Precast Concrete Composite Deck Components for Rehabilitation of Bridges

by Richard N. White

Synopsis: The first part of this paper provides a brief review of the critical issues met in replacing deteriorated bridge decks with precast deck systems, drawing on existing papers published by PCI and ACI. Recommended practices for typical bridge rehabilitation projects are provided.

The second topic focuses on the rehabilitation of the Tappan Zee Bridge over the Hudson River near New York City. Results are presented for an experimental program conducted on a 10 m span full-scale lightweight concrete slab-steel beam composite bridge deck unit. Loading history included 10 million cycles of flexural fatigue loading, followed by a flexural load capacity test. Measured values of capacity and mid-span deflection at this ultimate load level are compared with simplified analytical predictions.

Keywords: Bridge decks; bridges (structures); composite construction; cyclic loads; fatigue; flexural strength; lightweight concrete; precast concrete; reinforced concrete; tests

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A BRIEF SURVEY OF BRIDGE DECK REPLACEMENT STRATEGIES

Tens of thousands of highway bridges in the U.S. (and elsewhere) are in various stages of deterioration. Bridge decks have a particularly demanding environment. The decks must carry the dynamic, cyclic loading effects of trucks and other traffic, while also being exposed directly to the environmental elements, including thermal effects, rain and snow, freezing and thawing action, and the deleterious effects of de-icing salts in northern climates.

The criteria for successful bridge rehabilitation are quite simple; they include the following (where relative importance varies from bridge to bridge):

- provide a higher level of durability than in the original bridge
- minimize traffic disruption during rehabilitation
- reduce dead load, if possible, to increase load capacity
- provide a reasonable construction cost.

In recent years the use of full depth precast panels has become quite popular for replacing existing bridge decks. A typical application is shown in Fig. 1. A comprehensive review of various applications of both reinforced and prestressed full depth precast decks is given in the state-of-the-art paper by Issa et al (1). A survey of actual in-field performance of these deck systems is provided in (2), and construction procedures for rapid replacement of bridge decks are described in (3). Because of space limitations, only the most important design features and conclusions contained in these papers will be discussed here, as follows:

a. Disruption to traffic must be minimized, particularly on bridges located on major highways. Special construction procedures using precast deck units are based on having a partial width of the bridge closed to traffic overnight. A relatively short length of the closed deck is removed and replaced the same night, and the full bridge width is available for the morning traffic. The process is repeated until the full length of both halves of the bridge is rehabilitated.

b. Precast deck panels are usually quite short but with high loading effects near the ends of the panels. The necessary complete development of pretensioning strands is not possible, and a fully prestressed system cannot be utilized. Thus it is customary to utilize epoxy coated reinforcing steel for primary load-carrying capabilities in the long direction of the precast panel, often supplemented by a modest level of prestressing to prevent cracking from stresses induced during handling and installation .

c. Composite action of the new deck and the main supporting members is always advantageous and often necessary to provide adequate bridge capacity after rehabilitation. This requires blockouts (openings) in the deck panels for installation of shear connectors for transfer of horizontal shearing forces between the deck and the supporting girders. The top flanges of steel girders are fitted with either welded shear studs or welded channel connectors. The shear connection to the slab is completed by filling each opening with a high-quality, durable concrete (polymer or epoxy concrete if quick curing is required). Typical blockout and shear connections are shown in Fig. 2.

d. Joints between adjacent panels present one of the most vexing obstacles met in achieving a durable precast concrete deck system. The female-to-female joint detail shown in Fig. 3, with a polymer or epoxy concrete gap filler (which can reach strengths of 30 MPa in one hour) appears to give superior performance. The initial gap opening is on the order of 12 mm. Since panels are not perfect in their dimensions, lack-of-fit dimensions of up to 12 mm under or over can be accommodated with this detail. Design, materials, construction, and durability issues for the joint are presented by Gulyas (4). For panels spanning transversely to traffic direction, prestressing is provided in the direction of traffic to further secure good joints and to help resist the shearing and bending actions produced by traffic wheel loads.

e. Most reported cost analyses show that construction costs are slightly higher for precast systems than for cast-in-place slabs, but this cost differential is more than made up in favor of precast systems from the substantial reductions in traffic delays and re-routing as well as in construction time savings.

Variations on the full thickness concrete deck panels also find application in bridge rehabilitation, as will be shown in the following discussion of a system using lightweight concrete deck panels precast compositely with shallow steel beams.

No matter which system is used, concrete durability is a critical concern. Bridge decks present a particularly fruitful opportunity for the use of high performance concrete, and the use of precast deck systems offers the advantage of tight quality control and optimal curing conditions to achieve durability.

EXPERIMENTAL STUDY OF PRECAST DECK UNIT FOR THE TAPPAN ZEE BRIDGE, NEW YORK, USA

This second part of the paper summarizes the results of an experimental program conducted at the George Winter Structural Research Laboratory at Cornell University on a full-scale composite lightweight concrete slab-steel wide flange bridge panel, 2.44 m wide and 10.26 m long, supported on a simple span of 9.85 m. The tested section (Fig. 4) was quite similar to a design proposed for a replacement deck on the Tappan Zee Bridge in New York State. The primary reason for conducting the testing program was to qualify this new type of composite construction for use in New York State highway bridge decks.

The Tappan Zee Bridge

The 5-km long Tappan Zee Bridge crosses the Hudson River about 10 km above the New Jersey-New York state line, carrying more than 100,000 commuters each day, plus major truck traffic, on Interstate 287. This 41 year old steel truss structure is scheduled to have its entire cast-in-place concrete deck replaced (a $70 million project), along with repairs to steel members above and below the deck ($9 million) and additional structural protection against ship collision impact ($50 million).

According to Mr. Keith Giles, division director of the New York Thruway Authority, "Our goal is to keep all lanes open during peak hours. We have a movable barrier that we can use to make sure there are always four lanes in the commuter direction. Traffic stoppage is not in our game plan". The full rehabilitation project, which also includes a number of other smaller projects, may take up to 4 years.

The mid-1996 contract signed for the Tappan Zee Bridge rehabilitation calls for replacing a 15.2 m length of a single 4 m wide lane each night, using a 178 mm thick normal density concrete slab acting compositely with galvanized 27 WF beams. The initial installation, which began in early May 1997, involves replacement of a complete lane down the middle of the bridge, closing the lane at 9 pm and re-opening the next morning at 6 am. The penalty for re-opening later than 6 am is $1000 per minute.

Scope of Experimental Study

Phase I of the experimental program included applying 10 million cycles of loading to simulate the maximum flexural effects of HS 25 truck loading. Fig. 6 shows the panel in position for flexural fatigue testing. Loading was applied through two beams spaced 1.22 m apart and spanning transversely across the 2.44 m width of the panel.

Phase II of the study consisted of a flexural capacity test of the bridge panel. Static loading, applied through a beam system similar to that used in the fatigue loading, was increased gradually up to failure.

Bridge Panel Geometry and Material Properties

Critical cross-sectional dimensions for the bridge panel are shown in Fig. 4(c). The panel consisted of a 127 mm thick lightweight concrete deck supported on four steel wide flange W16X31 beams, with composite action between concrete and steel beams provided by two lines of 19 mm diameter by 76 mm long shear studs spaced uniformly at 165 mm on each beam. The unit was fabricated utilizing the ***Inverset*** process at the production facilities of The Fort Miller Company in Schuylerville, NY. The ***Inverset*** process involves supporting the beams at their ends, hanging slab forms from the beams, and placing the concrete deck under the beams (Fig. 5). After the concrete has gained sufficient strength, the entire unit (slab and beams) is inverted to produce a composite system with a high quality (bottom-cast) slab top surface and a modest prestressing action and upward camber resulting from the downward bending action of the fresh soft concrete when it was placed in the form suspended under the steel beams.

The deck slab concrete was made with Type I portland cement, sand, and lightweight aggregate, proportioned to have a unit density not less than 1840 kg/m^3 and a 28 day compressive strength of 34 MPa. Measured concrete strength at 139 days was 46.3 MPa (average strength of two 150 by 300 mm cylinders), indicating an approximate 28 day cylinder strength of 40 MPA. Concrete modulus was measured with six tests over a 222 mm gage length on each of two cylinders up to approximately 35% of their compressive strength, giving average values of E = 22.5GPa and 24.3 GPa at 139 days.

Two cylinders were tested just prior to the ultimate load test of the bridge, when the concrete was 1.22 years old. Results were as follows: compressive strength = 60 MPa and 60 MPa, and modulus of elasticity = 24.8 GPa and 21 GPa.

Steel girders were A572 Grade 50 (345 MPa yield), and the slab reinforcement was epoxy coated Grade 60 (414 MPa yield).

Test Set-up and Loading System

Flexural fatigue loading, applied at the rate of 1 cycle every 2 seconds, simulated the flexural effects an HS25 truck loading, including an impact factor of 0.30. The required load, including the weight of the load distribution system, was 236 kN. Load was applied with an MTS 245 kN capacity hydraulic actuator and was divided into equal loads with a three-level loading system as shown in Fig. 6. The net target actuator loading was 231 kN.

It should be noted that no attempt was made to simulate the absolute maximum end reactions and shear forces that would be produced by HS25 loading, nor were the local effects of concentrated wheel loads on the slab simulated by the loading. A parallel experimental program at the University of Oklahoma investigated the punching shear strength of an identically designed slab-beam unit.

The four steel beams of the bridge panel were supported at each end on W 5X16 steel beams resting on concrete pedestals as shown in Fig. 6. 13 mm thick neoprene pads of the type typically used for bridges -- 55-65 hardness, 10 MPa tensile strength and a temperature range of -29C to 77C -- were placed between the steel beams and the transverse supporting beams to help equalize loadings and to permit longitudinal motion of the beams during load application. At one end of the panel, each beam was restrained against longitudinal motion.

Instrumentation

49 channels of instrumentation were used -- 19 channels of displacement and load transducers to measure center span and end support deflections, slip of the concrete deck with respect to the top flanges of the steel beams, and load magnitude and displacement of the hydraulic actuator, and 30 electric resistance strain gages to measure strains on the steel beams, in the concrete in the plane of the reinforcing bars, and on the top and bottom surfaces of the concrete deck. All transducers and gages were connected to a computer-controlled data acquisition system which provided rapid data reading and ample capacity for digital storage.

Cyclic Loading Test Results

Deflections and Stiffness of Panel -- Maximum mid-span deflection was nearly constant at about 15 mm throughout the 10 million load cycles. The corresponding flexural stiffnesses of the bridge panel (applied load divided by net mid-span deflection) are plotted in Fig. 7. The general trend in response was a slight increase in stiffness up to about 2 million cycles, a small decrease from 2 million to 3.5 million cycles, and nearly constant stiffness during the remaining load cycles. The stiffness after 10 million cycles, 15.5 kN/mm, decreased 3.2% from the value of 16.0 kN/mm measured after 100,000 cycles.

Potential residual (permanent) deflections accumulated during the cyclic loading testing over a period spanning many months were measured with a surveyor's level. Measurements taken before (5/21/93) and after (8/2/93) the 2 million load cycles indicate that the residual net deflection at midspan was 0.15 mm. Similar measurements taken on 9/9/93 after additional cycling showed an additional 0.05 mm residual displacement. Readings taken after 8.5 million cycles (on 3/10/94) and after 10 million cycles (on 5/12/94) showed total residual displacements of 0.79 mm and 1.17 mm, respectively.

Recovery of displacement as a function of time after unloading was measured after 8.5 million load cycles, but results are not reported here because measurement reliability was questionable -- the measured small values of displacement were highly sensitive to temperature changes in the laboratory environment. Thus there is no credible data available on the issue of possible creep effects from the cumulative effects of having live load applied to the bridge for so many cycles. However, given that the residual displacement did not exceed 1.17 mm, cumulative live load creep effects were negligible.

Strains and Stresses in Beams and Slab; Fatigue Strength of Concrete -- Measured strains produced by a static load of 231 kN applied at the conclusion of two, six, and ten million load cycles for each of the four beam locations are plotted in Fig. 8. The beams are identified as E1, I1, I2, and E2, where E and I denote exterior and interior beams, respectively. Strain measurements were made with gages bonded to (a) the top of the concrete slab, (b) the top of the top flange of the steel beam, and (c) the bottom of the bottom flange of the steel beam. The strain values increased slightly during the progression of static loading cycles executed every 500,000 load cyles, which is consistent with the measured displacement data.

Midspan flexural stresses produced by the HS25 (+ impact) moment were about 103 MPa tension in the bottom flanges of the four steel beams, and about 5.5 MPa compression in the top of the concrete slab immediately above the beams.

Although fatigue strength of concrete tends to increase when loading is stopped for taking strain and displacement readings, as was done here, it is believed that this effect does not provide any appreciable enhancement of fatigue strength. Countering this strength enhancement effect is the fact that actual passage of trucks on the real bridge would not be nearly as rapid as the basic loading regime used in this laboratory study, where many years of truck traffic were simulated in about one year's time.

Composite Action Between Slab and Steel Beams -- Potential slip between the concrete deck and the top flange of the steel beams was measured at both ends of all four beams. With the exception of the right ends of beams I1, I2 , and E2, measured slips did not exceed 0.008 mm, which is the same order of magnitude as the sensitivity of the gages (noise level threshold). There was no discernible trend of increasing slip with increasing number of load cycles. Thus essentially full composite action was maintained during the 10 million load cycles.

Nondestructive Investigation of Concrete Slab Condition -- The concrete deck slab was analyzed with a commercially available impulse-echo flaw detection system. Three sets of data were taken -- before loading began, after 2 million load cycles, and after 10 million load cycles. This analysis was done by the developer of the equipment, Cornell faculty member Dr. Mary Sansalone, and two structural engineering graduate students. The impulse-echo equipment was used to detect possible slab cracking or delamination, to investigate potential changes in the quality of contact between concrete and steel beam flange at the deck-girder interfaces, and to reveal any other potential flaws. The three sets of data gave essentially identical results, indicating no separations of the deck from the top flanges of the steel beams and no measurable internal laminar cracking.

Flexural Capacity Testing Program

Loading System and Modifications to Instrumentation -- The bridge panel was moved to a stronger reaction frame consisting of two hold-downs anchored into the laboratory floor, with a transverse reaction beam spanning between the two hold-downs. Load was applied with a 1780 kN capacity Enerpac hydraulic ram acting between the hold-down system and a (stronger) load distribution system similar to that used for the fatigue testing program.

Load was applied in a load-control mode in increments of 90 kN for the first 623 kN and was then reduced to 45 kN increments as yielding was approached. At higher load levels, there was significant reduction of load at the end of each increment when loading was stopped to permit inspection.

Several changes were made to the instrumentation system used for the fatigue testing program, including: (a) beam deflection at mid-span was measured for one beam only, using a DCDT for displacements up to 100 mm, and an engineering scale to get approximate values of larger deflections, and (b) potential slip between the top flanges of the steel beams and the concrete slab was measured at 10 locations -- both ends of the four beams, and at 1.83 m and 3.66 m from the south end of Beam E1.

Load-Deflection Response; Panel Stiffness -- Mid-span displacement vs. applied load is plotted in Fig. 9. Response was linear to a load of about 490 kN (about 2.1 times flexural design load). At higher loads, gradual yielding of the steel beams and nonlinear action in the concrete resulted in increasing nonlinearity. At a load of about 1245 kN, problems with the loading system required an unloading. As shown in Fig. 9, the bridge panel unloaded with essentially the same stiffness as the original linear stiffness. The load system was repaired and reloading faithfully followed the unloading path. Reduction of stifffness is plotted in Fig. 10. Just before failure, with the load in excess of 1335 kN, and with a midspan deflection of 203 mm (L/50), the panel maintained about 40% of its original stiffness.

Strain Magnitudes and Distributions -- The bottom fiber strains in the exterior beam showed nonlinearity after a load of about 490 kN. Bottom fiber interior beam strains remained linear to higher load levels, approaching 670 kN, and then increased sharply because the exterior beams had yielded first and could not "take up" much additional load as the yielding interior beams tried to shed load.

The top fiber steel beam strain plots reached a maximum value of approximately 0.00013 to 0.00015 at a load of about 755 kN. At higher loads, strain increments became tensile, reducing the amount of compression until strains eventually reached zero at an applied load in excess of 1110 kN. This state of zero strain corresponded to a neutral axis located at the bottom of the slab thickness. At higher loads, tensile strains increased rapidly as the neutral axis continued to migrate upward.

Mid-span strain distributions at three different load levels (890, 1110, and 1245 kN) are plotted in Fig. 11. It can be seen that the assumption of plane sections remaining plane is satisfied very well even when the bridge panel had experienced severe plastic strains. The two strain values measured on the bottom flange were quite different at high loads because the steel beams were fabricated with an appreciable sweep in the bottom flange, which tended to straighten during plastification.

Concrete strains were measured on the top surface of the slab at three locations -- over the exterior beam E1, the interior beam I1, and at the center of the deck (Fig. 12). Response was linear up to a load of about 534 kN, with gradually increasing nonlinearity at higher loads. Maximum strains were about 0.0025 at a load level of about 1245 kN, the last opportunity to record reliable strain readings.

Behavior Near Ultimate; Failure Mode -- At the end of the test, with the load peaking at 1380 kN and the midspan deflection exceeding 200 mm, concrete strains above the girders were about 3500 microstrain and the average steel strain in the bottom flanges was over 10,000 microstrain (about 5 times the yield strain of the steel). The plane section assumption was still satisfied reasonably well at this advanced stage of behavior.

The failure mode was a "classical" underreinforced flexural mode, as had been predicted by a simple "back-of-the-envelope" analysis which gave estimated values of 1200 kN peak load with mid-span deflection = 180 mm. Continued yielding of the steel beams resulted in upward migration of the neutral axis and a more rapid increase in the concrete