicklungstendenzen der Praxis in den letzten vier Jahren, indem er verschiedene Verbindungen erläutert, die der Autor anläßlich des Lissaboner Kongresses im Jahre 1956 veröffentlichte. Der Verfasser ist der Ansicht, daß zusätzliche Maßnahmen zur Erreichung voller Kontinuität nur in absolut notwendigen Fällen getroffen werden sollten. Der Grund dafür liegt in der Einsparung an Kosten und Montagezeit. In dieser Auffassung läßt sich ein gewisser Gegensatz zu jener der früher genannten Autoren feststellen.

Schlußfolgerungen

1. Es ist notwendig, der Ausbildung von Auflagern und Verbindungen bei Verwendung von vorfabrizierten Elementen im Hinblick auf genügende Festigkeit und Steifigkeit sowie Wirtschaftlichkeit und Schnelligkeit bei der Montage besondere Aufmerksamkeit zu schenken.

2. Der Konstrukteur von vorfabrizierten Betonelementen muß mit dem Hersteller und Unternehmer eng zusammenarbeiten. Er muß Herstellung, Ausschalen, Transport sowie Sicherheit bei der Montage berücksichtigen.
3. Kontinuität zwischen aufeinanderfolgenden Spannweiten bringt offensichtliche Vorteile mit sich, beispielsweise weniger Feldmomente, weniger Durchbiegungen, geringere Anfälligkeit gegen sekundäre Einflüsse wie exzentrische Belastung sowie exzentrische Auflager.
4. Dehnungs- und Schwindfugen sind in gewissen Fällen notwendig.
5. Auflager und Verbindungen ohne Kontinuität, jedoch mit genügender Stabilität sind des öftern absolut genügend.
6. Viele Beispiele von Einzelheiten bei vorfabrizierten Konstruktionen sind angeführt und von der praktischen Seite her erörtert.
7. Die Möglichkeit der Kombination von Stahlkonstruktionen (als zusätzliche Hilfe bei der Montage, später ausbetoniert) mit vorfabrizierten Betonelementen ist für spezielle Fälle zu untersuchen.
8. Nur in der Arbeit von McHenry und Mattock wird anhand von sehr weitgehenden Versuchen über die Eigenschaften von Verbindungen berichtet. Weitere Studien in dieser Richtung sind höchst wünschenswert.

Vb. Spannungsumlagerung infolge Kriechen

J. N. Distefano behandelt in seiner Veröffentlichung grundsätzlich das Problem der Berechnung der Durchbiegungen eines Betonbalkens auf elastisch-plastischer Bettung. In diesem Fall entstehen elastisch-plastische Deformationen im Balken und in der Unterlage. Der Verfasser geht betreffend Einfluß des Alters des Betons auf die Kriechfunktion von verschiedenen Annahmen aus. Er ist der Ansicht, daß dieser Einfluß bei Konstruktionen in Ortsbeton beträchtlich ist, hingegen bei vorfabrizierten Betonelementen, die vor der Verwendung längere Zeit gelagert wurden, weniger ins Gewicht fällt. Zu diesem letzteren Fall macht der Autor ausführliche Berechnungen. Er bestätigt dabei die Richtigkeit der klassischen Methode, die einen reduzierten Elastizitätsmodul verwendet.

Betreffend des Themas Vb scheint es ratsam, keinerlei Schlußfolgerungen zu ziehen, sondern lediglich auf den Generalbericht zu verweisen.

Bemerkung: In Übereinstimmung mit dem Vorschlag im Generalbericht wurde während des Stockholmer Kongresses ein Unterausschuß der IVBH-Arbeitsgruppe III geschaffen, um Probleme im Zusammenhang mit vorfabrizierten Konstruktionen mit größerer Systematik zu untersuchen.

Va1

Development of Continuity in Precast Prestressed Construction

Développements dans la réalisation de la continuité dans des ouvrages préfabriqués en béton précontraint

Herstellung der Durchlaufwirkung in vorfabrizierten, vorgespannten Bauwerken

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Continuity in Precast Bridge Construction

The precast prestressed concrete girder combined with a cast-in-place deck slab has been used extensively in highway bridge construction in recent years. The majority of these bridges have consisted of a series of simple spans. Since continuity between adjacent spans leads to well-recognized advantages, a study was made of various methods of creating continuity in precast prestressed concrete bridges. Several forms of continuity connection were considered [1], and it was decided to investigate in detail that form of connection in which continuity is created for live loads only by the embedment of deformed rein-

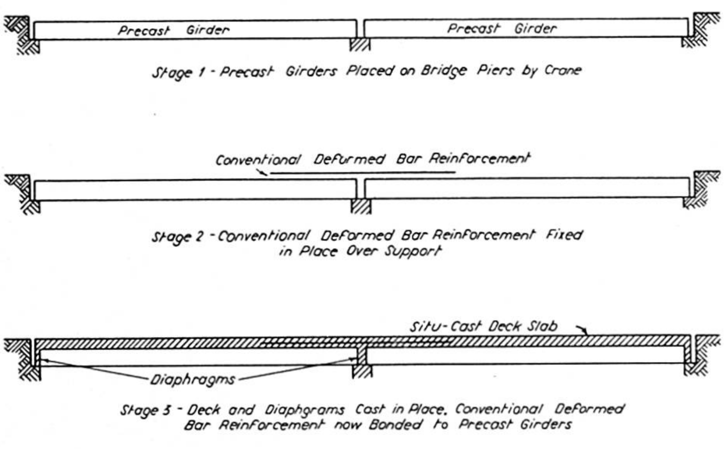

Fig. 1. Sequence of Construction of Continuous Precast Bridge.

forcing bars longitudinally in the cast-in-place deck slab over the interior supporting piers.

The sequence of construction when this form of continuity connection is used is set out in Fig. 1. The precast prestressed girders are placed on the bridge piers with the ends of girders in adjacent spans about six inches apart. The formwork for the deck slab and diaphragms is then erected, being supported by the precast girders. Conventional deformed reinforcing bars are next placed longitudinally in the deck slab over the intermediate piers. The transverse diaphragms, which enclose the ends of the girders and fill the space between them, are then concreted, together with the deck slab.

Experimental Program

The type of continuous construction described above combines precast, cast-in-place, prestressed, and normally reinforced concrete. The use of such a combination gives rise to several questions regarding the properties of structural concrete not covered by previous tests. A program of experimental work was therefore drawn up in nine stages to seek answers to these questions.

1. Girder Continuity

The first stage of the program was to check the behavior and strength of this type of connection when subject to static loading. As a case of extreme severity, a connection between girders without end blocks and without deflected strands was considered. In this case the precompression of the bottom flange at the ends of the girders is a maximum. The influence of prestress in the

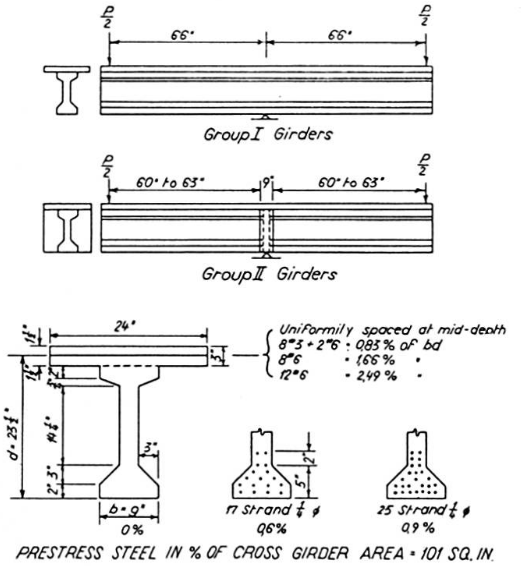

Fig. 2. Cross-Sections and Loading Arrangements of Test Specimens in Stage I.

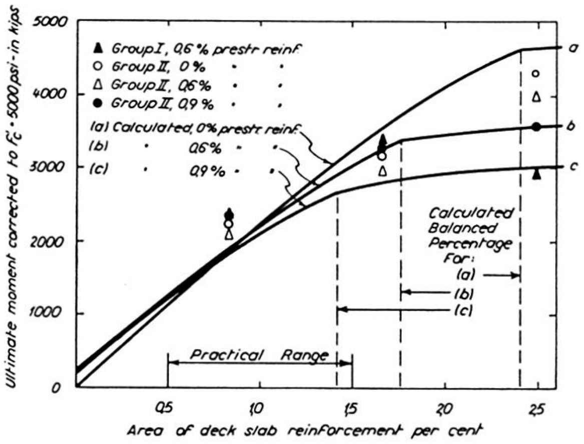

Fig. 3. Ultimate Moments for Groups I and II.

precast girders on the behavior of the connection would, therefore, be at a maximum.

In this first stage of the experimental program specimens were tested of the type shown in Fig. 2. The girders were supported at the mid-point and loaded at the two ends so as to produce negative bending. Loading was continued until failure occurred. The deck slabs contained various amounts of bar reinforcement, and three levels of prestress were used in the girders. The results of both groups of tests are shown in Fig. 3, where they are compared with the calculated relationships between ultimate strength of connection and the amount of continuity reinforcement in the deck slab. In these tests the amount of normal reinforcing bars in the deck, expressed as a percentage of (depth of girder x width of bottom flange), was 0.83, 1.66, or 2.49%. The initial precompression of the bottom flange of the precast girders was 0, 2100, or 3200 psi. The ultimate strength of each connection measured in the tests was compared with the ultimate strength calculated neglecting any influence of precompression of the bottom flange of the precast member, and also with the strength calculated taking this precompression into account. It was concluded that, for the practical range of continuity reinforcement of from 0.5 to 1.5%, the influence of precompression of the bottom flange may be neglected in the calculation of the ultimate strength of this type of connection.

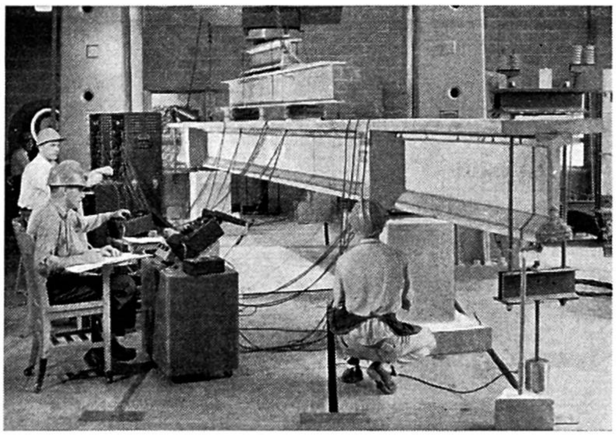

Fig. 4. Test of Continuous Girder from Group III.

Another group of specimens consisted of precast prestressed girders 22 ft. long joined to 6 ft. long precast girder stubs at their ends by deformed reinforcing bars placed in a deck slab cast on top of the precast girders. The 22-ft. spans were loaded at their third-points and loads were applied at the ends of the cantilevers so that no rotation of the ends of the 22-ft. spans occurred. A test is shown in progress in Fig. 4. The specimens behaved according to the elastic theory at service load level, and at ultimate strength redistribution of moments occurred so that ultimate strength as calculated by limit design was attained in all cases. It was therefore concluded that this type of connection can be designed to behave satisfactorily both at service load level and at ultimate strength.

2. Horizontal Shear Connection

The transfer of horizontal shear between the precast girder and the cast-in-place deck slab is of vital importance in composite construction. The second stage of this experimental program was therefore concerned with the effectiveness of various means of horizontal shear transfer. The means considered as variables in the tests were: adhesive bond, roughness, stirrups, and shear keys.

Pilot tests were first carried out in which a short length of deck slab was pushed off a section of precast girder, as may be seen in Fig. 5. The slab was cast in a horizontal position and the specimen was rotated through 90° before test. The load was applied to the slab in a direction parallel to the joint, the

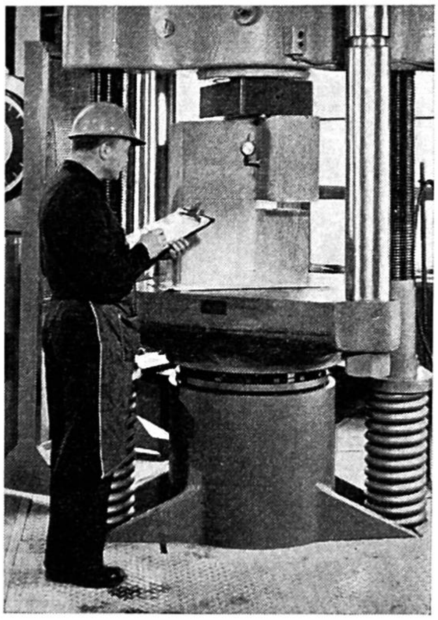

Fig. 5. Horizontal Shear Test using "Push-Off" Specimen.

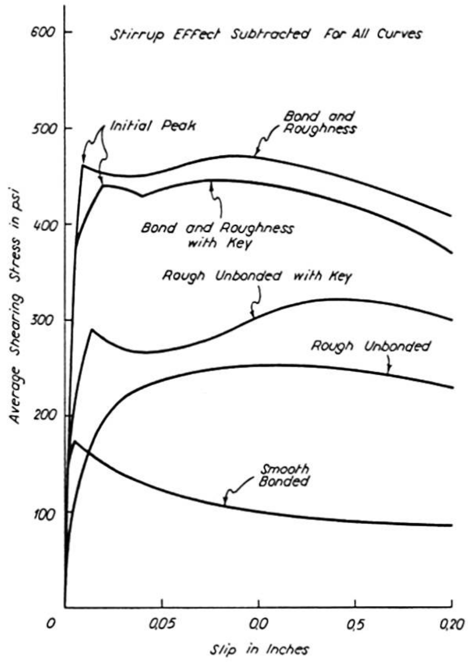

Fig. 6. Typical Shear-Slip Curves for "Push-Off" Tests.

line of action of the load being 1 in. from the contact face of the joint. The slip of the slab relative to the precast girder was measured using 0.0001 in. dial gauges. Typical shear-slip curves obtained in these tests are shown in Fig. 6.

The push-off tests were followed by tests of composite girders in which the slip of the deck slab relative to the precast girder was measured by a series of dial gauges along the length of the girders. The shear-slip curves obtained in the girder tests were found to have the same form as those obtained in the push-off tests, and to be closely related quantitatively.

From a study of the beam deflection curves, it was concluded that true composite action breaks down when the maximum slip reaches a value of about 0.005 inch. The girder tests indicate a maximum horizontal shearing stress for composite action of 500 psi for a rough bonded contact face, and 300 psi for a smooth bonded contact face. These limiting stresses correspond to concrete compressive strengths of 3000 and 5000 psi for the slab and girder respectively.

The tests demonstrated clearly that shear keys used with a rough bonded contact surface do not increase the strength of the connection. The slip required to develop the strength of the shear keys cannot occur until the bond between the precast and situ-cast concrete has been broken. On the other hand, stirrup reinforcement used in conjunction with a rough bonded contact surface was found to increase the limiting shear stress. For the half-inch diameter stirrups used, the increase in shearing stress was approximately 175 psi for each per cent of stirrup reinforcement crossing the contact surface.

The stage of the test program is described in detail in PCA Development Department Bulletin No. D 35, "Horizontal Shear Connections" [2].

3. Bridge Design Studies

At this stage, a tentative design was prepared for a two-span continuous highway bridge to carry the AASHO standard H 20-S 16 loading. This structure is shown in outline in Fig. 7. The design of the continuity connection,

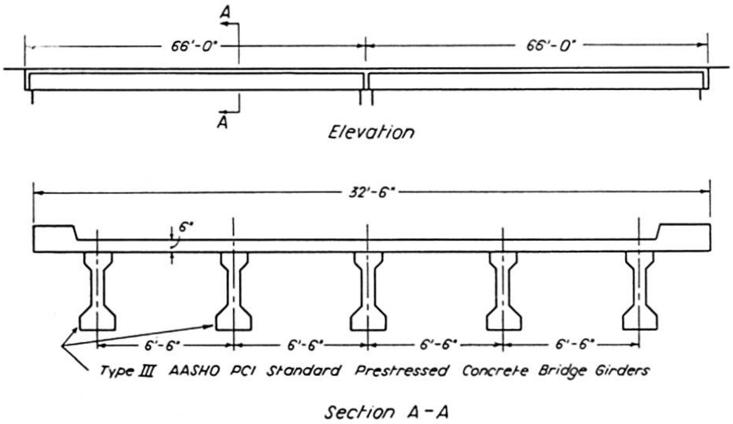

Fig. 7. Design Study Bridge.

and of the horizontal shear connection, were based on data obtained in the first two stages of this experimental program. As far as was possible, all other phases of the design were carried out in conformity with the American Association of State Highway Officials' "Specifications for Highway Bridges", or in accordance with the recommendations of the ACI-ASCE Committee on Prestressed Concrete. A two-span bridge was chosen for this study in order that it could be reproduced at half-scale on the test floor at a later stage in the experimental program. However, certain problems were considered which are inherent in the application of this form of construction to bridges of more than two spans. These design studies brought out the need for additional test data, and further stages of the experimental program were planned. This phase of the investigation, together with stages 4, 7, and 8 of the experimental program, is described in detail in PCA Development Department Bulletin D 43 [3].

4. *Flexural Strength*

It was decided next to extend the work carried out in the first stage of the investigation by testing a half-scale model of a single continuous girder. The member tested represented one girder and its portion of deck slab taken from the bridge design studied in stage three. The girder was loaded by hydraulic rams in a manner designed to simulate the distribution of loads in the H 20-S 16 equivalent lane loading. The loads were arranged so as to result in the most severe bending moment conditions at the center support section of the girder. The connection behaved in a satisfactory manner, and full redistribution of bending moments was achieved in the girder at ultimate strength. The variation of center support moment with applied load throughout the test is shown in Fig. 8. The maximum load sustained by the girder was approximately seven

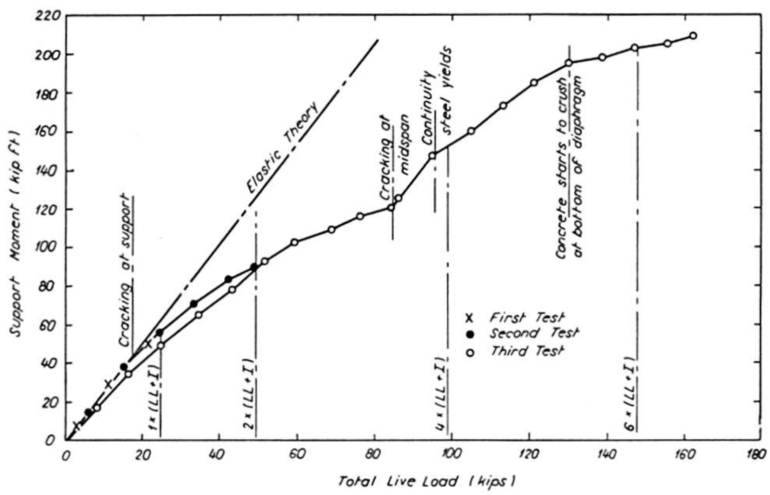

Fig. 8. Test of Continuous Composite Prestressed Bridge Girder, Variation of Center Support Moment with Increase in Load.

times the design load, plus the self-weight of the girder. It appears that substantial economies could arise from the development of a form of limit analysis suitable for use in bridge design, and also if some tension were permissible at the bottom face of a prestressed girder under design load. Further research is necessary on both these subjects before recommendations could be made which would cover all eventualities.

5. *Shearing Strength*

In the region of the continuity connection the girders were designed for flexure as reinforced concrete members, so the question arose as to how this same region of the girders should be designed for shear. Half-scale composite girders were loaded by a group of point loads to simulate the distribution of wheel loads of the standard H 20-S 16 design vehicle. The girders tested were single-span girders with a tied-down cantilever at one end. By suitably varying the tie-down force at the end of the cantilever independently from the vehicle loads, it was possible to simulate in the single span the conditions which would exist in one span of a two-span continuous girder, when subject to the same external loads in each span. A typical test is shown in progress in Fig. 9. The

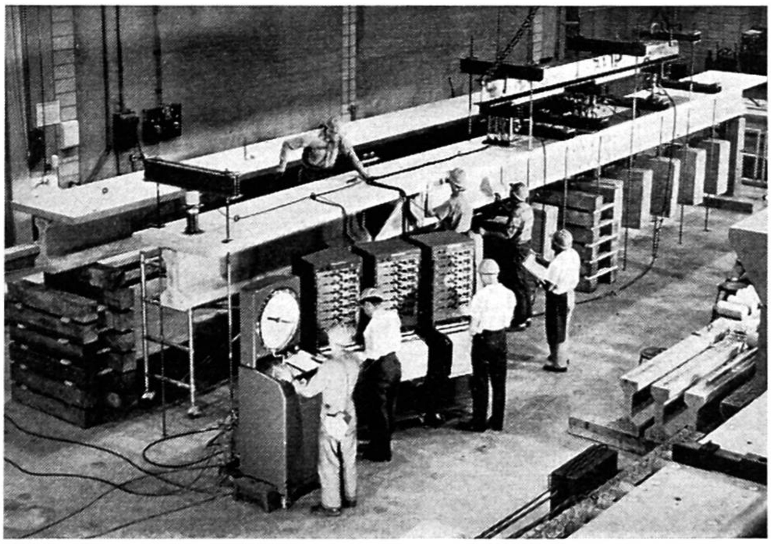

Fig. 9. Shear Test of Composite Prestressed Girder.

blocks seen hanging from the girder were used in many of the tests so that the dead-load stresses would be the same in the half-scale model as in the full-scale prototype. The variables considered in the shear tests were the amount of web reinforcement and the location of the applied loads.

Results from tests of 16 girders indicate that the presence of prestress in the precast member has a beneficial influence on the ultimate shear strength of the composite continuous girder. Flexural cracks due to negative bending at the support did not appear to accelerate the formation of the diagonal tension cracks which subsequently led to failure of the girders. This stage of the program is fully described in PCA Development Department Bulletin D 45, "Shear Tests of Continuous Girders" [4].

6. *Creep and Shrinkage Studies*

Two 66-ft. long continuous girders, similar to the one tested to destruction in stage (4), were subjected for approximately two years to sustained loading which simulated dead load of the prototype. One of the girders was provided with a connection designed to resist reverse moments at the center support. Extensive measurements were made of support reactions, girder deflections, and steel and concrete strains. Periodically the continuity behavior of the girders was checked by loading them and measuring the support reactions.

At the end of two years the girders were loaded to destruction in the manner described in stage (4). The ultimate strength of the girders was not affected adversely by the creep and shrinkage effects, but from the observations made it was concluded that to ensure fully continuous behavior at service loads after an extended period of time means should be provided to resist reverse moments set up at the interior support sections due to creep deformation of the precast prestressed girders. This can be readily accomplished by the use of suitable details.

7. *Repeated Loading*

Earlier work in this experimental program demonstrated the capacity of this type of connection to resist static loads. This work was supplemented by a study of the behavior of a typical connection when subject to repeated loading. The specimens used simulated at half-scale that part of a girder which extends 20-ft. either side of the center support, as taken from the design study bridge. In these tests, the only variable considered was the maximum value of the pulsating load. The specimens were supported at the diaphragm, and loaded at their ends by Amsler pulsating load rams, as may be seen in Fig. 10. The connection was designed to have a static ultimate flexural strength 2.5 times the service load moment.

The number of applications of load necessary to cause failure for various values of the upper load limit are shown in Fig. 11. In all cases the lower load

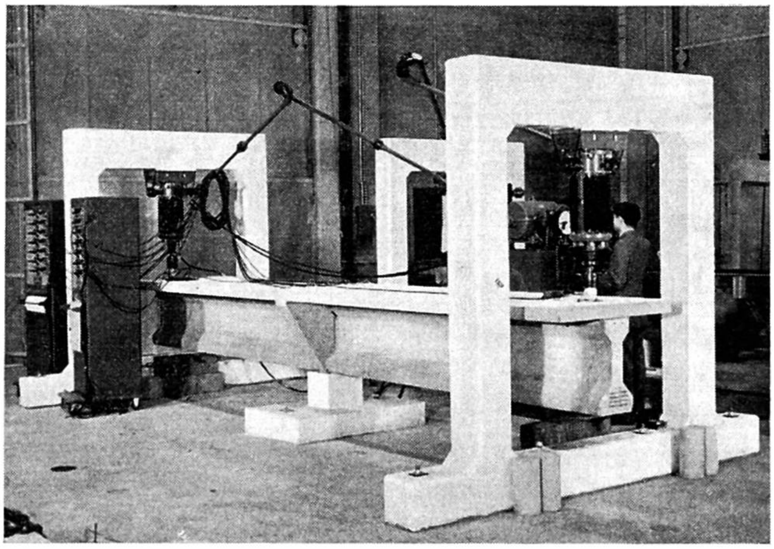

Fig. 10. Repeated Load Test of Connection between two Precast-Prestressed Girder Stubs.

limit was 0.28 times the service load moment. The failures were in every case due to fatigue failure of the continuity reinforcement. The bottom flanges of the girders remained sound despite the very high compressive stresses. It appears from these tests that repeated loading at service load level should not adversely affect the performance of this type of connection.

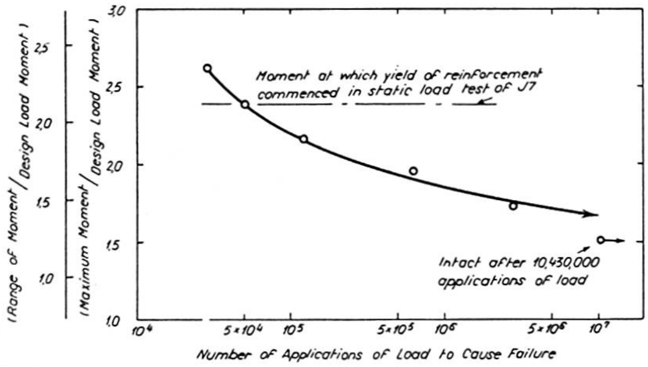

Fig. 11. Repeated Load Tests of Continuity Connection, Variation of Ultimate Moment with Number of Load Applications.

8. Reverse Bending

Continuous bridges of more than two spans, when subject to certain distributions of load, can develop positive moments at interior girder support sections. It has also been seen that positive moments can result from the effects of creep of the precast girders. Dynamic and static tests were therefore made on connections designed to resist this moment. A joint utilizing reinforcing bars embedded in the bottom flanges at the ends of the girders and welded to structural steel angle sections was found to be capable of developing the yield strength of the reinforcing bars and to have a satisfactory behavior as regards cracking.

9. Bridge Test

The final stage in this experimental program was a test of a complete half-scale model of the bridge considered in the design study. The testing of the bridge was carried out in two stages. Firstly, tests were carried out at service load level to check the continuity behavior of the structure, and also to investigate the extent of transverse distribution of concentrated loads applied to the bridge deck. A typical test is shown in Fig. 12. The 20,000-lb. dead weight loading block was placed at fifty locations on the bridge deck resting on a 12-in. square pad. Extensive measurements were made of deflections, strains, and reactions for each location of load. This type of test was carried out for three conditions of the deck slab: a) in its initial uncracked condition, b) after transverse cracks had been produced in the deck in the region of the center support, and c) after longitudinal cracks had been formed in the deck slab between all girders. The distribution of deflections in the last test series, when the loading block was placed at midspan over various girders, is compared in Fig. 13 with the distribution of deflections calculated using the GUYON-MASSONNET [5, 6, 7] theory. It is seen that there is good agreement between the experimental points and the theoretical curve taking into account the torsional stiffness of the bridge members. The degrees of continuity measured in these three series of tests were 92.3%, 86.8%, and 91.2% of the

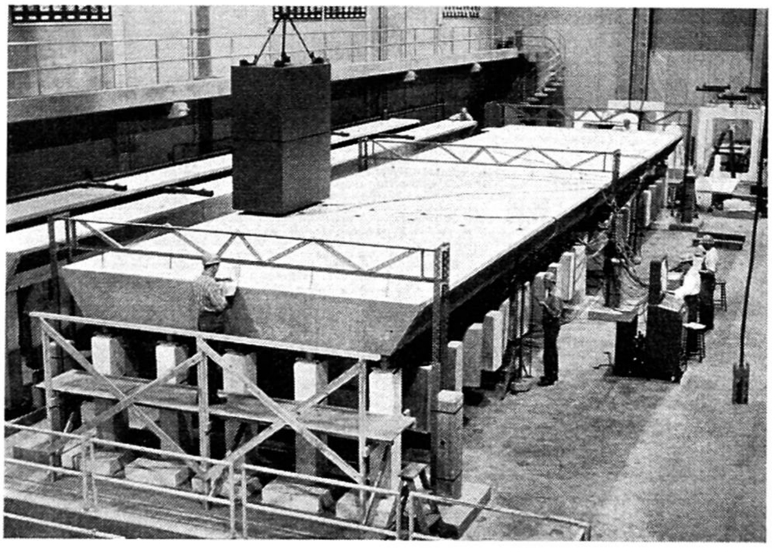

Fig. 12. Influence Surface Test of Half-Scale Highway Bridge.

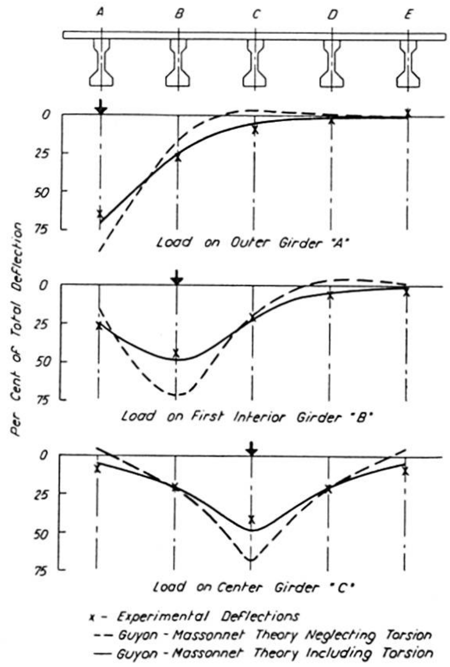

Fig. 13. Concentrated Load Tests — Series 3. Lateral Distribution of Deflections — Load at Midspan.

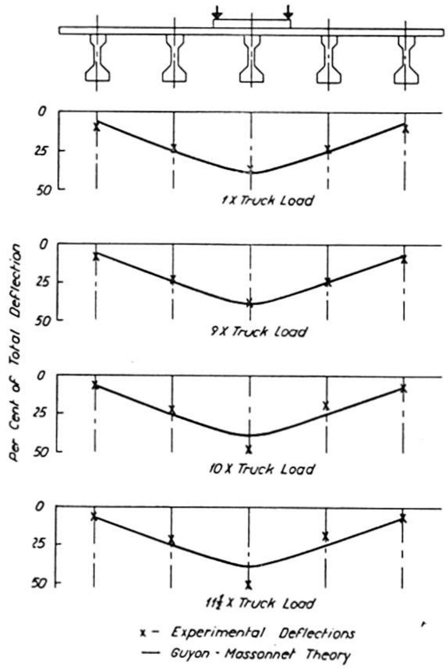

Fig. 14. Final Test, Lateral Distribution of Deflections.

theoretical continuity assuming constant stiffness of the structure over its entire length.

The second stage in the testing of the bridge involved a series of overload tests, culminating in a test to destruction by applying a group of four point loads centrally in one span. The grouping of loads used in this test was intended to simulate the grouping of loads in the equivalent military vehicle loading used in bridges on the Interstate Highway System. This loading system was chosen so that the lateral distribution of heavy loads could be examined, not to check the overall load factor for the structure. In these tests also, extensive measurements were made of strains, deflections, and reactions. The distribution of deflections for various levels of loading in the final test are shown in Fig. 14. The measured deflections are compared with the distribution of deflections calculated using the GUYON-MASSONNET theory including the effect of torsional stiffness. Excellent agreement was observed until the load reached 10 times the equivalent vehicle load, at which time a diagonal tension crack appeared in the slab around the loading heads. Further increase in load led to punching shear failure of the deck slab, which in turn resulted in a local failure of the precast girder below the punched-through zone, at a load of 11.5 times the equivalent vehicle load.

Concluding Remarks, Bridge Project

The soundness has been demonstrated, by extensive experimental investigation of that form of bridge construction in which continuity is created between precast prestressed girders by placing deformed bar reinforcement longitudinally in the deck slab across the intermediate piers. Results obtained during the tests are being studied to the end that design criteria may be evolved for the various aspects of the design of such a structure. The test program, and the conclusions drawn from it, are being reported in a series of Portland Cement Association Development Department Bulletins. The title of the series is, "Precast-Prestressed Concrete Bridges". Parts 1, 2, 3, and 4 have been published as Bulletins D 34 [1], D 35 [2], D 43 [3], and D 45 [4]. Other parts will be published in the near future.

Application to Building Construction

Continuous construction is fully as desirable in building construction as in bridge construction. A project has been initiated, therefore, to examine the applicability of this type of continuity connection to a construction consisting of precast prestressed double-tee members combined with a cast-in-place topping.

The first specimens tested consist of two short double-tee members joined together by a cast-in-place diaphragm around their abutting ends, and with conventional deformed reinforcing bars over the joint in the cast-in-place topping. These specimens were supported under the diaphragm, and were loaded at each end. Specimens so far tested indicate that the conclusions drawn from the tests of the connections of I-shaped girders are equally applicable to connection between the double-tee type of member.

The continuity behavior of this form of construction at various levels of loading is also being investigated using specimens consisting of two 27-ft. spans continuous with 6-ft. tied down cantilevers at each of the outer ends. Conditions in the 27-ft. spans simulate those existing in the center two spans of a six span continuous system loaded over the two center spans only. Testing of these specimens is only now commencing. The variables to be investigated are the design support moment for a given total load to be carried by the system, and use of various degrees of prestress in the double-tee members. In all cases the continuous members will be designed to give a constant over-all load factor, by the adjustment of the midspan design moment according to the principles of limit design.

Application of this type of connection to building construction results in sounder and more economic structures than those made from precast concrete members which do not utilize continuity.

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2. HANSON, N. W., "Precast-Prestressed Concrete Bridges, Part 2 — Horizontal Shear Connections". Portland Cement Association, Development Department Bulletin D 35.
3. MATTOCK, A. H., and KAAR, P. H., "Precast-Prestressed Concrete Bridges, Part 3 — Further Tests of Continuous Girders". Portland Cement Association, Development Department Bulletin D 43.
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5. GUYON, Y., «Calcul des ponts larges à poutres multiples solidarisées par des entretoises». Annales des Ponts et Chaussées. Paris 1946, September-October, pp. 553—612.
6. MASSONNET, C., «Méthode de calcul des ponts à poutres multiples tenant compte de leur résistance à la torsion». Publications, International Association for Bridge and Structural Engineering, Zurich 1950, Vol. 10, pp. 147—182.
7. MASSONNET, C., «Compléments à la méthode de calcul des ponts à poutres multiples». Offset Delporte, Mons.

Note: Since preparation of this paper was completed the following additional Bulletins have been published, which describe the final phases of the investigation.

8. MATTOCK, A. H., "Precast-Prestressed Concrete Bridges, Part 5 — Creep and Shrinkage Studies". Portland Cement Association, Developement Department Bulletin D 46.
9. MATTOCK, A. H., and KAAR, P. H., "Precast-Prestressed Concrete Bridges, Part 6 — Test of a Half-Scale Highway Bridge Continuous Over Two Spans". Portland Cement Association, Development Department Bulletin D 51.

Summary

The Research and Development Laboratories of the Portland Cement Association have conducted a comprehensive laboratory investigation of a simple method of developing continuity for live loads in precast prestressed concrete bridge construction. The type of bridge selected for study includes precast I-shaped girders, pretensioned with straight 7-wire strands without the use of end blocks. Continuity from span to span is obtained by the use of diaphragms at the girder ends together with a situ-cast deck slab which contains deformed bar reinforcement to carry the moments at intermediate supports. Tests of individual girder-slab members included studies of continuity performance, horizontal shear, diagonal tension, flexural strength, creep and shrinkage effects, fatigue effects, and reverse bending. Finally, a complete two-span, two-lane bridge was constructed at half scale in the laboratory and was tested to failure.

The findings of the bridge project have constituted a starting point for application of the same principles to building construction. A similar laboratory study of rather broad scope is now under way to investigate composite construction and continuity in the assembling of precast prestressed building elements.

Résumé

Le Laboratoire de Recherche et Développement de l'Association des Ciments Portland a entrepris de vastes recherches ayant pour but de trouver une méthode simple permettant de réaliser la continuité, pour les charges utiles, dans des ponts constitués d'éléments préfabriqués et précontraints. Ces recherches ont porté sur un type de pont comprenant des poutres en forme de I, précontraintes par des câbles comportant sept fils rectilignes sans tête d'ancrage. De travée en travée, la continuité est obtenue à l'aide de diaphragmes disposés aux extrémités des poutres et par une dalle de tablier, coulée sur place et munie de fers d'armature profilés servant à la reprise du moment de flexion au droit des appuis intermédiaires. Les essais sur des «poutres-dalles» isolées comprenaient des recherches sur la continuité, les efforts rasants, les tensions principales, la résistance à la flexion, le retrait et le fluage, la résistance à la fatigue et l'influence d'une flexion de signe inverse. Finalement, un pont entier à deux travées et deux voies, exécuté à l'échelle 1 : 2, fut sollicité jusqu'à la rupture, en laboratoire.

Les résultats obtenus ont constitué le point de départ pour l'application des mêmes principes à la construction des bâtiments. Des recherches similaires de grande envergure sont actuellement entreprises afin d'étudier la réalisation de la liaison avec des éléments coulés sur place et de la continuité dans les assemblages entre éléments de bâtiments précontraints et préfabriqués.

Zusammenfassung

Das Forschungslaboratorium des Portland-Zementkonzerns hat umfassende Untersuchungen in seinen Laboratorien ausgeführt, um eine einfache Methode zur Herstellung der Durchlaufwirkung für die Nutzlast in vorfabrizierten, vorgespannten Konstruktionen zu entwickeln. Untersucht wurde ein Brückentyp, dessen I-förmige Hauptträger mit 7-drahtigen Litzen ohne Ankerköpfe vorgespannt waren. Zur Herstellung der Durchlaufwirkung dienten an den Trägerenden angeordnete Scheiben sowie eine mit normalen Betoneisen bewehrte Ortsbetonfahrbahn, deren Armierung die Momente über den Zwischenstützen aufzunehmen hatte. Die an den einzelnen Plattenbalken, bestehend aus Hauptträger und Ortsbetonfahrbahn, ausgeführten Versuche umfaßten die Untersuchungen über die Güte der Durchlaufwirkung, die horizontale Scherbeanspruchung, die Hauptspannungen, die Biegefestigkeit, das Schwinden und Kriechen, die Ermüdungsfestigkeit und den Einfluß positiver Momente über den Stützen. Schlußendlich wurde eine vollständige zweifeldrige, zweispurige Brücke im halben Maßstab im Labor aufgebaut und bis zum Bruch untersucht.

Die aus diesen Versuchen gezogenen Folgerungen erlaubten, die gleichen Konstruktionsgrundsätze auf Hochbauten anzuwenden. Eine umfangreiche entsprechende Untersuchung wird nun unternommen, um die Verbundwirkung mit Ortsbetonelementen und die Durchlaufwirkung in den Anschlüssen von vorfabrizierten, vorgespannten Hochbauelementen zu ermitteln.

Va2

Discussion - Diskussion - Discussion

Assemblages des éléments dans les constructions composées préfabriquées (C. Fernandez Casado, L. Huarte Goñi, Va1) [1]

Verbindung der Konstruktionsteile bei zusammengesetzten vorfabrizierten Bauten (C. Fernandez Casado, L. Huarte Goñi, Va1) [1]

The Joining of Structural Members in Composite Prefabricated Structures (C. Fernandez Casado, L. Huarte Goñi, Va1) [1]

D. H. NEW
London, Great Britain

The Paper by CASADO and GOÑI is of particular interest as it gives an opportunity to discuss in detail some basic principles relating to the formation of joints between pre-cast concrete members. To those closely concerned with the development of this type of construction careful attention must be focused on the necessity for developing joints that give adequate strength and rigidity combined with economy and speed of erection.

The Paper states that the left-hand joint shown in Fig. 4 is made monolithic by welding and grout injection. While I would agree with grouting, preferably in the form of fine concrete, for the top part of the joint, the tension zone, I would not recommend this procedure for the bottom part, the compression zone. My proposal would be that having positioned the beam on its bearing on suitable packers and carried out the welding, grout-tight formwork should be fixed to cover the bottom of the vertical joint and vertically on each side an inch or so, to the top of the steel only, the void thus formed being filled with a stiff grout. The remainder of the lower vertical joint and the horizontal bearing can then be formed by means of dry packing and the workman will be able to see the efficiency of his work at all stages. My proposals

[1]) Voir «Publication Préliminaire» — siehe «Vorbericht» — see "Preliminary Publication", p. 731.

would, of course, apply if high compression were expected, if not, grouting would be quite adequate.

Presumably the Authors' intention is to make the joint shown in Fig. 9 fully rigid. Rigidity of joint can only be obtained by tightening the bolts sufficiently to give good contact between the concrete surfaces but as there must be adequate clearance to allow the beam to enter the forked end, this tightening would tend to crack the forks unless the clearance space had been taken up by grouting or packing. Furthermore, for complete rigidity it would be necessary to grout up the bolt hole clearance spaces unless high tensile bolts tightened by means of a torque wrench were used. There do, therefore, appear to be site expenses which might well bring the cost of the joint up to that which would be obtained if the joint were made by reinforcing bars from the ends of the joint-members projecting into a gap filled with a rapid-setting and hardening concrete mixture. Perhaps the Authors could give some information on their site experience with these joints and their views on the economy of their use.

It is gratifying to note that the example of a freely-supported joint shown in Fig. 13 indicates that the space between the main beam and the end of the supported beam is left entirely void. Any tendency towards filling voids of this description should be strongly resisted as not only is it entirely unnecessary but can be dangerous and might well act against the development of this form of construction. Several examples have come to my notice where this type of joint has been filled and the mortar has eventually dropped out, in some cases I trust, on the head of the Designer responsible.

To obtain continuity, frequent use is made by the Authors of butt welding of reinforcing bars projecting from individual pre-cast members. While this is obviously a most convenient method on the Drawing Board, considerable cost can be incurred on site if the precast members with their protruding steel are not jig made to very close tolerances. Some notes would be helpful on the order of the precision which the Authors have obtained in practice and the means used, together with the steps taken on site on the occasions when the distances between the butting ends of reinforcing rods were too great for straightforward welding.

Professor Wästlund makes an early mention in his General Report of the full duties of a Designer of pre-cast concrete structural members. It must be appreciated that it is not good enough for the Designer merely to design a structural element in its final position and entirely ignore fabrication, lifting off the beds, transport and erection. There have been too many examples where manufacturers have had to re-design units before they could be safely cast and handled. As we tend to construct pre-cast units of ever increasing size, it cannot be over-emphasised that if the Designer is carrying out his proper function, he must consider fully, manufacture, handling off the casting beds, transport to site and erection, in addition to the often comparatively simple matter of the design of the unit in its functional position. Full liaison between the Designer, the Fabricator and the Erector is essential if proper economy is to be assured.

Summary

Mr. NEW commented on various details of the joints between pre-cast elements shown in the Paper by CASADO and GOÑI. He emphasised the point made by Professor WÄSTLUND that the Designer should design any unit for handling in all its stages of manufacture and erection and not merely in its final position.

Résumé

L'auteur discute divers détails relatifs aux assemblages entre éléments préfabriqués, problèmes dont traite le rapport de MM. CASADO et GOÑI. Il insiste tout spécialement sur le fait, déjà relevé par M. le Professeur WÄSTLUND, que l'ingénieur doit tenir compte de tous les états de fabrication et de montage et ne peut se limiter à considérer les sollicitations de l'ouvrage terminé.

Zusammenfassung

Herr NEW behandelt verschiedene Einzelheiten von Stößen vorfabrizierter Elemente, die im Beitrage der Herren CASADO und GOÑI gezeigt werden. Wie Herr Professor WÄSTLUND, betont er nachdrücklich, wie wichtig es sei, daß der projektierende Ingenieur nicht nur den Endzustand eines Elementes vor Augen habe, sondern auch sämtliche Fabrikations- und Montagezustände erfasse.

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Vb1

Redistribution of Stresses in a Continuously Supported Beam, due to Creep

Redistribution due au fluage des contraintes dans une poutre sur appui continu

Kräfteumlagerungen in einem Balken auf durchgehender Bettung infolge Kriechen

J. N. DISTEFANO
Rep. Argentina

I. Introduction

In a so-called "foundation beam", the deformations due to creep will produce a variation of the reaction of the support, and consequently a redistribution of the stresses in the beam will take place.

In the present paper, this redistribution has been studied, taking into account not only the creep of the beam, but also a possible creep of the support. We shall assume linear behaviour of creep, i. e., the case in which creep strains are proportional at any instant to the applied stress.

Generally, linear or non-linear creep behaviour are studied by means of standard creep tests at constant stress; but it is important to note that such tests are not sufficient to assure linear behaviour at different rates of stress.

To admit linear behaviour is the same as to admit the generalized principle of superposition in BOLTZMANN's sense. So, it will be possible to investigate linear behaviour by means of superposition.

For materials like synthetic plastics, many papers, and our own experiences [1] show that the creep of these materials follows very closely the principle of superposition.

For concrete, McHENRY[1]), ROSS, BLACKSTON etc., [2] [3] [4] have shown

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[1]) McHENRY was the first — according to our opinion — to realize that the creep recovery curves could be interpreted like the superposition of the separated effects of

that the superposition method gives excellent results. In Russia, GWOSDEW[2]) and others [5] used this method of work successfully.

In this order of ideas, the theory of creep effects can be deduced from VOLTERRA's theory of hereditary phenomena [6] which naturally implies the BOLTZMANN principle of superposition.

II. General Formulation

According to the previous considerations, it is easy to show that in those kinds of materials the effect of a certain solicitation can be expressed by means of VOLTERRA's mentioned theory. In our particular case it is easy to show [7] that the curvature of a bent beam at the instant t, subject to a variable bending moment $M(t)$ applied at the instant τ_0, can be expressed by

$$\mu = -\frac{\partial^2 y}{\partial x^2} = \frac{M(t)}{E I} + \int_{\tau_0}^{t} M(\tau) f_1(t,\tau)\, d\tau, \tag{1}$$

where E is the elastic modulus of the material of the beam, and I the inertia moment of the cross section.

The function $f_1(t,\tau)$ is called the creep function. It can be easily connected with the curves that we obtain experimentally performing creep tests at different ages of the material, by means of

$$f_1(t,\tau) = -\frac{\partial}{\partial \tau}\bar{\epsilon}_0(t,\tau), \tag{2}$$

where $\bar{\epsilon}_0(t,\tau)$ is the specific creep strain at the time t, obtained when a unit stress was applied at the age τ.

We shall consider also, a linear viscoelastic behaviour of the support. The deflections which will take place when a pressure $q(x,t)$ is applied at the instant τ_0 will be expressed by means of

$$y(x,t) = \frac{q(x,t)}{k} + \int_{\tau_0}^{t} q(x,\tau) f_2(t,\tau)\, d\tau, \tag{3}$$

where $f_2(t,\tau)$ is the creep coefficient of the support, and k the classical coefficient of elastic reaction.

If we call $p(x,t)$ the external load acting on the beam, we can write the following fundamental relation between bending, external loads, and reaction of the support

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an initial load and a negative one of the same intensity, applied at the instant of unload. In such a way — and many experiments confirm the criterion — it is proved that creep in concrete obeys the same law, for increasing and decreasing stresses.

[2]) The very importance of the paper of this author in 1943 is to show that FREYSSINET's ideas about deformation of concrete are congruent with the linear-integral formulation of VOLTERRA's theory of hereditary phenomena.

$$\frac{\partial^2 M}{\partial x^2} = -p(x,t) - q(x,t). \tag{4}$$

Eliminating $y(x,t)$ and $M(x,t)$ between eqs. (1), (3), and (4), we obtain the following integral equation

$$E I \int_{\tau_0}^{t} \frac{\partial^4 q}{\partial x^4} f_2(t,\tau)\, d\tau + \frac{E I}{k}\frac{\partial^4 q}{\partial x^4} + E \int_{\tau_0}^{t} q(x,\tau) f_1(t,\tau)\, d\tau + q(x,t) = - p(x,t) - E \int_{\tau_0}^{t} p(x,\tau) f_1(t,\tau)\, d\tau. \tag{5}$$

To solve this equation we can imagine that functions $p(x,t)$ and $q(x,t)$ are expressed by the following expansions of orthogonal functions

$$\begin{aligned} p(x,t) &= \sum_1^\infty a_i(t)\,\varphi_i(x), \\ q(x,t) &= \sum_1^\infty b_i(t)\,\varphi_i(x), \end{aligned} \tag{6}$$

where $\varphi_i(x)$ are the Eigen-functions of the following differential equation

$$\frac{d^4 \varphi_i}{d x^4} - k_i \varphi_i = 0. \tag{7}$$

It is noted that the φ_i-functions are orthogonal functions, when conditions of free, or hinged or built-in ends are satisfied. Then, the coefficients $a_i(t)$ can be immediately calculated as follows

$$a_i(t) = \frac{1}{D}\int_0^l p(x,t)\,\varphi_i(x)\, dx; \qquad D = \int_0^l \varphi_i^2(x)\, dx. \tag{8}$$

Concerning the $b_i(t)$-coefficients, substituting eqs. (6) into (5) and taking into account (7), the following integral equation for the $b_i(t)$-coefficients can be written

$$b_i(t) + \lambda_i \int_{\tau_0}^{t} b_i(\tau)\,[f_1(t,\tau) + I k_i f_2(t,\tau)]\, d\tau = g_i(t), \tag{9}$$

where $\lambda_i = \dfrac{E}{1+\dfrac{EI}{k}k_i}$ and $g_i(t) = -\dfrac{1}{1+\dfrac{EI}{k}k_i}\left[a_i(t) + E\int_{\tau_0}^{t} a_i(\tau) f_1(t,\tau)\, d\tau\right].$

To solve the preceeding integral equation we shall apply some restrictions to the functions $f_1(t,\tau)$ and $f_2(t,\tau)$ which correspond to two important cases of practical applications.

III. Case of Invariable Creep

We consider the case in which the creep of the materials is independent of the age of loading, that is to say that the creep functions $f_1(t,\tau)$ and $f_2(t,\tau)$ will depend only on the difference of the parameters $(t-\tau)$. In this way the integral eq. (9) has an elegant solution [1] by means of the Laplace transformation, as follows

$$b_i(t) = -\frac{1}{1+\frac{EI}{k}k_i} L^{-1} \frac{L a_i(t)[1+E L f_1(t)]}{1+\lambda_i L[f_1(t)+I k_i f_2(t)]}, \tag{10}$$

where L represents the Laplace transformations defined by

$$L f(t) = \int_0^\infty e^{-st} f(t)\, dt$$

and L^{-1} represents the inverse transformation.

The problem is formally solved in a very general way. If the law of variation of external loads, and the analytical expressions of the creep coefficients are given, we are able to calculate the $b_i(t)$ coefficients, using tables or direct inverting methods, by means of expression (10).

However, the most practical interest is to know the convergence of deflections, contact-pressures, etc., when $t \to \infty$ that is to say the asymptotic behaviour. In order to do this we have to investigate the asymptotic behaviour of the functions $b_i(t)$. A theorem of PALEY and WIENER on the VOLTERRA integral equation [8] affirms that the $b_i(t)$ coefficients given by eq. (9) will converge to the limit

$$b_i(\infty) = \frac{\lim\limits_{t\to\infty} g_i(t)}{1+\lambda_i \int_0^\infty [f_1(t)+I k_i f_2(t)]\, dt}, \tag{11}$$

if and only if

$$\lambda_i \int_0^\infty [f_1(t)+I k_i f_2(t)]\, e^{-st}\, dt \neq -1; \quad s \geqq 0.$$

This condition is always fulfilled because λ_i and the integrand are positive. Morover $b_i(\infty)$ will tend to a finite limit, only if $g_i(t)$ tends to a finite one. This last condition is naturally dependant on the convergence of the external loads $p(x,t)$. If we suppose that the external forces tend to the finite limit

$$\lim_{t\to\infty} p(x,t) = p_\infty(x) \tag{12}$$

the $a_i(t)$ coefficients given by eq. (8) will also tend to a finite limit $a_i(\infty)$. Consequently the limit

$$\lim_{t\to\infty} g_i(t)$$

may be obtained by means of the mentioned theorem of PALEY and WIENER, using the expressions of $g_i(t)$ given in (9),

$$\lim_{t\to\infty} g_i(t) = -\frac{a_i(\infty)}{1+\frac{EI}{k}k_i}\left[1+E\int_0^\infty f_1(t)\, dt\right]. \tag{13}$$

Substituting (13) into (11) we obtain the following expression for the $b_i(\infty)$ coefficients

$$b_i(\infty) = -\frac{a_i(\infty)}{1+\frac{EI}{k}k_i\frac{1+k\gamma_2}{1+E\gamma_1}}, \tag{14}$$

where γ_1 and γ_2 are

$$\gamma_1 = \int_0^\infty f_1(t)\,dt; \qquad \gamma_2 = \int_0^\infty f_2(t)\,d\tau.$$

If we substitute $b_i(\infty)$ given by (14) in the expression (6) we obtain

$$q(x,\infty) = -\sum_1^\infty \frac{a_i(\infty)}{1+\frac{EI}{k}k_i\frac{1+k\gamma_2}{1+E\gamma_1}}\varphi_i(x). \tag{15}$$

If we solve the purely elastic (classical) problem of the foundation by means of the orthogonal functions (6) we obtain the following expansion

$$q(x,0) = -\sum_1^\infty \frac{a_i(0)}{1+\frac{EI}{k}k_i}\varphi_i(x). \tag{16}$$

Comparing (15) and (16) we find that the asymptotic solution of the visco-elastic problem is coincident with the solution of the purely elastic problem, in which the elastic modulus E and the coefficient of reaction of the support are substituted by the following effective values:

$$E^* = \frac{E}{1+E\gamma_1}; \qquad k^* = \frac{k}{1+k\gamma_2} \tag{17}$$

and the asymptotic values of the external loads, instead of its initial values is used.

Hence, to investigate asymptotic behaviour, when the creep of a material is independent of the age (invariable creep) the so-called method of "effective modulus" is perfectly correct.

IV. Concrete Beam on an Elastic Support

We shall study only the case in which the support is an elastic one, and the loads remain constant after application. The integral eq. (9) will be reduced to

$$b_i(t) + \lambda_i \int_{\tau_0}^{t} b_i(\tau) f_1(t,\tau)\,d\tau = -\frac{a_i}{1+\frac{EI}{k}k_i}\left[1+E\,\bar{\epsilon}_0(t,\tau_0)\right]. \tag{18}$$

It is known that creep in concrete depends on the age; generally this influence of age is studied by means of standard creep tests at constant stress and different ages of the concrete. We shall have a certain number of tests that will give us the specific creep curves $\bar{\epsilon}_0(t,\tau)$ obtained at different ages τ. Experience shows that diminution of creep with the age is an asymptotic phenomenon.

Many authors have proposed formulas in order to represent analytically the specific creep $\bar{\epsilon}_0(t,\tau)$. Among the most important we shall mention the Dischinger formula [9] which considers the age dependence in the following form. The specific creep of the concrete at the age of loading is represented by means of the following function

$$\theta(t) = \gamma_0(1-e^{-\delta t}). \tag{19}$$

Then, it is assumed that creep of concrete, when the load is introduced at a certain time τ, can be represented by means of

$$\bar{\epsilon}_0(t,\tau) = \theta(t) - \theta(\tau). \tag{20}$$

That is to say, as we can see in Fig. 1, the creep produced by a load introduced at the instant τ can be obtained by translating vertically the creep curve given for the reference age $\tau = \tau_0$.

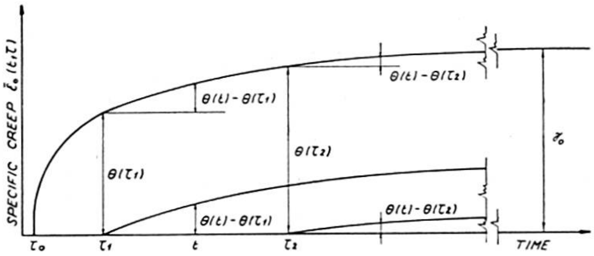

Fig. 1.

This assumption has one inconvenience, namely, the intensity of creep tends to zero when the age of concrete τ tends to infinite. Generally concrete always presents creep, and the experiments show that the intensity of creep is lower when age increases, but tending asymptotically to a limit when $\tau \to \infty$. In other words, function $\bar{\epsilon}_0(t,\tau)$ will tend asymptotically towards a not null function of the type $F(t-\tau)$ when $\tau \to \infty$.

For this reason the Dischinger formula gives always a pessimistic evaluation of the creep effects, when new loads are introduced in the structures at different ages [10].

In order to represent more closely the dependance of creep on age, the following function was proposed [11]

$$\bar{\epsilon}_0(t,\tau) = \psi(\tau)\, F(t-\tau), \tag{21}$$

where ψ is an always positive function that decreases monotonously towards the finite limit $\psi(\infty) = \gamma_0$ and represents the gradual diminution of creep due to age. Function $F(t-\tau)$ is also positive and monotonously growing towards an upper limit equal to unity, for great values of the parameter $(t-\tau)$. Taking

$$\psi(\tau) = \gamma_0 + \frac{C}{\tau}, \qquad F(t-\tau) = 1 - e^{-\delta(t-\tau)} \tag{22}$$

and choosing convenient constants γ_0, c, δ it is possible to follow as closely as desired the creep of concrete and its dependence on the age. In Fig. 2 this situation is shown. It is noted that γ_0 represents the asymptotic value of the specific creep of an aged concrete, i.e., of a concrete loaded when $\tau \to \infty$.

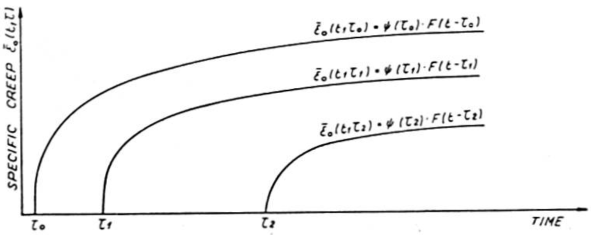

Fig. 2.

The creep function $f_1(t,\tau)$ can be calculated by means of eq. (2) and its values is

$$f_1(t,\tau) = -\frac{\partial}{\partial \tau}[\psi(\tau)(1 - e^{-\delta(t-\tau)})]. \tag{23}$$

Substituting (23) in (18) eventually leads to

$$b_i(t) - \lambda_i \int_{\tau_0}^{t} b_i(\tau)\psi'(\tau)\,d\tau + \lambda_i \int_{\tau_0}^{t} b_i(\tau)(\psi' + \delta\psi)e^{-\delta(t-\tau)}\,d\tau = h_i(t,\tau_0), \tag{24}$$

where h_i is the second member of (18).

Differentiating the preceeding equation with respect to t,

$$b_i'(t) + \lambda_i \psi(t) b_i(t) - \delta\lambda_i \int_{\tau_0}^{t} b_i(\tau)(\psi' + \delta\psi)e^{-\delta(t-\tau)}\,d\tau = h_i'(t,\tau_0). \tag{25}$$

Eliminating the integral

$$\int_{\tau_0}^{t} b_i(\tau)(\psi' + \delta\psi)e^{-\delta(t-\tau)}\,d\tau$$

between (24) and (25), and differentiating once again with respect to t, the following differential equation is obtained

$$b_i''(t) + \delta[1 + \lambda_i\psi(t)]b_i'(t) = 0 \tag{26}$$

with the following boundary conditions

$$b_i(\tau_0) = \frac{a_i}{1 + \frac{EI}{k}k_i}, \tag{27}$$

$$b_i'(\tau_0) = \frac{\frac{EI}{k}k_i}{\left(1 + \frac{EI}{k}k_i\right)^2} a_i E\delta\psi(\tau_0). \tag{28}$$

The general solution of eq. (26) is obtained by means of two integrations

$$b_i(t) = b_i(\tau_0) + b_i'(\tau_0) \int_{\tau_0}^{t} e^{-J_i(\xi)} d\xi, \tag{29}$$

where

$$J_i(t) = \delta \int_{\tau_0}^{t} [1 + \lambda_i \psi(\tau)] d\tau.$$

Substituting $\psi(\tau)$ by its equivalent (22), the preceeding integral is transformed into

$$J_i(t) = \delta(1 + \lambda_i \gamma_0)(t - \tau_0) + \ln\left(\frac{t}{\tau_0}\right)^{\lambda_i C\delta},$$

which substituted in (29) gives

$$b_i(t) = b_i(\tau_0) + b_i'(\tau_0) e^{\delta(1+\lambda_i\gamma_0)\tau_0} \tau_0^{\lambda_i C\delta} \int_{\tau_0}^{t} e^{-\delta(1+\lambda_i\gamma_0)\tau} \tau^{-\lambda_i C\delta} d\tau. \tag{30}$$

Introducing the incomplete gamma function $\phi(\alpha, t)$ defined by

$$\phi(\alpha, t) = \int_0^t e^{-\tau} \tau^{\alpha-1} d\tau,$$

eq. (30) can be written

$$b_i(t) = b_i(\tau_0) + b_i'(\tau_0) e^{r_i\tau_0} \tau_0^{1-\alpha_i} r_i^{-\alpha_i} [\phi(\alpha_i, r_i t) - \phi(\alpha_i, r_i\tau_0)], \tag{31}$$

where $\alpha_i = 1 - \lambda_i C\delta; \qquad r_i = \delta(1 + \lambda_i\gamma_0).$

The integral that appears in eq. (30) is convergent if and only if

a) $1 + \lambda_i \gamma_0 > 0,$
b) $1 - \lambda_i C\delta > 0.$

Condition a) is always satisfied because λ_i and γ_0 are positive. Condition b) is satisfied when

$$E C \delta < 1 + \frac{EI}{k} k_{min},$$

this inequality is generally fulfilled, because $E C \delta$ does not exceed in practical cases 0,5.

Now, if we substitute the value of the $b_i(t)$ coefficients given by eq. (31) in the second expansion (6) and substitute also the boundary values (27) and (28), we shall obtain

$$q(x, t) = q(x, \tau_0) - E\delta\psi(\tau_0) \sum_1^\infty \frac{a_i \frac{EI}{k} k_i}{\left(1 + \frac{EI}{k} k_i\right)^2} H_i(t) \varphi_i(x), \tag{32}$$

where

$$H_i(t) = e^{r_i\tau_0} \tau_0^{1-\alpha_i} r_i^{-\alpha_i} [\phi(\alpha_i, r_i t) - \phi(\alpha_i, r_i\tau_0)]$$

and $q(x, \tau_0)$ is the instantaneous contact pressure.

V. Comparison with the Dischinger Formula of Creep

At the beginning of the last section we have seen that, following DISCHINGER, we can write the specific creep by means of the following function

$$\bar{\epsilon}_0(t,\tau) = \theta(t) - \theta(\tau), \tag{33}$$

where $\theta(t)$ represents the creep curve obtained at the initial age. Function $\theta(t)$ can be represented by means of

$$\theta(t) = \gamma_0 (1 - e^{-\delta t}). \tag{34}$$

It follows that the creep function defined in (2) will be

$$f_1(t,\tau) = -\frac{\partial}{\partial\tau}\bar{\epsilon}_0(t,\tau) = \frac{d\,\theta(\tau)}{d\tau}. \tag{35}$$

Hence the creep function depends only on the variable τ.

Substituting expression (35) in the general integral eq. (18) we obtain

$$b_i(t) + \lambda_i \int_{\tau_0}^{t} b_i(\tau)\frac{d\,\theta(\tau)}{d\tau}\,d\tau = -\frac{a_i}{1+\frac{EI}{k}k_i}\left[1 + E\,\theta(t)\right].$$

Differentiating the preceeding equation with respect to the upper limit we shall obtain the following differential equation

$$b_i'(t) + \lambda_i b_i(t)\,\theta'(t) = -\frac{a_i}{1+\frac{EI}{k}k_i}\,E\,\theta'(t) \tag{36}$$

the solution of which is immediately obtained as follows

$$b_i(t) = -\left[1 - \frac{\frac{EI}{k}k_i}{1+\frac{EI}{k}k_i}\,e^{-\lambda_i\theta(t)}\right]a_i.$$

Substituting the preceeding expression in (6) we obtain the following expression for the contact pressure

$$q(x,t) = -\sum_{1}^{\infty}\left[1 - \frac{\frac{EI}{k}k_i}{1+\frac{EI}{k}k_i}\,e^{-\lambda_i\theta_i(t)}\right]\alpha_i\,\varphi_i(x). \tag{37}$$

In order to compare this solution with formula (32), we have solved a particular case represented in Fig. 3. A concrete beam, hinged at the ends and continuously supported on an elastic foundation, is loaded with a uniformly distributed load p_0 at $\tau_0 = 7$ days. It remains constant during three months. Then it is doubled, remaining thereafter constant during the following three months.

The specific creep was assumed to be

$$\bar{\epsilon}_0(t,\tau) = \left(0{,}9 + \frac{4.82}{\tau}\right)\left[1 - e^{-0.026(t-\tau)}\right]\cdot 10^{-5}.$$

so at 7 days it will be $\bar{\epsilon}_0(t,7) = 1{,}588\left[1 - e^{-0.026(t-7)}\right]\cdot 10^{-5}$.

In order to calculate with DISCHINGER's formula, creep at 7 days was assumed to be equal, that is to say, function $\theta(t)$ considered at 7 days will be

$$\theta(t) = 1.588\,[1 - e^{-0.026\,(t-7)}] \cdot 10^{-5}.$$

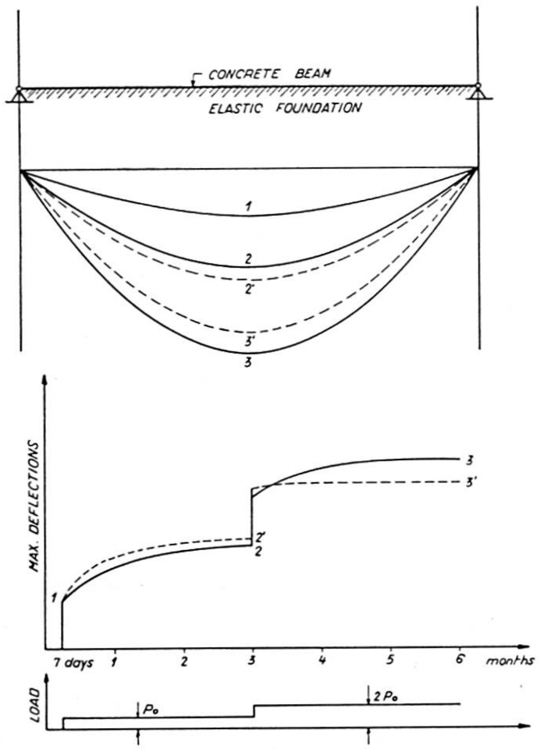

Fig. 3.

VI. Conclusion

In the third section we have seen that — to investigate asimptotic behaviour — the method of the effective modulus can be rigourously applied to structures the creep of which does not vary with the age of the materials. This conclusion [3]) is not only important for the more and more extended use of modern plastics, but also for the aged concrete, in which the specific creep, after a certain period of time — practically 4 or 5 months — tends to repeat at any age of loading the same creep curve. This observation is important when the structure is formed by pre-cast parts, long-time stored before use.

For young concrete, we think that the criterion used to represent the specific creep in Section IV permits us to evaluate more accurately creep effects than the DISCHINGER formula, which can only represent in one way the diminution of creep intensity due to age.

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[3]) This conclusion is valid for any non-homogenous structure — if the creep of each component material is invariable — (see the paper of the author "Sul comportamento asintotico di corpi viscoelastici a ereditarietà invariabile" in the "Atti dell'Accademia delle Scienze di Torino", Nov. 1960, Vol. 95).

List of Notations

M	Bending moment [kg cm].
μ	Curvature of the beam.
y	Deflection of the beam [cm].
E	Elastic modulus of concrete [kg cm^{-2}].
I	Inertia moment of the cross section of the beam [cm^4].
t	Time.
τ	Age of the viscoelastic materials.
$\bar{\epsilon}_0(t,\tau)$	Specific creep of concrete [cm^2 kg^{-1}].
$f_1(t,\tau)$	Creep function of concrete [cm^2 kg^{-1}/day].
$f_2(t,\tau)$	Creep function of the support [cm^2 kg^{-1}/day].
k	Coefficient of elastic reaction of the support [kg cm^{-2}].
k_i	Eigenvalues [cm^{-4}].
$p(x,t)$	External loads acting on the beam [kg cm^{-1}].
$-q(x,t)$	Reaction of the support [kg cm^{-1}].
E^*	Effective modulus [kg cm^{-2}].
k^*	Effective coefficient of reaction of the support [kg cm^{-2}].
γ_1	Asimptotic creep strain in aged concrete produced by a stress equal to 1 kg cm^{-2} [cm^2 kg^{-1}].
γ_2	Asimptotic creep deflection in the support produced by a distributed load equal to 1 kg cm^{-1} [cm^2 kg^{-1}].
$\phi(\alpha,t)$	Incomplete Gamma function.

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Summary

In the present paper the redistribution of stresses, due to creep in a foundation beam, is studied, considering not only the creep of the beam but also a possible creep of the foundation. Linear behaviour of creep is assumed, and the general integral-differential equation of deflections is solved for the most important cases of applications. Particularly, it is shown that when creep of materials composing the beam and foundation are independent of the age of loading, the problem is immediately solved by means of the classical (elastic) methods.

Finally, the case of materials, the creep of which varies with age, is studied, and a discussion of the Dischinger criterion of creep of concrete is made.

Résumé

L'auteur étudie la redistribution des contraintes dans une poutre de fondation, les matériaux de la poutre et de sa fondation étant considérés comme visco-élastiques. On a admis un comportement linéaire du fluage, conformément à la théorie des phénomènes héréditaires de Volterra. L'équation intégrale-différentielle régissant les déformations est résolue pour les cas d'application pratique les plus importants. L'étude a montré que, pour le calcul des déformations finales, la méthode du module effectif est parfaitement correcte quand le fluage du matériau n'est pas variable avec le temps.

Finalement, l'auteur étudie le cas du fluage variable avec l'âge du matériau et il discute le critère de fluage du béton proposé par Dischinger.

Zusammenfassung

Der vorliegende Artikel befaßt sich mit den durch das Kriechen bedingten Kräfteumlagerungen eines Fundationsträgers, wobei nicht nur das Kriechen des Trägers, sondern auch dasjenige des Untergrundes berücksichtigt werden. Zur Lösung der die Deformationen beherrschenden, allgemeinen Integral-Differential-Gleichung wurde ein linearer Zusammenhang zwischen Beanspruchung und Kriechen angenommen, wobei die Resultate für die wichtigsten Anwendungsfälle angegeben werden. Haben Träger und Untergrund ein vom Alter des Materials im Moment der Belastung unabhängiges Kriechmaß, so kann das Problem sofort mit Hilfe der klassischen (elastischen) Theorie gelöst werden. Zum Schluß wird der Fall des vom Alter des Materials abhängigen Kriechens untersucht sowie das Dischingersche Betonkriechkriterium diskutiert.

Discussion libre - Freie Diskussion - Free Discussion

Réponse à la communication de Mr. New [1])

Antwort auf die Mitteilung von Mr. New [1])

Reply to the Communication of Mr. New [1])

FERNANDEZ CASADO
Madrid

Nous remercions vivement Mr. New de l'attention prêtée à notre travail, et nous nous permettons de répondre aux trois observations faites dans sa communication.

La première concerne l'assemblage entre une poutre préfabriquée lourdement chargée et des consoles coulées sur place (voies de roulement des ponts-roulants du hall de laminage d'Ensides à Avilès). La partie supérieure du joint est remplie par des injections de mortier ou plutôt de béton à petits agrégats, puisque la largeur est d'environ 10 cm. On injecta ensuite du mortier jusqu'à mi-hauteur de la poutre afin de remplir tous les vides.

La deuxième observation concerne l'assemblage de poutres et de piliers préfabriqués par des boulons transversaux métalliques (fig. 9). L'assemblage fut terminé en injectant de la laitance afin de remplir les vides, de protéger le fer de l'oxydation et d'obtenir une bonne adhérence par contact entre les deux éléments.

Dans l'assemblage entre pannes et cadres (tous deux préfabriqués) (fig. 13) le remplissage a pour but, dans certains cas, de protéger les éléments métalliques et d'empêcher un déplacement longitudinal des pannes. Normalement, l'élément principal était surélevé à l'aide de béton coulé sur place, afin d'entretoiser les pannes puisque l'ouvrage ne comporte pas de dalle de couverture.

Nous sommes également d'avis de réduire au minimum la soudure au montage car elle revient chère; mais c'est le procédé le plus efficace et le plus

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[1]) Voir page 413 — siehe Seite 413 — see page 413.

rapide pour obtenir une solidarité des éléments lorsqu'on dispose d'armatures en saillies ou de petites plaques métalliques enrobées.

Nous sommes d'accord avec Mr. New ainsi qu'avec le Rapporteur, Monsieur le Professeur Wästlund, sur la grande responsabilité du constructeur d'ouvrages préfabriqués, qui doit prendre toutes les précautions nécessaires à la solidarité d'ensemble de la construction.

Résumé

En réponse aux questions de Mr. New, l'auteur discute trois détails relatifs aux systèmes d'assemblage des éléments préfabriqués.

Zusammenfassung

Drei Verbindungsdetails für den Zusammenschluß von vorfabrizierten Teilen werden als Antwort zu den von Mr. New aufgeworfenen Fragen behandelt.

Summary

As answer to Mr. New's questions, three details on the assembly of prefabricated elements are discussed.

Montage von Stahlbetonfertigteilen in Geschoßbauten

Mounting of Pre-Cast Reinforced Concrete Structural Units in Multy-storey Buildings

Montage des éléments préfabriqués en béton armé dans les immeubles à étages

HEINZ ZEIDLER
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Im Hinblick darauf, daß besonders bei Stockwerksbauten in Stahlbeton-Montagebauweise jede Verbindung eine schwache Stelle der Konstruktion darstellt und die Horizontallasten entweder durch Wandscheiben oder steife Ecken aufgenommen werden müssen, ist es erstrebenswert, Rahmenkonstruktionen auszuführen.

Vollrahmen in Geschoßhöhe und mit mehreren Stielen werden meist sehr schwer. Es wird daher oft zu der Notmaßnahme gegriffen, Rahmen aufzuschneiden, in einzelnen Elementen zu montieren und diese nachträglich wieder biegesteif zu verbinden. Das ist nur durch Schweißen oder Überdeckungsstöße möglich. Bis zur Kraftschlüssigkeit dieser Verbindungsstellen erfordern diese Konstruktionen bei der Aufstellung des Skelettes, bevor durch die Deckenplatte eine Längssteifigkeit eintritt, oft umfangreiche Absteifungshilfsgerüste.

Ich möchte daher die Aufmerksamkeit auf die Verbindung von Stahlkonstruktionen mit Stahlbetonelementen bei der Montage von Stahlbetonfertigteilen hinlenken. Diese Mischbauweise wird deshalb wohl bisher etwas vernachlässigt, weil die meisten Ingenieure der Stahlbeton-Montagebauweise eben aus dem Stahlbetonbau kommen und sich mit Stahlkonstruktionen nicht gern befassen.

Man kann bei Kombination von Stahlkonstruktionen mit Stahlbetonfertigteilen die Absteifungsgerüste als bleibende Konstruktionsglieder benutzen oder als Bewehrung später auszubetonierender Teile zur Tragkonstruktion werden lassen.

Bei einem 10stöckigen Krankenhaus wurde diese Mischbauweise — Stahlbeton mit Stahl — erfolgreich angewandt, indem auf eine Mittelstütze aus

Stahl zwei Halbrahmen aus Stahlbetonfertigteilen aufgelegt wurden. Die Stahlkonstruktion bildet somit für die Montage ein Hilfsgerüst und nimmt die Eigengewichte der Halbrahmen und der Deckenplatten eines Geschosses auf, so daß die Hebezeuge sofort nach Absetzen der Halbrahmen zur fortschreitenden Montage frei werden. Nach Abdecken des Rahmenskelettes mit vorgefertigten Deckenplatten ist es leicht, auf dieser Hilfskonstruktion einen

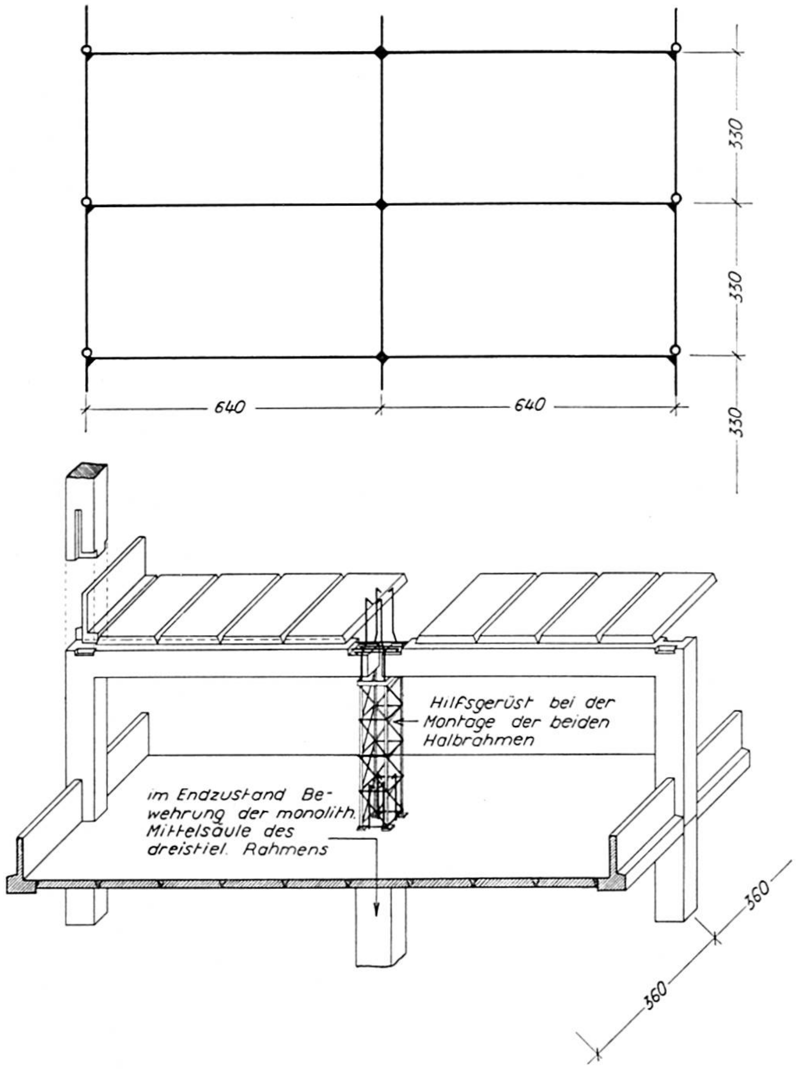

Fig. 1.

Überdeckungsstoß der beiden Halbrahmen auszuführen und die Stahlkonstruktion für alle zukünftigen Lasten zur Bewehrung einer monolithischen Stahlbeton-Mittelsäule werden zu lassen. Durch Ausgießen einer versetzbaren Schalung mit Ortbeton wird die Endkonstruktion ein dreistieliger Rahmen. Dadurch entfallen alle kostspieligen Absteifungen, die die Montage aufhalten, weil sie vor Kraftschlüssigkeit der Verbindungen nicht versetzt werden dürfen.

Selbstverständlich muß das Schwinden und Kriechen dieser Zusammenwirkung eines vorgefertigten und damit abgebundenen Teiles mit der monolithischen Säule in dieser berechnet und die Zusatzspannung, die sehr gering ist, aufgenommen werden.

Dieses soll nur ein Beispiel für das Zusammenwirken von Stahl und Stahlbetonelementen in der Montagebauweise sein und als Anregung dienen, solche Mischbauweisen zu entwickeln, die unnötige Hilfskonstruktionen vermeiden und die Zerlegung komplizierter und sehr schwerer Fertigteile in einfache Formen ermöglicht, die sich leicht herstellen und montieren lassen.

Zusammenfassung

Der Autor lenkt die Aufmerksamkeit der Fachwelt auf die Kombination von Stahlkonstruktionen mit Stahlbeton-Fertigteilen in der Montagebauweise bei mehrgeschossigen Bauwerken, wobei darauf hingewiesen wird, daß *Stahl*-Konstruktionen, die zunächst Hilfsgerüste für die Montage darstellen, später zu bleibenden Konstruktionen als Bewehrung monolithischer Bauteile werden.

Summary

The author directs the attention to the combination of steel elements with pre-cast reinforced concrete structural units in the "assembly" method of construction of multi-storey buildings, at the same time mentioning that the *steel* elements, which in the first place are auxiliary erection scaffolds, subsequently become permanent constructions as the reinforcement of monolithic structural parts.

Résumé

L'auteur montre comment on peut combiner, dans la construction en série des immeubles à étages, des parties métalliques avec des éléments préfabriqués en béton armé: les parties métalliques, servant d'abord d'échafaudages auxiliaires de montage, sont ensuite incorporées dans la construction définitive, en qualité d'armature des éléments monolithiques.

Leere Seite
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Verbindungen von Stahlbetonfertigteilen in der Montagebauweise

Assembly on Erection of Precast Units of Reinforced Concrete

Assemblage au montage d'éléments préfabriqués en béton armé

E. LEWICKI
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Freier Diskussionsbeitrag zum VI. Kongreß der IVBH (vorgetragen von Dr.-Ing. H. Rühle, Dresden)

Die von mir auf dem V. Kongreß in Lissabon 1956 vorgetragenen Verbindungsarten vorgefertigter Stahlbetonelemente in der Montagebauweise sind — bis auf eine — noch mehr oder weniger aktuell. Bezüglich ihrer Anwendung hat sich in den letzten 4 Jahren folgendes ergeben:

Mehr und mehr kommt man aus Gründen der Kostenersparnis und Verkürzung der Montagezeit besonders bei Hallen- und Stockwerkbauten davon ab, um jeden Preis durch geeignete nachträgliche Verbindungen der Fertigteile kontinuierliche oder Rahmentragwerke herzustellen, besonders bei Tragwerken, die eine im Verhältnis zur Eigenlast nur geringe Verkehrslast zu tragen haben, wie z. B. Dachpfetten, Deckenträger in Wohnungsbauten. Hier wirkt sich die erstrebte Abminderung der Feldmomente aus Verkehrslast gegenüber den Feldmomenten aus Eigengewicht nur wenig aus.

Bei Hallenbauten legt man die Binderriegel auf die Köpfe der im Boden fest eingespannten Stützen auf — entweder beiderseits gelenkig gelagert oder auch als Balken auf 2 Stützen. Bei Stockwerk-Skelettbauten werden sämtliche Stützen als Pendelstützen behandelt, und die Stabilität des Bauwerks wird durch starre senkrechte Scheiben in geeigneten Achsen erzielt. Für derartige Bauten verwendet man immer mehr Dollen oder Verschraubung mittels Stahlbolzen (Verbindungen 2 und 3 meiner damals angegebenen Systematik)[1].

Allerdings gibt es noch viele Tragwerksarten, bei denen die Durchlaufwirkung erforderlich — zumindest wünschenswert — ist, wie Kranbahnen,

[1]) Schlußbericht des V. Kongresses, Beitrag VIa6, S. 637—642.

Silos, Brücken u. a. mehr. Von den hierfür in meiner Systematik aufgeführten Verbindungen 4—10 hat sich nicht bewährt die «Keilverbindung» (9).

Am beliebtesten ist noch immer die Verbindung durch Überdeckung herausstehender Bewehrungsstäbe mit nachträglichem Einbetonieren derselben (6), da sie verhältnismäßig einfach auszuführen ist. Die relativ lange Zeitdauer für ihre Kraftschlüssigkeit bemüht man sich durch Verwendung eines schnellerhärtenden Betons abzukürzen.

Die Anwendung der Verschweißung herausstehender Stahlteile (7) hat sich wegen der sofort zu erzielenden Kraftschlüssigkeit in den sozialistischen Ländern sehr stark ausgeweitet. Vorbedingung ist die Knicksicherheit der geschweißten, zunächst freiliegenden Stahlstäbe auf der Druckseite der Stoßlücke durch Beschränkung der Länge der Stoßlücke im Beton und Verstärkung der Stabquerschnitte an der Schweißstelle. Bezüglich der Schweißung verweise ich auf meinen zusammen mit H. Löser geschriebenen Aufsatz in der Wissenschaftlichen Zeitschrift der Technischen Hochschule Dresden *7* (1957/58), H. 3, S. 479—486.

Die Verbindung durch Zusammenspannen nimmt an Beliebtheit stark zu. Viele Ausführungen sind bekannt geworden. Einfachheit und Wirtschaftlichkeit haben sich hierbei besonders bei weitspannenden Tragteilen erwiesen.

Die in Ungarn entwickelte und von W. Herrmann (Dresden) systematisch untersuchte Verbindung durch Stahlbolzen (4) hat sich bisher nicht eingeführt, obwohl sich bei einigen Bauausführungen keine Nachteile gezeigt haben. Vielleicht fehlt es an einer genügenden Propagierung.

Auch die Verbindungen 5 (Bewehrungsschleifen) und 3 (Verschraubung herausstehender Stahlteile) sind weniger beliebt.

Zusammenfassende Empfehlung

1. Nachträgliche Herstellung einer Kontinuitätswirkung soll auf die unbedingt notwendigen Fälle beschränkt werden.
2. Hierdurch ist die Anwendung einfacher Verbindungen (2 und 3) möglich.
3. Als biegesteife Verbindungen sind zu empfehlen: Schweißung (7) — Zusammenspannung (10) — Überdeckung der Bewehrungsstabenden (6).

Zusammenfassung

Es wird die Beliebtheit der Anwendung der verschiedenen Verbindungen vorgefertigter Stahlbetonteile besprochen. Nachträgliche Herstellung einer Kontinuitätswirkung soll auf die unbedingt notwendigen Fälle beschränkt werden, um einfache Verbindungen (Dollen und Verschraubung) zu erhalten. Als biegesteife Verbindungen sind zu empfehlen: Schweißung — Zusammenspannung — Überdeckung der Bewehrungsstab-Enden.

Summary

The author recalls the most suitable processes for the assembly of precast units of reinforced concrete. He recommends that continuous joints constructed in situ should be kept strictly to a minimum in order to enable simple means of assembly (pins and bolts) to be used. For constructing assemblies which remain rigid under bending stresses the following processes may be recommended: welding — connection by prestressing — overlapping of projecting reinforcement bars.

Résumé

L'auteur rappelle quels procédés sont les plus appréciés pour l'assemblage des éléments préfabriqués en béton armé. Il recommande de limiter au strict minimum les liaisons continues réalisées sur place, afin de pouvoir utiliser les moyens d'assemblage simples (goujons et boulons). Pour réaliser des assemblages rigides à la flexion, on peut recommander: la soudure — la liaison par précontrainte — le recouvrement des barres d'armature en saillie.

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