Precast Prestressed Concrete Panel Subdecks in Skewed Bridges

Interim Report

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Abstract

Precast prestressed concrete panels have been used in bridge deck construction in Iowa and many other states. To investigate the performance of these panels at abutment or pier diaphragm locations for bridges with various skew angles, a research program involving both analytical and experimental aspects, is being conducted. This interim report presents the status of the research with respect to four tasks. Task 1 which involves a literature review and two surveys is essentially complete. Task 2 which involved field investigations of three Iowa bridges containing precast panel subdecks has been completed. Based on the findings of these investigations, future inspections are recommended to evaluate potential panel deterioration due to possible corrosion of the prestressed strands. Task 3 is the experimental program which has been established to monitor the behavior of five configurations of full scale composite deck slabs. Three dimensional test and instrumentation frameworks have been constructed to load and monitor the slab specimens. The first slab configuration representing an interior panel condition is being tested and preliminary results are presented for one of these tests in this interim report. Task 4 involves the analytical investigation of the experimental specimens. Finite element methods are being applied to analytically predict the behavior of the test specimens. The first test configuration of the interior panel condition has been analyzed for the same loads used in the laboratory, and the results are presented herein. Very good correlation between the analytical and experimental results has occurred.

1. Introduction

This interim report has been written to provide the status of the research project entitled "Precast Prestressed Concrete Panel Subdecks in Skewed Bridges". As described in the research proposal, precast prestressed concrete panels are permanent forms that have been used for casting bridge decks. These slab panels eliminate the need for using conventional temporary formwork between the bridge girders. A reinforced concrete slab is cast on top of the panels forming a composite system with the prestressed panels to resist the applied highway loadings. The panels replace the bottom portion of a conventional full depth reinforced concrete deck, including the bottom layer of reinforcement in both the longitudinal and transverse directions. The total thicknesses of a composite slab and a conventional full depth cast-in-place slab are normally the same.

The research proposal identified four tasks. Task 1 involves a review of the literature and surveys of design agencies and panel producers. Field investigations of three bridges constructed with precast panel subdecks in Iowa are contained in Task 2. Task 3 involves an extensive experimental testing program of full scale specimens. Analytical investigations are contained in Task 4. The work completed to date and future efforts for each of these tasks are discussed in the following sections of this interim report.

2. Task 1 - Literature Review and Surveys

2.1. Background

The literature review has been essentially completed. Many articles have been located that address the use of precast prestressed concrete panels as permanent forms in bridge deck construction. These articles have discussed a variety of issues including reflective cracking in the cast-in-place topping slab at the panel joints, bond between the panels and the reinforced concrete slab, panel bearing, prestress strand extensions beyond the ends of the panels, strand transfer and development lengths, continuity conditions for the panels at the girders, and flexural and shear strengths for composite slabs whose supports occur only at the ends of the prestressed panels. The literature review to date has not revealed any studies that address the behavior of these slab systems at locations adjacent to abutment or pier diaphragms on non-skewed or skewed bridges. At these deck locations the ends of a precast panel are supported by the bridge girders and a longitudinal edge of the panel is supported by a diaphragm.

2.2. Questionnaires

Survey questionnaires were developed and distributed to both design agencies and precast prestressed concrete producers who are members of the Prestressed Concrete Institute. The survey that was sent to the 50 state departments of transportation, District of Columbia, tollway authorities, two United States provinces, and eight Canadian provinces contained nine parts and asked 83 questions. This questionnaire addressed topics related to general bridge geometry and conditions, general panel geometry and conditions, panel bearing details, prestressing strand description and conditions, design criteria, economy, experiences with panel usage, and panel details and specifications. The survey that was sent to 192 manufacturers in the United States and Canada contained ten parts and asked 80 questions. This questionnaire addressed topics related to the producers background, general bridge panel conditions and geometry, bridge panel bearing details, prestressing strand conditions and description for bridge panels, design criteria, economy, inspection, experience with panel usage, and panel details and specifications. Even though both surveys were extensive, the vast majority of the questions were multiple choice type. Seventy-one out of 121 questionnaires that were sent to the design agencies were returned and of those, 29 or about 41% stated that they allow or have allowed the use of precast panels in bridge deck construction. Seventy-three out of 192 questionnaires that were sent to the precast manufacturers were returned and of those, 27 or about 40% stated that they have produced precast panels for bridges.

2.2.1. Design Agency Questionnaire

Some of the results from the questionnaire returned by the design agencies are given in Table 2.1. The total of the responses to a given question may not equal 29, since multiple responses may have been given or the question may have been skipped by some respondents.

Table 2.1. Selected Survey Results from Design Agencies

The number in the parenthesis ( ) represents the number of design agencies having that particular answer.

I-3. Is your state or agency currently using or specifying panels for bridge deck construction?

(16) Yes (13) No

I-9. What type of panel support is provided for typical panels spanning perpendicular to the bridge span?

(1) Panels are not used to span in this direction
(16) Precast prestressed concrete girders only
(3) Steel girders only
(9) Either precast concrete or steel girders
(2) Other

II-3. Minimum panel thickness used:

(1) Not specified	(8) 3 in.	(3) 4 in.
(4) 2½ in.	(11) 3½ in.	(1) Other

II-5. Panel construction at skewed abutment or pier locations:

(8) Panels not used at these locations
(4) Panels sawn to match the skew only
(2) Panels cast to match the skew only
(12) Panels sawn or cast to match the skew
(3) Other

III-7. Permanent bearing material used to support panels:

(1) Not specified
(7) Continuous fiberboard, neoprene, polystyrene, or similar material only
(20) Continuous mortar, grout, or concrete bed only
(0) Steel shims at panel corners only
(1) Any of the above
(3) Other

IV-1. Total diameter of the strand that is used most often:

(0) 1/4 in.	(23) 3/8 in.	(3) 1/2 in.
(0) 5/16 in.	(2) 7/16 in.	(0) Other

IV-6. Are strand extensions used?

(18) Always (2) Sometimes (8) Never

V-17. Is the bridge deck designed as a continuous span across the girders when panels are used?

(24) Always (3) Sometimes (1) Never

V-21. Is two-way plate action considered in the design of the deck when the panels are supported along three edges?

(10) Three edge panel support not permitted (1) Yes (16) No

V-22. Is fatigue considered in the design of the deck when panels are used?

(1) Yes (26) No

VI-1. Have cost effectiveness studies even been performed to evaluate the economical advantages of using panels instead of full depth bridge deck?

(5) Yes (23) No

VI-2. What are the approximate cost savings realized (including costs associated with construction time), when panels are used for subdecks on a typical bridge compared to a conventional full depth bridge deck?

(18) Cost savings not known
(6) No cost savings
(3) $0-$1.00/ft^2 of deck area
(0) $1.00-$2.00/ft^2 of deck area
(0) $2.00-$3.00/ft^2 of deck area
(0) $3.00-$4.00/ft^2 of deck area
(0) Over $4.00/ft^2 of deck area

VII-5. How does your state or agency classify any problems associated with panel usage for bridge deck construction?

(12) Can not really comment since we have not used panels often enough
(1) Non-existent (7) Minor (6) Moderate (6) Significant
(0) Major

VII-6. Considering all aspects of manufacturing, transportation, erection, and performance of panels for bridge deck construction, how does your state or agency rate panel usage?

(11) Can not really comment since we have not used panels often enough
(1) Excellent (3) Very Good (7) Good (5) Fair (5) Poor

Sixteen out of 29 design agencies, who at some time permitted the use of prestressed concrete panels, are currently allowing panel usage. Considering the bridge girder material, 16, 3, and 9 agencies have specified that the panels are to be supported by precast concrete girders only, steel girders only, and either concrete or steel girders, respectively. As shown in Table 2.1, the panel thickness varies for the design agencies from a minimum of 2½ in. thick to a maximum of 4 in. thick. Eight agencies have stated that they prohibit panels at skewed abutment or pier diaphragm locations. When non-rectangular panels are permitted, 4, 2, and 12 agencies specify that the panels may be sawn to match the skew only, cast to match the skew only, and either sawn or cast to match the skew, respectively.

A significant majority of the design agencies specify a continuous mortar, grout, or concrete bed for the permanent bearing material of the panels, while some agencies have used continuous fiberboard, neoprene, polystyrene, or similar material. The most common strand diameter is 3/8 in. and most agencies required strand extensions. Regarding the behavior of the composite slab system, most agencies assume that the bridge deck has continuous spans across the bridge girders and that fatigue stresses and strains are not considered. Two-way plate bending is considered only by one design agency and neglected by 16 agencies when panels are supported along three edges. Ten agencies do not permit panels to be supported along three edges.

The survey revealed that five design agencies have performed some form of an economical analysis to evaluate any economical advantages of using panels instead of a conventional full depth reinforced concrete slab. When asked to place an approximate dollar value on any savings, all of the design agencies responded that the cost savings were unknown or less than $\$1.00/ft^2$ of the bridge deck area.

Each design agency was asked to classify any problems associated with precast panel usage for bridge deck construction. No agency thought that major problems existed with the panels; however, 12 agencies categorized problems as either moderate or significant, while 8 agencies classify problems as either non-existent or minor. Another 12 design agencies responded that they could not really comment since they had not specified panels often enough. Question VII-6 on the survey asked the respondent to consider all aspects of manufacturing, transportation, erection, and performance of panels for bridge deck construction and rate panel usage. Ten agencies gave panel usage only a fair or poor rating and 11 agencies gave the panels an excellent, very good, or good rating. Another 11 agencies stated that they could not really comment since they had not used panels often enough.

Standard details and specifications for precast panel subdecks have been provided by many of the design agencies that have specified panels for bridge deck construction.

2.2.2. Precaster Questionnaire

Some of the results from the questionnaires returned by the manufacturers of precast prestressed concrete panels are given in Table 2.2. The total of the responses to a given question may not equal 27, since multiple responses may have been given or a question may have been skipped by some respondents.

Table 2.2. Selected Survey Results from Panel Producers

The number in the parenthesis ( ) represents the number of precastors having that particular answer.

I-3. Has your company produced or submitted a bid to produce panels for a bridge project with the last two years?

(20) Yes (7) No

II-10. Top slab roughness and projections (not including lifting hooks):

(0) Smooth finish without bar projections
(0) Smooth finish with U-shaped bars or dowels
(3) Broom finish without bar projections
(1) Broom finish with U-shaped bars or dowels
(14) Raked finish without bar projections
(17) Raked finish with U-shaped bars or dowels
(2) Other

II-11. What is the direction of the raked depression with respect to the panel span?

(1) Raked depression not used
(6) Parallel to panel span only
(17) Transverse to panel span only
(1) Both parallel and transverse to the panel span
(2) Diagonal to panel span
(0) Other

II-23. Is additional steel provided in the panel ends to prevent splitting due to bond transfer:

(8) Always (8) Sometimes (11) Never

III-1. Temporary bearing material used to support panels:

(2) Temporary bearing material not used
(3) Unknown
(18) Fiberboard, neoprene, polystyrene, or similar material only
(2) Mortar, grout or concrete bed only

(2) Steel shims only
(2) Other

III-6. What is the minimum length of permanent bearing parallel to the panel span?

(3) Unknown
(6) 1 in.
(7) 1½ in.
(3) 2 in.
(3) 2½ in.
(4) Other

IV-12. What method is used to release the bridge panel prestressing strands?

(20) Acetylene torches
(6) Abrasive saw blades
(3) Wire (bolt) cutters
(2) Slow release of hydraulic pressure
(0) Other

VII-2. Does the state or agency for which your company is casting panels have a representative at your plant to observe strand detensioning, form stripping, and panel handling and storage?

(1) Not their responsibility (19) Always (6) Sometimes (0) Never

VII-3. Does your company send a representative to the bridge jobsite to inspect the panels after erection for cracks and proper bearing?

(5) Not our responsibility (5) Always (12) Sometimes (4) Never

VIII-1. Which of the following items of panel damage has your company directly experienced more than just a few times or occasionally?

(4) Can not really comment since we have not cast panels often enough
(6) Have not experienced any problems
(8) Broken corners
(9) Spalled or chipped edges
(9) Cracking parallel to strands along a significant portion of the panel length
(10) Cracking parallel to strands near the ends of the panel only
(2) Cracking transverse to the strands near panel midspan
(3) Diagonal cracks across panel surface
(1) Strand slippage
(4) Skew panels are difficult to detension properly
(0) Other

VIII-6. Which of the following casting techniques has your company established to minimize problems in panel fabrication?

(4) Can not really comment since we have not cast panels often enough
(4) Provide strand tie downs along prestress bed length
(10) Clean out header strand slots after each casting
(19) Allow for concrete preset prior to heat application for accelerated curing
(11) Institute special strand cutting sequence
(14) Provide steel headers
(2) Allow strands to oxidize by exposure to the weather for a few days
(2) Increase concrete release strength above minimum specified

(4) Increase concrete ultimate strength above minimum specified
(13) Provide a reinforcing bar transverse to strand at panel ends
(0) Apply compressed air when stripping panels
(2) Cast panels inside a structure to avoid exposure to weather
(1) Other

VIII-8. Considering all aspects of manufacturing, transportation, erection, and performance of panels for bridge deck construction, how does your company rate panel usage?

(1) Can not really comment since we have not cast panels often enough
(7) Excellent (7) Very Good (5) Good (3) Fair (2) Poor

Twenty of the 27 precastors who have manufactured precast panels have provided panels or submitted a bid to provide panels for bridge projects within the last two years, which indicates that designers and bridge contractors believe that precast panels provide a viable option for bridge deck construction. Questions II-10 and II-11 in Table 2.2 show that the treatment of the top surface of the precast panels to obtain composite behavior between the panels and the cast-in-place reinforced concrete slab varies amongst the panel producers. A raked finish is most common with the direction of the raking usually transverse to the panel span. U-shaped bars or dowels across the interface between the two slabs appear to be used about 50% of the time. To prevent splitting of the panels during strand release, some precastors place additional steel in the ends of the panels. As shown by Question IV-12, the majority of the panel producers use acetylene torches to release prestressing strands. Acetylene torches applied at a single point on a strand, abrasive saw blades, and wire cutters are all associated with quick strand release techniques. Two producers indicated that they release strands slowly using hydraulic pressure.

Table 2.2 lists two inspection questions (Questions VII-2 and VII-3) that were on the survey which was sent to the manufacturers. The responses indicate that additional inspection by both design agencies and panel producers may be warranted.

Experiences with panel usage were addressed in Part VIII of the questionnaire. Three of the eight questions asked and the producers responses are given in Table 2.2. Question VIII-1 was confined to panel damage only. The four types of damage experienced by the most panel producers are broken corners, spalled or chipped edges, cracking parallel to strands along a significant portion of the panel length, and cracking parallel to the strands near the ends of the panel only. To help eliminate problems with panel manufacturing, a variety of production techniques have been employed by panel producers as shown by Questions VIII-6. Those items receiving the greatest number of responses were clean out header strand slots after each casting, allow for concrete preset prior to heat application for accelerated curing, institute special strand cutting sequence, provide steel headers, and provide a reinforcing bar transverse to strand at panel ends. The manufacturers were also asked to rate panel usage considering all aspects of manufacturing, transportation, erection, and performance. Five producers rated precast panel usage as fair or poor, while 19 manufacturers rated panel usage as either excellent, very good or good.

Some of the manufacturers of precast concrete panels have provided drawings and specifications for deck panels.

3. Task 2 - Field Inspections

3.1. Bridge Descriptions

On October 19, 1989 field inspections of three Iowa bridges located in Hardin County near Eldora, Iowa were performed. All three prestressed concrete girder bridges are on the farm to market system and involve water crossings. The first bridge inspected was Bridge No. 9066 that is located 900 ft. south of the east 1/4 corner of Section 8-87-19 in Eldora Township of Hardin County over the Iowa River. This bridge has a 30 ft. roadway width, three spans (72 ft.-5 in. 81 ft.-6 in., and 72 ft.-5 in.), and no skew. The horizontal alignment is straight and the vertical alignment is at a 0.5% grade. The second bridge inspected was Bridge No. 8401 that is located 140 ft. north of the southwest corner of Section 36-88-19 in Clay Township of Hardin County over Pine Creek. This bridge has a 28 ft. roadway width, a single 80 ft. span, and no skew. The horizontal alignment is straight and the vertical alignment is at a 0.375% grade. The last bridge inspected was Bridge No. 7022 that is located 1320 ft. south and 1320 ft. east center of Section 12-88-20 in Jackson Township of Hardin County over the Iowa River. This bridge has a 30 ft. roadway width, three spans (68 ft.-3 in., 77 ft.-6 in., and 68 ft.-3 in.), and a 30 deg. skew angle. The horizontal alignment is straight and the vertical alignment is on a curve having grades of $\pm$ 1.000%.

The precast prestressed concrete panels for these bridges were cast by Precast Concrete Operations, a Division of Wheeler Consolidated, Inc., Iowa Falls, Iowa. The panels which span between the prestressed girders and extend along the entire length of each bridge were cast during the months of June 1983, March 1983, and June 1982 for Bridge Nos. 9066, 8401, and 7022, respectively. All three bridges have the same type of details for the precast panels. The 2½ in. thick by 8 ft. wide panels were set on 3/4 in. thick by 1 in. wide fiberboard strips to permit the concrete from the topping slab to flow under the panel ends for permanent bearing. The condition and extent of the concrete bearing could not be confirmed since the detail is hidden from view. At the abutment and pier diaphragms, the precast panels are supported along three edges. Steel channel intermediate diaphragms are provided at approximately the girder midspan locations. These diaphragms are attached to the precast girder webs and do not support the precast panels.

3.2. Inspection Results

The condition of the precast prestressed concrete panels in each of the three bridges is essentially the same. The slope of the grade beneath each bridge, the height of the bridge, and the presence of the waterways prevented inspection of the underside of the panels within the center span and many panels within the ends spans of the three span bridges (Bridge Nos. 9066 and 7022) and the panels within about the center third of the single span bridge (Bridge No. 8401). Many of the inspected panels for all three bridges have single and sometimes multiple hairline cracks running parallel to the panel span. These cracks which are located within the center half of the affected panels usually extend along the entire panel length and occur below a prestress strand. Also, for all three bridges, most of the observed panels had a slight discoloration (darker gray color) beneath the strands. For Bridge No. 9066, rust discoloration on the underside of the panels within the bridge end spans was not observed. For Bridge No. 8401, one panel located above the steel channel intermediate diaphragm along the west side of the bridge has rust strains about 3 in. long near the midspan of the panel. In addition, a diagonal crack at the southwest corner of the second panel from the south abutment along the west side of this bridge was observed. For Bridge No. 7022, several panels have rust discoloration about 6 in. to 12 in. long beneath strand locations. Two panels were observed to have significant rust staining. One panel, located along the north side of the bridge in the west end span, is the fourth panel from the west bridge abutment. The other panel, located along the south side of the bridge in the east end span, is the fourth panel from the east bridge abutment.

The top surface of the cast-in-place reinforced concrete slab for all bridges had been raked parallel to the panel span. The concrete deck on Bridge Nos. 9066, 8401, and 7022 was completely exposed, covered entirely by a sand and gravel layer, and partly covered by sand and gravel, respectively. The grooves from the raking and the presence of the sand and gravel fill prevent the observation of any reflective cracking in the topping slab.

3.3. Inspection Recommendations

Since panel cracking, discoloration, and rust strains concern the researchers, we recommend close inspection and documentation of the condition of the underside of the precast prestressed concrete panels that could not be observed in this study and reinspection at a future date of all of the panels for these bridges for comparison purposes and evaluation. In addition, since these three bridges are experiencing what appears to be initial rusting of prestressing strands, the researchers recommend that an inspection program be initiated for any other bridges containing precast panels.

4. Task 3 - Experimental Program

4.1. Specimens

As discussed in the research proposal, the experimental program involves testing five full scale specimens. Each composite specimen contains two 2½ in. thick precast, prestressed panels having a 5½ in. thick reinforced concrete topping slab and represents a different configuration of panel support and geometry. One specimen models a typical interior condition where the specimen is simply supported at the ends of the panel. Four specimens will be constructed to model the composite deck at locations adjacent to abutment or pier diaphragms, where one of the precast panels within these specimens will be supported along one longitudinal edge in addition to the end supports. Each of these four specimens will have a different skew angle (0,15,30, and 40 deg.).

The precast concrete panels are described in Sec. 4.3. The 5½ in. thick cast-in-place reinforced concrete slab is reinforced with a single layer of reinforcement which has the same sizes, spacings and location as the top layer of reinforcement specified for a conventional 8 in. thick bridge deck. No. 5 bars, which are positioned transverse to the span of the panels are spaced 9 in. on center and are supported on 1½ in. high individual bar chairs spaced at 3 ft. on center in both directions. The bar chairs rest directly on the top surface of the precast panels. No. 6 bars at 10 in. on center are positioned parallel to the span of the panels and are directly above the No. 5 bars. All reinforcing bars are A615 Grade 60 bars. Epoxy coated bars were not used. The concrete cover above the No. 6 bars is about 2½ in. The concrete for the topping slab is the Iowa Department of Transportation Mix No. D57 with the course aggregate satisfying Gradation No. 5 (1 in. maximum size) and the fine aggregate satisfying Gradation No. 1. The approximate quantities of dry materials per cubic yard of concrete are 710, 1413, 1413, and 291 lb of cement, fine aggregate, course aggregate, and water, respectively. Air entrainment is 6% and the slump should be between 1 to 3 in. with an absolute maximum of 4 in.

4.2. Tests

Initially, the test specimens will be subjected to a series of static loads positioned at various locations on the slab surface. The maximum magnitude for these loads are equal to an HS-20 wheel load including a 30% impact factor. The 20.8 kip load will be applied through an AASHTO wheel load footprint, having a rectangular area equal to 160 in^2 (8 in. by 20 in.). After completion of the static load tests, ultimate load tests will be performed to establish the flexural and shear strengths for the specimens. The ultimate flexural strength will be established first when the wheel load footprint is positioned over the joint between the two precast panels at the midspan of the panels. After the ultimate flexural strength test has been completed, the wheel load footprint will be repositioned to the mid-width of one of the precast panels adjacent to one end of the panel. For this load position, an ultimate shear strength test will be attempted on the same test specimen.

Instrumentation for all static and ultimate load tests consists of electrical resistance strain gauges to measure concrete strains on the top and bottom of the specimens at various points along the midspan of specimens, load cells to monitor the applied wheel loads, dial gauges to measure vertical slab deflection at various points on the slab surface, and displacement transducers to monitor three types of displacements. These displacements include vertical slab deflection at the load position, potential strand slip on selected prestressed strands, and possible end-slip between the reinforced concrete topping slab and a precast panel at some preselected locations.

4.3. Panels

Ten precast prestressed concrete panels have been provided and delivered to the Structural Engineering Laboratory at Iowa State University by Precast Concrete Operations, a Division of Wheeler Consolidated, Inc., Iowa Falls, Iowa. All panels are 2½ in. thick and 7 ft.-1 in. long. The rectangular shaped panels are 8 ft. wide and the four trapezoidal shaped vary in width along their length. Each of these panels is 8 ft. wide at their maximum width. The minimum widths are 6 ft.-1¼ in., 3 ft.-10 7/8 in., and 2 ft.-0 5/8 in. for the panels having skew angles of 15, 30, and 40 deg., respectively. The top surface of the panels has a raked finish that runs perpendicular to the span of the panel. All of the panels contain sixteen 3/8 in. dia., 7 wire, 270 ksi, low relaxation prestressing strands positioned at the mid-depth of the panel. The strands, which are spaced at 6 in. on center, extend beyond the ends of the panels by 5 in. and by 6 in. along any diagonal edge. Each panel has a single layer of 6x6-W5.5xW5.5 welded wire fabric located directly on top of the prestressing strands. The trapezoidal shaped panels have two No. 5 reinforcing bars placed along the diagonal edge of the panel. Some of the short strands in the trapezoidal shaped panels were sleeved to prevent bonding with the concrete. This debonding was done to prevent the triangular shaped corner of a panel from breaking during the detensioning of the prestressing strands.

After the strands were prestressed to about 17.2 kips, approximately 75% of the strand tensile strength, and before the concrete was cast, four panels were instrumented with PML-30 polyester model strain gauges. These gauges were wired between adjacent strands to position the gauges at the mid-depth of the panel and midway between two strands. These gauges were used to measure concrete strains during and after detensioning of the prestressing strands. After one day of heat curing, the concrete release strength was 4,810 psi. The 28 day concrete compressive strength was 7,532 psi.

4.4. Test Frame

To resist the test loads applied to the full scale specimens, a three dimensional structural steel frame has been designed, fabricated, and erected on the tie-down floor in the Structural Engineering Laboratory. Based on allowable stresses, the frame will resist 100 kips placed anywhere within a 6 ft.-6 in. by 22 ft. area. Many months of fabrication were needed to construct the frame. The steel members for the frame were cut from salvaged bridge beams provided by the Iowa Department of Transportation. The four columns, two girders, and three diaphragms are W30x108 shapes; the four tie-down girders and loading girder are W21x62 shapes; and the 16 tie-down beams and the four diagonal braces are S15x42.9 shapes. Eight 1-3/8 in. dia. Dywidag bars prestressed to 60 kips each fasten the frame to the floor.

4.5. Instrumentation Frame

To support the dial gauges and displacement transducers used to measure the vertical deflection of the top surface of the full scale test specimens, a three dimensional aluminum frame has been designed, fabricated, and erected. The frame will permit displacement to be measured anywhere on the deck surface. Construction of the instrumentation frame took several months to complete. The aluminum members for this frame were fabricated from salvaged guardrail and posts provided by the Iowa Department of Transportation.

4.6. Abutment Supports

The five different configurations of the full-scale composite slab test specimens required eight concrete abutments to be designed and constructed to accommodate the geometrical plan shapes for the specimens. Each abutment is 16 in. wide and 30 in. high. Four different lengths were required. An important design consideration was to prevent the joints between the abutments from matching the joint between the precast panels in the specimens. The abutments are tied together using steel plates and through bolts. Two steel diaphragms fabricated from S15x42.9 shapes tie together and stabilize the line of abutments which represent prestressed concrete girders in an actual bridge. The abutments are arranged in a horse shoe pattern to simulate bridge girders and an abutment or pier diaphragm.

5. Analytical Studies

5.1. Panel Model

The finite element model shown in Figure 5.1 contains 201 nodes and 180 elements. The model was developed to analytically establish the concrete strains induced in the trapezoidal shaped panel as a result of detensioning the prestressing strands. Plate elements were used to represent the concrete within the panel. The prestressing strands, welded wire fabric, and the two additional No. 5 reinforcing bars along the panel diagonal edge were not included in the analytical model. Loads representing the maximum strand forces, after elastic shortening but before concrete creep and shrinkage and strand relaxation occurred, were applied at the last nodes at the ends of the panel along the line where the strands exist in the panel. These forces were obtained from the concrete strains measured by the PML-30 polyester mold strain gauges which were embedded within the panels. A comparison of the analytical and experimental concrete strains are discussed in Sec. 6.1.

5.2. Bridge Deck Model

Finite element models of the full-scale composite bridge deck are being developed for the five test specimens. These analytical models are being assembled from a single layer of plate elements; therefore, the composite slab is being approximate as a homogeneous material having isotropic properties. Fig. 5.2 shows the finite element model containing 242 nodes and 200 elements for the test specimen representing an interior configuration for a bridge deck. The mesh size selected was established from a mesh size sensitivity study, involving comparisons of solutions by classical plate bending theory and finite element methods, of plate bending problems having known closed-form solutions. To maintain the desired degree of accuracy, the mesh size is smaller in the vicinity of the concentrated wheel load location. A comparison of the analytical

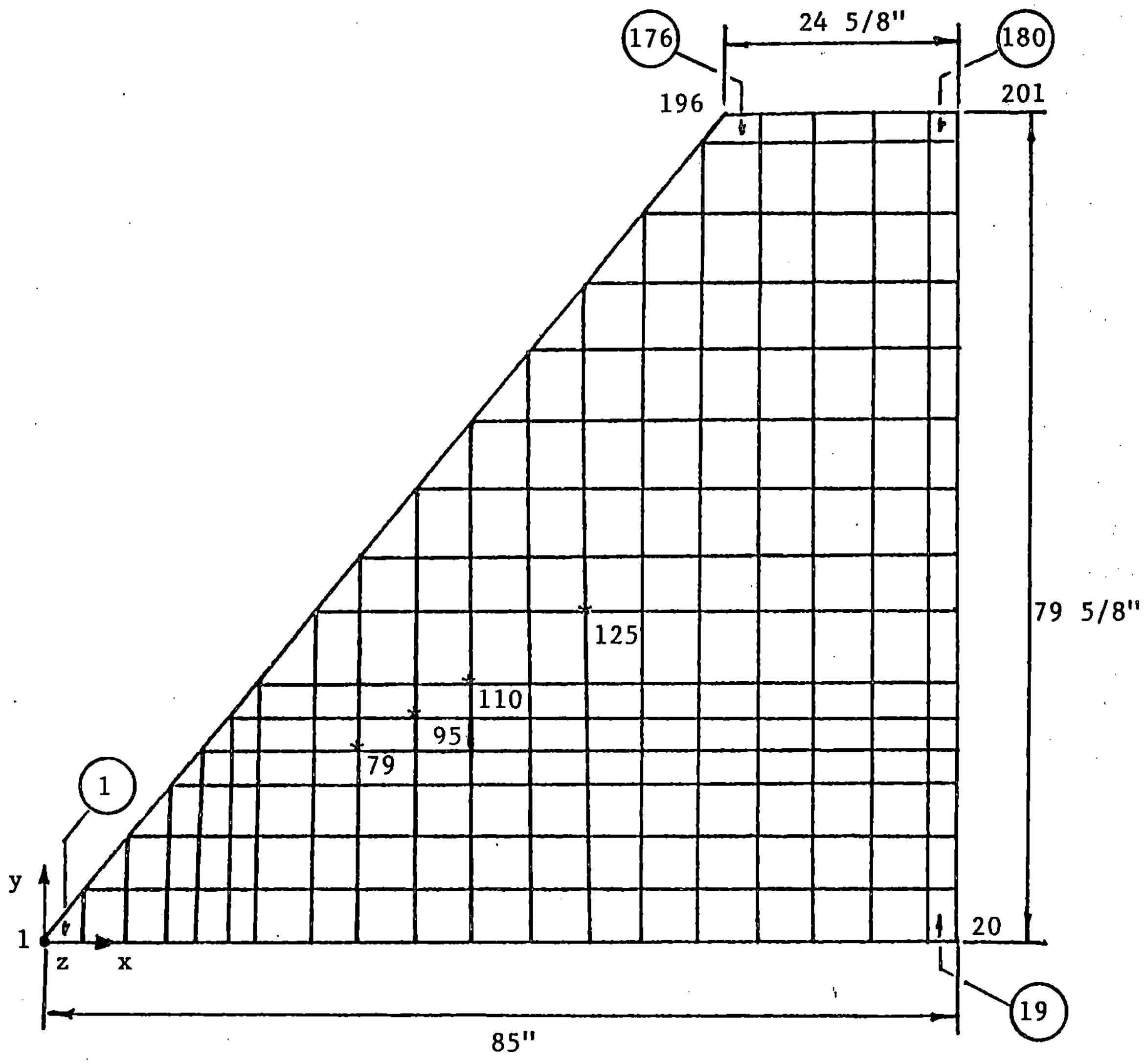

Figure 5.1. Finite Element Model for 40° Skewed Panel

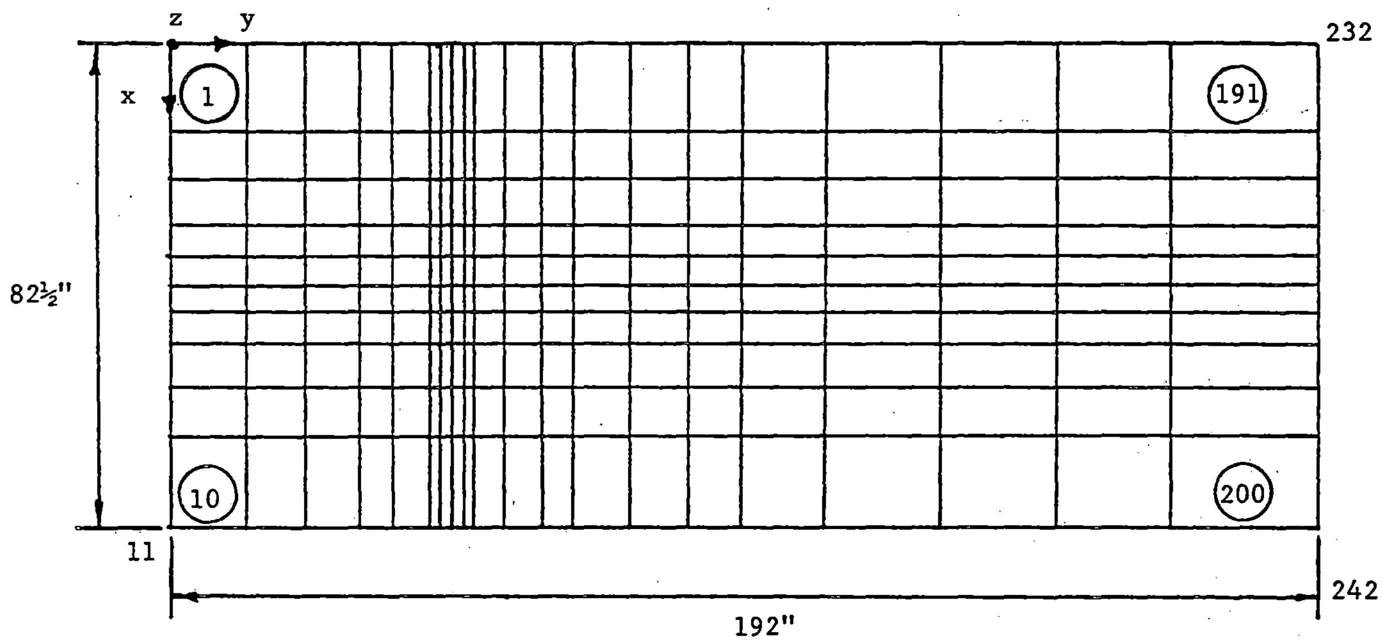

Figure 5.2. Finite Element Model for Bridge Deck at Interior Panel Configuration (Load at x = 41.25 in. and y = 48.00 in.)

and experimental results for one of the wheel load positions (x = 41.25 in. and y = 48.00 in.) on bridge deck are discussed in Sec. 6.2.

6. Experimental and Analytical Results

6.1. Strand Development Length

The strand development length is the total concrete embedment length required for the ultimate stress, f_{su}^*, in the strand to be obtained at the ultimate flexural strength of the percast member. This length consists of a strand transfer length and a strand flexural bond length. The strand transfer length, L_t, is the strand embedment length required to develop the effective strand prestress, f_{se}, which for pretensioned bonded tendons is the stress remaining in a strand after it has been released from the prestressing bed anchorages and includes all losses associated with elastic shortening, concrete shrinkage and creep, and strand relaxation. The effective prestress, f_{se}, for low relaxation prestressing strands is given by

$$f_{se} = 0.75\, f'_s - \Delta f_s \tag{6.1}$$

where, f'_s = ultimate strength at prestressing strand and Δf_s = total stress loss from all causes, excluding friction. The strand flexural bond length, L_b, is an additional embedment length, beyond L_t, required to increase the strand stress from f_{se} to f_{su}^*, occurring during flexural loading of the prestressed member. The total of the transfer and flexural bond lengths, referred to as the strand development length, L_d, is expressed as

$$L_d = L_t + L_b \tag{6.2}$$

Many researchers have proposed expressions to evaluate the development length, L_d. The AASHTO Specification Eq. (9-32) and the ACI Code equation in Sec. 12.9.1 are the same equations which are based on Hanson and Kaars' research. Both specification expressions for the development length can be written as

$$L_d = (f_{su}^* - \frac{2}{3} f_{se})D \tag{6.3}$$

where, D = nominal strand diameter in inches. The stresses f^*_{su} and f_{se} have units of ksi. A denominator not shown in Eq. 6.3 is equal to unity with units of ksi and provides for consistent units. The AASHTO Specification does not separate Eq. 6.3 into the two parts representing the transfer and flexural bond lengths, while the ACI Code Commentary Sec. 12.9 does make the distinction. The ACI Code Commentary equation can be rewritten as

$$L_d = \frac{f_{se}}{3} D + (f^*_{su} - f_{se})D \tag{6.4}$$

The first term in Eq. (6.4) represents the strand transfer length and the second term represents the strand flexural bond length.

Considering the strand transfer length immediately after cutting the prestressing strands, the only losses which have occurred are elastic shortening of the concrete and tendon relaxation during placing and curing of the concrete. For this condition, the transfer length can be expressed as

$$L_t = \frac{f_{si}}{3} D \tag{6.5}$$

where the initial prestress, f_{si}, for low relaxation strands is given as

$$f_{si} = 0.75 \, f'_s - f_{es} \tag{6.6}$$

with f_{es} = stress loss due to elastic shortening and initial strand relaxation. The AASHTO Specification provides an expression to compute prestress loss due to these effects. Rewriting the AASHTO Eq. 9-6,

$$f_{es} = \frac{E_s}{E_{ci}} (f_{cir}) \tag{6.7}$$

where, E_s = modulus of elasticity of the prestressing strand and E_{ci} = modulus of elasticity of the concrete at strand detensioning. The concrete stress, f_{cir}, at the center of gravity of the prestressing strands induced by the prestressing forces immediately after strand release is given by

$$f_{cir} = \frac{0.69\ f'_s\ A_s^*}{A_c} \tag{6.8}$$

where A_s^* and A_c are a prestressing strand and tributary concrete cross-sectional areas, respectively. Applying Eqs. 6.6 - 6.8 to the precast panels used in this research project, f_{si} = 195.0 ksi.

Another expression for the initial prestress, which only considers elastic shortening, can be obtained by equating the changes in length for the concrete and steel along a unit length of panel beyond the transfer length region, and knowing that the internal concrete compressive force equals the internal strand force. Applying these two conditions, the initial prestress can be expressed as

$$f_{si} = \frac{0.75\ f'_s}{[1 + \left(\frac{E_s}{E_{ci}}\right)\left(\frac{A_s^*}{A_c}\right)]} \tag{6.9}$$

Applying Eq. 6.9 to the precast panels used in this research project, f_{si} equals 194.5 ksi, which agrees closely with the results obtained from Eq. 6.6. This stress is represented by the horizontal portion of the ACI Modified curve shown in Fig. 6.1. The corresponding strand transfer length is obtained by substituting this stress into Eq. 6.5, which gives L_t equal to 24.3 in. The total panel length at the strand location would have to be twice this long in order for the strand stress to equal the maximum initial prestress of 194.5 ksi. If a shorter panel length exists, a decrease in strand stress occurs as shown by the straight line drawn between the origin and the Point A in Fig. 6.1.

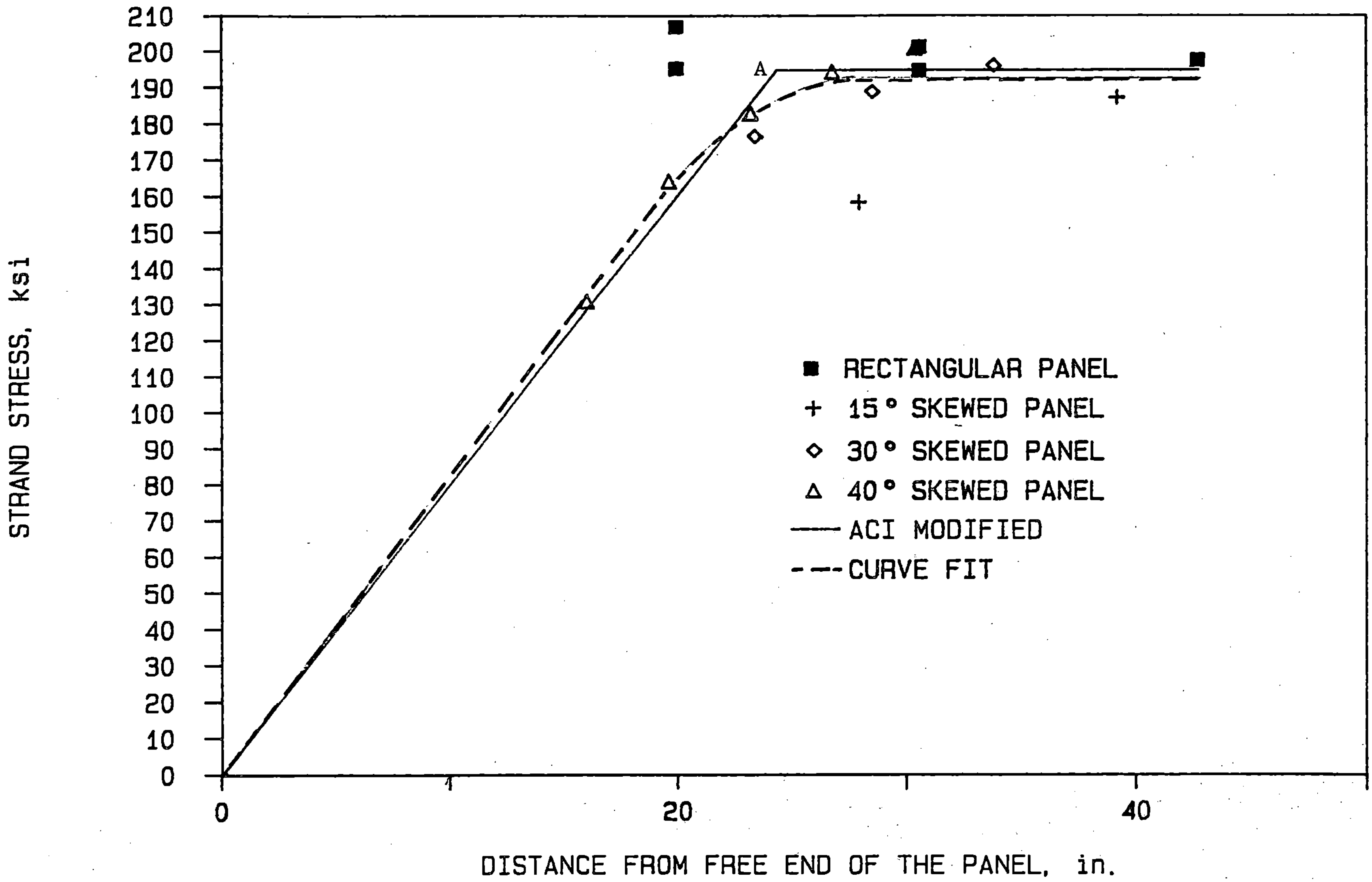

Figure 6.1. Transfer Length at Release

The data points shown in Fig. 6.1 were established from experimentally measured concrete strains at known locations in four precast panels. The strain readings, obtained from the PML-30 gauges which were embedded in the concrete, were taken after the strands were cut at the prestress plant. Since the gauges were between adjacent strands, curve linear interpolations and extrapolations were used to compute concrete strains at the prestressing strand locations. The strand stress was computed using Hooke's Law. The dotted line shown in Fig. 6.1 was visually curve fit through the computed data points. The correlation between the experimental and analytical transfer length functions is excellent.

Table 6.1 provides tabulated values for the transfer length, L_t, immediately after strand release, for 3/8 in. diameter, 7 wire, prestressing strands, that have been obtained by a number of researchers. The information presented in the table for Kear, Lafraugh, and Mass; Hanson and Kaar; and Base was obtained from an article by Zia and Mustafa.

Table 6.1. Transfer Length at Release of 3/8 in.-7 Wire Strands

f_{si} (ksi)	f'_{ci} (psi)	L_t (in.)	Researchers
191.7	1690	24.5	Kear, Lafraugh, and Mass
191.1	3400	28.5	
186.0	5000	25.5	
187.5	3250	32.0	
166.1	3450	26.0	
144.9	3400	23.0	
150.0	4000	29.4	Hanson and Kaar
165.0	3232[a]	8.0[a]	Base
165.0	4696[a]	8.0[a]	
194.5	4810	18.1[b]	Zia and Mustafa
194.5	4810	24.3[c]	ACI Modified
194.5	4810	30.0[d]	Abendroth and Pratanata

[a]Average value
[b]Eq. 6.5
[c]Eq. 6.10
[d]Approximation based on a visual curve fit.

Using a linear regression analysis of experimentally derived transfer lengths from various researchers, Zia and Mustafa have proposed that the strand transfer length should be given as

$$L_t = 1.5\left(\frac{f_{si}}{f'_{ci}}\right)D - 4.6 \qquad (6.10)$$

where f'_{ci} = the concrete strength at strand release. An approximate transfer length of about 30 in. established during this research is based on the visual curve fit through the data points shown in Fig. 6.1.

A confirmation of the accuracy of the experimentally measured strain values was obtained by applying the visual curve fit expression of the strand stress versus embedment length relationship to establish the prestress forces in the 40° skewed panel. Knowing the available transfer lengths for the bonded strands in the panel, the prestress force in each strand was computed. Applying these forces to the nodes representing the ends of the strands in the finite element model (Fig. 5.1) the concrete strains at Nodes 79, 95, 110, and 125 were computed. Table 6.2 shows that excellent correlation exists analytical and experimental strains at these points.

Table 6.2. Concrete Strains in 40° Skewed Panel

Node No.	Analytical Strain (μ in./in.)	Experimental Strain (μ in./in.)
79	-229	-228
95	-260	-254
110	-270	-270
125	-281	-286

Strand development length has recently become a subject of controversy and additional research on this topic is presently being pursued by others.

6.2. Bridge Deck Behavior for Interior Panel Configuration

The displacement and strain results shown for the specimen representing an interior panel condition are based on loads applied to an 8 in. by 20 in. wheel footprint placed on the top surface of the composite slab and located at the center of one of the two precast panels. With respect to the entire specimen dimensions, the load point coordinate occurs at the midspan and quarter length point (x = 41.25 in. and y = 48.00 in.). The 20 in. length for the wheel footprint was parallel to the panel span.

Fig. 6.2 shows a load versus deflection plot for both the finite element solution and the experimental test results. The vertical deflection corresponds to the load point. The deflections shown are net displacements which account for vertical movements at the ends of the specimen. Considering the potential tests scatter associated with deflections of reinforced concrete members and the small magnitudes of the deflection, the correlation between the analytical solution (solid line) and the experimentally measured displacements is very good.

Figs. 6.3 and 6.4 show specimen displacements along the panel span and specimen length, respectively, for an applied load equal to 20.8 kips (maximum AASHTO wheel load including 30% impact). The deflection variation along the specimen length (Fig. 6.4) indicates that a point of contraflexure occurs at the joint (y = 96.00 in.) between the two precast panels. The displacement wave pattern shown agrees with the anticipated results. Again, very good correlation occurs between the analytical and experimental results. The maximum experimental deflection was less than 0.012 in.

The distribution of the strains, along the midspan of the specimen, at the top of the reinforced concrete slab and at the bottom of the precast panel, for an applied load of 20.8 kips, are shown in Fig. 6.5. As expected, the maximum strain occurs at the load point. Comparing the top and bottom fiber strains and assuming that full composite behavior exists, the approximately equal magnitudes of the absolute value of the experimental strains indicates that the neutral axis occurs at the mid-thickness of the specimen. Therefore, the composite slab can be analytically modeled as a homogeneous material. The close

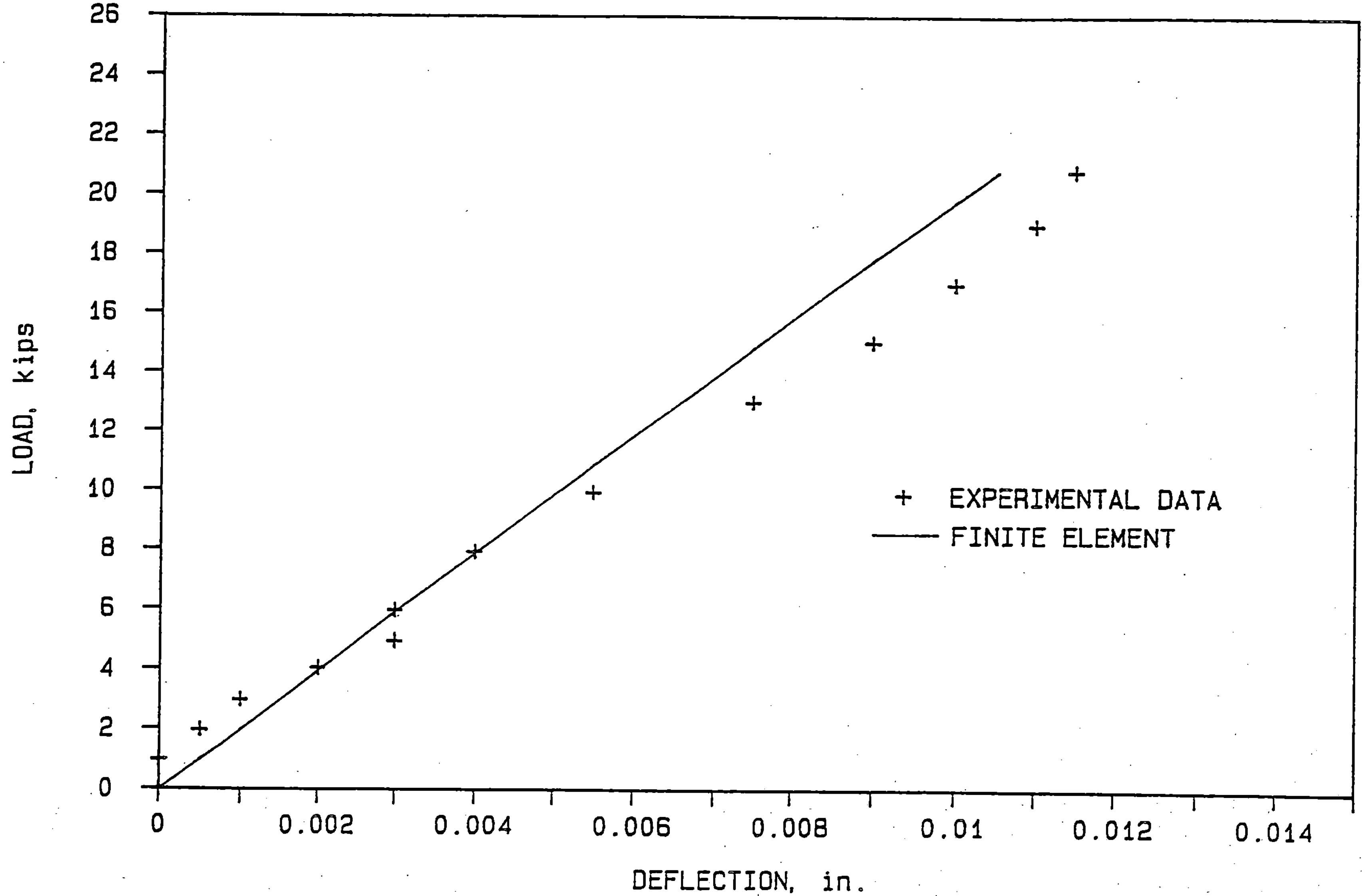

Figure 6.2. Load Versus Deflection for Interior Panel Configuration (Load at x = 41.25 in. and y = 48.00 in.)

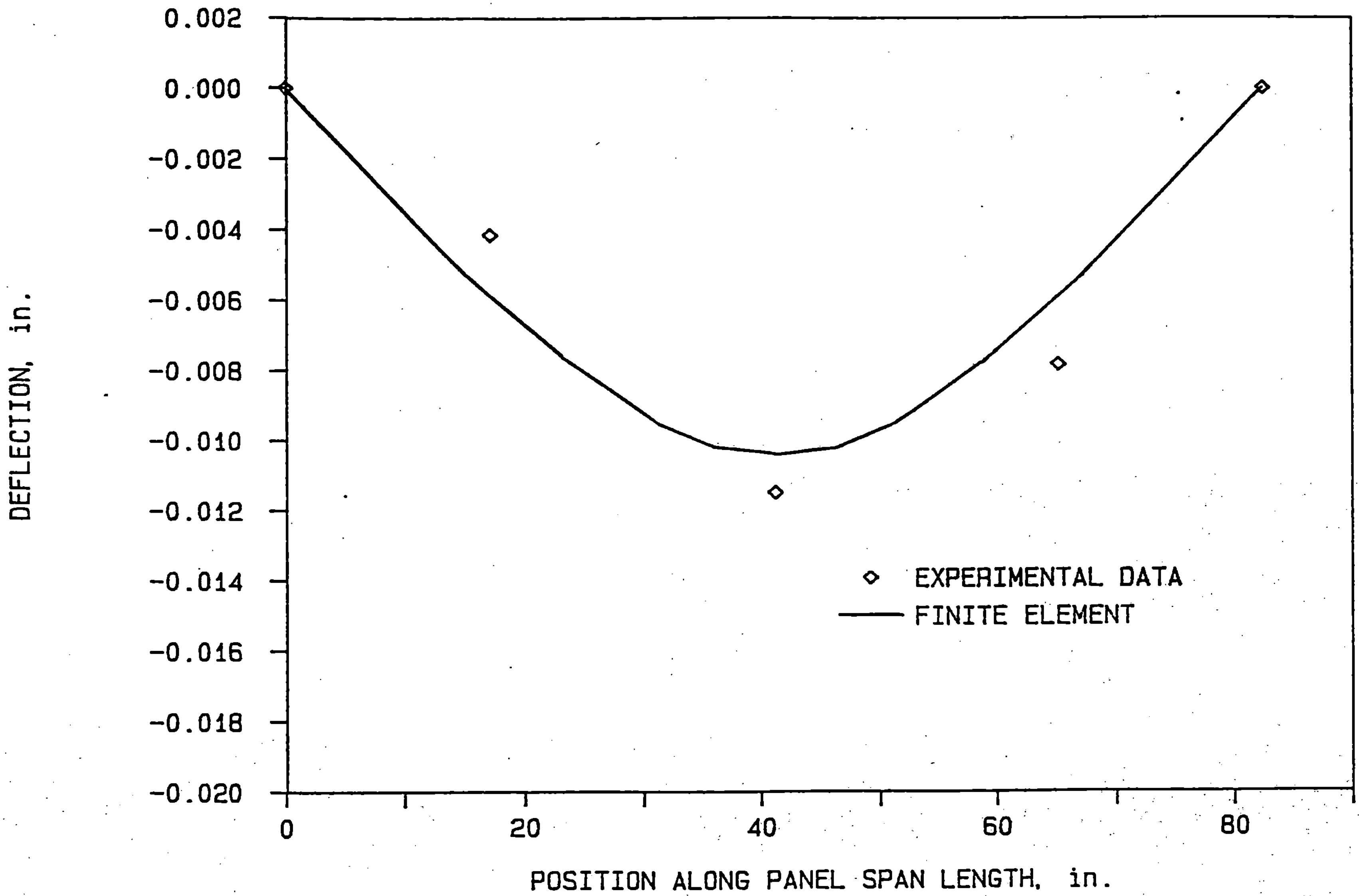

Figure 6.3. Deflections Along Panel Span at y = 48.00 in. for Interior Panel Configuration (Load at x = 41.25 in. and y = 48.00 in.)

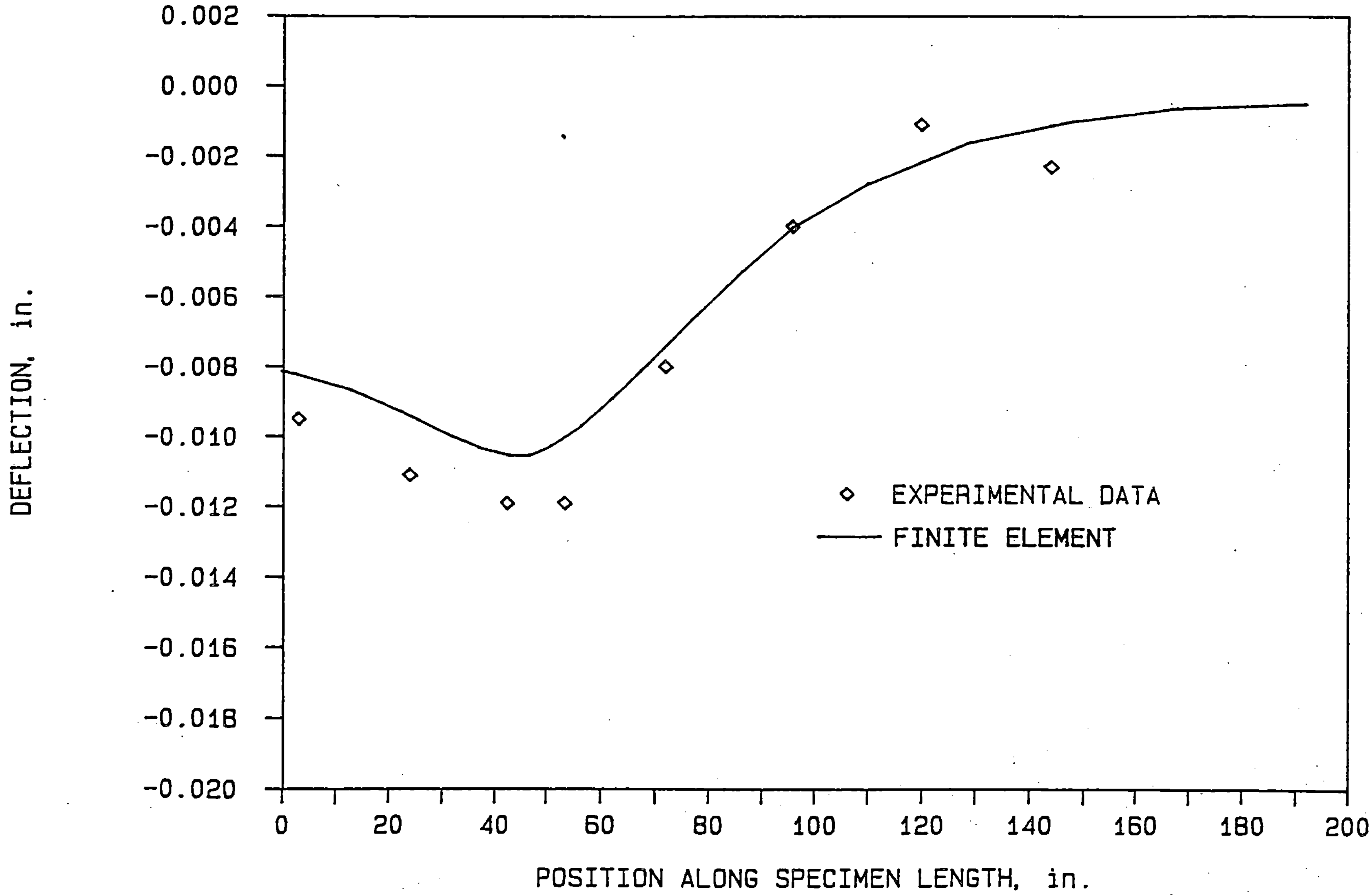

Figure 6.4. Deflections Along Specimen Length at x = 41.25 in. for Interior Panel Configuration (Load at x = 41.25 in. and y = 48.00 in.)

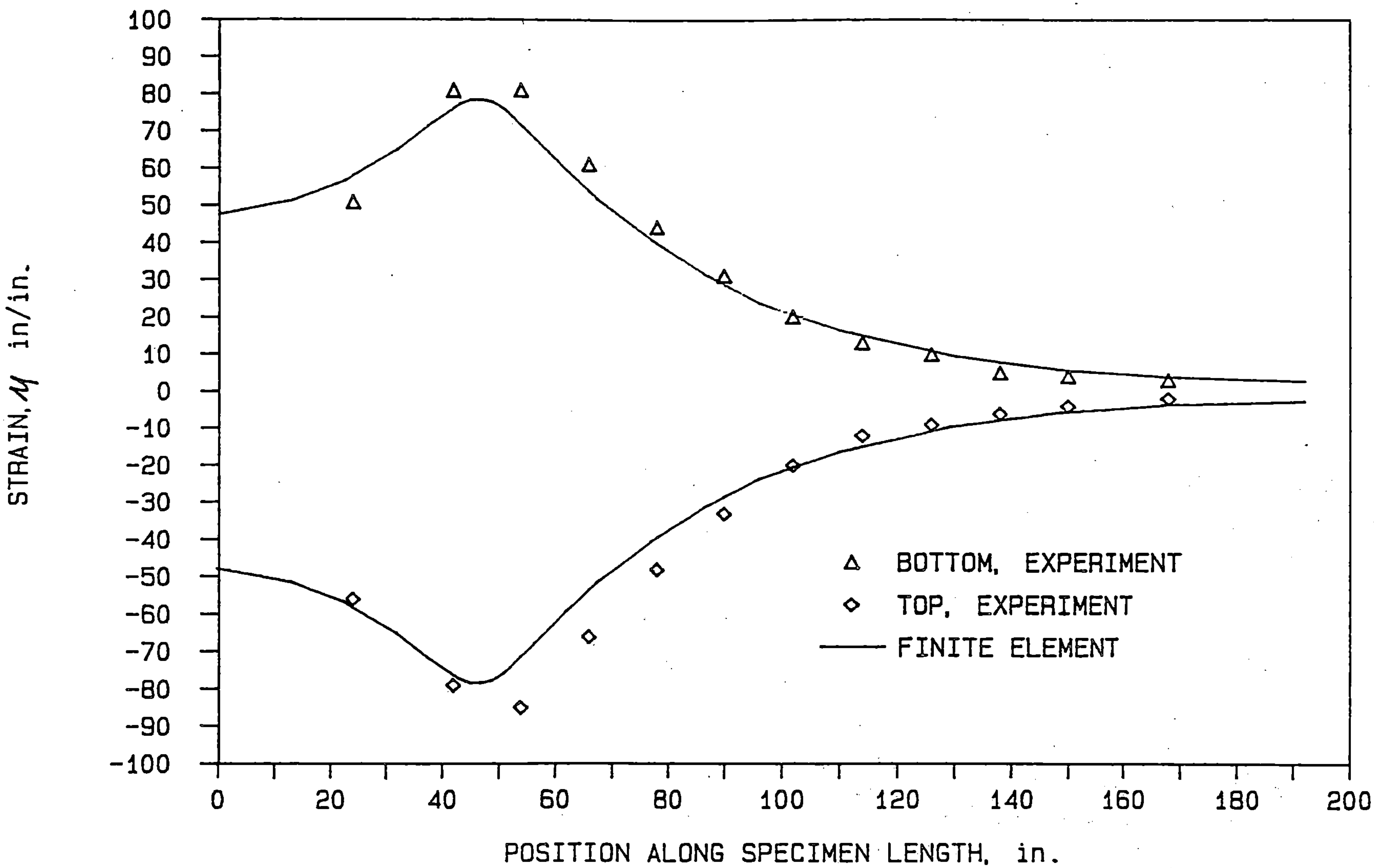

Figure 6.5. Concrete Strains Along Specimen Length at x = 41.25 in. for Interior Panel Configuration (Load at x = 41.25 in. and y = 48.00 in.)

correlation between the experimental and analytical strains confirms the accuracy of the finite element model for this magnitude of the applied load. Using a maximum experimental strain of about 85 microstrains, the corresponding stress due to the applied wheel load of 20.8 kips is about 386 psi compression at the top of the concrete slab and about 420 psi tension at the bottom of the precast panel. The stress difference between the top and bottom of the specimen is caused by differences in the modulus of elasticity for concrete in the reinforced concrete slab and the precast panel. Table 6.3 lists the stresses at the top of the reinforced concrete slab and at the top and bottom of the precast panel for all loadings. The minus sign indicates compression.

Table 6.3. Stresses in Bridge Deck for Interior Panel Configuration (load at x = 41.25 in. and y = 48.00 in.)

Location	Prestressing Force Including All Losses[a] (psi)	P/C Panel and R/C Slab Dead Load[b] (psi)	Wheel Load[c] (psi)	Total Stress (psi)
Top of R/C Slab	0	0	-386	-386
Top of P/C Panel	-978	-520	+158	-1340
Bottom of P/C Panel	-978	+520	+420	-38

[a]Based on prestressing to 0.75 f'_s and Δf_s = 29.9 ksi.
[b]Based on a concrete weight of 150 psf and a panel span of 6 ft, 7 in.
[c]Based on experimental strain readings and f'_c = 7,532 psi and 6,343 psi for P/C panel and R/C slab, respectively.

7. Future Work

7.1. Experimental

After the completion of the testing for the first configuration of the full scale composite slab, representing an interior panel condition, four more specimens will be prepared and tested. These specimens will have configurations representing panel conditions adjacent to abutment or pier diaphragms having skew angles of 0, 15, 30, and 40 degrees. The specimens will be constructed and tested as discussed in Sec. 4.

7.2. Analytical

Finite element models will be established to analyze the four remaining specimens in the experimental program. In addition, simplified analyses will be applied using basic engineering mechanics principles associated with a strength of materials approach. Comparisons of the displacements and strains between the experimental and analytical results will be made to establish the accuracy of the analytical models.

For the precast prestressed concrete panels in the slab configuration representing an interior panel condition, conventional prestressed concrete design principles will be applied to establish the concrete and steel tendon strains and stresses.

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