# Building and Rebuilding of Philadelphia's Walnut Lane Memorial Bridge

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*With a main span of 160 ft (49 m), the Walnut Lane Memorial Bridge in Philadelphia, Pennsylvania, was the first long-span prestressed concrete bridge built in the United States. It opened to traffic in 1951. Highlights of the design and construction of this post-tensioned structure are presented, showing innovations which have profoundly influenced the technology of prestressed concrete in the succeeding 40 years. A report on the service life of the bridge traces events that led to the replacement of the superstructure in 1990. Part 2, describing demolition work and design, fabrication and erection of the replacement structure, will appear in the July-August 1992 issue of the PCI JOURNAL.*

**Charles C. Zollman** participated in the original design and construction of the Walnut Lane Memorial Bridge from 1948 through 1950.

**Frank Depman** was responsible for the repair work conducted on the bridge in 1969.

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**Edward F. Hollander** was superintendent in charge during 1990 for the entire site construction operation.

# PART 1: A History of Design, Construction, and Service Life

The Walnut Lane Memorial Bridge in Philadelphia, Pennsylvania (completed in 1950), was the first long-span prestressed concrete bridge built in the United States. The concrete girders of the bridge were cast on site and post-tensioned. The structure provided almost uninterrupted vehicular service for nearly 40 years.

Apart from historical interest, the successful completion of the Walnut Lane Memorial Bridge was significant for two important reasons:

- Soon after the structure was completed, prestressed concrete became a preferred building material in American bridge construction.
- The bridge almost single-handedly created a new precast concrete industry which not only furnished pretensioned products for bridges but also for buildings and other structures.

During the service life of the Walnut Lane Memorial Bridge, some of its girders underwent major repairs and finally it was decided to replace the entire superstructure. Site preparations to remove the existing girders began in 1989 and actual construction of a new superstructure occurred the following year.

The reconstruction of the Walnut Lane Memorial Bridge and realignment of its approaches in 1990 stirred up nearly as much interest and curiosity in the engineering and construction community as did the original structure in 1949,[1,2] when it was the most publicized bridge in the world. Along with the interest in reconstruction, however, came questions as to why the replacement became necessary and what circumstances led to the decision for it to happen.

This paper answers some of those questions and records for posterity the remarkable saga of the Walnut Lane Memorial Bridge. It talks about the courageous and imaginative engineers who participated in the design and construction of the original bridge. Their decisions were to have a major impact on the future of bridge engineering practice.

Forty years later, the replacement of this famous bridge gave American engineers, once more, the opportunity to be bold, imaginative and innovative, and yet at the same time cautious and prudent. In contrast to the original bridge, which predominantly followed European engineering and construction practice, the reconstructed bridge was based on American-developed design and construction techniques.

The replacement superstructure, described in Part 2 of this paper, is an all-pretensioned design, consisting of three simply supported spans identical in length to those of the original bridge. However, the new girders,

> *"There must always be a better way to do a job."*
>
> **Roger Corbetta**
> Founder
> Corbetta Construction Company
> New York, N.Y.

based on a modified hybrid standard AASHTO Type VI girder, were all manufactured in a PCI-certified plant instead of the girders being constructed and post-tensioned on site.

Technological advances in the United States have made this change possible. Development of the seven-wire strand tendon, bonded to stronger concrete [6000 psi (41 MPa) or more], has made it economical to substitute assembly line production in a plant for the job-site casting of post-tensioned girders for longer spans to carry heavy loads. Another key to the success of plant precasting is that American engineers have developed specialized transportation techniques and erection equipment so that precast elements can be transported economically over long distances, 100 miles (160 km) or more.

Reconstruction at the site was completed in early fall of 1990 and the second Walnut Lane Memorial Bridge

Fig. 1a. The original Walnut Lane Memorial Bridge in Philadelphia's Fairmount Park had a post-tensioned main span of 160 ft (49 m). The superstructure is about 50 ft (15 m) above Lincoln Drive.

was opened to traffic in October 1990 after a construction period of about 14 months. As bid, the reconstruction cost, including improvements in bridge and approach road alignment, was about $4.3 million. Change orders during reconstruction increased the cost to approximately $5 million.

The competitive replacement of the original post-tensioned Walnut Lane Memorial Bridge superstructure with a more economical, fully pretensioned structure, clearly demonstrates the progress possible in a free competitive society. Striving to build a "better mousetrap" continues to be central to, and in line with, the American dream of building more efficient and economical structures. "There must always be a better way to do a job," as builder Roger Corbetta said many years ago. His zest for concrete and zeal for quality were unmatched and unparalleled.

## DESIGN OF THE ORIGINAL WALNUT LANE MEMORIAL BRIDGE

### Origins of the Bridge; People Involved

The story began in 1948 when the Preload Corporation of New York retained Gustave Magnel, professor of engineering at the University of Ghent, Belgium, as a special consultant for the development, design and construction of *linear* prestressed concrete structures. Preload Corporation was already an American leader in the construction of circular prestressed concrete structures and strategically well placed to promote linear prestressed concrete, particularly in the Philadelphia area.

Professor Magnel, representing the Belgian engineering and contracting community on behalf of the Belgian American Educational Foundation, had been invited in 1946 to visit the United States to gain insight into American construction technology developed during World War II. In return, he acquainted American engineers and constructors with European progress in linear prestressed concrete, a field in which he had personally pioneered following the footsteps of Eugene Freyssinet in France. (Freyssinet is generally acknowledged as the inventor of prestressed concrete.) Linear prestressed concrete had been nonexistent in the United States up to that time and was little known to American engineers.

It was this alliance between Preload and Magnel that brought about the design and construction of the original Walnut Lane Memorial Bridge (see Figs. 1a, 1b, 2 and 3). Magnel was responsible for the conceptual and final design of the bridge and also served as construction adviser.

Also from Belgium, Clement Atchit, chief superintendent for the Brussels contractor, Blaton-Aubert, was Magnel's assistant on prestressed concrete construction. He was assigned to the Philadelphia job site during construction of the first full size bridge girder which was tested up to and through failure.[3,4]

Several engineers for the City of Philadelphia were also instrumental in the pioneering effort. It is a testimony to the courage and vision of these men who had never seen a linear prestressed concrete structure, nor designed, nor constructed one, that they did not hesitate to proceed in an uncharted field.

Samuel S. Baxter, assistant chief engineer for the Philadelphia Bureau of Engineering, directed the project with authority, skill and diplomacy. E. R. Schofield, principal assistant engineer in charge of the day-to-day design operations of the Bureau, was responsible for the preparation of the detailed contract documents. And finally, Max Barofsky, the chief construction engineer for the Bureau, monitored the work to ensure that construction was proceeding according to plans and specifications.

For the Preload Corporation, Charles C. Zollman, a former student of Magnel and, at that time, staff engineer at Preload, and Ted Gutt, then a junior engineer for Preload (in 1987 he became Chairman of PCI), not only coordinated the Belgian and American design effort, but participated in design developments, adapted where possible European construction techniques to American practice, and developed the necessary shop drawings. A. G. Formel, an astute, capable and experienced engineer, was in charge of construction for Preload.

Fig. 1b. Underside view of Walnut Lane Memorial Bridge, showing post-tensioned I-girders. There were 13 such girders in the main span.

## Girder Design and Post-Tensioning

As shown in Fig. 4, 13 adjacent I-girders, 79 in. (2 m) deep and 160 ft (49 m) long, were required for the main span. Each girder weighed approximately 80 tons (73 t). Seven 74 ft (23 m) long girders, identical in concrete cross section to those of the main span but alternating with them (Fig. 4), were required for each approach span. Both of these girder arrangements allowed for the 44 ft (13.4 m) wide roadway and two 9 ft 3 in. (2.8 m) wide sidewalks. This gave an out-to-out bridge width of 63 ft $9^1/_2$ in. (19.4 m), including the aluminum handrailing and posts.

Each of the main span girders required 256 high strength, stress-relieved single wires having a 0.276 in. (7 mm) diameter. Stressed to produce an initial force of 7800 lb (35 kN) per wire, the wires produced a total force of 2,000,000 lb (9 MN) per girder. In each girder, the 256 wires were placed in four rectangular chases, each to receive 64 wires, arranged as shown in Fig. 5. Longitudinally, the two outer chases in the bottom flange remained straight throughout the entire span while the chases in the web plane curved up parabolically as shown in Fig. 6.

In the approach span, only three chases were required (see Fig. 7). Each of the outer chases housed 24 wires which remained straight throughout the span, while the center chase, housing 48 wires, curved up parabolically. The wires were stressed to produce a total initial force of approximately 750,000 lb (3.3 MN) per girder.

Transverse prestressing through the diaphragms cast integrally with the main span girders, approximately 14 ft


Fig. 2. Seen with a light dusting of snow, the bronze plaque on the bridge (remounted after reconstruction) acknowledges the pioneering design and construction effort on the structure.

6 in. (4.4 m) on centers, as shown in Fig. 6, completed the stressing operations. An 8 in. (200 mm) minimum reinforced concrete deck slab was placed over the approach span girders. In the main span, the cast-in-place concrete deck wearing surface varied from 2 in. (50 mm) thick along the curb to 6 in. (150 mm) at the centerline of the bridge. An overlay of 1 in. (25 mm) bituminous rectangular planks completed the deck.

All prestressing wires were anchored in pairs by Belgian (Magnel-Blaton) anchorage units (Fig. 8). The anchorage consisted of steel plates, called sandwich plates, with two trapezoidal grooves in the upper surface and two in the lower surface. A machined steel wedge fitted into each slot and fixed two wires in each groove. With four pairs of wires per plate, the anchorage required eight sandwich plates per duct at each end of the main span girders, or a total of 64 plates per girder. To protect the wires from corrosion, grout was fed into the ducts by gravity, and anchorages were encased in concrete.

Fig. 3. Primary participants in the bridge design and construction, seen in 1949. From left to right: E. R. Schofield principal assistant engineer, Philadelphia; Gustave Magnel, University of Ghent, Belgium; and Charles C. Zollman of the Preload Corporation, New York.

## ORIGINAL BRIDGE CONSTRUCTION HIGHLIGHTS

### Materials for Construction

**Concrete —** The bridge design was based on concrete having a 28-day compressive strength of 5400 psi (37 MPa). The determining factor for construction, however, was to achieve at least 40 percent of the 28-day strength, that is, 2160 psi (15 MPa), as soon as possible so that post-tensioning could begin. Low slump concrete and energetic vibration were prerequisites to achieving this goal.

Obtaining the low slump through the expected use of ready mixed concrete was to become a problem, as was the placing of concrete in a deep girder with small cross-sectional dimensions. A 2 in. (50 mm) maximum slump concrete was finally allowed.[5] The rubber cores that formed the ducts in the thin web width, along with the reinforcing steel in the web, reduced the effectiveness of internal vibration because only small vibrating heads could be used to avoid damaging the rubber. Therefore, vigorous external vibration was needed; yet, it was hardly possible because of the effect on the forms (described later). Thus, the effective placing and consolidation of the girder concrete was actually very difficult.

**Prestressing Steel —** Not only did the John A. Roebling Company meet specification requirements, but also its engineers were the first to understand the need for and technical advantage of using stress-relieved wires, which they produced in time for use in this bridge as a result of research they had undertaken. This marked the world's first use of the best quality single wire ever produced. It became widely used thereafter, only to be superseded by another American first, the stranded seven-wire tendon.

Assembly of the 64 wire units, 160 ft (49 m) long, was tedious as was their installation within the ducts, but no insurmountable problems were encountered.

**Reinforcing Steel —** Furnishing and installing standard reinforcing steel bars meeting ASTM requirements was not a serious problem, despite minimum clearances within the girder. The design did not call for stirrups in the bottom flanges of the girders. In retrospect, this omission of stirrups, particularly in the main girders, appears to have been an error in judgment. While stirrups would not have prevented cracking, they would have prevented some of the spalling of the concrete that occurred in later years. Stirrups might also have facilitated repairs, which engineers subsequently considered too difficult to undertake.

**Formwork** — Despite Magnel's insistence on steel forms rather than wooden forms, to allow for the energetic external vibration required, the contractor chose to use wood. Disregarding Magnel's recommendations for the use of minimum slump concrete in conjunction with steel forms was the source of most of the problems encountered during the service life of the bridge, to be described later.

The use of wooden forms prevented the *vigorous external vibration* necessary to obtain the dense concrete, free of honeycombs and cold joints, that the low slump mix would have otherwise produced. The contractor's expected economies from using wooden forms in place of steel failed to materialize. Even the reduced level of external vibration dislocated the wooden forms, and the costly, time-consuming repairs that were needed used up the potential savings the contractor had hoped for when deciding to use wooden forms.

**Chases for Prestressing Wires** — The use of rubber core assemblies to form the chases (ducts) necessary for wire unit trajectories was ingenious and economical. However, because the contractor had selected wooden forms, internal vibrators had to be used in addition to whatever external vibration could be produced. This internal vibration, combined with insufficient bracing of the rubber cores, caused the cores in some instances to be dislocated and to deviate from the intended straight lines. Such deviations, however small, pro-

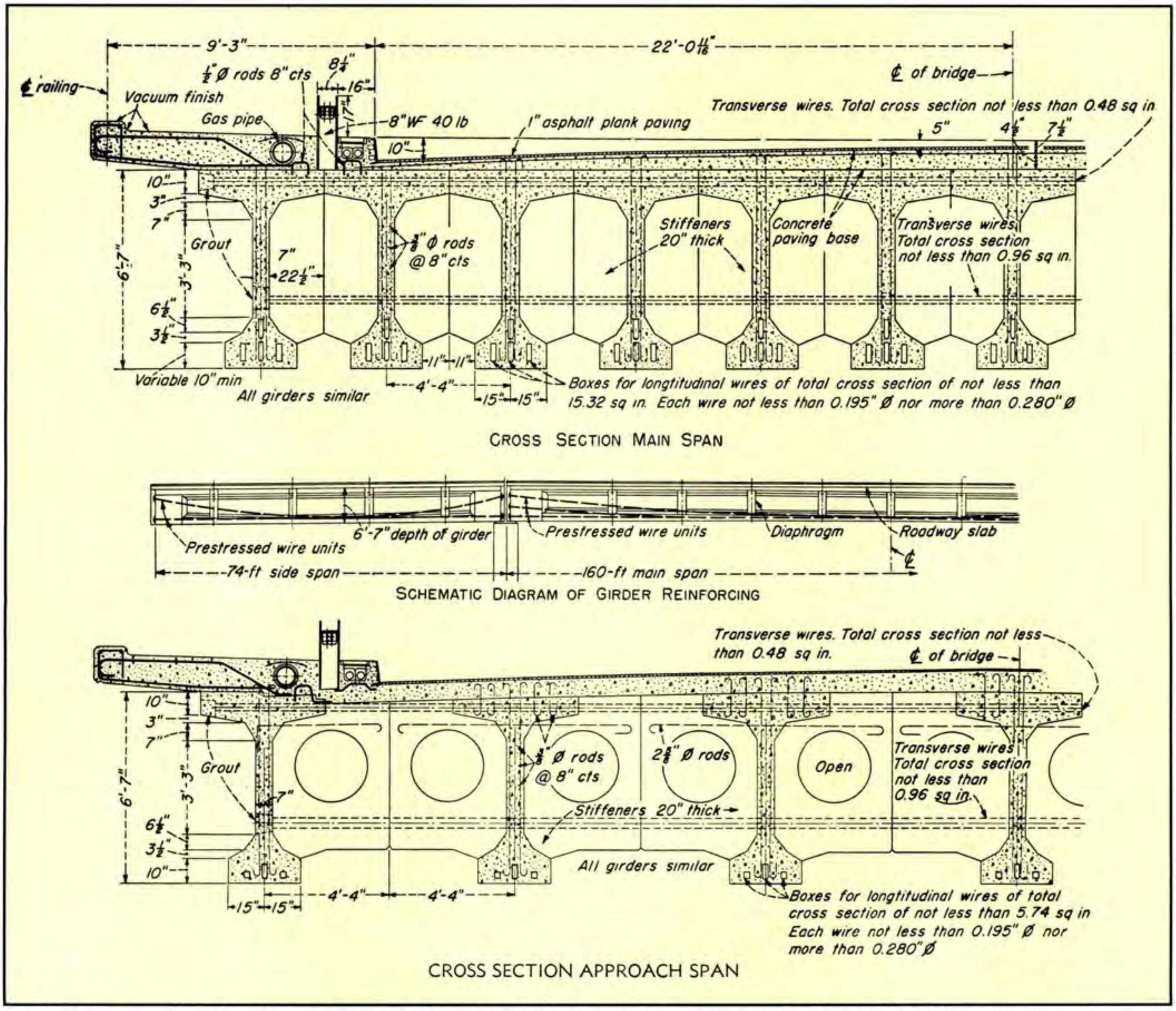


Fig. 4. Cross sections of main and approach spans of the Walnut Lane Memorial Bridge. Diaphragms for the main span were cast with the girders; for the approach spans, diaphragms were cast continuous with the deck slab (diagram is borrowed from *Engineering News-Record,* December 30, 1948).

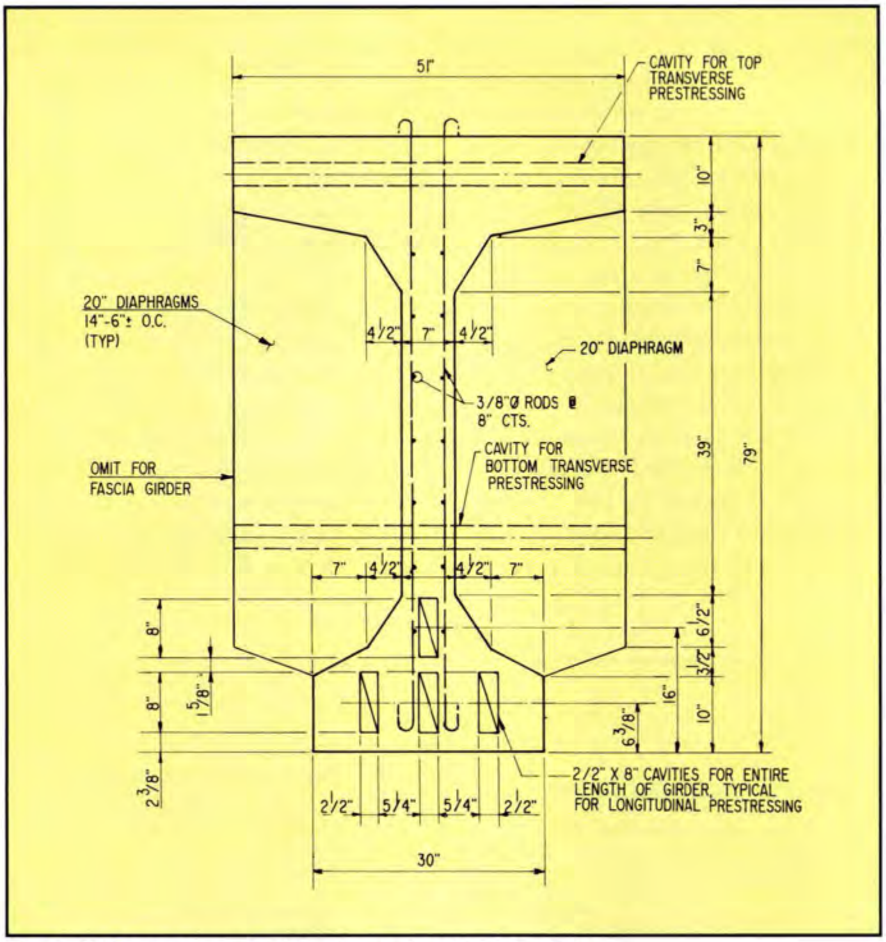


Fig. 5. Midspan cross section of typical main span girder.

duced undesirable stresses in the concrete as a result of the post-tensioning operations.

**Grouting** — To avoid material segregation and voids during the grouting required for corrosion protection of the prestressing wires, Magnel recommended the use of Colcrete, a proprietary material with particles in colloidal suspension. For reasons of his own, the contractor elected not to use this material. Instead, a standard grout mixture of water, cement and sand was introduced into the ducts by gravity flow.

It is reasonable to assume that, grouting being the next to last or even the last operation, the contractor in his haste to complete the project did not carry out the task with the care required. As a result, air pockets must have remained in the chases, despite all the precautionary measures which were supposed to have been taken.

## The Construction Team

The well-established Philadelphia general contracting firm of Henry W. Horst Company was low bidder in the amount of $750,000 for the entire bridge (1950s dollar values, of course). This firm was responsible for all substructure and foundation construction. As subcontractor, The Preload Corporation of New York was responsible for the casting and erection of the 27 prestressed concrete girders as well as all the related bridge work.

The first full size 160 ft (49 m) main span girder produced was tested to destruction. Arthur R. Anderson, who later became a founder and president of ABAM Engineers and Concrete Technology Corporation in Tacoma, Washington (Dr. Anderson also served as chairman of PCI in 1970-1971), was responsible for the instrumentation and measurements during testing of the main span girder.

## Testing the First Girder

Preload personnel cast the 160 ft (49 m) long test girder on the western approach road to the bridge, under the direct supervision of Clement Atchit from Belgium. Despite its novelty, no unusual problems were encountered. Fig. 9 shows the perfect appearance of the test girder after removal of the forms. However, prior to the post-tensioning operations, the south face of this test girder had been left exposed to the sun for several months.

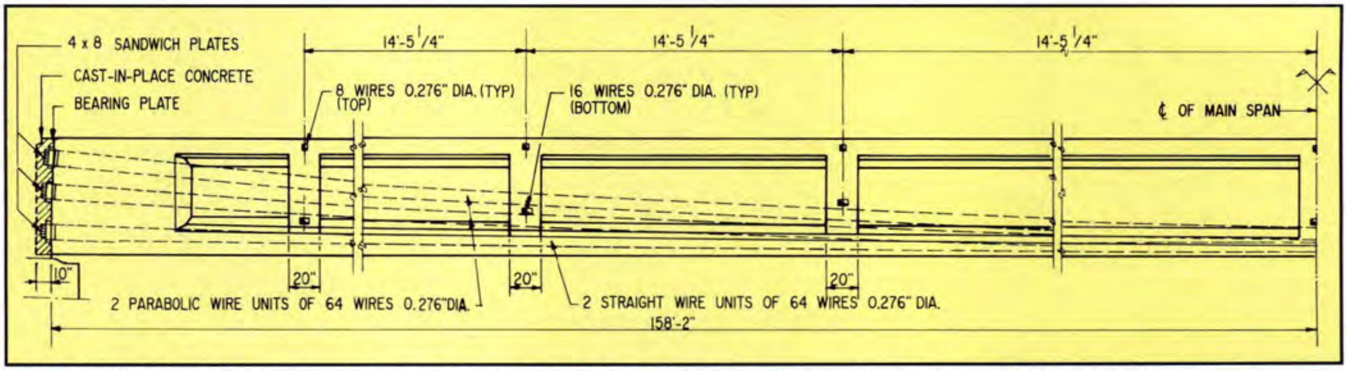


Fig. 6. Profile of longitudinal post-tensioning wire units in the main span girder of the bridge, and location of transverse prestressing wire units. Before the forms were removed, the longitudinal wires passing through the sandwich plates were prestressed and anchored with wedges.

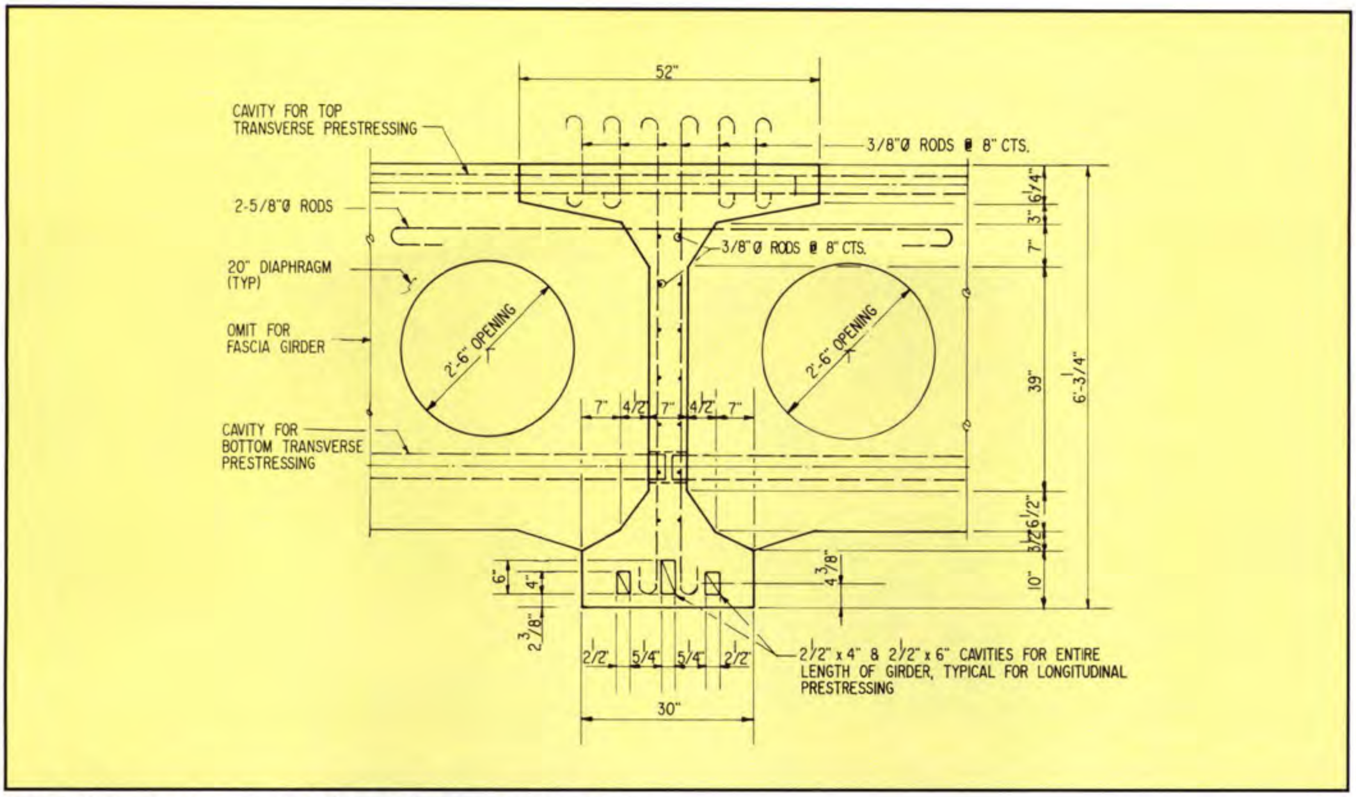

Fig. 7. Midspan cross section of typical approach span girder.

The other face was exposed toward the north, receiving little sun, and therefore remaining cooler. Consequently, the temperature differential within the girder produced more expansion along the south face than along the north face. This resulted in a bowing (sweep) of the test girder, an effect which was particularly pronounced since the test girder, being unerected, was unrestrained.

The load test of the girder was conducted in October 1949, witnessed by some 300 engineers who stood in the rain all day to witness the drama (see Ref. 3 for full details). Two longitudinal cracks appeared unexpectedly at about twice the total design load. One was on the south face of the girder at the junction of the sloped surface of the bottom flange with the vertical plane of the web. The other occurred in the soffit of the girder. Those in charge of the testing program attributed these cracks to the combined effect of horizontal deformation (sweep) of the girder before prestressing and a probable displacement of the outermost chase for the post-tensioning wires during placement of the concrete.[6,7]

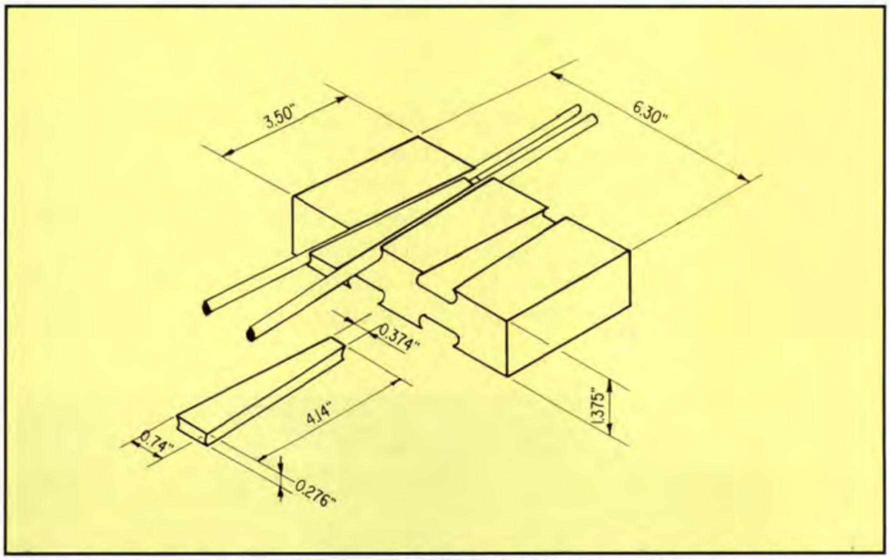

Fig. 8. Sandwich plates and wedges are the holding device in the Magnel-Blaton (Belgian) anchorage system. Exploded view shows how wires, plates and wedges are assembled in the anchorage.

Because of the bowing of the girder and the displacement of the cavity, the tendons were probably not straight. Upon increase of the external testing load, the tendons tried to straighten out during the stressing operations, and this produced a horizontal force which could have cracked the concrete. This cracking was not considered to be of any particular consequence, insofar as the ultimate test load was concerned, because the bottom flange, being in tension, had for all practical purposes become superfluous at this stage of the test.

Magnel always delighted in showing prestressed concrete resistance and flexibility by loading test beams beyond the calculated cracking load, thereby producing a crack. Upon removal of the load, the crack would close up and become invisible. Magnel

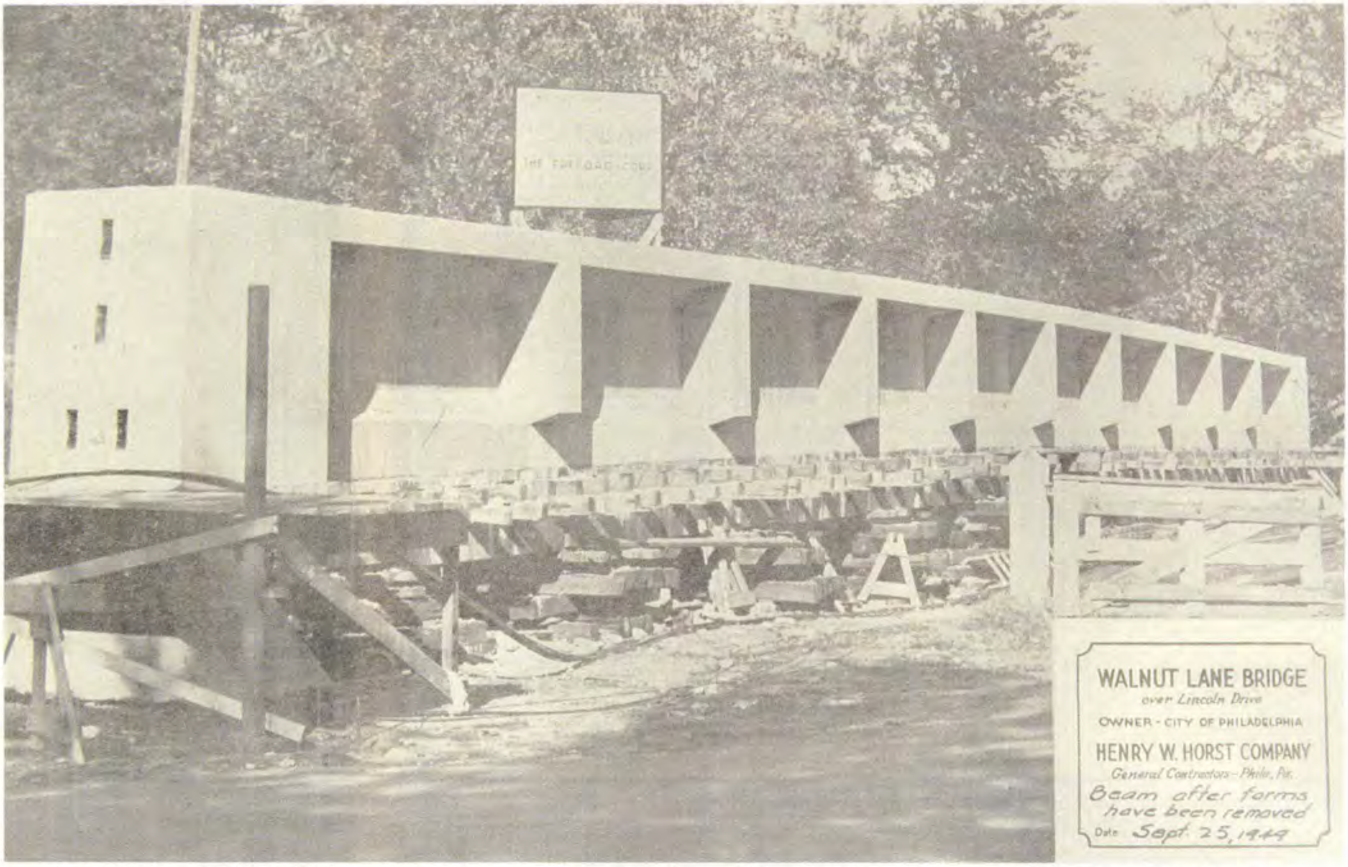


Fig. 9. Appearance of the first test girder after removal of the forms. Note that diaphragms were cast as part of the main span girders.

was denied this pleasure at the Walnut Lane Memorial Bridge because of the longitudinal cracks; he feared spalling during the removal of the loads. Structurally, such spalling would not have had adverse consequences but would have been unsightly. (At total design loads, stresses in the bottom flange are nominal.)

Since the test girder supported about 10 times the design load before total failure, after deflecting about 25 in. (635 mm), the structural strength of the girder was confirmed. Not much attention was paid to the possible significance of the sweep, the secondary stresses and the resulting longitudinal crack. No thought was given to the possible long term reduction in *durability* of the girder. In retrospect, it is now apparent that these were in fact unheeded warning signals of the events which were to come.

Fig. 10. Appearance of south side of first girder cast, after removal of forms. Note reinforcement displacement.

## The South Fascia Girder

The subcontractor's major problem throughout the project was placing concrete to achieve the early strength of 2160 psi (15 MPa) required before the prestressing operations could begin. As stated elsewhere, this could only be achieved through the use of low slump concrete and intense external vibration. To maintain the required low slump of 2 in. (50 mm) in ready mix trucks for one hour of transportation from the batching plant to the site was extremely difficult, with drivers unwilling to make the extra efforts required. It was also difficult to obtain the extra effort needed for external vibration on wooden forms.

To diminish the placing problem, windows for placing and vibrating concrete had been built into the forms at regular intervals along the girder, where the bottom flange joined the web. That the subcontractor had not mastered the concrete placing techniques required for low slump concrete was evident when the forms were removed after casting the first bridge girder, the south fascia girder (see Figs. 10 and 11). Honeycombs, cold joints in the web and bottom flange, large and small holes, discontinuities, and local displacement of the reinforcing steel and chases were found throughout the entire length of the girder. The sight was appalling.

Serious consideration was given to rejection of the girder. After evaluating the alternatives, it was decided to restore the girder and to proceed with prestressing. Since the highest concrete stresses would occur during the stressing operations, the stressing would, in effect, test the structural capabilities of the girder.

The test should have revealed any structural shortcomings that might be present in the girder. The prestressing operations were, therefore, carefully carried out and critically observed. There was no indication of potential structural distress; the origin of distress which was to come later was unexpectedly found to be elsewhere in the girder. Fig. 12 shows the rehabilitated south fascia girder erected in the bridge in 1950.

Based on a study of the concrete placement for this first girder, different concrete placing procedures were developed for the other girders. Substantially more care was exercised and the difficulties experienced with the

Fig. 11. Honeycombs in the first 160 ft (49 m) girder cast in 1949 for the Walnut Lane Memorial Bridge.

Fig. 12. Appearance of the south fascia girder after restoration and erection in 1950.

first girder were almost eliminated. The mentality of haste, of "get it done," of carelessness, which characterized the casting of the first girder, and the attitude of "we know it all" were studiously avoided. The need for high quality workmanship and concrete quality control had been learned the hard way.

### Diaphragms and Transverse Prestressing

To achieve monolithic behavior of each span, concrete diaphragms and transverse prestressing were provided. In the main span, the 20 in. (508 mm) thick diaphragms were cast integrally with the precast girder.[3] They were approximately 14 ft 6 in. (4.4 m) on centers and equal to the width of the top flange (51 in. or 1.3 m) of the girder.

After erection, the 1 in. (25 mm) gap between girders, provided as a construction tolerance, was filled with a high strength grout. Subsequently, the one eight-wire transverse tendon in the top flange and the one 16-wire transverse tendon near the bottom flange (see Fig. 4) were stressed and grouted, thereby establishing the monolithic action of the deck.

Along the girder supports, the concrete encasement of the prestressing anchorages was cast monolithically for the entire bridge width, thereby acting also as a transverse diaphragm adding to the stiffness of the deck.

A similar procedure was adopted for transverse post-tensioning of the approach spans; however, because of the girder spacing, the diaphragms were cast integrally with the concrete deck slab (see Fig. 7).

Although structural rigidity of the deck was achieved, this construction (as explained in Part 2) was to create serious problems during removal of the deck.

## FAR-REACHING INFLUENCE OF THE WALNUT LANE MEMORIAL BRIDGE

*Engineering News-Record*'s editorial, December 30, 1948, commented on the Walnut Lane Memorial Bridge: "In undertaking a modern prestressed concrete bridge, Philadelphia is serving American engineering admirably, and it may be hoped that the benefits . . . may be even greater than expected." The city engineers and others involved in the project can take pride in knowing that the benefits in a wide variety of areas were indeed immeasurably greater than anticipated, and that in participating in the design and construction of this bridge, they were instrumental in the development of the prestressed concrete industry in the United States.

### Post-Tensioned Construction

In the area of post-tensioned construction, the Walnut Lane Memorial Bridge was the incentive for the nearly immediate construction of:

- The 110 ft (34 m) span Arroyo Seco Pedestrian Overpass in Los Angeles, California. This, the first post-tensioned bridge west of the Mississippi River, was to be the forerunner of the prestressed concrete construction industry on the West Coast.
- The first 17,424 ft (5.3 km) long Sunshine Skyway trestle connecting St. Petersburg to Bradenton, Florida, marking the beginning of prestressed concrete construction in Florida and in the southern United States.
- The early warehouses in the Tulsa, Oklahoma, area.

### High Quality Concrete

The growing viability of prestressed concrete compelled the concrete construction industry to seek ways and means for improving its methods for producing high quality concrete. At the time, 2500 psi (17 MPa) concrete was common, and 5000 psi (34 MPa) was unheard of for structural concrete. Demands for higher quality concrete led to development of the concrete quality assurance program described in Part 2.

### Impact on the Steel Industry

The inspiration for the prestressed concrete industry, brought about by the Walnut Lane Memorial Bridge construction, forced the steel industry to commence a research program in Pennsylvania to produce better steels to meet the growing competition from prestressed concrete. As a consequence of the competitiveness of prestressed concrete, the steel industry developed higher strength steels and weathering steels.

### Precast, Pretensioned Concrete

Most importantly, the anticipated construction of the Walnut Lane Memorial Bridge prompted Robert Peterson and B. J. Baskin (then president and chief engineer, respectively, of Concrete Products Company of America, Pottstown, Pennsylvania) to join a Philadelphia group on their European inspection trip in June of 1949. Peterson and Baskin were originators of bridges made with precast concrete channel slabs. To overcome the 36 ft (11 m) span limitations for these precast slabs, they decided to investigate prestressed concrete, in particular the design and manufacturing process of pretensioned beams used in Europe.

As a result of their trip, in less than a year (May 1950), the first American pretensioned concrete bridge beam, 30 ft (9 m) long, 3 ft (914 mm) wide and 17 in. (432 mm) deep, was produced by assembly line procedures in Pottstown, Pennsylvania, in what was to become the first pretensioning production plant in the United States. This plant established several milestones in pretensioned work:

- The first use of strands of $^1/_4$ in. (6 mm) diameter, and soon thereafter of $^3/_8$ in. (9.5 mm) diameter, instead of single smooth wires.
- The first use of the temporary grip to hold the strand force by use of the Nicopress sleeve developed for this specific pretensioning operation by the American Steel & Wire Company, Pittsburgh, Pennsylvania.
- The first use of circular cardboard tubes to reduce the weight of box beams.

This was the origin of today's precast, pretensioned concrete industry. It is ironic that it was post-tensioning in the Walnut Lane Memorial Bridge which gave the impetus to pretensioned concrete construction in the United States, and that the same bridge has been rebuilt as a pretensioned structure 40 years later. The impact of the Walnut Lane Memorial Bridge on the American construction industry is thus crystal clear.

## SERVICE LIFE OF THE BRIDGE — FROM 1950 TO 1989

### First Appearance of Distress in the Late 1950s

At the 1958 PCI Convention in Chicago, Illinois, Zollman discussed the Walnut Lane Memorial Bridge with Noel Willis, who was then bridge engineer for the Philadelphia Department of Streets. When questioned about a 30 ft (9 m) long longitudinal crack in each of the fascia girders of the main span, Willis replied:

> . . . the cracks are not due to an inherent defect in prestressed concrete design or construction but perhaps a detail, perhaps a misassignment of load. All of the girders, for example, carried on that bridge exactly the same load according to the design calculations; while actually the fascia girders carry a heavy cantilevered sidewalk, and we think that the cracks, while they are minor, and while we are watching them very carefully, and while we are now, probably at this very moment, patching them, they are not significant. We think they reflect no stigma on the prestressed concrete industry. We think prestressing is a good technique. We will use it again when a situation warrants it. We have every confidence that we can expect the Walnut Lane Memorial Bridge to serve for the next 100 years!

What was happening at the Walnut Lane Bridge? Why were longitudinal cracks appearing in the main span fascia girders only? Why were they more pronounced in the south fascia girder than in the north fascia girder? At that time, no longitudinal cracks had appeared in the 11 *interior* main span girders. Yet the design and construction procedures of all the 13 main span girders were about the same.

There were hardly any cracks of any size to be found in the 74 ft (23 m) long approach spans. Yet, the cross sections of the approach span girders were identical to those of the main span, except for the reduced depth of the top flanges and the reduced number and size of the tendons.

Was it then reasonable to assume that the exposed location contributed to the cracking of the fascia girders? Was it possible that the concrete durability of the repaired south fascia girder had been impaired? Had moisture, seeping through the multitude of joints of the bituminous plank atop the concrete wearing surface, somehow penetrated into the curved chases, accumulated in the lower part of the parabolic curve and froze, thereby cracking the concrete near the midspan of the girder?

> *"For lack of a nail the shoe was lost; for want of a shoe the horse was lost."*
>
> **Benjamin Franklin**
> in *Poor Richard's Almanac*

In 1957, the cracks appeared to be innocuous. However, it was evident that if moisture penetration was causing the distress, the longitudinal cracks would enlarge with time and eventually extend to the massive end blocks. Since the cracks were still incipient, the City of Philadelphia decided in 1957 to apply a cement coating to the exposed areas of the girders for the sake of appearance, after recording the size, type and location of the cracks.

Fig. 13. Longitudinal crack in bottom flange of south fascia girder (1968).

Fig. 14. Close-up view of crack shown in Fig. 13.

## In-Depth Investigations and Corrective Measures in the 1960s

**State Monitoring of the Bridge —** For several years after the 1957 events, the behavior of the bridge girders was monitored by the Pennsylvania Department of Transportation (PennDOT) maintenance personnel. By 1968, the structural integrity of the south fascia girder had become a matter of concern because of the location of one of the longitudinal cracks on the outer sloping face of the bottom flange.

This particular crack (see Figs. 13 and 14) now extended the full length of the center span, varying in width from $^1/_4$ to $^1/_2$ in. (6 to 13 mm). In places, it was $2^1/_2$ in. (64 mm) deep. The average distance from the crack to the vertical face of the flange was about 5 in. (127 mm). The crack in question appeared to extend downward into the area of the prestressing tendon. Furthermore, the other longitudinal crack, which was located in the soffit of the bottom flange, appeared to extend upward into the same area of the prestressing tendon.

Rainwater flowing down the vertical face of the web of the girder could penetrate into the upper crack and, upon freezing, exert an outward force. With alternating freezing and thawing cycles over the years, the outer portion of the concrete flange was likely to spall off the girder sooner or later, especially since there was no conventional reinforcing steel, such as stirrups, in the bottom flange.

In other words, there was nothing to hold the outer chunk of concrete to the main body of the girder except the bond between it and the grout injected into the wire cavity. It was imperative that this bond — or whatever remained of it — be preserved and its effectiveness maintained to prevent spalling off of the outer portion of the flange.

**Repair Contract —** Even though the Walnut Lane Memorial Bridge, and maintenance responsibility for it, were transferred from the City of Philadelphia to PennDOT in 1963, the city maintained an interest in the historic structure and the safety of the traveling public. In 1968, the city retained the consulting firm of Zollman Associates, Inc., to study the flange crack problem in depth and prepare recommendations for repair.

By the fall of 1968, PennDOT was very concerned that, unless immediate action was taken, the exterior bottom flange of the south fascia girder could spall during the winter with potentially fatal consequences to the traffic on busy Lincoln Drive (below the bridge). Therefore, PennDOT entered into an emergency contract with Kaufman Construction Company, Inc., of Philadelphia, an engineering-oriented heavy construction contractor. The contract signed October 21, 1968, provided for scaffolding, inspection, testing and repair as directed.

Scaffolding was erected under the full length and width of the main span while maintaining four lanes of unimpeded traffic on Lincoln Drive. The cementitious coating over a polyvinyl acetate binder coat that had been applied in 1957 was removed by abrasive blasting. A tabulation of all visually observable cracks was made. Though by now there were some cracks in all the main span bottom flanges, the two fascia girders had the most and, by far, the most serious cracks. The south fascia girder had a continuous longitudinal crack up to 0.7 in. (18 mm) wide on the outside bottom flange for its entire length. The flange was cracked from its top surface to the exterior tendon chase and also from that chase to the bottom of the flange. This was truly a precarious condition, considering that the girder had no reinforcing steel stirrups in this flange.

**The Perennial Water Seepage Problem —** As with most concrete structures that develop problems, water was identified as causing the cracks. It was seeping through the cracks. It was suspected that the water could be coming from the deteriorated expansion joints immediately over the end anchorages of the post-tensioning tendons, specifically in the vicinity of the vertical joint between the end face of the girder proper and the 10 in. (254 mm) thick concrete cast over the anchorages to protect them from corrosion.

To confirm this suspicion, and because of the winter weather, alcohol containing green dye was poured into an area chipped out around the anchorages. Some of this alcohol was later observed seeping out of the joint at the girder bottom flange.

A review of the original construction procedures indicates that the grout used to fill the tendon chases was poured into the girders from their two ends, *without pressure and without an expansive additive.* Apparently, this left voids in the chases, and water flowed down the joint into the chases. Subsequent freezing and thawing cycles produced longitudinal cracks observed along the line of the prestressing tendons.

**Epoxy Injection —** The contractor was directed to inject epoxy into all visible cracks and to try to fill all voids in the chases. This first prestressed concrete bridge in the United States became the site of the first major pressure injection epoxy repair project in the eastern United States, using the structural concrete bonding process licensed by Structa-Bond, Inc., Philadelphia, Pennsylvania, for the Adhesive Engineering Company of San Carlos, California.

The system, unique at the time, combined two epoxy components in a special mixing head just prior to injection into the crack. Since the components were stored in separate reservoirs, which were refilled without interrupting the flow of epoxy, it was possible to inject 16.5 gal. (62.5 l) of epoxy in one continuous 12 hour pumping operation for the entire 152 ft (46 m) of the south fascia crack. A low-exothermic-action epoxy was used in all cracks and voids more than $^1/_8$ in. (3 mm) wide. A less viscous epoxy was used in the narrower cracks.

Subsequent core drilling of these epoxied cracks indicated penetration success beyond expectations. Hairline cracks, almost invisible to the naked eye, were filled with epoxy. This epoxy injection started January 20, 1969, after enclosing the entire work platform with polyethylene sheeting. Propane heaters warmed the enclosed area to 60° F (16° C). Epoxy crack and void injection continued throughout the winter, coming to a halt about April 10, 1969, even though the work was not completed. Day-by-day job records disclosed that the owner discontinued repair work due to monetary constraints.

**Other Repairs in the 1960s** — Other work performed during this 1968-69 repair project included removal of the bituminous deck planking, which had contributed to the deterioration of the cast-in-place concrete wearing surface (Fig. 15) because the planking was not impervious and had innumerable joints. At the abutment joint, the low point of the roadway profile (Fig 16), water and melted snow collected.

Repair work also included expansion dam seal replacement, deck scaling and miscellaneous work on paving, railings, curbs and bearings. The project was under the supervision of District 6, Pennsylvania Department of Transportation. Paul Peterson was district bridge engineer, Henry Garfield, project engineer, and Sergei Borichevsky, resident engineer.

## Reappearance of Distress in the 1970s

After repairs previously described, the bridge was inspected regularly by

Fig. 15. Deterioration of concrete wearing surface in low point area of roadway, near joint.

Fig. 16. Condition of abutment joint at roadway's low point, where water and snow collected. Note repair work.


Fig. 17. Continued growth of major cracks noted in the 1970s.


Fig. 18. New hairline diagonal crack appeared in end block of fascia girder.

PennDOT district personnel. In 1973, new cracks were observed in both fascia girders, and girder monitoring was scheduled three times a year. Continued growth of major cracks (Fig. 17) was noted along with new hairline diagonal cracking on one end of the south fascia girder (Fig. 18).

In 1978, Structa-Bond, Inc., at the request of the PennDOT Bridge Unit, injected several cracks on the bottom flange of the fascia girder on the northwest corner and of the fascia and first interior girders on the southwest corner, as well as a crack about midspan of the north fascia girder. John Miller and Jim Mastrilli of the PennDOT Bridge Unit supervised the operation, carried out at no charge to PennDOT. In the years following this minor repair effort, additional crack progression was observed primarily in the main span fascia girders.

### The 1980s: Decision to Replace the Superstructure

Sometime in 1981, after further years of routine inspection and continuing deterioration, PennDOT, owner of the Walnut Lane Memorial Bridge, began to consider replacing the entire superstructure of this historic bridge even though only the main span fascia girders were of major concern. PennDOT had been unable to find a repair scheme that would preserve the authenticity of the existing bridge structure. The stainless steel boot suggested to contain the anticipated spalling of the bottom flange of the girders in question did not meet environmental criteria.

Concurrently, it was hoped that the Portland Cement Association and other organizations, such as the American Concrete Institute, would be interested enough to seek funds for a well-considered study, testing and engineering evaluation program by experienced personnel who would make appropriate recommendations to remedy the situation while still maintaining authenticity of the bridge.

Professor F. G. Riessauw of the Magnel Laboratories of Ghent University, Belgium, who had considerable experience and familiarity with distress and repair of early European prestressed concrete bridges, was willing to be available as a consultant. Unfortunately, funds were not forthcoming for a Walnut Lane Memorial Bridge testing and evaluation program.

Finally, it was resolved by the PCI Board of Directors on June 23, 1983, to issue a statement of support for the PennDOT program to replace the Walnut Lane Memorial Bridge superstructure with a new and modern prestressed concrete bridge. The PCI statement further indicated "that the Prestressed Concrete Institute views the positive aspects of the bridge's significance to the industry and service to society and so promotes it, while supporting its replacement."

Later in 1983, PennDOT decided to replace the superstructure. A contract was awarded to A. G. Lichtenstein & Associates of Langhorne, Pennsylvania, in 1986, for the design of a new superstructure (described in Part 2) to be constructed of prestressed concrete girders "of similar size and shape" to the existing prestressed concrete girders.

In June of 1987, during PennDOT's periodic monitoring, the north fascia girder was found to have a 1 in. (25 mm) wide crack extending 25 ft (8 m) at midspan along the line of the exterior prestressing tendon chase. The loose concrete was removed by state forces to preclude its falling on traffic and the exposed areas were painted to preserve the aesthetics of the bridge.

Bids for the superstructure replacement were finally taken in June of 1989.

## POSSIBLE CAUSES OF THE CRACKING PROBLEM

With the benefit of hindsight, the following causes of the cracking problem can be identified:

**1.** Water entering the girders at the anchorages was a basic problem compounded by the many voids left in the chases after the completion of the grouting operation. However, this water leakage was not the only factor because only the fascia girders were in severe cracking distress although all girders were grouted using the same method (gravity) and material (water, sand and cement). Fig. 19 clearly shows the joint between the girder's end block face and the cast-in-place concrete encasement.

**2.** Incorporation of the steel channel expansion dam into the end block introduced impact forces from traffic which caused the girder end block face to separate from the 10 in. (254 mm) wide cast-in-place concrete, allowing water to travel down the joint and into the tendon chases.

**3.** The most severe cracking was in the exposed portion of the bottom flange of both fascias and nowhere else. Why? Water infiltration from the expansion dam and through the prestressed cable anchorages should have been of the same order of magnitude in all bottom flange chases. The differential in temperature on the two sides of each fascia (as evidenced by the bow in the test girder) could have compounded the cracking in the bottom flange along the vertical plane of weakness at the edge of the outer chase. A hairline crack here would have been a natural entrance point for rain running down the exposed face of the fascia girder, and could account for the more severe cracking of the two fascias. Had the grouting of the chases been perfect, and had no water infiltrated from the expansion dams, the temperature differential alone might have caused a longitudinal hairline crack to appear along the tendon chase.

**4.** Stirrups would certainly have been a safety factor as hairline cracks would have remained hairline cracks. An opportunity for repair would have been available. Hindsight suggests that their omission was an error in design judgment.

**5.** We must learn to avoid the mistakes of the past. The lack of stirrups was a "state-of-the-art" problem, readily overcome in new construction. Stirrups have been provided in all bridge girders constructed since the Walnut Lane Memorial Bridge as well as in all the standard AASHTO/PCI type bridge girders. The infiltration of water at the end anchorages resulted from misuse of grouting material and grouting procedures. Unsuitable detailing in the vicinity of the anchorages must be prevented.

Fig. 19. Opened joint at contact between girder end block and cast-in-place concrete encasement protection over tendon anchorages.

**6.** The differential temperature on the two faces of a fascia girder, particularly on very long and deep girders, is a somewhat different problem. The stresses induced are not conducive to crack-free flanges. Stirrup reinforcement will help reduce the problems resulting from temperature differential stresses, but will not eliminate them totally. How can temperature differential be reduced? One possible way would be to proportion the sidewalk cantilever to the girder depth — the greater the girder depth the greater the sidewalk cantilever. This would give some shade to the exposed face, reducing the temperature differential. A secondary benefit would be to minimize the fascia exposure to inclement weather.

**7.** Although cracks in the webs of the main span girders were not a major problem, some webs of the main span interior girders did have cracks with water marks. These cracks extended into the prestressing tendon chases. These girders also had some form tie holes from the original construction that had never been patched. It is evident that some of these tie holes acted as weep holes. It is most interesting that the diary of Borichevsky, PennDOT's project engineer, records the fact that *no web cracks were found in areas where unpatched tie holes remained, as they permitted infiltrated water to drain.* It also appears that bridge engineers involved with the repair considered drilling weep holes into the bottom chases so as to drain the three main chases. However, fear of damaging the prestressing wires caused abandonment of this meritorious thought.

## CONCLUSIONS

Should we conclude that the water issue, the primary cause of most concrete structure problems anyway, must be addressed during the design, construction and maintenance phases of all concrete structures? Absolutely. Strategically placed weep holes along each prestressing chase possibly could have prevented some of the cracking and extended the useful life of the Walnut Lane Memorial Bridge despite unanticipated problems associated with the lack of stirrups, improper chase grouting, and temperature differentials on the fascias.

Finally, were the design/construction blunders — there were, indeed, blunders — later compounded with the then prevailing philosophy and orientation toward new construction, rather than toward correction and repair? There appears to have been little or no interest, nor incentive, for finding either engineering solutions to the problems, or the necessary funding to complete the tasks begun in repair or restoration.

Indeed, "For lack of a nail the shoe was lost; for want of a shoe the horse was lost." And for want of a horse the rider was lost.

**Note:** Part 2, "Demolition and Rebuilding of the Superstructure," with the carefully developed demolition work plan and the implementation thereof, will appear in the next issue of the PCI JOURNAL.

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