SCHOOL OF
CIVIL ENGINEERING

INDIANA

DEPARTMENT OF TRANSPORTATION

JOINT HIGHWAY RESEARCH PROJECT

Part 1 Final Report

FHWA/INDOT/JHRP-92-24

Strand Debonding in Pretensioned Beams
- Precast Prestressed Concrete Bridge
Girders with Debonded Strands

Continuity Issues

O.A. Abdalla, J.A. Ramirez, and R.H. Lee

PURDUE UNIVERSITY

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Strand Debonding in Pretensioned Beams - Precast Prestressed Concrete Bridge Girders with Debonded Strands.

Part 1, Continuity Issues

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To: Vincent P. Drnevich, Director

Attached is Part 1, of 2, Final Report of a research project entitled, "Strand Debonding in Pretensioned Beams" By O.A. Abdalla, J.A. Ramirez, and R.H. Lee. The report considers the comments of the advisory committee.

Respectfully submitted,

Julio A. Ramirez and R.H. Lee, Co-Principal Investigators

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16. Abstract <p>This report summarizes an experimental investigation carried out to evaluate the effects of strand debonding on the behavior of precast pretensioned bridge members made continuous with a cast-in-place slab and diaphragm. Shear and flexural capacity were evaluated and the experimental results compared to the results obtained using the PCA and CTL (proposed) analytical methods.</p> <p>Four continuous specimens were fabricated and tested. Three specimens consisted of Type-I AASHTO girders continuous with a cast-in-place slab and diaphragm. The fourth specimen consisted of Indiana Type CB-27 box girders also continuous with cast-in-place slab and diaphragm.</p> <p>The effect of time-dependent creep and shrinkage deformations on the capacity of the girders at the continuous supports was investigated in this study. Also addressed in this report is the effect of limiting the stress at the extreme compression fiber, near the continuous support, to allowable working stress values on the load carrying capacity of continuous members.</p>			
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NOTATIONS

A_c	= cross sectional area of composite section
A_d	= cross sectional area of deck slab
A_g	= cross sectional area of precast girder
A_{ps}	= total area of prestressing steel
A_v	= cross-sectional area of the stirrups
b_w	= web width of the girder
d	= distance from the extreme compression fiber to the centroid of the longitudinal tension reinforcement
e	= base of the Napierian logarithms (2.7183)
E_{di}	= modulus of elasticity of deck concrete at time t_i
E_d	= modulus of elasticity of deck concrete at 28 days
E_g	= modulus of elasticity of girder concrete
e_c	= distance between the top of the girder and the centroid of composite section
e_s	= eccentricity of prestressing force from the centroid of composite section
f_d	= stress due to unfactored dead load, at extreme fiber of section where tensile stresses are caused by externally applied loads
f_r	= modulus of rupture of concrete
f_{pc}	= compressive stress at the centroid of the composite section, or at the junction of the web and flange when centroid lies within the flange, due to both prestressing and the moment resisted by the precast member

acting alone

- f_{pe} = compressive stress, due to prestressing, at extreme fiber of section where tensile stresses are caused by externally applied loads
- f_{pu} = specified tensile strength of prestressing strands, psi
- f_{py} = specified yield strength of prestressing strands, psi
- f_y = yield strength of nonprestressed reinforcement
- f'_c = compressive strength of concrete
- h = deck slab thickness
- H = total depth of composite girder
- I = moment of inertia of the composite section
- I_{cr} = cracked transformed moment of inertia of composite section
- M_{cr} = moment causing flexural cracking at section due to applied loads
- M_d = mid-span dead load moment
- M_{max} = maximum factored bending moment at section due to externally applied loads
- M_p = mid-span prestressing moment on composite section
- M_r = restraint moment
- M_s = differential shrinkage moment
- n = ratio of modulus of elasticity of slab concrete to the modulus of elasticity of girder concrete
- s = stirrup spacing
- t = time in days
- V_{ci} = nominal shear strength provided by concrete when diagonal cracking

results from combined shear and moment

V_{cw} = nominal shear strength provided by concrete when diagonal cracking

results from excessive principal tensile stress in the web

V_d = shear force at section due to unfactored dead load

V_i = factored shear force at section due to externally applied loads

V_s = nominal shear strength provided by web reinforcement

y_t = distance from the centroid of the section to extreme fiber in tension

ΔM_{si} = change in differential shrinkage moment at time step i

ϵ_s = differential shrinkage strain

ϵ_{sdi} = shrinkage strain in deck at time t_i

ϵ_{sdu} = deck concrete ultimate shrinkage strain

ϵ_{sgi} = shrinkage strain in girder at time t_i

ϵ_{sgu} = girder concrete ultimate shrinkage strain

$\Delta \epsilon_{si}$ = change in differential shrinkage strain at time step i

Ψ = change in creep coefficient

ABSTRACT

This report summarizes an experimental investigation carried out to evaluate the effects of strand debonding on the behavior of precast pretensioned bridge members made continuous with a cast-in-place slab and diaphragm. Shear and flexural capacity were evaluated and the experimental results compared to the results obtained using the analytical methods.

Four continuous specimens were fabricated and tested. Three specimens consisted of Type-I AASHTO girders continuous with a cast-in-place slab and diaphragm. The fourth specimen consisted of Indiana Type CB-27 box girders also continuous with cast-in-place slab and diaphragm.

The effect of time-dependent creep and shrinkage deformations on the capacity of the girders at the continuous supports was investigated in this study. Also addressed in this report is the effect of limiting the stress at the extreme compression fiber, near the continuous support to allowable working stress values on the load carrying capacity of continuous members.

CHAPTER 1

INTRODUCTION

The economical benefits that can be gained from the construction of bridges composed of simple span precast prestressed I-beams made continuous with cast-in-place concrete decks have made this type of construction a very popular one. The determination of time-dependent and ultimate moments at the continuous support presents a major difficulty in the design of this type of bridge structure. Several methods have been proposed to estimate the restraint moments which will develop at the continuous support. The method most commonly used to date was first published by the Portland Cement Association in 1961 and extended in 1969 , since then, various computer programs have been developed to further refine the analysis and design of this type of structure (Sinno and Furr [1972], Tadros et al [1975], Suttikan [1978] and Glikin et al [1987]).

Debonding of strands at end regions of pretensioned members is a technique which is used extensively to control normal stresses in pretensioned members. By debonding the strands, expensive draping hardware can be eliminated and labor costs can also be reduced. However, the end regions of pretensioned beams with debonded strands at interior supports of continuous multi-span bridges deserve further consideration. Current AASHTO [1989] design specifications limit the stress in the extreme compression fiber at interior girder ends to $0.6 f'_c$. This requirement specifies consideration of effects of prestressing and negative live load bending. This limitation often results in additional debonding requirements. Debonding strands to meet the stress requirement may be counterproductive and cause deleterious effect on the shear

strength.

Recent analytical studies (Oesterle et al [1989]) indicated that time-dependent effects and construction timing can be of major influence on the effective continuity at pier supports under vehicle loads. Herein, the PCA [1969] and the proposed Construction Technology Laboratory [1989] analytical methods will be discussed and their predicted level of continuity compared to the experimental results from four prestressed precast two-span continuous beams with a composite concrete slab and various amounts of debonding. The first three specimens consisted of Type-I AASHTO girders made continuous with a 48×4 inch cast-in-place concrete slab and diaphragm. The fourth specimen consisted of Indiana State Type CB-27 box girders made continuous with a cast-in-place 36×4 inch composite slab and diaphragm.

The results from the continuous tests reported herein, will be used to evaluate the normal stresses induced by prestressing with various debonding schemes, time-dependent effects, as well as those caused by the applied superimposed load on the composite structure in the region adjacent to the continuous supports of multi-span bridges. In addition, the flexural and shear behavior of this type of structure at the continuous support will be evaluated. This report is Part-1 of a two part final report for the research study "Behavior of Pretensioned Bridge Members with Debonded Strands". Part two will include the results on the performance of simply supported pretensioned bridge members with debonded strands.

CHAPTER 2

EXPERIMENTAL PROGRAM

2.1 Objective and Scope

The objectives of the tests conducted in this phase of the research study were to evaluate, at continuous supports of pretensioned precast beams, (a) the combined effects of time-dependent deformations; (b) the stress in the extreme compression fiber at the ends of the girder due to prestressing and superimposed loads; and (c) the ultimate capacity of precast prestressed bridges with debonded strands and a cast in place composite concrete deck (see Figure 2.1). The test specimens are shown in Figures (2.2-2.5). The specimen details are shown in Figures (2.6-2.8). The precast beams were virtually identical except for the strand debonding scheme near the ends (A-B) of the beams tested as shown in Figures (2.2-2.5). The prestressing strands of the second beam (C-D) were always fully bonded throughout the entire length. The first two specimens in this study were tested by Schmid [1991] and Ogg [1991].

All the continuous I-beam specimens consisted of two full scale Type-I AASHTO girders made continuous with a cast-in-place slab and diaphragm. In Specimen 1 (Schmid [1991]) beam A-B had 6 strands (50%) debonded at each end as shown in Figure (2.2). Specimen 2, tested by Ogg [1991], had 6 of the strands (50%) debonded at end A, and 8 strands (67%) debonded at end B (see Figure 2.3). Beam A-B in Specimen 3 had 8 of the strands (67%) blanketed at end A, while 10 of the strands were blanketed (83%) at the continuous end B as shown in Figure (2.4). The precast beams in Specimen 4 were Indiana State Type CB-27 box girders. Beam A-B in Specimen 4

had 50% debonding at the interior support B and at support A (see Figure 2.5).

The precast beams were made continuous by means of a cast-in-place slab and diaphragm. The slab was reinforced with #6 bars in the longitudinal and transverse directions. In addition, four strands from each girder were embedded into the diaphragm following standard specifications of the Indiana Department of Transportation. The continuous structure was tested under a static two-point load system.

The continuous test set-up was designed such that the continuous test would cause appreciable damage only to the interior seven-feet of the beam adjacent to the continuous support as shown in Figures (2.2-2.5). The outside seventeen feet towards the simply supported end of the beam remained elastic and uncracked after the completion of the continuous test. Upon completion of each of the continuous tests the continuity between the two beams was broken and each undamaged portion of the beams was further tested over a simply supported span.

2.2 Description and Fabrication of Test Specimens

2.2.1 Precast Beams Construction and Instrumentation

The precast pretensioned beams were manufactured at the Hydro Conduit Corporation plant in Lafayette, Indiana. The test beams were cast in pairs, each individual beam being 308 inches long. The beams were cast in a single prestressing bed.

The prestressing steel, in both girders, consisted of stress relieved (Specimen 1) and Lo-Lax (Specimens 2, 3 and 4) Grade 270, uncoated seven-wire strands, 0.5 inch

diameter. All the strands in the test beams were straight throughout the entire length. Each strand was initially tensioned with a force of 2200 pounds (Specimen 1) and 3000 pounds (Specimens 2,3 and 4). Prestressing of the strands was performed from one end only, each individual strand at a time. The surface condition of the strands was considered as weathered.

After the initial stressing, electrical resistance strain gages were then affixed onto the strands, at the desired locations. The strain gages were aligned along one helix of the strand. The resulting strain in the prestressing steel, due to the initial pull force of 28,900 pounds (Specimens 1, 2 and 3) and 30,983 pounds (Specimen 4), was then measured using an automated data acquisition system. Figures (2.9-2.12) show the locations of the strand instrumentation for the test specimens.

The prestressing steel in the beam with debonded strands was then encapsulated to the desired length using plastic tubing. Both ends of the tubes covering the strands were taped shut to effectively isolate the strand from the concrete. The locations of the debonding points for the test specimens are shown in Figures (2.2-2.5).

The shear reinforcement for all the specimens consisted of vertical stirrups made of #3 Grade 60 bars, spaced at 4 inches on center over the entire length of the beams. The stirrups were assembled in the form of a cage by tack welding them to the mild reinforcement at the top of the precast beams as shown in Figures (2.13) and (2.14). The stirrup instrumentation is shown in Figures (2.15-2.17).

The reinforcing cage was then placed in the casting bed, and all the stirrups were tied to the strands using plastic ties. Lifting loops were provided at the ends of each

beam. Concrete was then poured in a single layer and consolidated using portable vibrators. After the forms of both beams were filled, the concrete was struck even with the top of the steel forms and then roughened to a depth of 0.25 inch. Along with the beams, thirty 6×12 inch cylinders and nine $6 \times 6 \times 22$ inch flexural beams were also cast and cured in the same conditions as the beams.

The prestressing force was transferred to the beams using the standard flame cutting procedure. The prestressing strands were detensioned one strand at a time. After the prestress release, the strain in the strands was recorded and the girders were then removed from the prestressing bed and shipped to the laboratory. Upon arrival, the beams were set on four supports in the testing area awaiting the casting of the deck slab and diaphragm.

2.2.2 Slab and Diaphragm Construction

The formwork for the cast-in-place concrete deck slab and diaphragm was constructed from lumber and plywood. The forms were then placed on the precast beams so that the slab self-weight, together with the forms, was supported entirely by the simply supported precast beams following unshored construction practice. (see Figures 2.18-2.19). The forms for the continuity connection were tailored to the diaphragm dimensions. The forms were sealed and oiled before placing the deck slab reinforcing mat.

The slab longitudinal reinforcing steel consisted of 8 #6 bars. The slab transverse reinforcement, for Specimens 2, 3 and 4, consisted of #6 bars at 18 inches on centers tied to the longitudinal steel with plastic ties. The deck slab in Specimen 1 had no

transverse reinforcement. The diaphragm was reinforced with a cage made of #3 bars tied to the strands projecting from the ends of the precast beams. Three bars of the slab longitudinal steel were instrumented using electrical resistance strain gages at five locations along the specimens as shown in Figures (2.20), (2.21) and (2.22).

A batch of concrete was used for the cast-in-place slab and diaphragm. Compaction was accomplished using portable vibrators. After the forms were filled, the surface of the concrete was leveled using a lumber screed and finished with steel trowels. Thirty 6×12 inch concrete cylinders and nine 6 × 6 × 22 inch flexure beams were cast from the same batch of concrete. The slab, the concrete cylinders and flexure beams were cured in the laboratory. The forms were removed three days after the casting of the slab.

2.3 Materials

2.3.1 Concrete

The girders were cast using the standard 6000 psi concrete mix for pretensioned bridge members in the State of Indiana. The concrete compressive strength was monitored using the standard 6×12 test cylinders. The compressive strength of the concrete used in the precast beams and the cast-in-place slab and diaphragm, for the different test specimens, is shown in Figures (2.23-2.26).

2.3.2 Prestressing Steel

The prestressing steel, in both girders, consisted of stress relieved (Specimen 1) and Lo-Lax (Specimens 2, 3 and 4) Grade 270, uncoated seven-wire strands, 0.5 inch diameter (cross-sectional area of 0.153 in²). The stress-strain behavior of the strand is

shown in Figures (2.27-2.29). Strains were measured by means of electrical resistance strain gages attached to the strand as in the test beams. The yield stress, the ultimate strength, and the modulus of elasticity of the strand were determined from these tests.

2.3.3 Non-Prestressed Reinforcement

Standard deformed Grade 60, #6 bars were used as the nonprestressed top reinforcement in the precast beams and the deck slab reinforcing mat. The properties of these bars were determined from tension tests. The stress-strain behavior of the #6 bars is shown in Figures (2.30-2.32).

The stirrup reinforcement consisted of deformed Grade 60, #3 bars. The stress-strain curve for the stirrup steel is shown in Figures (2.33-2.35).

2.4 Continuous Tests

The continuous tests arrangement is shown in Figures (2.2-2.5). The ends of the beams were resting on four concrete blocks fixed to the laboratory floor by gypsum grout. At the interior supports, each beam was resting on greased rollers placed between two steel plates. One of the plates was grouted to the concrete block and the other was attached to the bottom of the beam. The reactions at the outer ends of the beams were transmitted to the concrete blocks supporting them, through load cells resting on steel plates. The magnitude of the end reactions was monitored at different intervals as the slab and diaphragm concrete reached the desired strength. This allowed the evaluation of time-dependent effects related to creep due prestressing and differential shrinkage between deck and girder concrete.

External loads were applied by hydraulic rams mounted on a vertical reaction frame anchored to the laboratory floor. The loading system employed in this study is shown in Figure (2.36). The applied loads were monitored using load cells resting on steel plates grouted to the top surface of the slab as shown in Figure (2.37). Equal static loads were applied, in small increments, at two symmetrical locations. After each increment, the loads were kept constant, while careful observation of cracks was made. Applied loads, end reactions, strains and vertical displacements were measured and recorded using an automated data acquisition system.

2.4.1 Specimen 1

The continuous beam test for Specimen 1 was conducted in three phases. In the first phase, the applied load P was increased up to a load of 100 kips, then the specimen was completely unloaded. This loading sequence will be referred to as the initial load phase. In the second phase, the load P was increased by 10 kip increments up to 100 kips, then decreased in 10 kip steps to 50 kips. The load was then increased using 10 kip increments up to 100 kips to complete one cycle. This procedure was repeated twice ending at 100 kips. The beam was completely unloaded in 10 kip steps. In the final loading phase, the specimen was loaded up to 100 kips, then completely unloaded and loaded back to 100 kips. This was followed by two cycles between 100 and 50 kips in 50 kip intervals. Upon completion of the second cycle back to 100 kips, the load was increased up to a load of 140 kips. At this point the continuous test was concluded. This last stage will be referred to as the final load phase. At each load increment, data was recorded using an automated data acquisition system. The recorded data included strain gage readings, applied loads, end reactions, and deflections. Deflections were monitored with LVDT's along with manual readings of mechanical deflection gages.

During the final load phase, the loading frame swayed out of plane at a load of 140 kips, which was near the maximum capacity of the hydraulic rams. At this point it was decided to end the test.

2.4.1.1 Cracking

Flexural cracking of the deck occurred at a load of 24 kips over the continuous support. Web-shear cracking occurred near stirrup STIB1R for the 50% debonded

girder and near stirrup STIC1R for the fully bonded girder. Crack patterns of the deck at the end of the final load phase are shown in Figures (2.38) and (2.39). Web-shear cracking occurred in both girders at an applied load of 90 kips. The crack patterns for the two beams were similar as shown in Figures (2.40) and (2.41). In the 50% debonded girder, a flexure-shear crack originating from the bottom flange occurred near the strand debonding point at a load of 140 kips. No flexure-shear cracks occurred in the fully bonded girder.

2.4.1.2 Deflections

Deflections were measured on both sides of the beam at the centerline of each span and under the applied load. Placement of the LVDT's and dial gages is shown in Figure (2.42). Load-deflection curves for the initial and final phases of this test are shown in Figures (2.43-2.46). Deflections for the fully bonded girder and the 50% debonded girder are plotted on the same graph for comparison. As can be seen from these figures, there is no significant difference in the load-deflection relationship for the two girders up to a load of 140 kips. At the 140 kip load level, a flexure-shear crack developed in the 50% debonded beam near the debonding point. This crack caused a small increase in the deflection when compared to the 0% debonded beam.

2.4.1.3 Concrete Bottom Fiber Strains

Bottom fiber concrete strains were monitored using surface strain gages at the bottom of the girder near the interior support. Location of these gages is shown in Figure (2.47). Figures (2.48) and (2.49) show the concrete strain versus the applied load for the initial and final load phases respectively. Surface gages 4 and 6 were located on

the 50% debonded beam, while gages 7 and 8 were on the fully bonded girder. These gages were affixed on to the beam after continuity was established. Therefore, only strains due to superimposed load P were recorded. This explains the similar results obtained for both beams.

2.4.1.4 Stirrup Strains

Four stirrups were instrumented in each beam near the interior support (see Figure 2.15). The strain readings for the four stirrups in each beam are plotted on the same graph for the initial and final load phases. Typical load versus strain curves are shown in Figures (2.50-2.53). By analyzing Figures (2.50) and (2.52), the web-shear cracking load of 90 kips in both beams is verified. From the figures it is apparent that higher strains occurred in the web reinforcement of the 50% debonded girder. Stirrup IC4 in the 0% debonded beam showed little increase in strain compared to stirrup IB4 in the 50% debonded beam; this reflects the greater extent of shear cracking in the debonded girder.

2.4.1.5 Longitudinal Bar Strains

Three bars in the cast-in-place slab over the continuous supports were instrumented to measure the strains in the reinforcing bars. Location of these gages is shown in Figure (2.20). Load versus strain curves for the deck bars, at the centerline of the diaphragm, are shown in Figures (2.54) and (2.55). Figure (2.54) is for the initial load phase, while Figure (2.55) is for the final load phase. It can be seen that none of these bars reached yield.

2.4.1.6 Strand Strains

Typical strains in the prestressing strand are shown in Figures (2.56-2.58) for the debonded beam and Figures (2.59-2.61) for the fully bonded beam. The instrumentation scheme is shown in Figure (2.9). As expected, the strains in the strands showed no significant change throughout the entire range of loading with the exception of the strands on the beam with 50% debonding. A large sudden increase in strain was indicated by the strain gages near the debonding point due to flexure-shear cracking as shown in Figure (2.56)

2.4.2 Specimen 2

Specimen 2 was also loaded in three stages. In the initial stage, the continuous beam was loaded in increments from zero load to 100 kips to induce cracking. The second load phase consisted of three cycles of loadings between 50 and 100 kips. After unloading, the final load stage starting at zero continued to 100 kips then back to 50 kips then to the maximum load of 162 kips. The test was terminated at the 162 kips load level because the available loading system capacity was exhausted. A description of the test results is presented next.

2.4.2.1 Cracking

The first crack appeared on the deck above the continuous support at a load of 25 kips. The first diagonal crack appeared at 70 kips in both the fully bonded and the 67% debonded beams. These cracks appeared in the web near the ends of the girder at the continuous support. Additional web-shear cracks formed as the load was increased to 100 kips. The subsequent cycles of load up to 100 kips caused no additional new cracks, only growth of existing ones.

Subsequent loading up to 162 kips caused additional shear cracking to occur. The increase in loading to 130 kips caused a flexure-shear crack to originate at the bottom flange near the second debonding point at a distance of 77 inches from the centerline of the interior support (see Figure 2.62). The fully bonded beam never displayed any positive moment flexure-shear cracks. Figures (2.63-2.64) illustrate the crack patterns at the continuous support.

2.4.2.2 Deflections

Figure (2.65) shows the location of the LVDT's and the dial gages used to measure the vertical deflection. The load-deflection behavior of the continuous structure is illustrated in Figures (2.66-2.69). The fully bonded beam deflected less than the debonded beam. The fully bonded girder has a relatively linear load-deflection relationship up to 162 kips. The debonded girder displayed the same type of behavior up to 125 kips. After 125 kips, larger deflections were observed at smaller load increments for the debonded beam. The 125 kip load level corresponds to the load level just prior to the first visible sign of a flexure-shear crack in the debonded girder.

2.4.2.3 Concrete Bottom Fiber Strains

The location of the surface strain gages used to measure the compressive strains at the continuous supports is shown in Figure (2.70). The bottom fiber strains of the precast sections are shown in Figures (2.71-2.72). The gages were placed on the girders 24 days prior to the casting of the slab. The measured strains are due to the applied superimposed load, the added weight of the unshored deck, and shrinkage and creep effects between the deck and precast sections. These readings do not reflect the effect of the initial prestressing.

2.4.2.4 Stirrup Strains

Figures (2.73-2.76) show the load stirrup-strain relationships. Figure (2.16) shows the location of the instrumented stirrups for Specimen 2. None of the stirrups yielded during this test. Overall, the 67% debonded beam initially displayed larger stirrup

strains than the fully bonded beam. Also due to flexure-shear cracking the stirrup into the span (IB4) showed significantly large strains in the debonded beam.

2.4.2.5 Longitudinal Bar Strains

The negative moment reinforcement in the top slab over the continuous support consisted of 8 # 6 bars. The load-strain relationship is illustrated in Figures (2.77-2.78) and the location of the strain gages is shown in Figure (2.21). None of the instrumented bars yielded during the test.

2.4.2.6 Strand Strains

Figures (2.79-2.84) show some typical strain readings from both the debonded and fully bonded beams during the final load phase. Figure (2.10) shows the locations of the strand strain gages for Specimen 2. The debonded girder showed linear load-strain relationship up to the 130 kips load level. The 130 kips load level corresponds to the first flexure-shear crack in the debonded beam (see Figure 2.62). Sudden increase in the strain can be seen when the flexure-shear crack occurred.

As expected, the fully bonded beam displayed linear load-strain relationship throughout the test. The gages located near the applied load typically indicated greater strain increases.

2.4.3 Specimen 3

The continuous test for Specimen 3 was conducted in three phases. In the initial phase the load was increased up to 100 kips. In the second phase the structure was unloaded to 50 kips. The load was then increased to 100 kips and unloaded to 50 kips. This was followed by another cycle between 50 kips and 100 kips. The specimen was then completely unloaded. In the final loading phase the beam was loaded incrementally up to a maximum load of 164 kips.

2.4.3.1 Cracking

The first flexural crack was noticed on the deck slab over the interior supports at a load of 25 kips. The first inclined crack in the web was observed in the beam with 83% debonding near the interior support at a load of 70 kips. Additional cracks formed in the web as the load was increased to 90 kips. These cracks were parallel to the first inclined shear crack. Figure (2.85) shows the end IB of the beam with 83% debonding after the completion of the first test.

The first inclined crack in the beam with 0% debonding developed at a load of 77 kips near the continuous support. No new cracks were observed in this beam until a load of 97 kips was reached, at which time three parallel inclined cracks formed in the web. The end IC of the beam with 0% debonding after the initial loading phase is shown in Figure (2.86).

A flexure-shear crack formed in the beam with 83% debonding at a load of 91 kips. This crack originated as a flexural crack at the second debonding point, a distance of 66

inches from the center line of the interior support (see Figure 2.87).

In the second and in the final load phase, additional inclined cracks formed in the web of both beams near the continuous support. These cracks were parallel to the cracks from the first phase. The interior region of the continuous beam at the end of the final load phase is shown in Figures (2.87) and (2.88). The cracking patterns of the slab over the continuous support after the completion of the initial and final load phases of the test are shown in Figures (2.89) and (2.90) respectively.

2.4.3.2 Deflections

The vertical deflection under the point loads and at the mid-span of each beam, was measured using linear variable differential transformers (LVDT's), and mechanical dial gages as shown in Figure (2.65). The load-deflection behavior of the continuous beam at the point of application of the superimposed load and at midspan of each beam is shown in Figures (2.91-2.92) and Figures (2.93-2.94) respectively. The curves show that the behavior of the two beams is similar in every respect up to the initiation of the flexure-shear cracking. The reduction in the stiffness upon flexure-shear cracking of the beam with 83% debonding is clearly indicated in these figures.

2.4.3.3 Concrete Bottom Strains

The concrete strains were measured using surface gages placed at the bottom of the precast beams near the continuity connection as shown in Figure (2.70). Three gages were affixed on both sides of each girder. The measured strains were due to creep and shrinkage and the superimposed load. Strains due to the prestressing force

were not included in these readings. The relationship between the applied load and the concrete strains near the continuous support is presented in Figures (2.95) and (2.96). Large changes in strain were shown in the strain near the continuous support when web-shear cracking occurred as shown in Figure (2.95). Away from the support the debonded beam showed larger strains when the flexure-shear crack opened as indicated in Figure (2.96). Figure (2.97) shows the measured bottom fiber strains for the final load phase.

2.4.3.4 Stirrup Strains

The instrumented stirrups near ends IC and IB are shown in Figure (2.17). Typical load versus measured strain behavior is shown in Figures (2.98-2.101). All stirrups showed no significant strain until they were crossed by inclined cracks. However, none of the stirrups reached its yielding point. The increase in strain at web-shear cracking is clearly indicated in these figures.

2.4.3.5 Longitudinal Bar Strains

The longitudinal reinforcement in the deck slab was instrumented as shown in Figure (2.21). The applied load versus strain curves for the longitudinal steel in the slab at the continuous supports are shown in Figures (2.102) and (2.103). The resulting strain was approaching the yield value of these bars.

2.4.3.6 Strand Strains

The measured strains in the prestressing strand (see Figure 2.11) near the continuous supports during the final load phase are shown in Figures (2.104-2.107).

Sudden increase in the strain occurred in the beam with 83% debonding near the point load at 134 kips. This increase in strain was due to a flexure-shear crack that opened near the third debonding point a distance of 84 inches from the centerline of the interior support. However, the resulting stress did not reach the yielding strength of the strand. In the beam with 0% debonding, the strains varied linearly with the applied load as shown in Figures (2.105-2.107).

2.4.4 Specimen 4

Specimen 4 was tested under a loading sequence similar to that of Specimens 2 and 3. In the initial phase the load was increased up to 100 kips. In the second loading phase the structure was unloaded to 50 kips. The load was then increased to 100 kips and next unloaded to 50 kips. This was followed by another cycle between 50 kips and 100 kips. The specimen was then completely unloaded. In the final loading phase the continuous box girder was loaded incrementally up to a maximum load of 176 kips.

2.4.4.1 Cracking

The first flexural crack occurred on the top of the deck slab over the centerline of interior support diaphragm at a load of 27.5 kips. The first inclined shear crack was observed in the beam with 50% debonding near the interior support at a load of 94.5 kips. This inclined crack appeared on one side of the beam. No additional cracks formed in the initial loading phase. A web-shear crack occurred on the 50% debonding side during the final loading phase at a load of 118 kips. The crack patterns of the beam with 50% debonding near the continuous support at the end of the initial and final load phases are shown in Figures (2.108-2.109).

The first inclined shear crack in the beam with 0% debonding developed at a load of 136 kips on one side of the beam only. On the opposite side of the beam the first inclined shear crack did not occur until a load of 150 kips.

Additional inclined cracks formed in the webs of both beams near the continuous support when the load was increased to the maximum level of 176 kips. The crack

pattern of the beam with 0% debonding at the continuous end during the initial and the final loading phases is shown in Figure (2.110). The cracking patterns of the slab over the continuous support after the completion of the first and the second phases of the test are shown in Figures (2.111) and (2.112) respectively. It can be seen that, in both girders, no flexure-shear cracks formed under this loading.

2.4.4.2 Deflections

The vertical deflection under the point loads and at mid-span of each beam, was measured using LVDT's on one side of the beam, and mechanical dial gages on the other side as shown in Figure (2.113). The load-deflection behavior of the continuous beam measured at the points of application of the superimposed load is shown in Figures (2.114-2.115). The deflection at midspan of both beams is given in Figures (2.116-2.117). The curves show that, up to a load of 176 kips the behavior of the two beams is similar in spite of the difference in the amount of debonding. This can be explained by the absence of flexure-shear cracking in the positive moment region of the continuous beam.

2.4.4.3 Concrete Bottom Strains

The location of the strain gages used to measure the concrete strains at the bottom fibers of the precast beams near the continuity connection is shown in Figure (2.70). These gages registered strains due to the time-dependent deformations in addition to the effect of the superimposed load. The concrete strains due to prestressing were not recorded. The relationship between the applied load and the concrete strains near the continuous support is presented in Figures (2.118) and (2.119).

2.4.4.4 Stirrup Strains

The instrumented stirrups near ends IC and IB are shown in Figure (2.17). Typical load versus measured strain in the stirrups during the final loading phase is plotted in Figures (2.120) and (2.121). The initial loading phase caused no significant strain in the stirrups. The beam with 50% debonding showed higher strains in the final load phase. However no yielding was observed in the stirrup reinforcement.

2.4.4.5 Longitudinal Bar Strains

The longitudinal reinforcement in the deck slab was instrumented as shown in Figure (2.22). The applied load versus strain curves for the longitudinal steel in the slab at the continuous supports are shown in Figures (2.122) and (2.123). Since, none of these bars reached its yielding strength it can be concluded that the ultimate capacity of the continuity connection was not reached in this test.

2.4.4.6 Strand Strains

Figures (2.124-2.129) show the relationship between the applied load and the strains in the strand at different locations near the interior support and under the point load. Linear increase in the strand strains with the applied load was shown by these gages. No substantial increase in strains were recorded. This is explained by the absence of flexure-shear cracking.

2.5 Summary

This chapter contains the description, fabrication and testing of the specimens in this study. The behavior of continuous-composite precast prestressed bridges with debonded strands and a cast-in-place composite slab was examined. The continuous structures were tested under the effect of monotonic concentrated loads applied at two symmetrical locations.

Crack patterns, load-deflection curves, stirrup strains, strand strains, longitudinal bar strains and the concrete bottom fiber strains near the continuous supports are presented in this chapter for all the specimens. The load-deflection curves showed that the behavior of the beams with debonded strands is similar to the behavior of the fully bonded beams up to the initiation of flexure-shear cracking. After flexure-shear cracking the debonded beams showed larger deflections. The strains at the extreme compression fiber near the continuous supports also indicated the same behavior with considerable increase upon flexure-shear cracking.

The debonded beams showed larger stirrup tensile strains. Also higher strains were induced in the prestressing strand of the debonded beams when flexure-shear cracking occurred.

Chapter 3 will deal with the evaluation of the effect of creep and shrinkage at the continuous supports. The time-dependent restraint moments obtained from the test will be compared to the predicted values using the PCA and CTL methods.

In chapter 4, a comparison of the experimental results of the superimposed load

tests with the theoretical analysis based on the PCA and CTL methods, will be presented. The flexural as well as the shear capacity near the continuous supports will be evaluated and discussed.

Chapter 5 contains a summary of this phase of the research, the conclusions drawn from the test data and future research needs with respect to the use of strand debonding in continuous pretensioned bridge members.

CHAPTER 3

TIME-DEPENDENT EFFECTS

3.1 PCA Method

In a pretensioned bridge girder, prestress will usually cause the member to camber. If the member is simply supported, the ends will tend to rotate. When members are made continuous, their ends are restrained from any further rotation due to creep deformations resulting from the prestressing. As a result a positive restraint moment may occur at the pier (positive moment produces tension in the bottom of the girder). Furthermore, in this type of composite construction, the slab is cast some time after the girders. The subsequent shrinkage of the girders will be less than that of the slab. The rotation caused by the moment resulting from the differential shrinkage strain, and creep effects due to dead load would produce a negative restraint moment at the continuous supports. The final restraint moment is the sum of the previously mentioned effects.

The Portland Cement Association conducted experimental and analytical research to determine the long term-effects of creep and shrinkage at the continuous support of a two-span beam (see Mattock [1961]). This investigation was extended later by Freyermuth in 1969, for the design of multi-span continuous highway bridges. This work also offers guidelines for the design of the continuity connection between adjacent girders. The 1969 PCA method proposed by Freyermuth is used in current INDOT design practice for predicting the restraint moments at the interior supports of precast pretensioned girders.

As suggested by Mattock [1961] and Freyermuth [1969] the differential shrinkage moment at any time is given by:

$$M_s = \epsilon_s E_d A_d \left(e_c + \frac{h}{2} \right) \quad (3.1)$$

ϵ_s = differential shrinkage strain (assumed uniform over the thickness of the slab)

E_d = modulus of elasticity of deck concrete

A_d = cross-sectional area of deck slab

$\left(e_c + \frac{h}{2} \right)$ = distance between middepth of deck slab and centroid of the composite section.

h = slab thickness

e_c = distance between the top of the girder and the centroid of composite section

The final restraint moment, M_r , at the interior support of a two-span continuous beam is given by:

$$M_r = \left(\frac{3}{2} M_p - M_d \right) (1 - e^{-\Psi}) - \frac{3}{2} M_s \frac{1 - e^{-\Psi}}{\Psi} \quad (3.2)$$

Where

M_p = moment caused by the prestressing force about the centroid of the composite section.

M_d = mid-span moment due to dead load.

Ψ = increase in creep coefficient after continuity was created.

e = base of the Naperian logarithms (2.7183)

3.2 CTL Method

The Construction Technology Laboratory method developed by Oesterle, Glikin and Larson [1989], for the analysis of precast prestressed beams made continuous, incorporates the effect of the stiffness and length of the connection between the precast girders at the continuous supports. Also, the calculation of the shrinkage restraint moment component accounts for the compatibility between the girders and the deck when shrinkage occurs. This procedure is a modified version of the original PCA method. The computer program BRIDGERM was developed at CTL to calculate the time dependent restraint moments. Different time dependent functions were presented for girder concrete creep, deck concrete shrinkage and girder concrete shrinkage. These functions were suggested by ACI Committee 209 [1982].

The analysis in BRIDGERM is conducted by superimposing the restraint moment increments calculated over a series of time intervals. For each time step, the three components of change in restraint moments, differential shrinkage, dead load creep, and prestress creep, are calculated using the rate of creep method. The calculated increment of restraint moment is then added to the sum from the preceding time step to determine the restraint moment at the end of an interval.

3.3 Evaluation of Time-Dependent Effects

The evaluation of the predicted restraint moment at continuous supports due to creep and shrinkage deformations by the 1969 PCA method proposed by Freyermuth, and the recently developed CTL approach, was conducted using the results from the experimental program previously described. The restraint moments, determined from

the experimental program, were calculated using the end reaction measurements at supports A and D shown in Figures (2.2-2.5). The end support reactions were measured at suitable intervals after the deck and diaphragm were cast. The changes with time in the reaction values, due to differential shrinkage and creep effects, were used to calculate the restraint moments at the interior support up to the time of application of the concentrated superimposed loads.

In the case of a symmetric two-span continuous girder, the restraint moments at the interior support are directly proportional to the change in the reaction at the end support. A decrease in the end support reaction corresponds to a negative restraint moment at the interior support, on the other hand, an increase in the reaction indicates a positive restraint moment. The measured end reactions and the resultant restraint moments due to the measured end reactions are presented in Tables (A.1) through (A.4).

The predicted interior support restraint moments using the CTL and the 1969 PCA methods are shown in Figures (3.1-3.4) together with the experimental values. The vertical axis represents the value of the restraint moment at the interior support. The horizontal axis starts with the age of the girder corresponding to casting of the slab up to the first application of the superimposed load P. It can be seen from Figures (3.2) and (3.4) that the results obtained using the PCA method are in close agreement with the test results for early ages of beam at the time continuity is established.

The CTL method values were calculated using the Equations (3.3) and (3.4) given below, for the increment of differential shrinkage restraint moment: (Equation (3.3) is

used when the deck slab age is less than 28 days, and after 28 days Equation (3.4) is used)

$$\Delta M_{si} = \Delta \epsilon_{si} E_{di} A_d \left(e_c + \frac{h}{2} \right) \quad (3.3)$$

$$\delta \Delta \epsilon_{si} = (\epsilon_{sdi} - \epsilon_{sdi-1}) - (\epsilon_{sgi} - \epsilon_{sgi-1})$$

ϵ_{sdi} = shrinkage strain in deck at time t_i

ϵ_{sgi} = shrinkage strain in girder at time t_i

E_{di} = modulus of elasticity of deck concrete at time t_i

A_d = cross-sectional area of deck slab

$\left(e_c + \frac{h}{2} \right)$ = distance between middepth of deck slab and centroid of the composite section

h = slab thickness

e_c = distance between the top of the girder and the centroid of the composite section

and,

$$\Delta M_{si} = \frac{\Delta \epsilon_{si} E_{di} A_d}{1 + \frac{E_{di} A_d}{E_g A_g}} \left(e_c + \frac{h}{2} \right) \quad (3.4)$$

where:

A_g = cross sectional area of precast girder

E_g = modulus of elasticity of girder concrete

This modification is not available in the PCA method. At the end of the evaluation

time, deck slab age was 44 days for specimen 1, 38 days for specimen 2, 22 days for specimen 3, and 34 days for specimen 4.

Further improvement of the CTL method was achieved by using the formula suggested by Dischinger (see Oesterle [1989]), to account for the restraining action against deck shrinkage of the reinforcement in the deck slab. The estimate of the restraining moments obtained using the CTL method with the Dischinger modification starting 3 days after continuity was established and the PCA method values are compared with the test results in Figures (3.5), (3.6), (3.7) and (3.8). With this modification the CTL predicted restraint moments showed better agreement with the experimental results for both young and old girder ages at the time continuity was established.

Figure (3.4) indicates that positive restraint moment due to creep and shrinkage deformations developed at the continuous supports of Specimen 4. It is deemed that the larger amount of prestressing acting in the precast box girders caused higher positive restraint moments at the interior supports. The box girders were prestressed by 20 strands, whereas the I-beams were prestressed by 12 strands. The larger effect of creep deformations under the prestressing force was reflected by the positive restraint moment. Furthermore, the cross-sectional area of the deck slab in the box specimen is less than that in the I-beams. Consequently, the shrinkage restraint moment, which counteracts the positive moment due to creep under prestressing, was less in the box beams. It can be concluded that positive moment reinforcement could be needed in the cast-in-place diaphragm if tensile stresses exceed the concrete cracking capacity. It

must be noted that the girder age at the time continuity was established was a lot higher for the I-beam specimens than for the box beam specimen. This allowed for larger shrinkage induced restraint moments in the I-beam specimens.

3.4 Summary

The time-dependent restraint moments, determined using the end reaction measurements were compared to the results obtained using the PCA and CTL methods. The comparison showed that the restraint moments calculated using the PCA method were in good agreement with the measured values when continuity was established at early ages of the precast girders. The CTL method showed improved agreement with the test results when the restraint of the slab reinforcing steel was accounted for at an earlier age of continuity. In this case, the CTL method had a better agreement with test results in the instances where continuity was established at later ages of the precast girders.

In the next chapter the analysis of the results obtained from testing the continuous members under the effect of the superimposed load will be presented. Summary, conclusions and future work are presented in chapter 5.

CHAPTER 4

SUPERIMPOSED LOAD EFFECTS

4.1 Introduction

The superimposed load test results obtained in this study are compared to the theoretical values predicted using the PCA and CTL methods. The PCA method assumes full structural continuity for the calculation of the restraint moments due to superimposed loads. The beam support in the diaphragm region is assumed to be a knife edge support. The CTL method on the other hand considers the finite length and stiffness of the diaphragm between the precast girders. Figure (4.1) shows the two different interior connection models used in this study for the I-beam specimens. With the CTL method, one model employs an uncracked section of the composite girder and the other a cracked section. Figure (4.2) shows the interior connection modeling used to analyze the results of Specimen 4 (box-beam specimen). The composite beam cross-section was used in the analysis for both types of specimens.

4.1.1 Effective Strand Stress

Strains in the prestressing strands were measured by electrical resistance strain gages. The measured strain was converted to stress by multiplying it by the modulus of elasticity of the prestressing strand. Table (4.1) gives the effective strand stress at the time of application of the superimposed load. These values were used in all the calculations performed in this investigation.

Table (4.1)
Effective Strand Stress

Specimen No.	Beam	Effective Strand Stress (ksi)
1	50% Debonded	133.7
	Fully Bonded	126.1
2	67% Debonded	153.7
	Fully Bonded	140.7
3	83% Debonded	164.0
	Fully Bonded	160.0
4	50% Debonded	164.1
	Fully Bonded	161.1

4.1.2 Continuity for Superimposed Loads

The behavior of precast girders made continuous with a cast-in-place diaphragm and slab was examined by studying the variation of the continuity moment developed at the interior support due to the superimposed load P . The moment at the interior support was calculated based on the end reaction measurements due to the applied load during the first phase of loading. A comparison between the predicted values, by linear-elastic analysis for the negative moment at the interior support using a rigid connection assumption (current approach) and a flexible connection (CTL Method), with the measured experimental values is shown in Figures (4.3-4.6). It can be seen that prior to the observed cracking of the diaphragm region, the predicted continuity moment from the rigid connection approach (PCA), the CTL approach, and the measured values are in reasonable agreement.

After cracking, the PCA method (rigid connection) considerably overestimates the moment at the interior support. With the flexible connection approach (CTL), the calculated values after diaphragm region cracking are obtained using the cracked transformed section of the composite girder and modifying the stiffness of the short span between girder ends (flexible connection). As can be seen from Figures (4.3-4.6), this modification results in an improved conservative estimate of the experimentally determined values.

4.1.3 Flexural Cracking

The predicted continuity moment required to produce flexural cracking in the diaphragm region was calculated using the following equation:

$$M_{cr} = \frac{f_r I}{n y_t} \quad (4.1)$$

where:

M_{cr} = moment causing flexural cracking at section due to applied loads

f_r = modulus of rupture of deck slab concrete

$$f_r = 7.5 \sqrt{f'_c}$$

I = moment of inertia of the composite section

y_t = distance from the centroid of the section to the extreme fiber in tension

n = ratio of modulus of elasticity of slab concrete to the modulus
of elasticity of girders concrete

The predicted flexural cracking loads according to the PCA and CTL methods are compared to the observed cracking loads in Table (4.2). The effects of creep and

shrinkage were included in this analysis.

Table (4.2)
Flexural Cracking Loads

Specimen No.	Model	Predicted Load (kips)	Observed Load (kips)	obs/pred
1	PCA	35	24	0.69
	CTL	44	24	0.55
2	PCA	38	25	0.66
	CTL	43	25	0.58
3	PCA	36	25	0.69
	CTL	41	25	0.61
4	PCA	60	27.5	0.46
	CTL	69	27.5	0.40

It can be seen that both methods overestimated the flexural cracking load in the diaphragm region. The PCA method gives slightly closer values to the test results, since higher negative bending moments are obtained by this method at the interior supports.

4.1.4 Web-Shear Cracking

After further cracking of the deck had occurred, the next stage of cracking observed in the continuous members was inclined shear cracking near the continuous support. Web-shear cracks are diagonal cracks that form in the web near the centroid of the member. The critical section for web-shear cracking was taken as $H/2$ away from the face of the interior support (H is the total depth of the composite beam). The ACI [1989]/AASHTO [1989] web-shear cracking capacity is determined as follows:

$$V_{cw} = (3.5 \sqrt{f'_c} + 0.3 f_{pc}) b_w d \quad (4.2)$$

where:

V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in the web

f'_c = compressive strength of concrete

f_{pc} = compressive stress (after allowance for prestress losses) at the centroid of the composite section, or at the junction of the web and flange when centroid lies within the flange, due to both prestressing and the moment resisted by the precast member acting alone

b_w = web width of the girder

d = distance from the extreme compression fiber to the centroid of the longitudinal tension reinforcement

Using the calculated value of V_{cw} , the superimposed load required to produce web-shear cracking was found by conventional linear-elastic analysis using the analytical models specified by PCA and CTL methods (see Figures 4.1 and 4.2). Tables 4.3 and 4.4 present the results of the web-shear cracking analysis, and give a comparison of the web-shear cracking loads for the beams with debonded strands and the fully bonded beams. Table (4.3) gives the web-shear cracking loads at the critical section, $H/2$ away from the continuous support. Table (4.4) gives the web-shear cracking loads based on the web-shear cracking capacity at the locations where these cracks observed. The two methods used in the analysis underestimated the web-shear cracking capacity of both the debonded and fully bonded I-beams. However, these methods overestimated the web-shear cracking loads for the continuous box girder. The better agreement for the I-beam specimens was obtained, as expected, using the cracked section for the diaphragm region. This was not the case for the box beam specimen.

Table (4.3)
Web-Shear Cracking Loads at Critical Section (H/2)

Specimen No.	Beam	Predicted Load (kips)	Observed Load (kips)	obs/pred
1	50% Debonded			
	PCA	57	90	1.58
	CTL-UC	56	90	1.61
	CTL-CR	59	90	1.53
	Fully Bonded			
	PCA	68	90	1.32
	CTL-UC	67	90	1.34
	CTL-CR	71	90	1.27
2	67% Debonded			
	PCA	54	70	1.30
	CTL-UC	54	70	1.30
	CTL-CR	56	70	1.25
	Fully Bonded			
	PCA	63	70	1.11
	CTL-UC	63	70	1.11
	CTL-CR	66	70	1.06
3	83% Debonded			
	PCA	49	70	1.43
	CTL-UC	49	70	1.43
	CTL-CR	51	70	1.37
	Fully Bonded			
	PCA	66	77	1.17
	CTL-UC	66	77	1.17
	CTL-CR	68	77	1.13
4	50% Debonded			
	PCA	116	94.5 (118)	0.81
	CTL-UC	116	94.5 (118)	0.81
	CTL-CR	122	94.5 (118)	0.77
	Fully Bonded			
	PCA	144	136 (150)	0.94
	CTL-UC	144	136 (150)	0.94
	CTL-CR	151	136 (150)	0.90

Table (4.4)
Web-Shear Cracking Loads at Initial Crack Location

Specimen No.	Beam	Predicted Load (kips)	Observed Load (kips)	obs/pred
1	50% Debonded			
	PCA	56	90	1.61
	CTL-UC	56	90	1.61
	CTL-CR	58	90	1.55
	Fully Bonded			
	PCA	67	90	1.34
	CTL-UC	67	90	1.34
	CTL-CR	70	90	1.29
2	67% Debonded			
	PCA	53	70	1.32
	CTL-UC	53	70	1.32
	CTL-CR	55	70	1.27
	Fully Bonded			
	PCA	65	70	1.08
	CTL-UC	65	70	1.08
	CTL-CR	68	70	1.03
3	83% Debonded			
	PCA	50	70	1.40
	CTL-UC	50	70	1.40
	CTL-CR	52	70	1.35
	Fully Bonded			
	PCA	58	77	1.33
	CTL-UC	58	77	1.33
	CTL-CR	61	77	1.26
4	50% Debonded			
	PCA	122	94.5 (118)	0.77
	CTL-UC	122	94.5 (118)	0.77
	CTL-CR	129	94.5 (118)	0.73
	Fully Bonded			
	PCA	131	136 (150)	1.04
	CTL-UC	131	136 (150)	1.04
	CTL-CR	138	136 (150)	0.99

The early web-shear cracking in the box-beam specimen was caused by the difference in the thickness of the walls on the two sides of the beam. In both beams, one wall had a thickness of 4 3/8 inches while the other had a thickness of 5 5/8 inches. The values in parentheses in Tables 4.3 and 4.4 were the observed loads when web-shear

cracking occurred in the thicker walls. In general, the better agreement was obtained for fully bonded specimens. For the I-beam specimens with debonded strands the predicted values were more conservative than for the fully bonded specimens.

4.1.5 Flexure-Shear Cracking

The final stage of cracking that occurred in the continuous members was flexure-shear cracking. Flexure-shear cracking results from diagonal cracks that extend from already existing flexural cracks. The ACI/AASHTO flexure-shear capacity is given by the following expression:

$$V_{ci} = 0.6 \sqrt{f'_c} b_w d + V_d + \frac{V_i M_{cr}}{M_{max}} \quad (4.3)$$

where:

V_d = shear force at section due to unfactored dead load

V_i = factored shear force at section due to externally applied loads

M_{max} = maximum factored bending moment at section due to externally applied loads

M_{cr} = moment causing flexural cracking at section due to externally applied loads

$$M_{cr} = \frac{I}{y_t} (6\sqrt{f'_c} + f_{pe} - f_d)$$

I = moment of inertia of composite section

y_t = distance from centroid of composite section to extreme tension fiber

f_{pe} = compressive stress, due to effective prestressing, at extreme fiber of section where
tensile stresses are caused by externally applied loads

f_d = stress due to unfactored dead load, at extreme fiber of section where
tensile stresses are caused by externally applied loads

Using the calculated values of V_{ci} , the flexure-shear cracking loads were evaluated using elastic analysis with the diaphragm idealized as suggested by PCA and CTL methods. Table (4.5) summarizes the predicted flexure-shear cracking loads and the corresponding test results for the four test specimens.

For Specimen 1, the flexure-shear cracking loads were calculated at the point of application of the superimposed load. The flexure-shear crack occurred near the debonding point, a distance of 44.5 inches from the continuous support. Since the debonding point was near the point of inflection, very high loads would be required to produce flexure-shear cracking at that location. Flexure-shear cracking loads for the beam with 67% debonding in Specimen 2 were calculated at the second debonding point, located at a distance of 77 inches from the continuous support. For the fully bonded beam, flexure-shear cracking loads were calculated at the point load. For Specimen 3, flexure-shear cracking occurred at the second debonding point, located at a distance of 66 inches from the continuous support. The predicted loads required to produce flexure-shear cracking at that location were calculated. For the fully bonded beam the analysis was carried at the point load. Flexure-shear cracking loads for both beams in Specimen 4 were evaluated at the location of the applied load.

The results obtained using CTL method with the cracked transformed section of the diaphragm region, coupled with the ACI/AASHTO equations, although providing the better agreement, substantially overestimated the flexure-shear cracking loads for the debonded I-beams. This finding in the continuous specimens agrees with the earlier flexure-shear cracking observed in simply supported members with debonded strands.

Table (4.5)
Flexure-Shear Cracking Loads

Specimen No.	Beam	Predicted Load (kips)	Observed Load (kips)	obs/pred
1	50% Debonded			
	PCA	215	140	0.65
	CTL-UC	220	140	0.64
	CTL-CR	173	140	0.81
	Fully Bonded			
	PCA	204	-	-
	CTL-UC	209	-	-
	CTL-CR	164	-	-
2	67% Debonded			
	PCA	272	130	0.48
	CTL-UC	272	130	0.48
	CTL-CR	194	130	0.67
	Fully Bonded			
	PCA	237	-	-
	CTL-UC	237	-	-
	CTL-CR	193	-	-
3	83% Debonded			
	PCA	325	91	0.28
	CTL-UC	331	91	0.27
	CTL-CR	179	91	0.51
	Fully Bonded			
	PCA	232	-	-
	CTL-UC	234	-	-
	CTL-CR	191	-	-
4	50% Debonded			
	PCA	439	-	-
	CTL-UC	442	-	-
	CTL-CR	343	-	-
	Fully Bonded			
	PCA	432	-	-
	CTL-UC	434	-	-
	CTL-CR	337	-	-

note:

(-) means no flexure-shear cracking was observed

4.1.6 Ultimate Shear Strength

In these tests, shear failure of the girders did not occur since the maximum applied loads were below the shear failure loads of the beams. The shear force at which shear failure was likely, was calculated by adding the lower value of V_{cw} and V_{ci} , to the contribution of the web reinforcement, V_s . The shear strength provided by the web reinforcement was calculated by the following equation:

$$V_s = \frac{A_v f_y d}{s} \quad (4.4)$$

where:

V_s = nominal shear strength provided by web reinforcement

f_y = yield strength of web reinforcement

A_v = cross-sectional area of the stirrups

d = distance from extreme compression fiber to the centroid of longitudinal tension reinforcement

s = stirrup spacing

Table (4.6) gives the shear failure loads calculated using the PCA and CTL models. As mentioned before shear failure did not occur in these tests. Specimen 3 was loaded up to 164 kips. However shear failure did not occur as predicted by PCA and CTL methods.

Table (4.6)
Shear Failure Loads

Specimen No.	Beam	Predicted Load (kips)	Observed Maximum Load (kips)	obs/pred
1	50% Debonded			
	PCA	177	140	0.79
	CTL-UC	176	140	0.80
	CTL-CR	183	140	0.77
	Fully Bonded			
	PCA	196	140	0.71
	CTL-UC	194	140	0.72
	CTL-CR	205	140	0.68
2	67% Debonded			
	PCA	172	162	0.94
	CTL-UC	172	162	0.94
	CTL-CR	179	162	0.91
	Fully Bonded			
	PCA	191	162	0.85
	CTL-UC	191	162	0.85
	CTL-CR	199	162	0.81
3	83% Debonded			
	PCA	167	164	0.98
	CTL-UC	167	164	0.98
	CTL-CR	174	164	0.94
	Fully Bonded			
	PCA	194	164	0.84
	CTL-UC	194	164	0.84
	CTL-CR	201	164	0.82
4	50% Debonded			
	PCA	226	176	0.78
	CTL-UC	226	176	0.78
	CTL-CR	238	176	0.74
	Fully Bonded			
	PCA	255	176	0.69
	CTL-UC	255	176	0.69
	CTL-CR	268	176	0.66

note:

Shear failure did not occur ,however, maximum load can still be used for comparison purposes

4.1.7 Continuity Moment Evaluation

The continuous cast-in-place deck slab was reinforced with 8 #6 Grade 60 bars over the interior supports. The strain gages on this reinforcement indicated that yielding did not occur at the maximum applied loads for all beams tested.

Based on the assumptions that plane sections remain plane after bending, and that no slip occurs between the concrete and steel, the negative moment at the section near the diaphragm was calculated from the readings of the strain gages placed on the longitudinal slab reinforcement. These values are compared in Table (4.7) with the required continuity moments, without load factors and with $\phi=1.0$, according to PCA and CTL design methods. The negative bending moment calculated based on end reaction readings and the predicted nominal flexural capacity in accordance with current AASHTO [1989] specifications are also shown in Table (4.7).

Although, the forces induced in a composite structure by creep and shrinkage of concrete would be relevant in checking the allowable stress levels, before cracking of the diaphragm region, they need not be included in ultimate strength checks. These stresses are relieved by cracking of concrete and yielding of reinforcing steel. It has been shown that (Mattock [1961]) the deformations due to creep and shrinkage do not influence the ultimate load capacity of continuous beams of the type considered in this study.

Table (4.7)
Continuity Moment at Diaphragm

Specimen	Continuity Moment Based on P_{max} (ft-k)			Measured Moment Based on Strain Gage Readings (ft-k)	Measured Moment Based on End Reaction Readings (ft-k)	Predicted Nominal Moment Capacity (AASHTO) (ft-k)
	PCA	CTL				
		uncracked diaphragm	cracked diaphragm			
$(P^1_{max}=140k)$	652	513	375	318	260	492
$(P^2_{max}=162k)$	748	651	510	373	391	501
$(P^3_{max}=164k)$	759	659	521	481	507	499
$(P^4_{max}=176k)$	814	707	517	341	335	485

It can be seen from Table (4.7) that, the CTL method with cracked diaphragm showed better agreement with the measured continuity moment. These findings confirm the results shown in Figures (4.3-4.6). It is also clear that the PCA method is extremely conservative for design purposes in the negative moment region if no redistribution is accounted for, and it could also lead to unconservative estimate of the bending moment in positive moment regions. Finally, when compared to current AASHTO predicted capacity for all specimens, both the PCA and CTL methods are conservative. Actual failure of the diaphragm region did not occur in these tests.

Table (4.8) shows, that in spite of the reduction in the measured maximum positive moment caused by flexure-shear cracking in the debonded beams compared to the fully bonded ones of Specimens 2 and 3, the PCA method resulted in lower estimates of the positive bending moments in the span of these beams. It must be noted that the flexure-shear cracking in Specimen 3 occurred during the first loading phase ($P=91$ kips) while in Specimen 2 occurred during the final loading phase. This would explain the larger difference between the measured positive moments in Specimen 3. The CTL method with cracked section gives a better estimate of the measured values in the positive moment region.

Table (4.8)
Positive Bending Moment at the Point Load

Specimen No.	Positive Bending Moment Based on P_{max} (ft-k)			Measured Positive Moment Based on End Reaction Readings (ft-k)	
	PCA	CTL		Debonded Beam	Fully Bonded Beam
		uncracked diaphragm	cracked diaphragm		
1 ($P_{max}=140k$)	339	331	429	503	514
2 ($P_{max}=162k$)	393	390	488	538	598
3 ($P_{max}=164k$)	398	395	491	439	548
4 ($P_{max}=176k$)	428	424	556	676	682

4.1.8 Bottom Fiber Stress Evaluation

Current AASHTO Specifications limit the service load stresses in the extreme compression fiber at interior girder ends to $0.6 f'_c$. This requirement specifies consideration of "effects of prestressing and negative live load bending". However, the specifications do not clearly define how far from the girder ends this allowable stress should apply. The Indiana Department of Transportation in trying to promote uniformity in design, requires that the allowable compressive stresses shown in Table (4.9) be followed.

Table (4.9)
Allowable Compressive Stresses at Girder Ends (INDOT)

Location	Allowable stress
At the girder end	$0.6 f'_c$
2 ft. from the girder end	$0.5 f'_c$
All other locations	$0.4 f'_c$

Debonding of strands in the end regions of a prestressed concrete beam is often necessary to meet this stress requirement. However, to prevent excessive reduction in the shear strength near the girder ends, INDOT further requires that no more than 50% of the total number of strands be debonded. It also suggests not to debond an entire row of strands.

The compressive stresses at the girder ends in the continuous specimens were monitored using the surface gages shown in Figure (2.47) and Figure (2.70). The measurements obtained from the surface gages due to time-dependent deformations and the applied loads during the final loading phase were added to the calculated strains due to the effective prestressing at the same locations. The corresponding value of stress was calculated using the well known Hogenstad stress-strain relationship for concrete (see Lin and Burns [1981]). These stresses were then converted to a dimensionless quantity K , the stress factor, which is equal to the calculated bottom fiber stress divided by the compressive strength of the girder concrete.

The strain gages labeled 4 and 6 were placed on the beams with debonded strands. The gages 7 and 8 were placed on the fully bonded beams. In Specimen 1, gages 6 and 7 were 3 inches away from the ends of the beams, and gages 4 and 8 were 25 inches away from the ends. In Specimens 2, 3, and 4 gages 6 and 7 were at a distance of 10 inches from the ends, while gages 4 and 8 were at 29 inches from the end.

To evaluate the behavior of the compression region near the interior supports, the K factors calculated from strain gage readings and from measured end reactions are compared in Figures (4.7-4.22). The measured end reactions included the effects of creep and shrinkage as well as the applied superimposed load.

In each specimen the stresses calculated from the readings of gages 6 and 7 are higher than those from the measured end reactions. However, the stresses at these locations are affected by the compressive reaction at the interior supports.

To compare the stresses from analysis to the measured values, the stresses were calculated using the PCA method and CTL method, with cracked diaphragm, and compared to the experimental results obtained from end reactions measurement in Figures (4.23-4.38). It can be seen that CTL method shows better agreement with test, based on reaction measurements, however gives low estimate when compared to strain gages due to effect of reaction near the supports.

It is worth mentioning that, the time-dependent restraint moment due to creep and shrinkage need not be included in computing the bottom fiber stresses after flexural cracking of the diaphragm region had occurred. The stresses due to creep and shrinkage deformation were relieved by cracking on top of the concrete diaphragm. Table (4.10)

compares the measured end reactions at three stages: before continuity was established, before cracking of the diaphragm region and after cracking of the diaphragm region. It can be seen that on cracking of the diaphragm, the end reactions changed towards the initial values before continuity was established. The measured values of the end reaction shown in Table (4.10) were taken under zero applied superimposed load. This confirmed that the stresses due to creep and shrinkage were relieved after cracking of the deck above the diaphragm region.

Table (4.10)
Variation of End Reactions

Specimen No.	Measured End Reaction (kips)		
	Before Continuity	Before Cracking	After Cracking
1	1.2	0.75	2.09
2	6.49	4.28	6.55
3	6.23	4.50	6.48
4	8.16	9.92	9.35

The measured compressive stress at the location of Gages 4 and 8 in Specimen 1 did not exceed the allowable limit of $0.4 f'_c$. However, the stresses computed using the PCA method as shown in Figures (4.23) and (4.25) exceeded the allowable limit. Therefore it can be concluded that evaluating stresses at these locations using the PCA method would be quite conservative.

In Specimens 2 and 3 the measured stresses based on end reaction measurements at Gage 8 location exceeded the allowable limit of $0.4 f'_c$ specified by INDOT (see

Figures 4.29 and 4.33). The compressive stress obtained using the PCA method at this location was found to be $0.6 f'_c$. This confirmed that exceeding the allowable limit of $0.4 f'_c$ did not influence the service load behavior of the fully bonded beam. The results obtained from Specimen 4 confirmed the previously mentioned observation that the PCA method overestimated the bottom fiber stresses at the interior supports of continuous beams.

The bottom fiber stresses based on strain measurements at the location of Gages 6 and 7 in Specimens 2 and 3 exceeded the allowable stress limit of $0.6 f'_c$. It can be concluded that exceeding the allowable stress limit did not affect the linear behavior of the fully bonded beams. It is worth noting that at these locations the lateral restraint provided by the diaphragm provided beneficial confinement to the end region of the beams at the continuous support.

4.1.9 Summary

The behavior of the test specimen under the effect of the superimposed load is presented. Shear as well as flexural behavior were examined in this chapter. The PCA and CTL methods were used to predict the test results. The PCA method assumes full continuity at the interior supports. The CTL method considers the finite length and stiffness of the diaphragm between the precast girders. The CTL method gave closer values to the test results after flexural cracking of the diaphragm by incorporating the cracked transformed section of the composite girders at the diaphragm region.

Measured values based on end reaction readings showed that the time-dependent restraint moments due to creep and shrinkage were released after cracking of the

diaphragm region. A comparison of values from strain gage readings and end reaction measurements indicated that the presence of a support reaction results in an increase on the compressive stress at girder ends above that induced by bending moments. The PCA method resulted in an overestimation of the bottom fiber stresses at end regions located 25 and 29 inches from the beam ends at the continuous support. Measured values of the bottom fiber stress using strain gage readings at 3 and 10 inches from the end of the girder showed values in excess of $0.6 f'_c$ in specimens 2 and 3. In this region of the member, the PCA method provided a conservative estimate of the bottom fiber stresses. The CTL method provided the better agreement with measured values based on end reaction readings at all gage locations. A summary and conclusions drawn from the experimental program in this report will be presented in the following chapter.

CHAPTER 5

SUMMARY AND CONCLUSIONS

5.1 Summary

This report presents the results of an experimental investigation directed towards evaluating three types of behavior in pretensioned concrete bridges. Four two-span composite specimens with different strand debonding schemes were fabricated and tested to evaluate: (1) time-dependent effects due to creep and shrinkage deformations of prestressed precast bridge girders made continuous with a cast-in-place slab, (2) shear and flexural behavior of prestressed girders with debonded strands at continuous supports, and (3) compressive stresses near the ends of pretensioned girders at continuous supports.

The time-dependent behavior was examined by establishing continuity between two prestressed precast girders using a cast-in-place slab and diaphragm. The restraint moments that developed at the continuous supports due to prestress induced creep of the precast girders, and of differential shrinkage between the precast girders and the cast-in-place slab were experimentally determined. The predicted restraint moments obtained using the PCA and CTL methods were compared with the measured values from the variation in the end support reactions.

The continuity behavior of the test specimens for the applied superimposed loads was examined by comparing the measured continuity moments with the corresponding values predicted using the PCA and CTL analytical methods.

The results obtained from testing the continuous structures under the effect of a static two-point loading were used to evaluate the ACI/AASHTO provisions for shear and flexural cracking loads at continuous supports of multi-span precast-pretensioned bridges.

The bottom fiber stresses at the continuous supports of pretensioned girders, due to effective prestressing and applied loads, were estimated using the PCA and CTL models. The predicted stresses were compared with the measured test values to evaluate both methods.

5.2 Conclusions

From the observations and analysis of the continuous tests the following conclusions can be made:

1. The time-dependent creep and shrinkage deformations in this type of construction induced restraint moments at the continuous supports. Positive restraint moments were measured at the interior support of the continuous composite box-beam specimen.
2. The time-dependent restraint moments computed using the PCA method were in good agreement with the test values when continuity was established at early ages of the precast girders. Large differences were observed when continuity was established at late ages of the precast members. The CTL method gave improved correlation with measured results when the modification to account for restraint of the top steel was introduced at an earlier age of the cast-in-place slab concrete.

3. Similar behavior was exhibited by both the debonded girders and the fully bonded girders, in regard to deflections under the effect of the superimposed loads, up to the formation of flexure-shear cracking. The debonded beams had larger deflections after flexure-shear cracking than the fully bonded beams where flexure-shear cracking did not occur.
4. Before cracking of the top slab in the diaphragm region both the PCA and CTL methods yielded reasonable and conservative estimates of the measured continuity moment due to the applied superimposed loads based on end reaction measurements. After cracking, however, the CTL method using the cracked transformed section of the composite girder at the diaphragm region gave better results than the PCA method. The PCA method significantly overestimated the bending moments at the continuous supports after flexural cracking. Consequently, using the PCA method resulted in the flexural requirements in the positive moment regions being underestimated, even though flexure-shear cracking led to a reduction in the positive moment due to further redistribution.
5. The flexural capacity at the continuous support was further examined using the results of the strain gages in the slab negative moment continuity reinforcement over the interior supports. It was confirmed that the PCA method gave very conservative estimates of the continuity moment. The theoretical results obtained by the CTL method with the cracked diaphragm assumption gave an improved conservative estimate of the negative moments at interior supports for all the beams tested.

6. The time-dependent restraint moment due to creep and shrinkage need not be included in computing the bottom fiber stresses after flexural cracking of the slab over the diaphragm region. Also the time-dependent restraint moments should not be included in the calculation of the ultimate load of the continuous beams. These moments were relieved by flexural cracking of the concrete slab over the diaphragm.
7. The ACI/AASHTO equations coupled with the PCA and CTL models gave conservative estimates of the web-shear cracking loads for the fully bonded as well as the debonded I-girders . These models yielded slightly unconservative estimates of the test values for the box girders.
8. Flexure-shear cracking developed in the I-shaped beams earlier than predicted by PCA and CTL methods in the positive moment region. It was noted that debonding the prestressing strands in the positive moment regions of these beams resulted in the premature opening of these cracks. All flexure-shear cracks originated in the bottom flange of the girders at the debonding points located near the applied loads. It was noticed that flexure-shear cracking reduced the positive bending moment at the applied superimposed load location indicating further redistribution of moments.
9. Strand debonding in the negative moment region near the continuous supports did not significantly influence the web-shear cracking capacity of the girders. Direct comparison of the ultimate shear capacity can not be made since no shear failure was observed in these tests.

10. The measured compressive strains at the continuous supports were affected by the compressive reaction at the supports. However, away from the supports the measured bottom stresses from the surface gages were in excellent agreement with the values computed from end reaction measurements. Comparison of the measured stresses with the theoretical results, indicated that both the PCA and CTL methods underestimated the compression fiber stresses, based on strain measurements, near the continuous supports. However, away from the support the PCA method substantially overestimated the bottom fiber stresses, while the CTL method with cracked diaphragm showed a much improved agreement.
11. Based on strain gage readings at the end regions near the continuous support, the compressive stresses exceeded the $0.6 f'_c$ limit. However no detrimental effect on the service load behavior of the beams was observed during the tests.

5.3 Future Work

Future research is needed to determine the available capacity for redistribution at the continuous support of precast pretensioned concrete girder bridges with debonding members.

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Appendix A - Time-Dependent Restraint Moments

Table (A.1)
Restraint Moments for Specimen 1

Age of girder (days)	Measured end Reaction (kips)	Change in Reaction (kips)	Restraint Moment (ft-k)
113	1.2	0.0	0.0
123	-0.49	-1.69	-40.6
130	-0.47	-1.67	-40.1
140	0.17	-1.03	-24.7
157	0.75	-0.45	-10.8

The end reaction due to self-weight of girder=3.44 kips

The end reaction due to slab weight (simple supports)=2.39 kips

(-) Restraint moment represents tension at the top of the cross-section

Table (A.2)
Restraint Moments for Specimen 2

Age of girder (days)	Measured end Reaction (kips)	Change in Reaction (kips)	Restraint Moment (ft-k)
49	6.49	0.00	0.0
50	5.62	-0.87	-21.17
51	5.85	-0.64	-15.57
52	5.58	-0.91	-22.14
53	5.21	-1.28	-31.15
54	4.99	-1.50	-36.50
55	4.87	-1.62	-39.41
56	4.52	-1.92	-47.93
57	4.26	-2.23	-54.26
58	4.12	-2.37	-57.66
59	4.02	-2.47	-60.10
60	3.82	-2.67	-64.96
61	3.87	-2.62	-63.72
62	3.83	-2.66	-64.72
86	4.28	-2.21	-53.77

The end reaction due to self-weight of girder=3.69 kips

The end reaction due to slab weight (simple supports)=2.80 kips

(-) Restraint moment represents tension at the top of the cross-section

Table (A.3)
Restraint Moments for Specimen 3

Age of girder (days)	Measured end Reaction (kips)	Change in Reaction (kips)	Restraint Moment (ft-k)
91	6.23	0.0	0.0
92	5.15	-1.08	-26.28
93	5.38	-0.85	-20.68
94	5.19	-1.04	-25.31
95	4.74	-1.49	-36.26
96	4.42	-1.81	-44.04
97	4.58	-1.65	-40.15
98	4.44	-1.79	-43.56
99	4.18	-2.05	-49.88
100	4.06	-2.17	-52.80
101	3.99	-2.24	-54.51
102	3.81	-2.42	-58.89
103	3.98	-2.25	-54.75
104	4.09	-2.14	-52.07
105	3.97	-2.26	-54.99
106	4.05	-2.19	-53.17
107	4.26	-1.97	-47.94
108	4.29	-1.94	-47.13
110	4.32	-1.91	-46.48
111	4.26	-1.97	-47.94
113	4.50	-1.73	-42.09

The end reaction due to self-weight of girder=3.84 kips

The end reaction due to slab weight (simple supports)=2.39 kips

(-) Restraint moment represents tension at the top of the cross-section

Table (A.4)
Restraint Moments for Specimen 4

Age of girder (days)	Measured end Reaction (kips)	Change in Reaction (kips)	Restraint Moment (ft-k)
26	8.16	0.0	0.0
27	7.48	-0.68	-16.5
28	7.48	-0.68	-16.5
29	7.75	-0.41	-10.0
30	7.85	-0.31	-7.5
31	7.96	-0.20	-4.9
32	8.05	-0.11	-2.7
33	8.03	-0.13	-3.2
34	8.21	0.06	1.5
36	8.77	0.61	14.8
38	9.17	1.00	24.3
39	9.10	0.94	22.8
40	9.43	1.27	30.9
41	9.47	1.31	31.9
42	9.44	1.28	31.1
43	9.48	1.32	32.2
45	9.46	1.30	31.6
46	9.50	1.35	32.9
47	9.56	1.40	34.1
48	9.53	1.37	33.3
49	9.34	1.18	28.7
50	9.36	1.20	29.2
51	9.36	1.20	29.2
52	9.46	1.30	31.6
53	9.42	1.26	30.7
54	9.59	1.43	34.8
55	9.75	1.59	38.7
57	9.44	1.28	31.1
58	9.65	1.49	36.3
59	9.92	1.76	42.8

The end reaction due to self-weight of girder=6.67 kips

The end reaction due to slab weight (simple supports)=1.93 kips

(-) Restraint moment represents tension at the top of the cross-section

STAGE 1 - PRECAST GIRDER IN PLACE

STAGE 2 - REINFORCEMENT AT SUPPORT

STAGE 3 - COMPLETED STRUCTURE

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Beam With 0% Debonding

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Figure (2.10) Strand Debonding Scheme and Instrumentation (Specimen 2).

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Beam With 0% Debonding

Beam End IC
0 % Debonding

Beam End ED
0 % Debonding

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Figure (2.33) Measured Stress-Strain Behavior of Mild Steel

#3 Bar, Grade 60 (Specimen 1)

Figure (2.34) Measured Stress-Strain Behavior of Mild Steel

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Beam With 83% Debonding (Specimen 3)

Figure (2.100) Stirrup Strains at Continuous Support, Initial Load Phase
Beam With 0% Debonding (Specimen 3)

Figure (2.101) Stirrup Strains At Continuous Support, Final Load Phase
Beam With 0% Debonding (Specimen 3)

Figure (2.102) Strain in Slab Longitudinal Steel at Centerline of Diaphragm
Initial Load Phase (Specimen 3)

Figure (2.103) Strain in Slab Longitudinal Steel at Centerline of Diaphragm
Final Load Phase (Specimen 3)

Figure (2.104) Strand Strain at 84 in. from Continuous Support, Final Load Phase
Beam with 83% Debonding (Specimen 3)

Figure (2.105) Strand Strain at 42 in. from Continuous Support, Final Load Phase
Beam with 0% Debonding (Specimen 3)

Figure (2.106) Strand Strain at 66 in. from Continuous Support, Final Load Phase
Beam with 0% Debonding (Specimen 3)

Figure (2.107) Strand Strain at 84 in. from Continuous Support, Final Load Phase
Beam with 0% Debonding (Specimen 3)

Figure (2.108) Beam with 50% Debonding at Completion of
Initial Load Phase (Specimen 4)

Figure (2.109) Beam with 50% Debonding at Completion of
Final Load Phase (Specimen 4)

Figure (2.110) Beam with 0% Debonding at Completion of
Final Load Phase (Specimen 4)

Figure (2.111) Deck Cracking over Continuous Supports at Completion
of Initial Load Phase (Specimen 4)

*Dial Gage and LVDT's located on bottom flange of the girder. Concrete surface gages located on cast in place deck.

- -Dial Gage
- X -LVDT
- -Concrete Surface Gages

Figure (2.113) Top View of Location of Dial Gages, LVDT's, and Surface Gages (Specimen 4).

Figure (2.114) Load-Deflection Relationship, Initial Load Phase
Specimen 4

Figure (2.115) Load-Deflection Relationship, Final Load Phase
Specimen 4

Figure (2.116) Load-Deflection Relationship, Initial Load Phase
Specimen 4

Figure (2.117) Load-Deflection Relationship, Final Load Phase
Specimen 4

Figure (2.118) Compressive Strain at Continuous Support
Initial Load Phase (Specimen 4)

Figure (2.119) Compressive Strain at Continuous Support
Final Load Phase (Specimen 4)

Figure (2.120) Stirrup Strains at Continuous Support
Final Load Phase, Beam With 50% Debonding (Specimen 4)

Figure (2.121) Stirrup Strains At Continuous Support
Final Load Phase, Beam With 0% Debonding (Specimen 4)

Figure (2.122) Strain in Slab Steel
at Centerline of Diaphragm, Initial Load Phase
(Specimen 4)

Figure (2.123) Strain in Slab Longitudinal Steel
at Centerline of Diaphragm, Final Load Phase
(Specimen 4)

Figure (2.124) Strand Strain at 48 in. from Continuous Support, Final Load Phase
Beam with 50% Debonding (Specimen 4)

Figure (2.125) Strand Strain at 60 in. from Continuous Support, Final Load Phase
Beam with 50% Debonding (Specimen 4)

Figure (2.126) Strand Strain at the Point Load, Final Load Phase
Beam with 50% Debonding (Specimen 4)

Figure (2.127) Strand Strain at 48 in. from Continuous Support, Final Load Phase
Beam with 0% Debonding (Specimen 4)

Figure (2.128) Strand Strain at 60 in. from Continuous Support, Final Load Phase
Beam with 0% Debonding (Specimen 4)

Figure (2.129) Strand Strain at the Point Load, Final Load Phase
Beam with 0% Debonding (Specimen 4)

Figure (3.1) Variation with Time of Support Restraint Moment
Considering Shrinkage Modification after 28 Days
and Effects of Slab Top Steel after 30 Days

Specimen 1

Figure (3.2) Variation with Time of Support Restraint Moment
Considering Shrinkage Modification after 28 Days
and Effects of Slab Top Steel after 30 Days
Specimen 2

Figure (3.3) Variation with Time of Support Restraint Moment
Considering Shrinkage Modification after 28 Days
and Effects of Slab Top Steel after 30 Days
Specimen 3

Figure (3.4) Variation with Time of Support Restraint Moment
Considering Shrinkage Modification after 28 Days
and Effects of Slab Top Steel after 30 Days
Specimen 4

Figure (3.5) Variation with Time of Support Restraint Moment
Considering Shrinkage Modification and
Effects of Slab Top Steel after 3 Days

Specimen 1

Figure (3.6) Variation with Time of Support Restraint Moment
Considering Shrinkage Modification
and Effects of Slab Top Steel after 3 Days
Specimen 2

Figure (3.7) Variation with Time of Support Restraint Moment
Considering Shrinkage Modification after 3 Days
and Effects of Slab Top Steel after 3 Days
Specimen 3

Figure (3.8) Variation with Time of Support Restraint Moment
Considering Shrinkage Modification and
Effects of Slab Top Steel after 3 Days
Specimen 4

Beam Model

Diaphragm Cross-section

Figure (4.1) Beam Model Used in the Analysis of I-beams.

Beam Model

Diaphragm Cross-section

Figure (4.2) Analytical Models of Specimen 4.

Figure (4.3) Variation of Continuity Moment due to Superimposed Load (P), Initial Load Phase Specimen 1

Figure (4.4) Variation of Continuity Moment due to
Superimposed Load (P), Initial Load Phase
Specimen 2

Figure (4.5) Variation of Continuity Moment due to Superimposed Load (P), Initial Load Phase Specimen 3

Figure (4.6) Variation of Continuity Moment Due to Superimposed Load (P), Initial Load Phase Specimen 4

Figure (4.7) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 4 (Specimen 1)

Figure (4.8) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 6 (Specimen 1)

Figure (4.9) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 8 (Specimen 1)

Figure (4.10) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 7 (Specimen 1)

Figure (4.11) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 4 (Specimen 2)

Figure (4.12) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 6 (Specimen 2)

Figure (4.13) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 8 (Specimen 2)

Figure (4.14) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 7 (Specimen 2)

Figure (4.15) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 4 (Specimen 3)

Figure (4.16) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 6 (Specimen 3)

Figure (4.17) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 8 (Specimen 3)

Figure (4.18) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 7 (Specimen 3)

Figure (4.19) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 4 (Specimen 4)

Figure (4.20) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 6 (Specimen 4)

Figure (4.21) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 8 (Specimen 4)

Figure (4.22) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 7 (Specimen 4)

Figure (4.23) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 4 Location (Specimen 1)

Figure (4.24) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 6 Location (Specimen 1)

Figure (4.25) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 8 Location (Specimen 1)

Figure (4.26) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 7 Location (Specimen 1)

Figure (4.27) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 4 Location (Specimen 2)

Figure (4.28) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 6 Location (Specimen 2)

Figure (4.29) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 8 Location (Specimen 2)

Figure (4.30) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 7 Location (Specimen 2)

Figure (4.31) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 4 Location (Specimen 3)

Figure (4.32) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 6 Location (Specimen 3)

Figure (4.33) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 8 Location (Specimen 3)

Figure (4.34) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 7 Location (Specimen 3)

Figure (4.35) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 4 Location (Specimen 4)

Figure (4.36) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 6 Location (Specimen 4)

Figure (4.37) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 8 Location (Specimen 4)

Figure (4.38) Compressive Stress Distribution at
The Bottom of Girder, Final Load Phase
Gage 7 Location (Specimen 4)

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