PRECAST, PRESTRESSED CONCRETE FOR BRIDGE DECKS

JULY 1970 - NUMBER 17

BY

M.J. GUTZWILLER

R.H. LEE

C.F. SCHOLER

JHRP

JOINT HIGHWAY RESEARCH PROJECT

PURDUE UNIVERSITY AND
INDIANA STATE HIGHWAY COMMISSION

Technical Paper

PRECAST, PRESTRESSED CONCRETE FOR BRIDGE DECKS

TO: J. F. McLaughlin, Director
Joint Highway Research Project

July 16, 1970

File No.: 7-4-14

FROM: H. L. Michael, Associate Director
Joint Highway Research Project

Project No.: C-36-56N

Attached is a Technical Paper titled "Precast, Prestressed Concrete for Bridge Decks" by Professors M. J. Gutzwiller, R. H. Lee and C. F. Scholer of our staff. The Paper resulted from research in progress on the HPR, Part II project of the same title. Some of the material in the Paper has been previously reported in Progress Reports on this research but other information, such as that pertaining to construction of two field installations, is reported for the first time.

The Paper will be presented by Professor Lee at the Summer Meeting of the Highway Research Board in Sacramento, California. It is planned for publication by the Highway Research Board.

The Paper was presented to the Advisory Board of the JHRP on July 16 and approved for presentation and publication. It is now submitted to the ISHC for review, comment and similar approval and for submission to the BPR for similar handling.

Respectfully submitted,

Harold L. Michael

Harold L. Michael
Associate Director

HLM:ms

cc: F. L. Ashbaucher
W. L. Dolch
W. H. Goetz
W. L. Grecco
M. J. Gutzwiller
G. K. Hallock

M. E. Harr
R. H. Harrell
V. E. Harvey
M. L. Hayes
E. M. Mikhail
R. D. Miles

C. F. Scholer
M. B. Scott
W. T. Spencer
H. R. J. Walsh
K. B. Woods
E. J. Yoder

Digitized by the Internet Archive
in 2011 with funding from
LYRASIS members and Sloan Foundation; Indiana Department of Transportation

Technical Paper

PRECAST, PRESTRESSED CONCRETE FOR BRIDGE DECKS

by

M. J. Gutzwiller

R. H. Lee

C. F. Scholer

Research Engineers

Joint Highway Research Project

Project: C-36-56N

File: 7-4-14

Prepared as Part of an Investigation

Conducted by

Joint Highway Research Project

Engineering Experiment Station

Purdue University

in cooperation with the

Indiana State Highway Commission

and the

U. S. Department of Transportation

Federal Highway Administration

Bureau of Public Roads

The opinions, findings and conclusions expressed in this publication are those of the authors and not necessarily those of the Bureau of Public Roads.

Not Released for Publication

Subject to Change

Not Reviewed By
Indiana State Highway Commission
or the
Bureau of Public Roads

Purdue University
Lafayette, Indiana
July 16, 1970

PRECAST, PRESTRESSED CONCRETE
FOR BRIDGE DECKS

Abstract

A rapid method of replacing deteriorated bridge decks is desirable to minimize inconvenience and delays to the traveling public. A research project at Purdue University under the auspices of the Indiana State Highway Department and the Bureau of Public Roads has demonstrated the feasibility of using precast, prestressed concrete slabs for deck replacement. Plant manufacture of the slabs should result in a high quality, durable specimen.

The pretensioned slabs (presently of 4'-0" width) are placed transversely to the bridge girders. Pretensioning stress level is intended to maintain compression stress in the concrete under full design loads. The slabs are post-tensioned in the longitudinal direction with cables strung through preformed slab ducts. A thin rubber pad placed in the joints between slabs minimizes stress concentrations due to surface irregularities. The pad and nominal post-tensioning stress level prevent water movement through the joint. Slab units are tied down to the supporting beams using spring clips and bolts screwed into preset anchors in the concrete.

Laboratory research has dealt with the behavior of several joint shapes, with a flat key-type joint selected for field use. The number of tie-down bolts used provides adequate longitudinal slip resistance between the slab and the supporting beams. Static vertical load tests with load applied at a joint showed little shear transfer between loaded units and the adjacent unloaded slabs. An initial set of repeated load tests to simulate wheel passage over a joint showed distress at the joint, indicating the need for careful attention to joint detailing and forming. A second set of laboratory specimens, constructed with an improved

joint detail withstood ten million cycles of repeated load application and showed no adverse effects.

Two field installations are planned for the Spring of 1970, one as a replacement deck for a heavily traveled bridge and the other on a newly installed bridge.

Introduction

The problem of concrete bridge deck spalling on many of our newer as well as older bridges is severe, somewhat perplexing as to cause, and difficult to solve. Furthermore the cost of repairs to highway agencies and cost of delays to the highway area can be extremely large.

The use of precast, prestressed concrete for bridge decks is a concept of construction or reconstruction which we hope will contribute considerably to the durability of concrete bridge decks and greatly reduce the time required for replacement of deteriorated bridge decks.

A New Concept

The origin of this concept is directly related to the spalling of bridge decks. The Joint Highway Research Project at Purdue University, with the Indiana State Highway Commission, has long been concerned with the durability of concrete. The Indiana Highway Commission, and most other highway agencies, have through the development of adequate designs, specifications, inspections and tests achieved high quality concrete with adequate durability for all highway uses except bridge decks. Their lack of success in this case is often dramatic and always obvious to the traveler over such a bridge.

When we consider the problem we find that several conditions exist on bridge decks which are seldom found elsewhere. First, the construction situation and environment are frequently the worst ever accepted for placing concrete. What may have been excellent concrete when mixed may be subject to unanticipated delays between mixing and placement. The placement, finishing, and curing of the concrete on bridge decks all too often involve hand labor, poor finishing techniques, high wind, extreme temperatures and questionable curing. It is small

wonder that the product resulting from placing concrete in forms in the sky is not of the best quality.

These undesirable field construction conditions are difficult to control and it was while pondering ways of negating them, hence improving the resultant concrete that the idea of taking the concrete to the environment rather than the environment to the concrete developed. This leads to the concept of precasting the concrete bridge deck. It is possible under the controlled conditions of a precast plant to obtain the highest quality of concrete and of steel placement.

It is well to realize that simply because the product is manufactured in a plant, there is no guarantee that high quality will automatically result. Proper control must still be exercised.

Feasibility

The initial planning for this research project started in early 1967 with it developing into an approved HPR research project in cooperation with both the Indiana State Highway Commission and the Bureau of Public Roads in October 1967. The initial feasibility study soon showed that the project had promise and that a laboratory investigation of certain aspects was both warranted and technically desirable.

The concept as further developed in the feasibility study is to use precast and pretensioned slabs which are placed on top of the bridge girders, then post-tensioned and tied mechanically to the underlying girders to achieve a deck of satisfactory structural characteristics. This is illustrated in Figure 1.

Among the many questions which require an answer for the utilization of this concept are:

What shape should the joints be?

What about joint material, both a sealer and one through which the post-tensioning load will be transmitted?

TYPICAL TEST SECTION

TYPICAL CROSS-SECTION AT
TIE DOWN

FIGURE 1. CONCRETE JOINT TESTING MODEL

Can the necessary cables and cable ducts be placed within the depth of the slab?

What about discontinuous support of the slabs on the girders - how important is this and how can voids be filled with a load bearing material?

How will the slab be transported and assembled in the field?

How will it be tied to the underlying girder?

What will be the load distribution of such a bridge deck compared to a conventional bridge deck?

Many, many more questions existed, many still exist and certainly some have not yet been asked.

Laboratory Investigations

Laboratory investigations in the JHRP laboratories at Purdue University showed that satisfactory slab tie down to the girder could be obtained using a spring steel clip attached by a high strength bolt to a steel anchor insert in the precast concrete slab. Such a clip has successfully been used by the Association of American Railroads to fasten rails to precast concrete ties. Such clips are commercially available.

Others* have made detailed studies of anchor pull out, loss of bolt torque with loading and other factors involved with the rail to concrete fasteners. It is anticipated that clips will be only lightly fastened until after the slabs are post-tensioned and they will then be fastened securely by means of a torque wrench.

*Hsu, T. T. C. and Hanson, N. W., "An Investigation of Rail-to-Concrete Fasteners," Journal of the PCA Research and Development Laboratories, Portland Cement Association, Vol. 10, No. 3, September 1968, pp. 14-35.

The shape of the joint was investigated by means of a photoelastic investigation which indicates that the "flat" joint of Figure 2(a) was superior to either a semi-circular joint of Figure 2(b) or circular sector joint of Figure 2(c) insofar as minimizing stress concentrations due to longitudinal loading.

Laboratory tests of concrete joints were made by approximating the curvature of a steel beam (30 WF 124) span of 50'-0". This simply supported beam was loaded with an HS 20-44 live loading and subsequent deflection calculation made to develop a deflection curve. This curve was then used to develop the required deflection of the laboratory test beam holding the joint test specimen as shown in Figure 3.

The "flat" joint was tested under a 40 psi post-tensioning stress, with repetitive loads through 2-1/4 million cycles without distress.

In this test the male and female sections had been cast with a common form hence a good fit was assured. It seemed wise as indicated from the photoelastic studies to consider a joint material to reduce the stress concentrations in the joint. Another function of such a material was to prevent moisture from entering or passing through the joint.

One material known to have been used for a similar purpose was asbestos cloth, both plain and reinforced. Such a material was fastened to the concrete with an AC-70 asphalt cement, post-tensioned and then flooded. Capillary movement of water within the cloth proved to be considerable and the asbestos was not considered further.

A 1/16 inch, V60 neoprene sheet proved to be the most promising material for the purpose and is being used in the first field evaluation. However the joint recommended for field application does not have contact along the upper and lower

(a) Flat Joint

(b) Semicircular Joint

(c) Circular Sector Joint

FIGURE 2. PHOTOELASTIC MODELS

VIEW FROM STRESSING END

VIEW FROM "DEAD" END

FIGURE 3. LABORATORY JOINT TESTING ARRANGEMENT

portions, hence a polyurethane joint sealing will be used on this portion of the joint in the field. It might be mentioned that joints on a post-tensioned deck will not vary in width as do pavement joints so the sealing problem is much less difficult.

Post-tensioning will be done by greased cables coated with a plastic sleeve. The plastic sleeves are to insure that corrosive water cannot attack the post-tensioning cables.

Following the investigations of the joint shape and joint material the first commercially constructed pretensioned slabs were made and used in a laboratory investigation seen in Figure 4. This slab simulated a portion of the deck of a dual lane bridge in which the length of the slab (longest dimension) is 32 feet. Its width is 4'-0" which happens to be the width of a standard casting bed, and its depth is six inches. Such a slab was designed to take AASHTO loading with maximum distance between underlying girders being 8'-0". When the girder spacing is smaller a reduction can be made in the amount of prestressing steel. For the 8-foot beam spacing 12-7/16 inch diameter 270 kip strands were used at 6 inch spacing with 6 strands in the top and 6 in the bottom layers. For a 4-foot beam spacing the steel was reduced to 8-7/16 inch diameter 270 kip strands with 4 in the top and 4 in the bottom layers. This amount was based on allowing zero tension in the concrete under full dead load, live load and impact. Slabs were 6 inches deep with 1-1/2 inch of cover above and below the center line of the strands. These first slabs were cast in forms within a precasting bed as shown in Figure 5. Wood blocks were used between the forms forming the joints and the edge of the precasting bed. This discontinuous support proved to be insufficient to keep joint forms aligned which is absolutely essential for the successful use of these precast concrete bridge slabs. Even though the joints

FIGURE 4. PRECAST SLAB TEST SETUP

LONGITUDINAL VIEW OF PRECASTING BED
WITH PRESTRESSING STEEL IN PLACE

CLOSE-UP OF PRECASTING BED SHOWING TWO ROWS
OF PRESTRESSING STEEL, METHOD OF LOCATING
BOLT INSERTS, LIFTING LOOP, VOID SPACE FORMER,
AND SIDE FORM FOR TRANSVERSE CONTINUITY JOINT.

FIGURE 5. PRECASTING METHOD FOR LABORATORY SPECIMENS

were not aligned we did use these slabs for a phase of our laboratory investigation. They worked well for measurement of slip tests between the girders and the concrete slabs and it was found that they readily met the AASHTO requirements. The next phase of the laboratory tests required that specimens be post-tensioned together prior to static and subsequent repetitive loading as shown in Figure 6. Immediately upon post-tensioning some incipient spalling developed at "high spots" along the joints of the concrete slabs. Recognizing that this was in reality an unsatisfactory set of slabs and that such work would have to be rejected in the field we continued to utilize these slabs within the laboratory. They were subjected to repetitive loads whereupon it was found that severe spalling occurred after a very few cycles of loading where the irregularities or "high spots" caused severe stress concentrations. The post-tensioning force was again adjusted to make up for any loss in the stress or post-tensioning stress due to a failure of a concrete adjacent to the joints and continued testing produced no additional failures adjacent to the joint and no signs of distress at all in other areas of the slab. These four initial slabs were constructed on two separate days, the first day the operation was closely observed by a graduate student, Mr. Jim Ford, who was present on this project. The second day the construction and the subsequent cutting of the steel was not observed. On the unobserved prestressed slabs light cracking occurred near the ends of the prestressing strand. It is possible that the necessary strength of the concrete had not been obtained at the time that the prestressing was released, causing a slippage at the strand. In any case this is the only occurrence of cracks along the prestressing strand that was observed on either this set of slabs or a subsequent casting of three additional concrete slabs.

TESTING ARRANGEMENT

SPALL AT CENTER JOINT

FIGURE 6. REPEATED LOAD TEST WITH 8'-0" BEAM SPACING

A second series of commercially cast prestressed slabs was made and proper alignment of joints was obtained. These slabs performed most adequately and have undergone more than 10 million cycles of simulated 18 thousand pound single axle load application on adjacent sides of a joint without any apparent deterioration of the slab or the joint.

Longitudinal Forces

The laboratory simulation of the deck system was subjected to longitudinal forces tending to move the slab relative to the beam in order to measure friction between the slab and supporting beams. Several tie-down bolt arrangements were used to evaluate the longitudinal response with differing vertical forces.

The supporting beams were restrained against movement during application of the longitudinal loads to the slab, such that only relative movement between the slab and supporting beams occurred. The slab system was post-tensioned to close the joints and ensure that the entire deck moved as a unit. Movement of the slab units was measured by dial gages mounted at the corners of the slab units. Occurrence of slip was indicated by the dial gages.

Three test series were run, and total bolting force was variable in each series. The three series differed in the manner in which contact between the slab and beam was achieved.

Series I - The top flanges of the beams were leveled prior to the test (compensating for the beam flange skew due to mill production tolerances). Contact between slab and beam surface was essentially uniform. Three support beams were used at 4'-0" spacing.

Series II - Leveling of the support beams (three at 4'-0" spacing) was not done. This condition provided an uneven and variable bearing surface.

Series III - Support similar to Series II, except two beams at 8'-0" spacing were used.

Total bolting force was found to have the greatest effect on longitudinal slip force, rather than the actual arrangement of bolts to supply the force. Results of the tests are shown in Figure 7, which plots horizontal force at slip versus total bolting force for each of the series. Different bolt arrangements providing the same bolting force give rise to the several sets of results for the individual test series. As would be expected, reduction of bearing surface reduced the slip resistance. Superimposed on the plot is the design longitudinal force of five percent of the applied live load as specified by AASHO.

Vertical Loads

Static testing of the deck system under vertical loads was conducted with the following variables:

Vertical loading:	0, 5, 10, 15 kips
Longitudinal prestress:	0, 20, 35 psi (gross section)
Bolting force:	0, 32, 48 kips
Beam spacing:	4'-0" and 8'-0"

Figure 8 shows the loading arrangement used in the 4'-0" beam spacing configuration. Vertical loads were applied directly over the joint between the center slabs, with circular steel plates and a neoprene pad providing pressure distribution.

Strain gages mounted on the concrete surface at various points indicated little participation of the slab sections which were not loaded directly with the vertical load. In other words, shear transfer at unloaded joints was minimal.

FIGURE 7. HORIZONTAL LOAD AT SLIP VS. BOLTING FORCE

FIGURE 8(a). PLAN VIEW OF STATIC TEST FOR SECTIONS REINFORCED FOR 4'-0" CENTER-TO-CENTER BEAM SPACING

SECTION A-A

SECTION B-B

FIGURE 8(b). SECTIONS A-A AND B-B, STATIC TEST WITH 4'-0" BEAM SPACING

Continuous support was supplied for the beams, such that the deflections and strains were due to slab action alone.

Figure 9(a) shows the deflections of the specimen with 4'-0" beam spacing due to the 5, 10, and 15 kip vertical loads applied, with a longitudinal prestress of 34 psi on the gross section and zero force in the tie-down bolts. Figure 9(b) gives similar results with a total tension force of 48 kips present in the tie-down bolts. (The load was applied between points labeled 3 and 4. Refer to Figure 10 for locations of the deflection dials).

Figure 11(a) and 11(b) give deflections at a longitudinal section for the specimen supported on an 8'-0" span. (Load was applied between gage locations 11 and 12.) Figure 11(a) shows results when bolting force was zero, while Figure 11(b) gives data for a total bolting force of 32 kips.

It may be noted that bolting significantly reduced deflections under similar loadings, for both the 4'-0" and 8'-0" beam spacing. One other effect which was noted during the tests was that increasing the level of post-tensioning force increased the deflections due to the applied load. This was apparently due to the fact that the post-tensioning cables were in contact with the member only at the ends. The member deflections did not bring the slab into contact with the cables, which would have provided a restoring force component.

Repeated Load Tests

The deck system was subjected to repeated load tests with supporting beams placed at 8'-0" spacing and 4'-6" spacing. For this set of tests the supporting beams were supported on simple spans of 20'-0". A jack arrangement, shown in Figure 12 permitted simulation of a wheel load passing from one side of the joint to the other, by varying the jacking loads on each side of the joint. The

(a) Bolting Force - Zero

(b) Bolting Force - 48 Kips

Vertical Load Key: $P = 5.04$ k ———x——— $P = 10.08$ k ———■——— $P = 15.12$ k ———●———

FIGURE 9. LONGITUDINAL CENTERLINE DEFLECTION, 4'-0" BEAM SPACING, 34 PSI POST-TENSIONING (GROSS SECTION)

FIGURE 10. LOCATIONS OF DEFLECTION MEASUREMENT, STATIC TEST

(a) Bolting Force - Zero

Vertical Load Key: $P = 5.04$ k —x— $P = 10.08$ k —■— $P = 15.12$ k —●—

(b) Bolting Force - 32 Kips

FIGURE II. LONGITUDINAL CENTERLINE DEFLECTION, 8'-0" BEAM SPACING, 34 PSI POST-TENSIONING (GROSS SECTION)

FIGURE 12. REPEATED LOAD TEST SETUP

repeated load was cycled at a rate of 250 cpm, with a maximum jack load of nine kips and a minimum load of one kip.

Longitudinal post-tensioning stress was maintained at a level of 32 psi on the gross section. The load test for the 8'-0" spacing proceeded satisfactorily for only 32,000 cycles of load at which time a spall developed (Figure 6) at the joint. This spall was apparently due to a poor joint fit which has been described previously. The load repetitions continued to 715,000 cycles without further cracking or spalling, at which time the test was discontinued.

The deck sections were then interchanged to provide a "fresh" joint, beams were moved to provide 4'-6" spacing, and another test was performed. No cracking or joint deterioration occurred during two million cycles of loading. Concrete strains remained essentially at a constant level and the post-tensioning stress remained constant during the test.

As a result of the spalling which occurred, the joint was slightly redesigned and the casting method was changed in order to provide a better joint fit. A second set of specimens made to the new specification have been made and tested under ten million cycles of repeated loads with no apparent joint distress.

Field Tests

Two structures are intended to become an integral part of this project. The results of the laboratory tests were such that it seemed feasible to proceed with several monitored prototypes and the performance of these structures will help dictate the direction for future research work in this area.

Structure I will be a replacement deck on an existing structure on Indiana State Road 37 approximately 3 miles north of Bloomington, Indiana across Bean Blossom Creek. The existing structure is an eight-panel thru type pony truss

approximately 125 feet in length. This structure with its deck was constructed in the late forties and the deck has been patched several times. Electric strain gages are to be welded to several stringers and floor beams prior to removing the old deck. A test load vehicle will be used to produce strains in these members which will be closely monitored. The old deck will then be removed leaving the bare stringers and floor beams intact. The new deck will consist of 32 pretensioned slabs. After the new deck is installed additional strain gages will be placed at critical positions on the bottom surface of the deck. Strains will then be monitored once again for the stringers and floor beams and also for the slabs. The behavior of the entire system will be closely observed over a period of several years and this information will be reported at a later date.

Structure II is a new structure which will be placed on Indiana State Road 140 over the Big Blue River just south of Knightstown. This 200 foot structure will have 3 spans of continuous steel beams with the pretensioned slabs as the deck. There are two 70 foot spans and a single 60 foot span. The deck will consist of 51 individual slabs, each approximately 39 feet long positioned on the steel beams which will be spaced at 6 feet on centers. Strain gages will be positioned on the steel beams and on the slabs at critical positions. The traffic anticipated on this structure is considerably less than that expected on Structure I and of a different nature due to its location. The performance of this structure will likewise be closely observed and reported.

These two structures cover two situations which are entirely different in the design approach. The first structure is the design of a replacement slab under other than ideal conditions while the second structure is specifically designed to receive the precast slabs as a deck.