PRECAST, PRESTRESSED CONCRETE FOR BRIDGE DECKS

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A research project at Purdue University has demonstrated the feasibility of using precast, prestressed concrete slabs for deck replacement. The pretensioned slabs (presently 4 ft wide) 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 by using spring clips and bolts screwed into preset anchors in the concrete. 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 10 million cycles of repeated load application and showed no adverse effects.

The problem of concrete bridge deck spalling on many new as well as old bridges is severe, somewhat perplexing as to cause, and difficult to solve. Furthermore, the cost of repairs and of delays can be extremely large. The use of precast, prestressed concrete for bridge decks is a concept of construction or reconstruction that 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. Through the development of adequate designs, specifications, inspections, and tests the Indiana State Highway Commission, and most other highway agencies, have achieved highquality 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.

Several conditions exist on bridge decks that 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 placing, 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.

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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 led 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, however, 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. 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 that are placed on top of the bridge girders and then post-tensioned and tied mechanically to the underlying girders to achieve a deck of satisfactory structural characteristics. This is shown in Figure 1.

Among the many questions that require an answer for the utilization of this concept are the following: What shape should the joints be? What about joint material, including a sealer and one through which the post-tensioning load will be transmitted? 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 more questions existed, many still exist, and certainly some have not yet been asked.



TYPICAL TEST SECTION



TIE DOWN

Figure 1. Concrete joint testing model.

LABORATORY INVESTIGATIONS

Laboratory investigations showed that satisfactory slab tie-down to the girder could be obtained by 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 (1). 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.

The shape of the joint was investigated by means of a photoelastic investigation that indicates that the "flat" joint shown in Figure 2a was superior to either a semicircular joint shown in Figure 2b or a circular sector joint shown in Figure 2c 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 ft. This simply supported beam was loaded with an HS 20-44 live loading, and a subsequent deflection calculation was made to develop a deflection curve. This curve was then used to develop the required deflection of the laboratory test beam. The joint test specimen was held as shown in Figure 3. The flat joint was tested under a 40-psi, post-tensioning stress with repetitive loads through $2\frac{1}{4}$ million cycles without distress.

In this test the male and female sections had been cast with a common form, hence a good fit was ensured. 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 $\frac{1}{16}$ -in., 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 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 ensure 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 (Fig. 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 ft. Its width is 4 ft, which happens to be the width of a standard casting bed, and its depth is 6 in. Such a slab was designed to take AASHO loading, and maximum distance between underlying girders is 8 ft. When



Figure 3. Laboratory joint testing arrangement (stressing end on left and "dead" end on right).



Figure 4. Test setup.

the girder spacing is smaller, a reduction can be made in the amount of prestressing steel. For the 8-ft beam spacing, twelve $\frac{7}{16}$ -in. diameter 270-kip strands were used at 6-in. spacing with 6 strands in the top and 6 in the bottom layers. For a 4-ft beam spacing, the steel was reduced to eight $\frac{7}{16}$ -in. 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 in. deep with $1\frac{1}{2}$ in. of cover above and below the centerline 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 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 AASHO requirements.

The next phase of the laboratory tests required that specimens be post-tensioned together prior to static and subsequent repetitive loading. Immediately on 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 low stress concentrations (Fig. 6). 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 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 4 initial slabs were constructed on 2 separate days; the first day the operation was closely observed by a graduate student, but on 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



Figure 5. Precasting method for laboratory specimens (precasting bed with prestressing steel in place on left; and 2 rows of prestressing steel, method of locating bolt inserts, lifting loop, void space former, and side form for transverse continuity joint in precasting bed on right).



Figure 6. Spall at center joint during repeated load test with 8-ft beam spacing.

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 3 additional concrete slabs.

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-kip, single-axle load application on adjacent sides of a joint without any apparent deterioration of the slab or of the joint.

LONGITUDINAL FORCES

The laboratory simulation of the deck system was subjected to longitudinal forces in order to measure friction between the slab and the supporting beams. Several tiedown 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, such that only relative movement between the slab and the 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 3 series differed in the manner in which contact between the slab and the beam was achieved. In series 1 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-ft spacing. In series 2 leveling of the support beams (3 at 4-ft spacing) was not done. This condition provided an uneven and variable bearing surface. In series 3 the support was similar to that in series 2 except that 2 beams at 8-ft spacing were used.



Figure 7. Horizontal load at slip versus bolting force.

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 shows horizontal force at slip versus total bolting force for each of the series. As would be expected, reduction of bearing surface reduced the slip resistance. Superimposed on the plot is the design longitudinal force specified by AASHO.

VERTICAL LOADS

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

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

Figure 8 shows the loading arrangement used in the 4-ft 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 that were not loaded directly with the vertical load. In other word, shear transfer at unloaded joints was minimal. Continuous support was supplied for the beams, such that the deflections and strains were due to slab action alone.

Figure 9a shows the deflections of the specimen with 4-ft 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 9b shows similar results with a tension force of 48 kips present in the tie-down bolts. (The load was applied between points 4 and 9. Figure 10 shows the locations of the deflection dials.)

Figure 11 shows deflections at a longitudinal section for the specimen supported on an 8-ft span. (Load was applied between gage locations 7 and 12.) Figure 11a shows results when bolting force was zero, while Figure 11b shows results for a bolting force of 32 kips.



SECTION B-B

Figure 8. Static test for sections reinforced for 4-ft center-to-center beam spacing.

It may be noted that bolting significantly reduced deflections under similar loadings for both the 4-ft and 8-ft beam spacing. One other effect that 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-ft and $4\frac{1}{2}$ -ft spacing. For this set of tests the supporting beams were supported on simple spans of 20 ft. 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 repeated load was cycled at a rate of 250 cpm, with a maximum jack load of 9 kips and a minimum load of 1 kip.



Figure 9. Longitudinal centerline deflection for 4-ft beam spacing and 34 psi post-tensioning on gross section.



Figure 10. Locations of deflection measurement in the static test.



Figure 11. Longitudinal centerline deflection for 8-ft beam spacing and 34 psi post-tensioning on gross section.



Figure 12. Repeated load test setup.

Longitudinal post-tensioning stress was maintained at a level of 32 psi on the gross section. The load test for the 8-ft spacing proceeded satisfactorily for only 32,000 cycles of load at which time a spall developed (Fig. 6) at the joint. This spall was apparently due to a poor joint fit that 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 $4\frac{1}{2}$ -ft spacing, and another test was performed. No cracking or joint deterioration occurred during 2 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 that 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 have been made to the new specification and tested under 10 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 1 will be a replacement deck on an existing structure on Ind-37 approximately 3 miles north of Bloomington, Indiana, across Bean Blossom Creek. The existing structure is an 8-panel through type of pony truss approximately 125 ft in length. This structure and its deck were constructed in the late forties, and the deck has been repaired numerous 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 floors 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 2 is a new structure that will be placed on Ind-140 over the Big Blue River just south of Knightstown. This 200-ft structure will have 3 spans of continuous steel beams with the pretensioned slabs as the deck. There are two 70-ft spans and a single 60-ft span. The deck will consist of 51 individual slabs, each approximately 39 ft long positioned on the steel beams, which will be spaced at 6 ft 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 1 and of a different nature because of its location. The performance of this structure will likewise be closely observed and reported.

These 2 structures cover 2 situations that are entirely different from the design aspect. 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.

FUTURE RESEARCH

The concept of precast bridge decks lends itself to the possible use of materials other than conventional portland cement concrete. It is hoped that serious consideration will be given to many of these after the field evaluation of the concept has established its practicality.

Among the items for future consideration are lightweight concrete, fiber-reinforced concrete, polymer concrete, slabs with surfaces cast to have exceptional skid resistance, surfaces composed of material to impart exceptionally high resistance to wear

by studded tires, and sandwich type of construction with materials. Such possibilities will need to be evaluated from both economic and performance aspects. Future work should also include consideration of methods of connecting adjacent lanes of precast bridge decks on multilane structures. Of immediate application would be provision for a deck to be replaced while traffic was utilizing part of the structure.

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