Full Depth Precast, Prestressed Concrete Bridge Deck System

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This study presents a new full depth precast, prestressed concrete bridge deck panel system. The newly developed system includes stemmed precast panels, transverse grouted joints, longitudinal post-tensioning and welded threaded and headless studs. The purpose of the system is to provide a high quality bridge deck that can be built rapidly for new construction or for rehabilitation. Rapid reconstruction time reduces the traffic disruption and annoyance that bridge users may encounter. The new system is shown to have better crack control and to be 10 percent more slender and 20 percent lighter in weight than solid conventional reinforced concrete decks.

The goal of this research project was to develop a precast, prestressed concrete bridge deck for rapid construction. Rapid replacement of bridge decks is becoming increasingly important in high traffic areas due to public intolerance to extended bridge closures. A review of the literature[1-18] and the results of a questionnaire to bridge owners, designers, and contractors have revealed that prefabricated panel systems are advantageous in improving the quality and speed of bridge deck replacement.

The large majority of highway bridges in the United States are of short to medium span with lengths up to 100 ft (30.5 m). Most of the deficiencies of existing bridges are attributable to substandard deck conditions. Deteriorated decks not only cause major traffic delays and public annoyance, but also result in a reduction of the bridge capacity.

Over the past several years, many strategies have been developed to combat the shortcomings of traditional methods of concrete deck replacement. A popular method is to incorporate prefabricated elements to achieve economy through reduced on-site construction time and labor. Each of these methods has advantages and disadvantages.

Although a considerable number of bridge deck replacements have been completed with full depth precast reinforced concrete panels, very few full depth precast, prestressed concrete panel systems have been used for deck replacement projects. The primary reasons for this are: (1) difficulty of accommodating crowned sloped sections due to pretensioning in the long-line bed; and (2) difficulty in transferring and developing strands in the first maximum negative moment zone located at the external girders.

A major advantage of the proposed system is the creation of cross-sloped precast, prestressed concrete panels with indented strands, which reduce the transfer and development lengths.

The impact of this research is promising. Bridge owners will greatly benefit from the more efficient decking system with rapid construction than with existing deck systems, i.e., cast-in-place, precast conventionally reinforced concrete, exodermic, steel grid, and other systems.

Table 1. Comparison of prefabricated deck systems.*

Type	Precast reinforced concrete	Exodermic	Proposed precast prestressed concrete
Thickness of deck, in. (mm)	9.0 (230)	9.6 (245)	8.1 (205)
Equivalent solid thickness, in. (mm)	9.0 (230)	5.9 (150)	5.7 (145)
Weight of deck, psf (kPa)	110 (5.3)	75 (3.6)	70 (3.4)
Dead load saved, psf (kPa)	—	35 (1.7)	40 (1.9)

* Based on HS-25 truck loading and 12 ft (3658 mm) girder spacing.

Table 2. Comparison of materials for prefabricated deck systems.*

Material per sq ft (m^2)	Precast reinforced concrete	Exodermic	Proposed precast prestressed concrete
Concrete, cu ft (m^3)	0.75 (0.23)	0.38 (0.12)	0.54 (0.17)
Reinforcing bars, lb (N)	10.3 (493)	4.7 (230)	1.7 (82)
Steel grid, lb (N)	—	20.0 (958)	—
Post-tensioning strand, lb (N)	1.0 (48.4)	—	0.5 (23.7)
½ in. (13 mm) diameter strands, lb (N)	—	—	1.3 (62.4)

* Based on HS-25 loading and 12 ft (3658 mm) girder spacing.

DECK COMPARISON

In the early stages of this research, a detailed study was performed of different precast bridge deck systems. A brief summary of the study is given below.

Precast Reinforced Concrete

Most applications of precast reinforced concrete decks have been used for replacement of deteriorated concrete decks on steel girders. The applications have been typically in situations where rapid construction was required.

Two layers of biaxial reinforcement have usually been provided for the panels. Grout-filled shear keys have also typically been provided for transverse joints, and the haunches and the transverse joints have been filled with epoxy or non-shrink grout. Use of fast setting grout is important for rapid construction because a precast concrete panel system requires step-by-step construction sequencing, such as: (1) place panels; (2) fill transverse joints with grout; (3) apply post-tensioning; (4) fill haunches and pockets for shear connectors with grout.

Longitudinal post-tensioning has been successfully applied for load transfer and to avoid water leakage over the transverse joints. Composite action between panels and girders has been developed by welded or bolted connectors in the grout-filled pockets.

Precast, Prestressed Concrete

Only few bridge construction and re-decking projects have been done using precast, prestressed concrete panels in the United States. However, precast panels are used for many new and re-decking construction projects in Japan. Two bridge examples will illustrate this.

The first bridge is the Tochigi-Gawa Bridge, Nagano, Japan. The bridge owner is the Japan Public Highway Corporation. This is a four-span bridge with a total length of 525 ft (160 m). The four spans are 125.0, 137.2, 137.2, and 125.0 ft (38.1, 41.8, 41.8, and 38.1 m). The deck is a combination of full depth precast and cast-in-place concrete. Out of the 80 ft (24.4 m) deck width, 34.9 ft (10.6 m) is precast deck and the remaining portion is cast-in-place deck. The precast panels are longitudinally post-tensioned. The bridge has been in service for 4 years. No problems have been reported on the bridge deck.

The second bridge is the Shin-Kotoni Bridge, Hokkaido, Japan. This bridge is also owned by the Japan Public Highway Corporation. This is a five-span, 541.3 ft (165 m) long bridge. The spans are 109.3, 109.3, 109.3, 107.6, and 106.3 ft (33.3, 33.3, 33.3, 32.8, and 32.4 m). The bridge deck is a full depth precast concrete deck. The total deck width is 32.9 ft (10 m). The deck is fully post-tensioned in the longitudinal direction. The bridge has been in service for 5 years with no significant problems reported to date.

The overall features of precast, prestressed concrete panels are very similar to precast reinforced concrete panels except that transverse pretensioning is incorporated. Because of the transverse pretensioning, the panels can achieve a smaller slab thickness, better crack control, and better handling characteristics.

Exodermic Bridge Deck

An exodermic bridge deck consists of a fabricated steel grid for the bottom portion and a reinforced concrete slab for the top portion. A part of the steel grid portion extends upward into the reinforced concrete in order to achieve a composite deck.

An exodermic system can be either cast-in-place or precast. Embedding the shear connectors in the concrete haunch allows the exodermic decks to be made composite with the steel girders. The haunches can be poured simultaneously with the reinforced concrete deck when a cast-in-place deck is used, or separately when using a precast deck. The advantages of an exodermic deck system include light weight, rapid erection, and simplified construction staging.

The exodermic deck system has been in use since 1984. The Kentucky DOT, Illinois DOT and other DOTs have recently selected an exodermic deck for bridge rehabilitation projects.

Steel Grid

A steel grid bridge flooring system mainly consists of flat bearing bars (main bars) in the transverse direction and secondary bars placed perpendicular to the main bars. Steel grid floors are either welded or bolted to the supporting members to achieve composite action. The steel grid can be filled with concrete at the shop or at the construction site to create a smooth riding surface. Concrete filled steel grid floors have been installed on bridges for more than 50 years.

Inverset

The New York State DOT recently solved a bridge repair problem by using Inverset precast, precompressed concrete deck panels. The Inverset bridge deck system is cast in forms suspended from wide flange steel girders to which stud shear connectors have been welded. The combined weight of the forms and the concrete induce reversed stresses in the beams, producing a prestressed effect on the girders. When the cured units are turned over, the concrete in the deck is precompressed, yielding a deck of high density and nearly complete resistance to cracking.

Comparison of Systems

A comparison of the prefabricated deck systems is shown in Tables 1 and 2. Precast reinforced concrete panels, precast, prestressed concrete panels, and exodermic systems were chosen for comparison as representative systems. This comparison is performed based on HS-25 loading, 12 ft (3.66 m) girder spacing and a 2 in. (51 mm) thick wearing surface.

As can be seen from Table 1, the precast, prestressed concrete system is about 10 percent thinner than reinforced concrete systems. Also, it is about 30 percent lighter than the reinforced concrete system and comparable in weight to the exodermic system. Also, as shown in Table 2, concrete volumes for the precast, prestressed concrete system fall in between the precast reinforced concrete and exodermic systems. However, the amount of steel reinforcement is much less than that of the other systems.

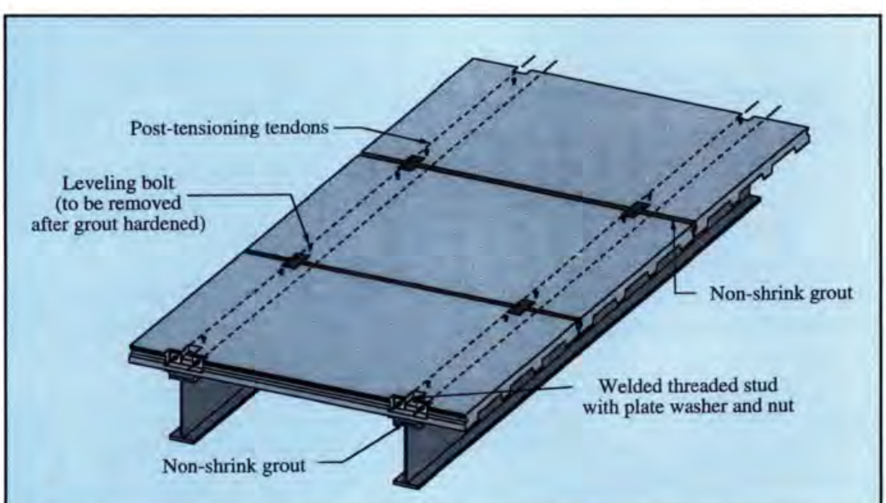

Fig. 1. Overview of proposed system.

GENERAL DESCRIPTION OF PROPOSED SYSTEM

An overview of the proposed system is shown in Fig. 1. This system consists mainly of precast, prestressed concrete panels, welded headless studs, welded threaded studs, grout filled shear keys, leveling bolts, and threaded bars for post-tensioning. The panels are transversely pretensioned and longitudinally post-tensioned.

Details of a typical precast panel are shown in Fig. 2. The overall shape of the panel is determined by the arrangement of prestressing strands for positive moments and to provide an adequate compressive zone for negative moments. One layer of steel mesh reinforcement is provided in the upper slab for flexural performance of the slab between stems. Strands are arranged in two layers and the eccentricity is minimized because the panel is subjected to both negative and positive moments. The main reasons for two layers of strands instead of one layer are to:

- Control cracking of the precast concrete surface.
- Maintain the required ultimate strength.
- Avoid a conflict with the longitudinal post-tensioning.

Additional transverse reinforcement is provided at both edges of the panels to make up for the short development length of the pretensioning strands.

Materials

The specified compressive strength of the concrete for the precast panels is recommended to be between 6000 and 10,000 psi (41.4 and 69.0 MPa). High strength concrete is useful in reducing the overall thickness of the slab. A specified strength of 5000 psi (34.5 MPa) at transfer of prestress and a 28-day strength of 7500 psi (51.7 MPa) have been chosen for design calculations.

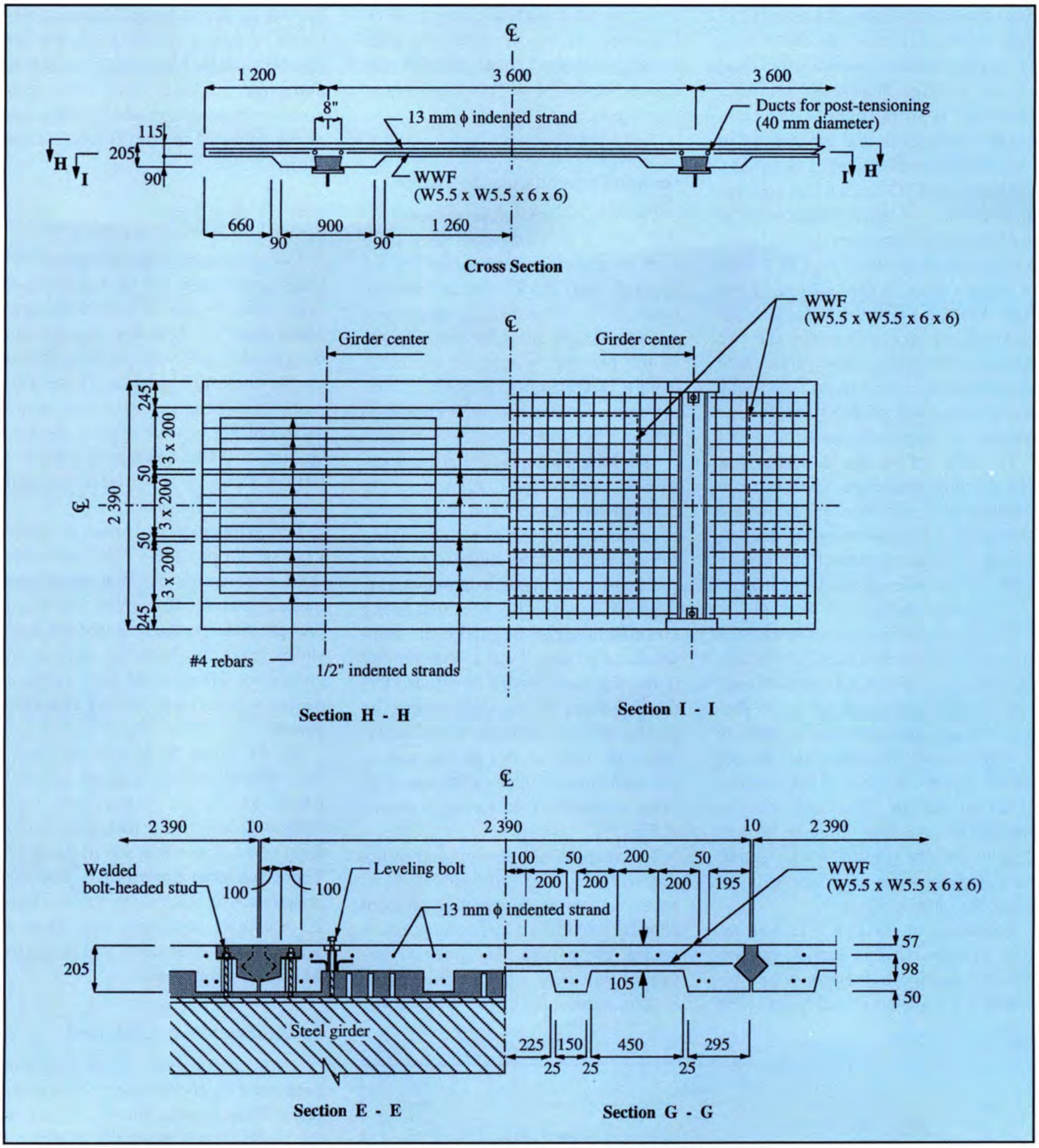

Fig. 2. General view and steel arrangement of proposed system. **Note:** 1 mm = 0.0394 in.

For prestressing steel, $^1/_2$ in. (12.7 mm) diameter 270 ksi (1.86 GPa) low-relaxation indented strand has been chosen. Strands with a $^3/_8$ in. (9.5 mm) diameter are commonly used for thin or small members to reduce transfer and development length and to avoid excessive stress concentration.

However, the required number of strands for $^3/_8$ in. (9.5 mm) diameter strand is 70 percent higher than that required for $^1/_2$ in. (12.7 mm) diameter strand in order to satisfy the requirements for prestressing force required in the bottom layer. This would increase the stem width to accommodate the strands and, hence, increase panel weight. Therefore, utilizing the capability of indented strand to reduce transfer and development length, $^1/_2$ in. (12.7 mm) diameter indented strand has been chosen for the transverse pretensioning. The mild reinforcing bars and the welded wire fabric are Grade 60.

Typical Section in the Direction of Traffic

A typical transverse cross section of the precast panel has a $4^1/_2$ in. (115 mm) thick solid slab, 11.6 in. (295 mm) wide external stems and 5.9 in. (150 mm) wide internal stems, as shown in Fig. 2. Use of a multi-stemmed section reduces the self-weight of the deck and the amount of longitudinal post-tensioning required.

The AASHTO Specifications require $2^1/_2$ in. (63 mm) of clear cover at the top of the slab when deicing compounds are used and 1 in. (25.4 mm) of clear cover at the bottom of the slab. The solid slab thickness was determined to accommodate the top strands and welded wire fabric with the required clear cover. Two-way shear (punching shear) strength was checked to support this decision.

The width of the external stems was set to accommodate two bottom strands and provide an adequate blockout for a post-tensioning anchorage device and threaded studs. The width of the interior stems was set to accommodate two bottom strands.

The longitudinal cross-sectional area (as a part of the composite section of girders) is about half that of conventionally reinforced concrete sections because the stems are oriented in the transverse direction and do not take compressive force in the longitudinal direction. This allows the amount of post-tensioning to be less than half of that required for longitudinal compression in conventionally reinforced concrete panels.

Reduction of the concrete area results in a decrease in section properties as a composite section when replacing a conventional reinforced concrete deck with the new system. However, the higher elastic modulus of the prestressed panel coupled with the reduction of deck weight offset this impact.

Section Perpendicular to Traffic

The longitudinal cross section consists of 8.1 in. (205 mm) thick solid sections at each girder location and 4.5 in. (115 mm) thick sections between them, as shown in Fig. 2. Although a prismatic shape along the entire length of the product would be desirable from a production standpoint, a nonprismatic section has been chosen to optimize the system.

The thick portion at the girder location is necessary to accommodate post-tensioning steel and to eliminate eccentricity of the post-tensioning forces. Because the highest possible position for the post-tensioning steel is limited due to the pretensioning strands, a large negative moment would be produced due to eccentricity if the thickness were constant. The thick portion of the slab lowers the neutral axis to eliminate eccentricity. Also, the width of this portion can be adjusted for different girder spacings using adjustable void forms, as shown in Fig. 3.

The shape of the section perpendicular to traffic also efficiently reduces reinforcement at the negative moment zone by providing a large compression area at the bottom. The positive impact of this section on constructability is also significant in providing a flat bottom surface at girder locations. Because of this, a simple grout stopper can be installed between the panels and girders. This is done with oversized continuous expanded polystyrene at the edges of the top flanges of the girder.

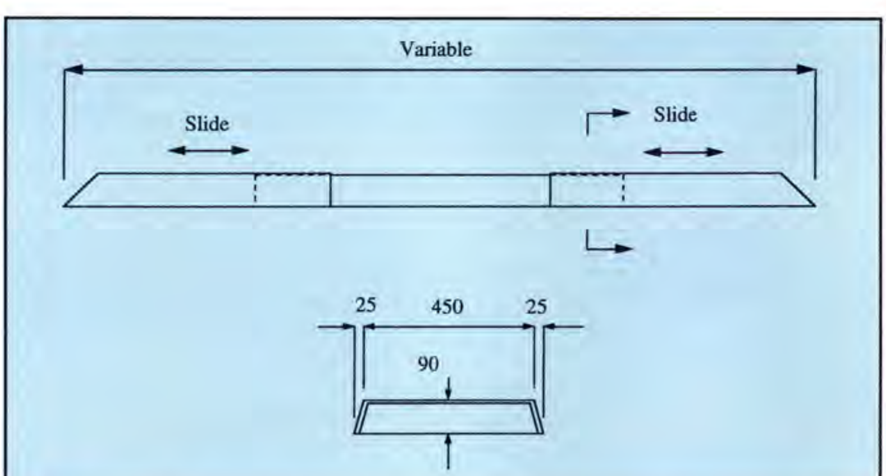

Fig. 3. Adjustable void form. **Note:** 1 mm = 0.0394 in.

Transverse Joint

Two important requirements of the transverse joints are to transfer live loads and to prevent water leakage. Many kinds of shear key shapes have been tried in previous projects. Based on the literature study of these projects, the shear key size and shape shown in Fig. 4 were chosen. A clear spacing of 0.4 in. (10 mm) is provided between panels for production and construction tolerances.

The grout material used in shear keys should provide a rapid set to reduce construction duration, must adequately transfer live loads, and must prevent water leakage. Rapid-set nonshrink grout has been the material of choice for filling shear keys on most projects involving precast concrete panels.

Set 45, made by Master Builders, Inc., was chosen as a grout material based on the study performed by Gulyas et al.[19] and a technical report from construction projects in Alaska.* There are other appropriate grouting materials. The reader may refer to Ref. 20 or contact companies such as Five Star, Fosroc, Tamstech and Sika for additional information.

Longitudinal Post-Tensioning

Longitudinal post-tensioning has been used in coordination with grout filled shear keys in many projects in the United States, primarily to prevent water leakage and to transfer loads at transverse joints. Longitudinal post-tensioning has also been used to prevent transverse cracking. Longitudinal post-tensioning has typically been provided at or near the mid-depth of the panels.

There are two options for applying post-tensioning to precast panels, depending on the details of the bridge

* Dennis Nottingham, Presentation at ACI Spring Convention, Salt Lake City, Utah, March 1995.

and allowable traffic restrictions during construction. One is full bridge post-tensioning and the other is staged post-tensioning. Because the details of staged post-tensioning are more intricate than that of full bridge length post-tensioning, this section discusses staged post-tensioning only.

Post-tensioning tendons for this new system are located above the top flanges of girders, as shown in Fig. 5. Blockouts are provided for anchorages and couplers at both transverse edges of panels. The number of post-tensioning bars is based on design calculations, which provide 200 psi (1.38 MPa) of longitudinal compressive

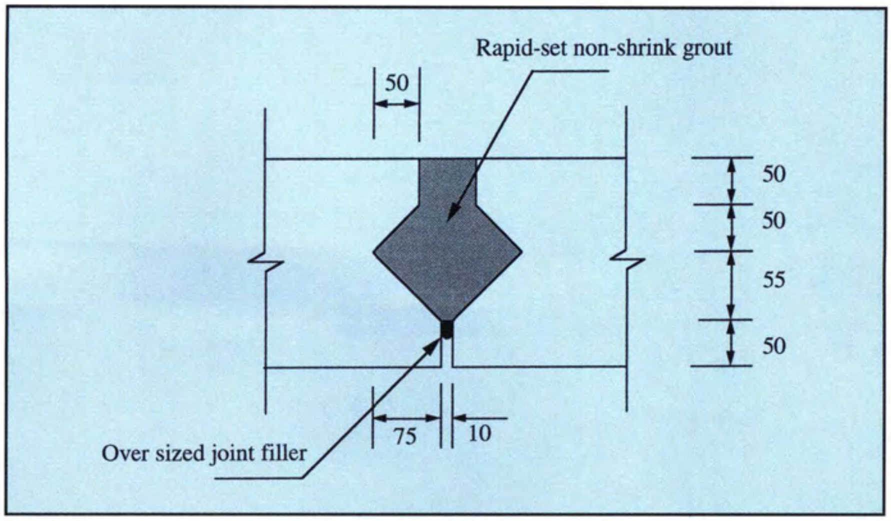

Fig. 4. Transverse joint. **Note:** 1 mm = 0.0394 in.

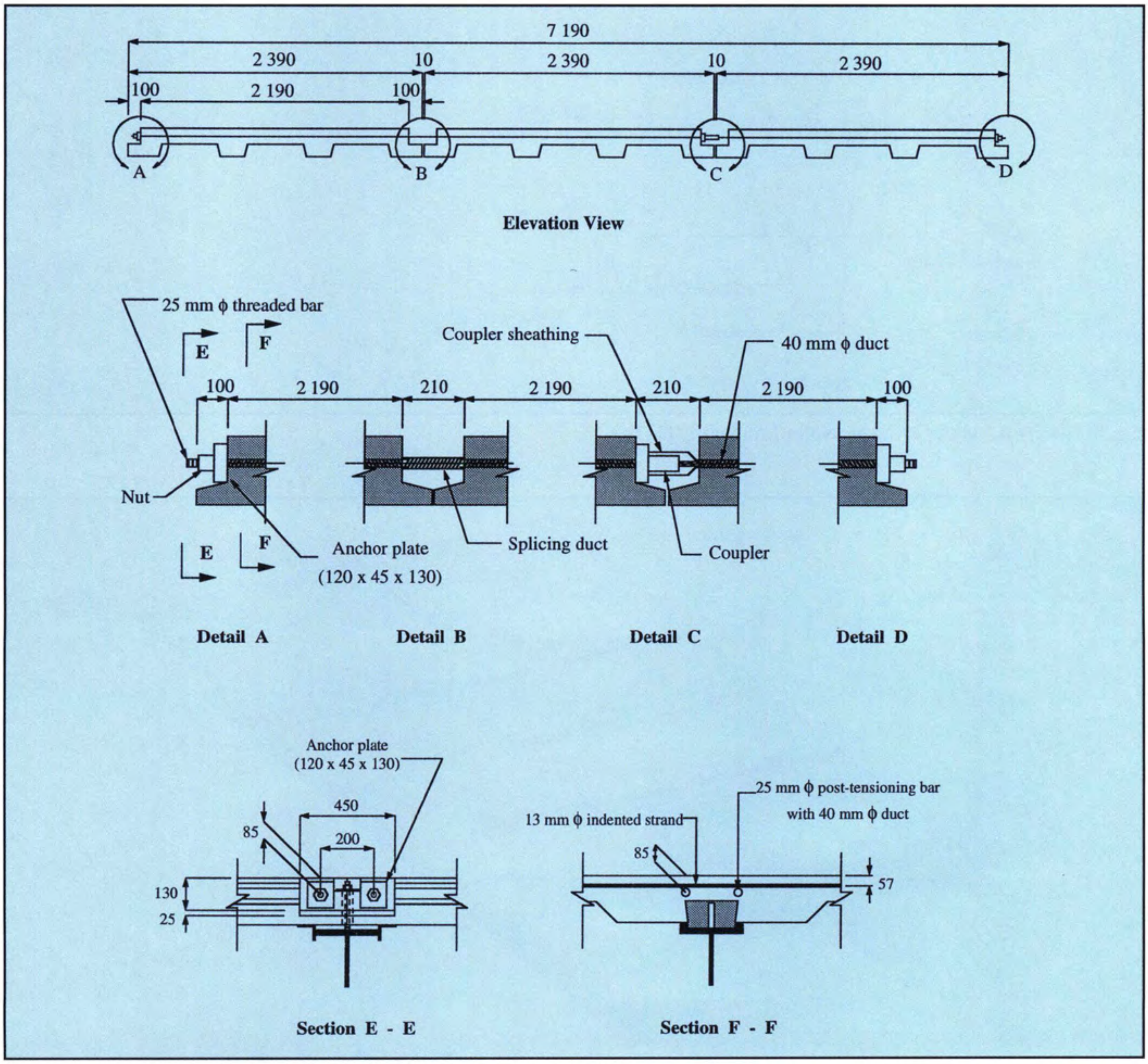

Fig. 5. Details of longitudinal post-tensioning. **Note:** 1 mm = 0.0394 in.

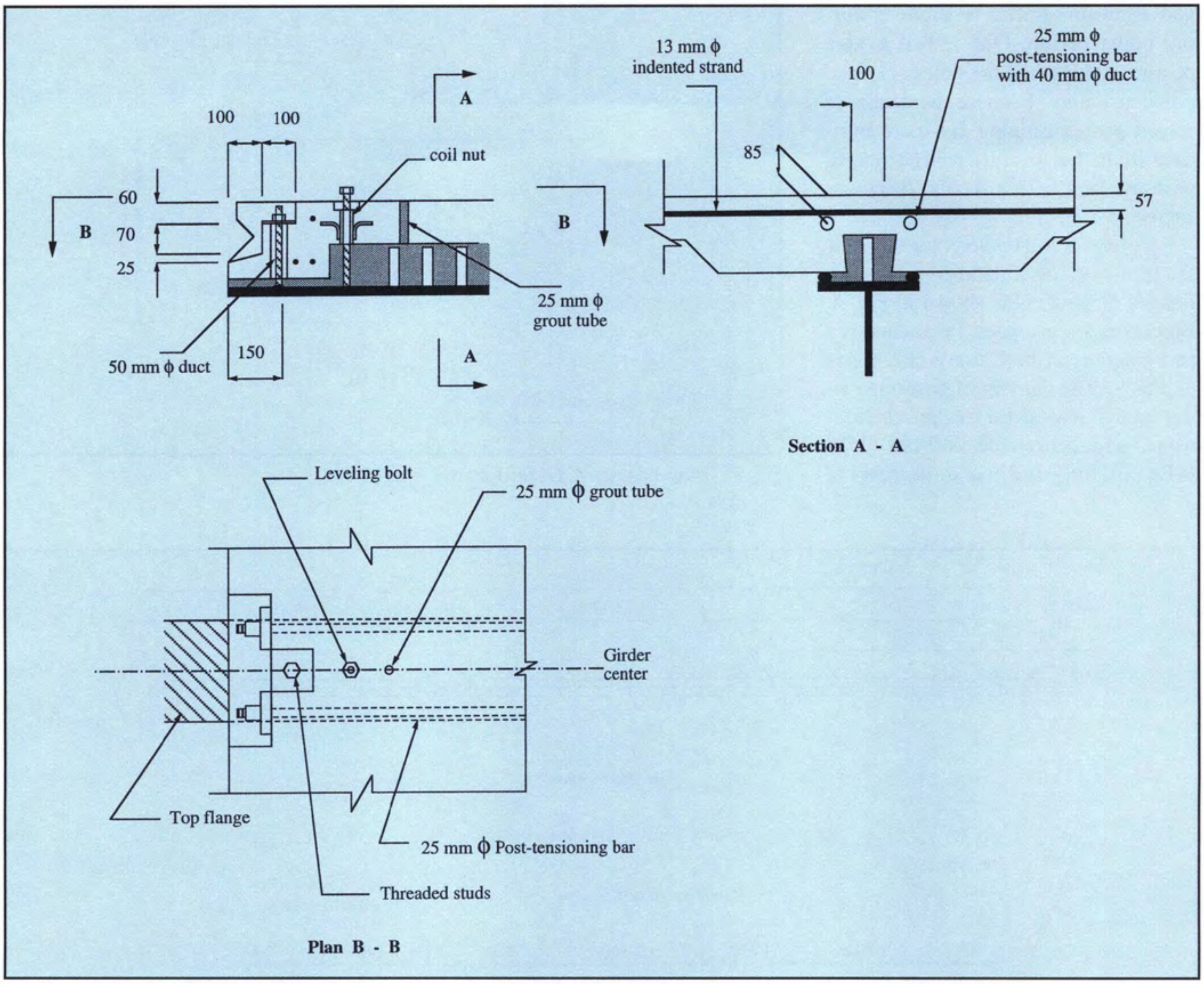

Fig. 6. Details at a girder location. **Note:** 1 mm = 0.0394 in.

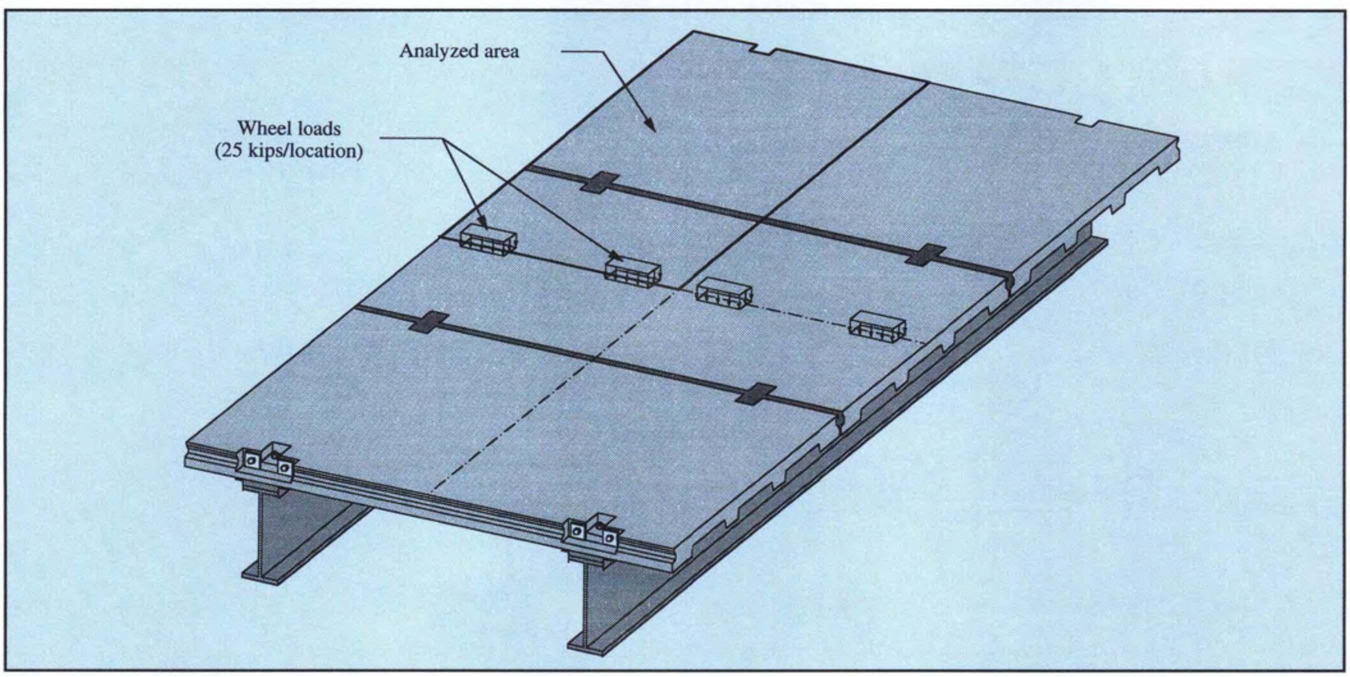

Fig. 7. Finite element analysis model.

stress in the panels. Only two 1 in. (25.4 mm) diameter 150 ksi (1.03 GPa) reinforcing bars are required for the new system because of the reduced cross-sectional area discussed earlier. However, the cost and time for installation of post-tensioning steel and grouting are substantial.

To eliminate the grouting process yet still provide corrosion protection, galvanized steel bars or fully encapsulated unbonded steel strands were considered. Unbonded steel, which is surface greased and covered with plastic tubing, has a larger outside diameter than conventional steel strand; hence, a larger post-tensioning duct is required. This is not desirable for thin members such as precast panels. Unbonded steel also requires extra field work to uncover tubes at both ends for anchorage devices. Therefore, galvanized bars were recommended for post-tensioning and the elimination of the grouting process.

Another alternative to longitudinal post-tensioning is the use of fiber reinforced plastic (FRP) materials. Composite cables, such as Carbon FRP, Aramid FRP, and Glass FRP are being developed and researched extensively in Japan as prestressing materials. FRP materials are corrosion free, have high ultimate strength, and low elastic moduli.

Sudden failure of the material due to no-yield characteristics in the stress-strain relation is an undesirable disadvantage when this material is used as the main reinforcement for beams and slabs. However, the purpose of the post-tensioning in this particular system is not ultimate strength but serviceability, such as crack control and water leakage prevention; hence, the disadvantages are minimized. The advantages of this material, especially resistance to corrosion, can be attained if it is used as external cables beneath the panels.

Even though galvanized post-tensioning bars without grout were chosen for this project, bare steel bars with grout, conventional steel strand without grout, and FRP materials are all possible alternatives for this system. The final choice of the post-tensioning material should be based on the decision by bridge engineers depending on the given situation.

The amount of post-tensioning required in a bridge deck varies with different projects. The minimum compressive stress of 200 psi (1.38 MPa) is empirically provided in the longitudinal direction to keep the transverse joints intact for leakage and also provide residual compression in the deck after stress redistribution due to differential creep and shrinkage has taken effect. In actual bridges, areas of negative moments over piers due to superimposed dead and live loads can create higher tensile stresses that must be balanced by longitudinal post-tensioning as high as 200 to 800 psi (1.38 to 5.52 MPa).

Horizontal Shear Connection

Shear connectors provide two functions. First, they provide a method for transferring the horizontal shear between the girders and the deck as a girder bends. Secondly, they provide a vertical clamping force between the top of the girder and the bridge deck. The shear connectors for the new system consist of welded headless studs and welded threaded studs with nuts, as shown in Fig. 6.

After panel erection, the short headless studs are welded on the top flange through grout pockets. These studs take only horizontal shear developed when load is applied to the composite girder. The long threaded studs are welded onto the top flanges through grout pockets in each panel to match openings in the panels. The nuts provide a means to clamp the precast panel to the steel girders. These studs resist horizontal shear and uplift. A precast concrete girder design would utilize either threaded inserts or studs grouted into the tops of girders as shear connectors.

ANALYSIS AND DESIGN

Because this system is unique, a detailed analysis was performed to determine the stresses in the various sections. The finite element method was used to determine the critical stresses. The computed stresses were then compared with experimental results.

Finite Element Analysis

Because of the multi-stem shape, the precast panel's stiffness in the transverse direction differs from that in the longitudinal direction. Therefore, the behavior of the panel is different from a solid slab. This fact results in a concentration of bending moments and shearing forces around the stem on which a wheel load is applied. Because AASHTO's formulas in Article 3.24 obtain moment in the deck slab based on a solid slab, using these formulas for a multi-stem deck required verification for their use with this system.

A finite element analysis program, ANSYS 50A, by Swanson Analysis Systems, Inc., was used to investigate the stress distribution in the panel. The results from this analysis were compared with the AASHTO formulas for solid slabs and also with experimental test results.

The test specimen was modeled as a three-dimensional structure, as shown in Fig. 7. Because the structure is symmetric about both the x and z axes, one quarter of the structure was modeled with linear-isoparametric eight-noded three-dimensional elements. Selected loading cases were: (1) post-tensioning force; (2) HS-25 wheel loads; and (3) both Items (1) and (2) combined to check the level of stresses and the stress distributions over the panels.

The finite element result shows that stresses due to longitudinal post-tensioning were distributed almost equally at the transverse joint nearest to the edge of the panels where post-tensioning forces were applied.

In the transverse direction, stresses due to wheel loads were 50 to 60 percent of those obtained from AASHTO formulas. Therefore, the AASHTO formulas for slab design can be used for the proposed precast panels.

The critical load case as determined by the finite element analysis resulted in excessive tensile stresses at the bottom surface of the slab, located underneath the wheel load between stems in the longitudinal direction. The result of the analysis indicated 617 psi (4.26 MPa) of tensile stress at this location. This stress exceeds the allowable tensile stress of 520 psi (3.59 MPa) $(6\sqrt{f_c'})$.

TEST PROGRAM

A full-scale prototype of the proposed precast panel system was constructed and tested to confirm the feasibility of the design. An overall view of the test specimen is shown in Fig. 1. The test specimen consisted primarily of three 20 ft (6.0 m) long, 8 ft (2.4 m) wide precast, prestressed panels, two 26 ft (7.9 m) long girders, welded headless shear studs, welded threaded studs, non-shrink grout, and threaded bars for longitudinal post-tensioning. The program and objectives of this experimental test were:

- Precasting of the prestressed concrete panels to confirm production feasibility.
- Construction of the bridge deck system to confirm feasibility and speed of construction.
- Experimental load testing to examine the following performance factors:
  - -Serviceability of precast panels
  - -Fatigue resistance of precast panels
  - -Flexural strength of precast panels
  - -Performance of post-tensioned transverse shear keys
- Removing precast panels to confirm ease of removability for the deck system.

PANEL PRODUCTION

The precast, prestressed concrete panels were produced by Wilson Concrete Company at the company's Bellevue, Nebraska, plant. The panels were cast on a long-line bed and a prestressing force was applied in the bed prior to casting the panels. The following sequence describes the typical order of production.

The bottom layer of strands was installed and stressed after side and end forms were affixed to the bed. Side shear key forms and end forms were then installed using wood bulkheads. Styrofoam was used to form the stemmed shape. Welded wire fabric, post-tensioning ducts and blockouts for post-tensioning anchorages were then installed, and next the top layer of strands was installed and stressed. After the locations of post-tensioning ducts were carefully checked and affixed to the top layer of strands, leveling bolts, lifting bolts, and grouting ducts at each girder location were installed at the designated locations.

The mix design utilized Type III cement to produce 7500 psi (51.7 MPa) concrete. Prestress was transferred to the panels by torch cutting the strands 3 days after casting the concrete. Both sides of the panels were sandblasted to provide a clean and roughened surface for the transverse grout joint. Fig. 8 shows the completed steel arrangement.

CONSTRUCTION

The construction process followed a segmental redecking process, assuming night-time closure and daytime opening. Two panels were placed on the steel stringers and the transverse joint between the panels was filled with non-shrink rapid-set grout (Set 45). Longitudinal post-tensioning was applied after the grout obtained a minimum 2000 psi (13.8 MPa) compressive strength. Haunches and grout pockets for headless studs were then filled with non-shrink grout.

The third panel was installed in a similar manner. Post-tensioning bars were installed and coupled at the

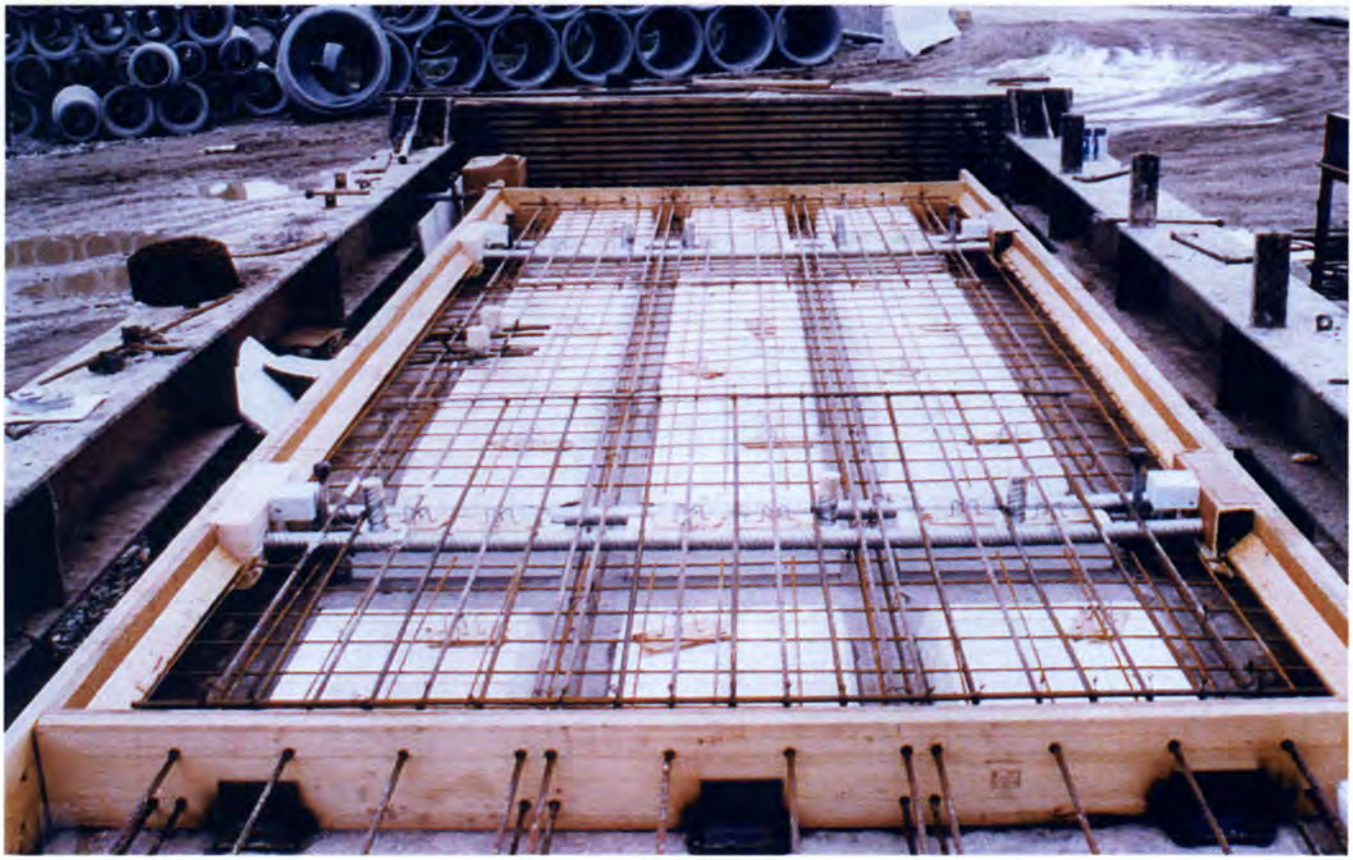

Fig. 8. Completed steel reinforcement arrangement.

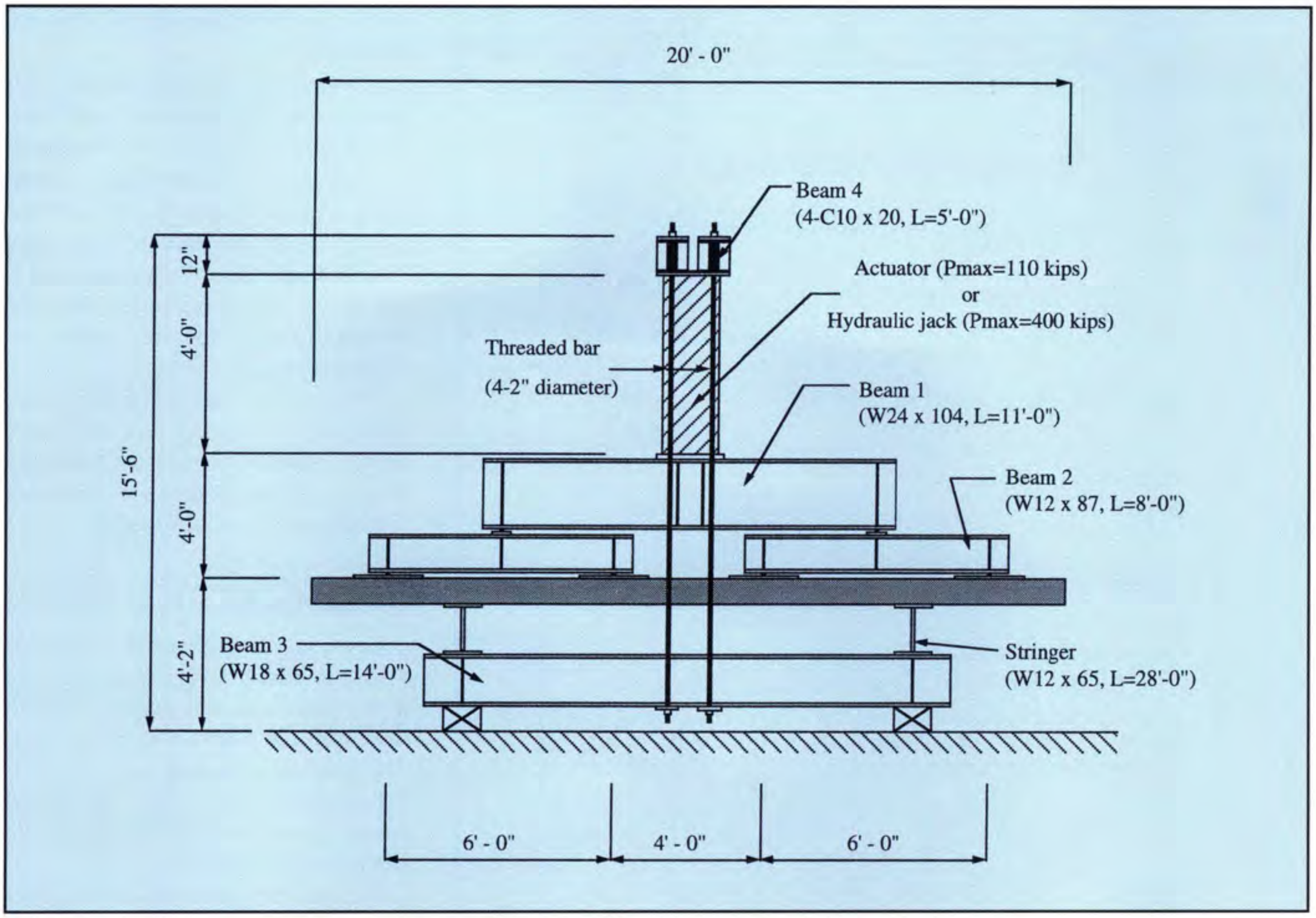

Fig. 9. Elevation of test setup. **Note:** 1 ft = 304.8 mm.

transverse joint between the second and third panels. The same process for grouting was employed for the second stage as was used for the first stage.

LOAD TEST

The simulated axle load consisted of four concentrated loads in accordance with AASHTO Specifications and was applied as shown in Fig. 9. Fig. 10 shows the load locations. Location 1 was adjacent to a transverse joint, Location 2 was centered between transverse joints, and Location 3 was at the edge of a precast panel.

The first load at Location 1 consisted of a 25 kip (111 kN) load per wheel location to simulate the rear wheels of HS25 vehicle with impact. The loads were applied monotonically to determine the stress distribution over the precast panels due to the concentrated loads. A 2 million cycle fatigue loading was then conducted at this location. The monotonic service load was again applied at Location 1 to compare the results with those before fatigue loading. A water pool was provided at the transverse joint (see Fig. 10) to check for water leakage during the fatigue loading.

At Location 3, only service loads were applied to check stress levels in the panels. The purpose of this load was to simulate a truck load at a free joint between an existing deck and a newly constructed deck panel for daytime opening and night-time closure construction. Finally, the monotonic ultimate load was applied at Location 2.

A series of strain gauges were installed on the specimen before loading. These strain gauges were used to measure stresses corresponding to the loads at each location. Displacement gauges were also installed to measure deflections at the tip of the cantilever and midspan between girders.

DISCUSSION OF TEST RESULTS

An experimental investigation was performed to determine the behavior of this system. The experimental investigation included fatigue loading and ultimate loading simulating the rear wheel of HS-25 vehicle load plus impact.

Fatigue Loading (2 million cycles) at Load Location 1

Figs. 11 and 12 show the relationship between the applied monotonic service load of 25 kips (111 kN) per loading point (simulating the rear wheel of HS-25 vehicles plus impact), and the resulting concrete stresses in the deck panels. The locations of strain measurements were at maximum positive and negative moment zones. Each figure shows the load-stress relationship before and after the fatigue loading.

Although concrete stresses due to service load after fatigue loading measured slightly higher than before fatigue loading, at most locations load-stress lines were almost identical before and after the fatigue loading. In addition, there were no cracks or water leakage during the fatigue loading.

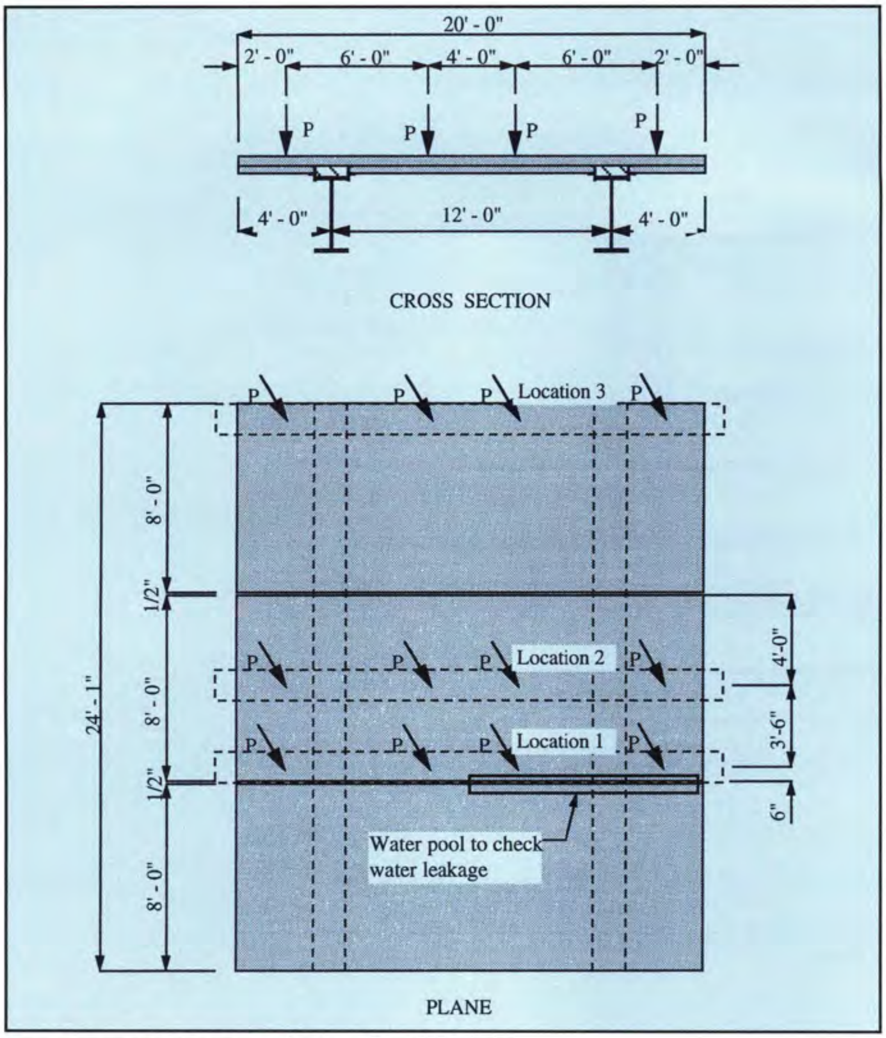

Fig. 10. Loading locations. **Note:** 1 ft = 304.8 mm.

ity reinforcement in the longitudinal direction.

Stresses are approximately four times larger than those at Load Location 1, at which panels are continuous via a grouted transverse joint. Transverse cracks occurred at the free-end side of the second stem from the edge. This indicates that the maximum tensile stress in the longitudinal direction occurred at this location where the thickness changes.

These results show that the free joint between an existing and new deck needs a scheme to maintain continuity or to avoid edge loading when the deck is temporarily opened to traffic.

Ultimate Loading at Load Location 2

Upon initial application of service load, the measured stress distribution in the transverse direction agreed with that from the finite element analysis. The stresses measured were approximately 50 percent of those computed based on the AASHTO formulas. No cracking was found at this stage.

The load was increased up to a factored load of 55 kips (244.6 kN) per load point. At 45 kips (200.2 kN) per load point, transverse cracking occurred at the bottom surface underneath the load point between the girders; however, there were no additional cracks at the factored load. At 80 kips (355.8 kN) per load point, some flexural cracks occurred, as shown in Fig. 15.

Longitudinal cracks at the girder locations were most likely due to the

Fig. 13 shows the stresses in the concrete due to service loading after fatigue loading. Stresses for the loaded side and the unloaded side of the transverse joint are about the same for maximum positive and negative moment zones. This would indicate that the transverse joint detail effectively transfers loads from one panel to an adjacent panel.

Deflections due to service load were 0.083 and 0.008 in. (2.11 and 0.20 mm) at midspan and at the tip of the overhang, respectively. Span-deflection ratios are less than 1/1000. These results indicate that the performance of the transverse joints meets all requirements for a precast panel bridge deck system, including serviceability, ability to transfer loads, and no water leakage.

Edge Loading at Load Location 3

Very high stresses occurred at the edge of the panel due to service load, as shown in Fig. 14. The figure also shows that cracks occurred at 80 percent of service load. This is due to the discontinuity of the panel and the lack of stiffness and continu-

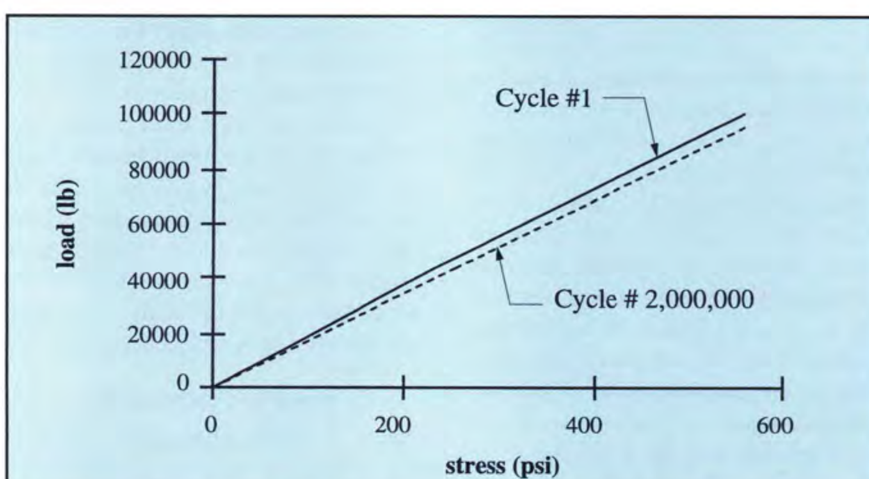

Fig. 11. Load-stress relationship at maximum positive moment location (top surface). **Note:** 1 psi = 6.895 kPa; 1 lb = 4.448 N.

negative moment developed at the ungrouted post-tensioning ducts. Oval-shaped cracks around two load points between the girders and a bell-shaped crack around each load point on the overhanging slab can be attributed to the slab responding in two-way action for flexure.

Because the specimen carried more than the capacity of the hydraulic jack, the test setup was modified by removing the load point at the overhangs and applying the loading at the two interior points only. The panel suddenly failed at 122 kips (542.7 kN) per load point due to punching shear at one of the load points. The computed punching shear capacity was 70 kips (311.4 kN) per load point.

From the test results, it is apparent that the AASHTO formula predicts a very conservative punching shear capacity. The AASHTO formula is based on a truss model with a concrete strut and the steel reinforcement as a tension chord. Based on this mechanism, the depth of steel reinforcement controls punching shear capacity because a deeper reinforcement gives a deeper truss element.

In the AASHTO formula, however, some important factors are missing. They include the amount of reinforcement and the effect of prestress. In reviewing shear-friction theory, the shear capacity can be determined by compression in the concrete from the tension steel across the shear plane and the friction coefficient depending on the shear plane conditions.

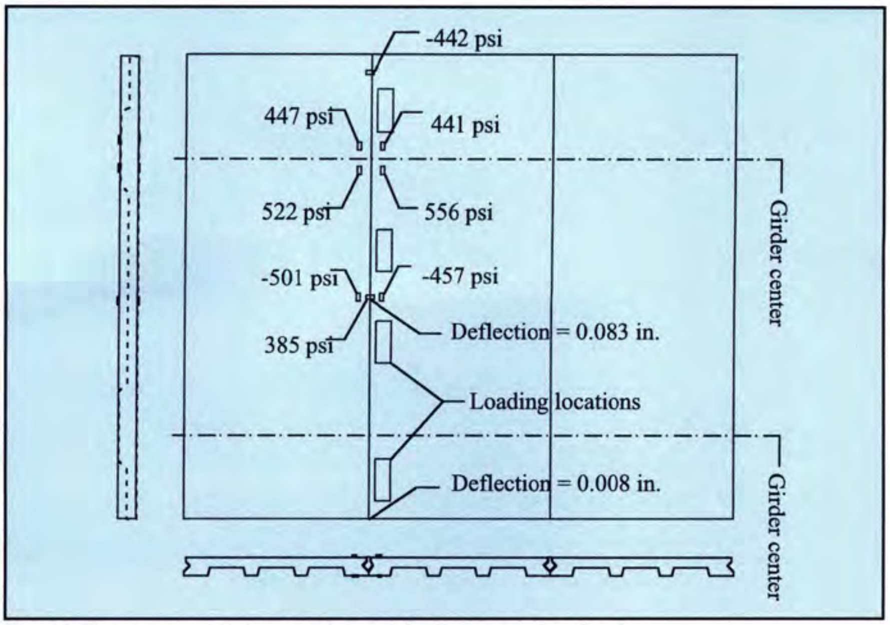

Fig. 13. Stresses and deflection in panels due to service load.
Note: 1 psi = 6.895 kPa; 1 lb = 4.448 N.

Based on this method, the precompression of the concrete slab due to prestress provides more shear capacity; however, the AASHTO formulas do not include this effect. When the slab is heavily prestressed like the test specimen, the prestress may significantly affect the punching shear capacity.

By inspecting the specimen after failure, it was found that the two stems adjacent to the loading points failed in shear at about 1 ft (0.305 m) from a girder location. This indicates that the stems behaved like a beam after the transverse crack occurred between the stems. The specimen obviously carried load even after the shear failure of the stems in the surrounding solid slab and other stems.

From this result, it was found that the system has a highly optimized loadbearing mechanism at the ultimate stage. Figs. 16 and 17 show the punching shear failure in the solid slab between the stems and the shear failure in the stems, respectively.

The ultimate load of this specimen was approximately 190 percent of the calculated factored load and overall behavior of the deck system was excellent. In addition, no cracks or unusual distress were found at the transverse joint during the ultimate loading.

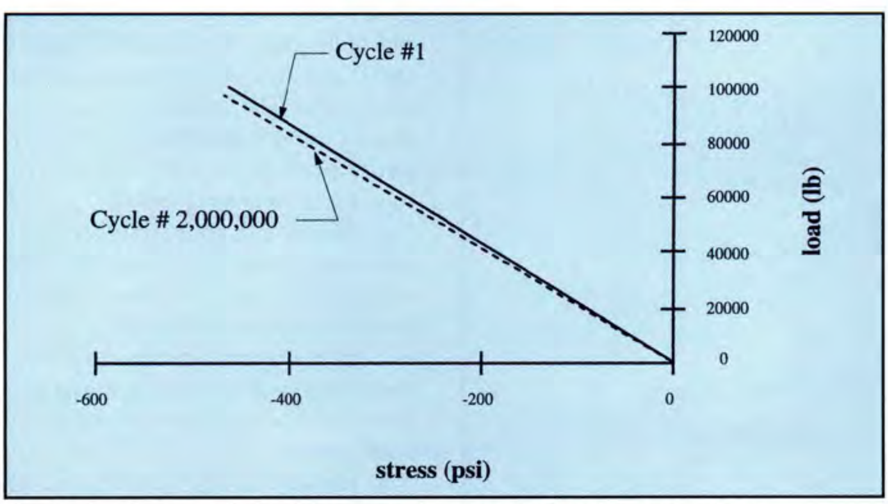

Fig. 12. Load-stress relationship at negative moment location (top surface).
Note: 1 psi = 6.895 kPa; 1 lb = 4.448 N.

PANEL REMOVAL

The precast panels were removed one by one after the post-tensioning bars were unstressed and removed. Nuts installed on the threaded studs were also removed before starting the panel removal process. A schematic of the panel removal process is shown in Fig. 18.

The initial attempt to remove the panels started with Panel 3 by jacking upward at Location 1 without first demolishing the transverse joint between Panels 2 and 3. However, this technique failed because the transverse joint was strong enough to prevent separation of Panel 3 from Panel 2. In addition, the free edge of Panel 3 was

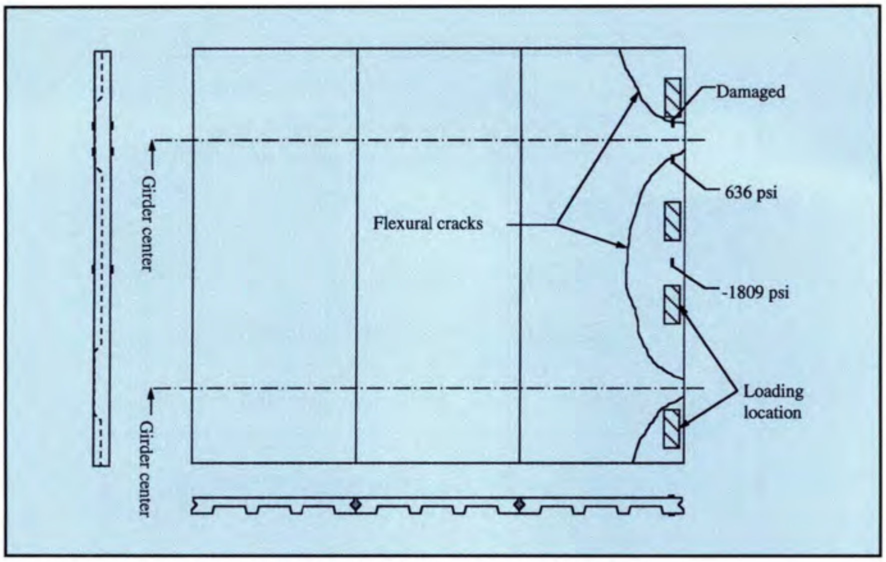

Fig. 14. Cracks and stresses on top surface under service load.
Note: 1 psi = 6.895 kPa.

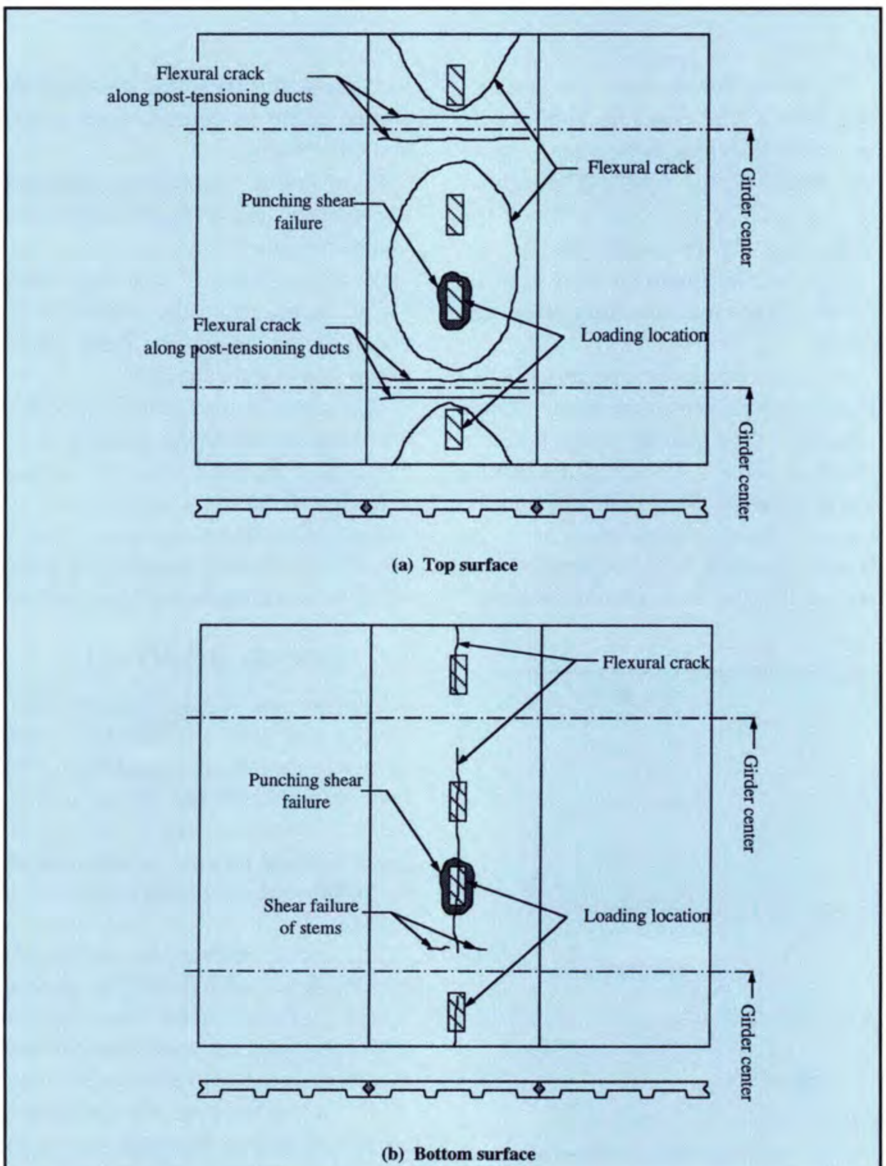

Fig. 15. Behavior of panels at ultimate stage.

too weak to support the jacking load. The jacking location was then moved to Location 2, and the transverse joints between panels were demolished by jackhammer. Using this scheme, Panel 3 was removed from the steel girders.

For the removal of Panel 2, hydraulic jacks were applied at Location 3 and that side of the panel was separated from the corresponding girder. The jacking location was then moved to Location 4 and the other side of the panel was separated from the girder. Finally, jacking locations were moved to Location 5 and Panel 1 was removed. During this removal, the jacking force was measured, and the maximum force required to remove the panel against the steel girders was about 30 kips (133.5 kN) per jack.

Overall removability of the panels was good and short headless studs allowed for easy removal of the precast panels. However, it was found that threaded studs provided an unexpected resistance to removal of the panels from the girders. This may be due to the confined grout of the steel pipes, which was used for the vertical ducts to accommodate the threaded studs.

NEW MODIFICATIONS

Because the full-scale test was performed, few simplified details had been worked out for this system. Those simplified details are given below:

Details at Girder Locations

A large amount of hardware was concentrated in the precast panels at girder locations. This included leveling bolts, ducts for threaded headless studs, and blockouts for post-tensioning anchorage. The details at these locations can be simplified to improve panel productivity. The following modifications are suggested:

1. Move the locations of post-tensioning ducts away from the girder centerline to provide more space between the ducts (see Fig. 19).

2. Use external temporary post-tensioning, and use strand tendons for permanent post-tensioning within the deck panels.

3. Furnish insert anchors and tie-down bolts at the bottom surface of panels to tie down the panels to girders instead of the threaded headless studs. This would eliminate the necessity for field welding of the threaded studs after the panel placing and would also facilitate deck removal (see Fig. 18).

External Temporary Post-Tensioning

To simplify the construction process, external temporary post-tensioning could be used. A schematic of an external post-tensioning system is shown in Fig. 20. With this scheme, strand tendons can be used for permanent post-tensioning after redecking is completed. This provides the following advantages:

1. Blockouts are eliminated for post-tensioning anchorages from precast panels.

2. Installation of final post-tensioning tendons is easier (no splicing required).

3. Strands are more flexible to install and less expensive than high strength bars.

4. The permanent post-tensioning process is removed from the critical path.

Details for the Joint Between New and Existing Decks

Details to avoid a free-edge loading should be developed for temporary opening of the deck to traffic. The details must either create continuity at the joint between new and existing decks or keep the loading location away from the edge.

When the bridge is temporarily opened to traffic, bridge girders are non-composite at the joint between new and existing decks. This may result in unacceptable stresses in the girders at the open joint location; therefore, details to maintain composite action also need to be developed.

Details for Longitudinal Joints

If the bridge needs to be opened to traffic for 24 hours a day, some lanes of the bridge must be used for traffic while the rest of the lanes are under construction. In this situation (transverse segmental construction), the bridge decks would have longitudinal joints and the decks must be integrated with the transverse joint. Details for

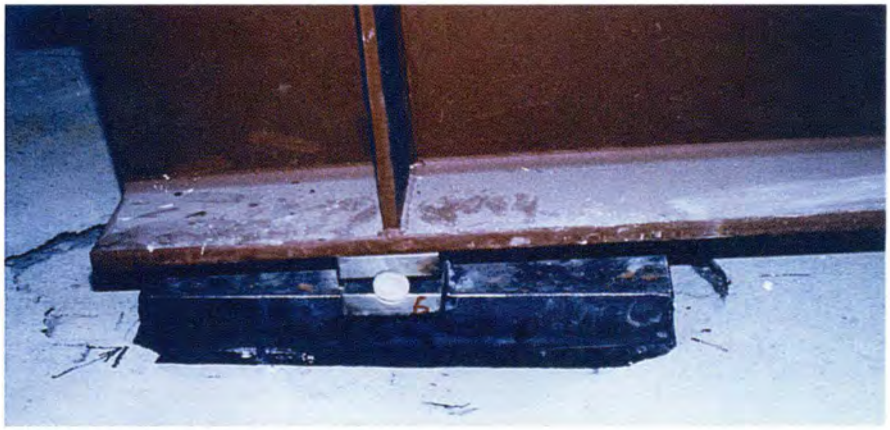

Fig. 16. Punching shear failure of panel.

Fig. 17. Shear failure of stems.

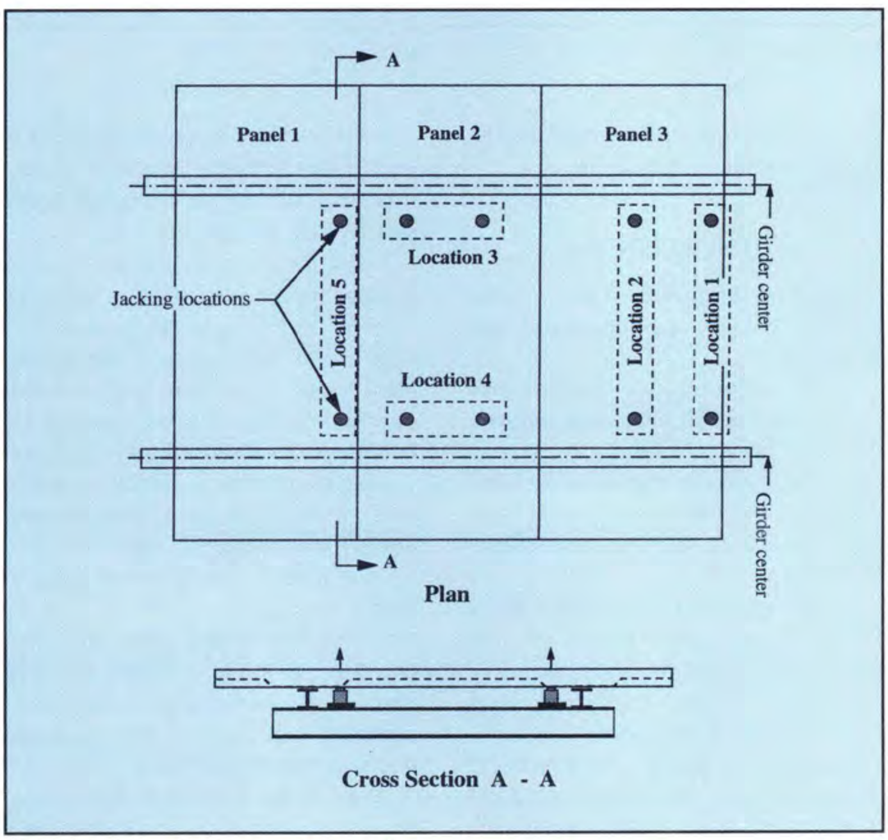

Fig. 18. Schematics for panel removal.

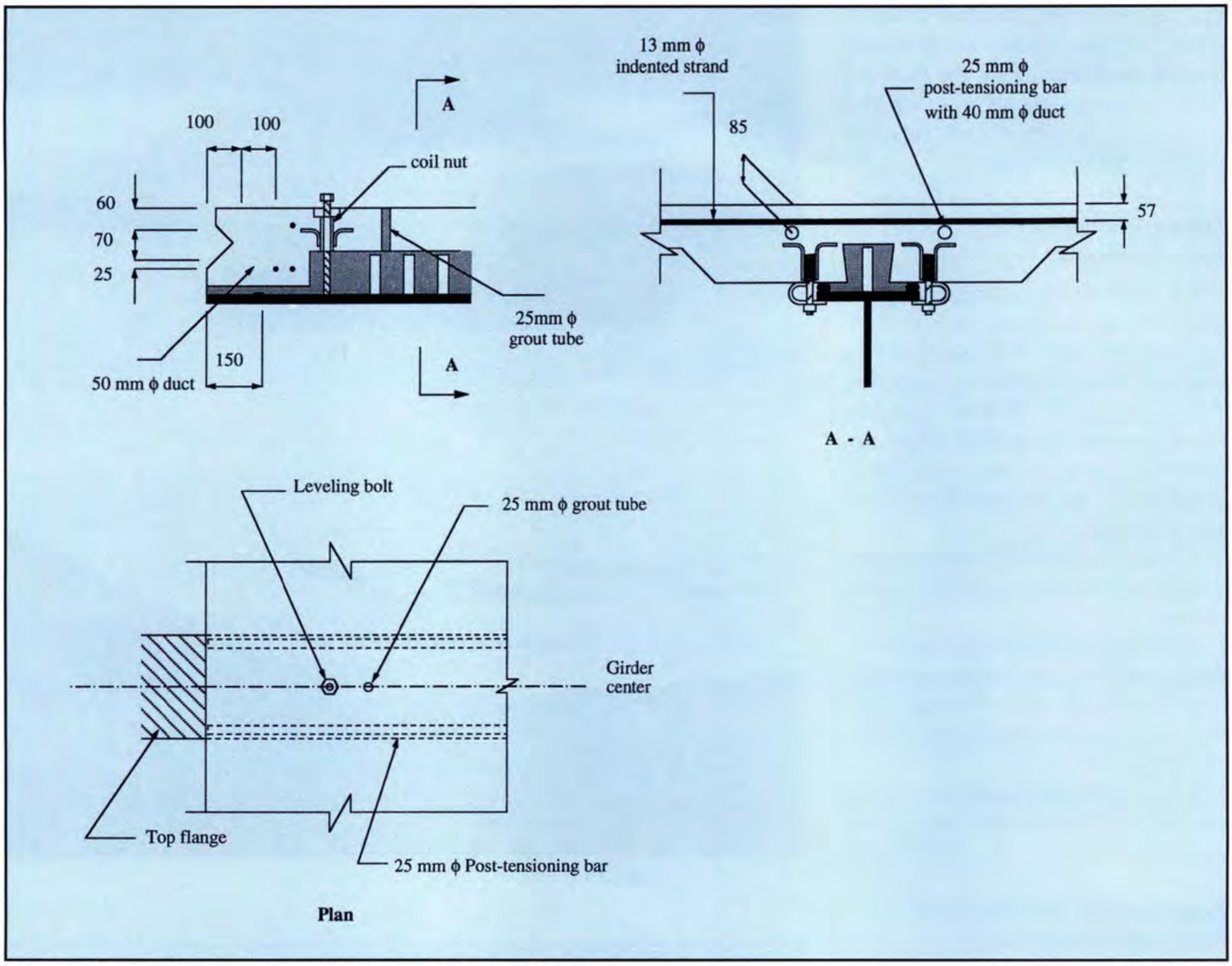

Fig. 19. Modified details at girder location. **Note:** 1 mm = 0.0394 in.

the longitudinal joints should be developed for this construction.

CONCLUSIONS

Based on the results of this investigation, the following conclusions can be drawn:

1. The performance of the proposed system meets all structural requirements for bridge decks.

2. The proposed system is cost competitive with other concrete panel systems yet it is 10 to 30 percent lighter in weight.

3. The system is comparable to an Exodermic system in weight yet it is thinner and much less expensive.

4. Panels can be rapidly produced, constructed, and removed.

5. Indented transverse pretensioning strands provide satisfactory prestress transfer and bond to the concrete in this system.

6. Grouted post-tensioned transverse joints between precast panels exhibit satisfactory performance under service load and fatigue load.

7. For deck replacement projects, a temporary condition may exist in which the new panels have a free edge under traffic. Under this condition, large deflections and flexural cracks may develop in the precast panel due to discontinuity. It is advisable to develop a scheme to maintain continuity at the existing-to-new deck joint to avoid edge loading when the deck is temporarily open to traffic.

8. Punching shear, rather than flexure, was the mode of failure, the 115 mm (4.5 in.) thick flange carried a concentrated load equal to 190 percent of that required by AASHTO. This is indicative of the very large flexural capacity of bridge decks and the potential for reducing flexural reinforcement.

9. Stresses measured in the deck panels due to wheel loads agreed with those from finite element analysis but were only approximately 50 percent of those computed based on the AASHTO Specifications. The AASHTO provisions appear to be too conservative.

10. Headless studs can help facilitate panel removal.

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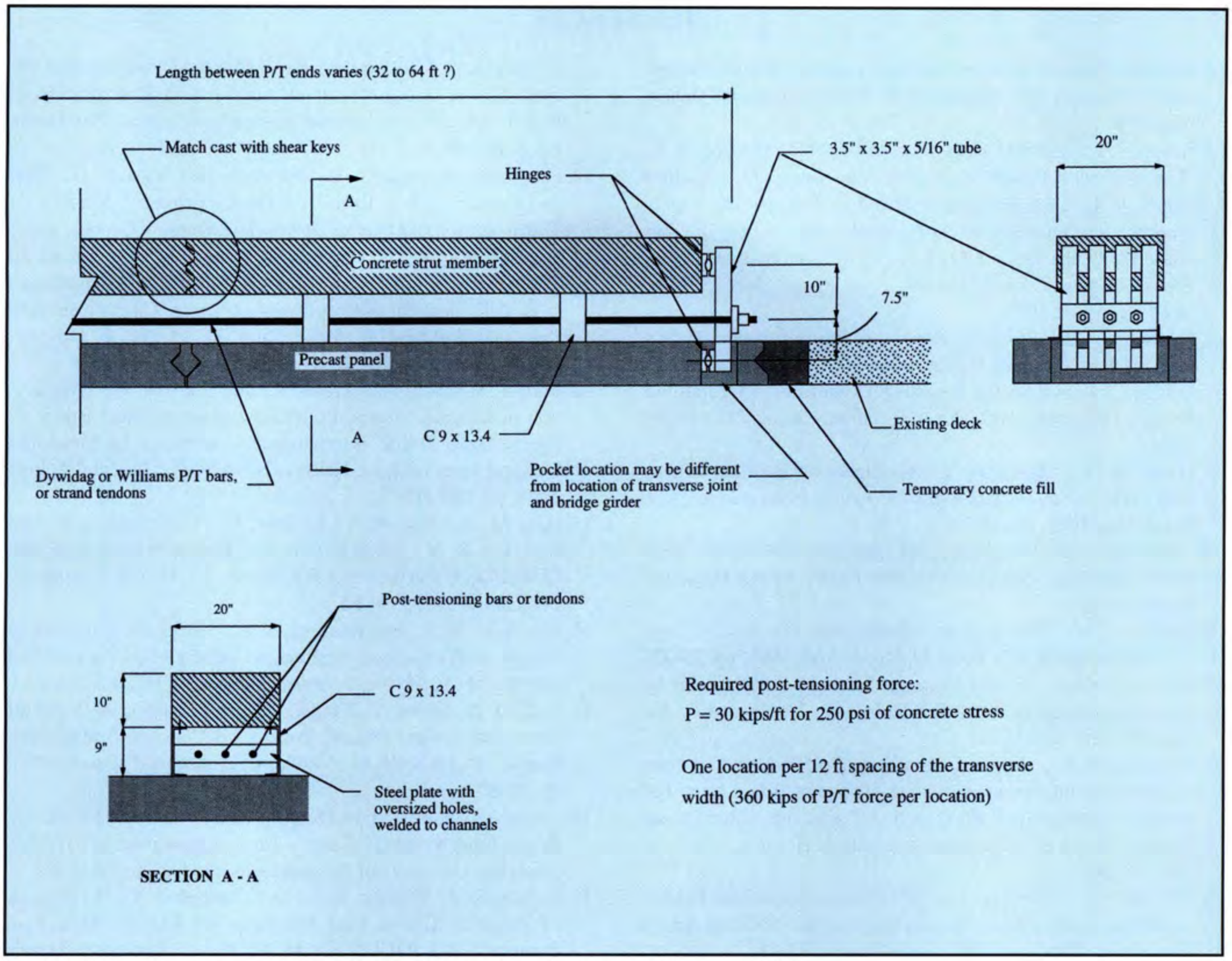

Fig. 20. External temporary post-tensioning. **Note:** 1 in. = 25.4 mm; 1 kip = 4.448 kN.

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