SOME CONSIDERATIONS IN WIDENING AND REHABILITATION OF BRIDGES

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Bridge rehabilitation, whether required for repairs, strengthening, or widening, requires an insight into unique structural problems. These problems are compounded if modifications must be accomplished while maintaining traffic and if modifications alter the structural characteristics of the existing structure. This paper deals with solutions employed on two widening and rehabilitation projects. Widening the Hackensack River Bridge on the New Jersey Turnpike required integrating the superstructure with the existing superstructure. Techniques of jacking the widening main members against the existing system, inducing compatible cambers and ensuring proper load distribution, are described. This paper describes foundation additions to main river piers which were integrated by temporarily leaving a gap between the foundations. This permitted elastic shortening of new piles under pier dead load, and prevented overloading the existing piles. A classic example of fatigue failure is illustrated. Welding of fills caused a geometrical notch at the toe of welds, producing a crack through the main members of brackets and floorbeams. Widening the I-83 bridge over the Susquehanna River in Harrisburg required an inspection which revealed rivet failures at bracket tie plates. These failures appear to correlate with findings by researchers at Lehigh University (1). Unfastening the bracket tie plates from the main girders is discussed as well as other design considerations resulting from unfastening tie plates near the girder supports.

Bridge rehabilitation, whether required for repairs, strengthening, or widening as part of a safety update program, provides an insight into unique structural problems---some quite unexpected. These problems are often compounded if modifications must be accomplished under traffic and more particularly if such modifications alter the characteristics of the existing structure. Quite apart from the ricochet effect of details when modifications are made to a member or members, there is much insight to be gained about how the various structures and their components actually function. Such matters as fatigue, corrosion, and joint details relate to an understanding of actual structural action. This paper addresses several design and construction features in the widening and rehabilitation of some conventional bridge structures.

In the widening or rehabilitation of a structure, certain obvious constraints will often dictate the overall plan, design scheme and construction methods. Alignment may be predicated on right of way, physical features or proximity of interchanges. The volume and type of traffic will materially affect the modification should it be necessary to maintain traffic on the structure. Run-arounds, detours, or temporary structures permit modifications without the inconvenience and hazards of traffic, but only under rare and fortunate circumstances are these devices found both feasible and cost effective.

Widening the Hackensack River Bridge

Widening the Hackensack River Bridge carrying the New Jersey Turnpike is an example of maintaining the center-line alignment and widening the structure symmetrically. See Figures 1 & 2.

Figure 1.

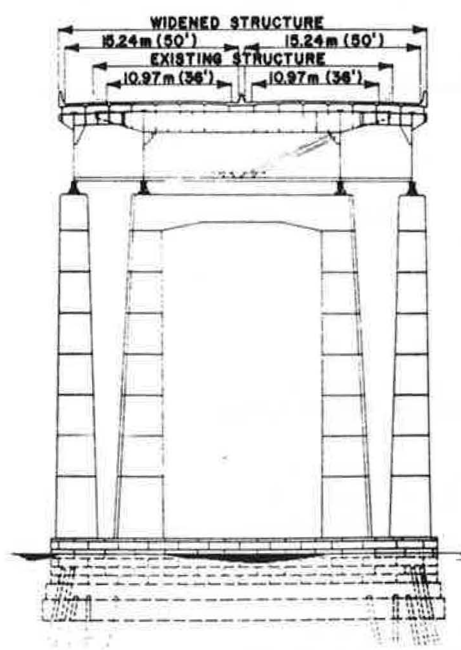


Figure 2.

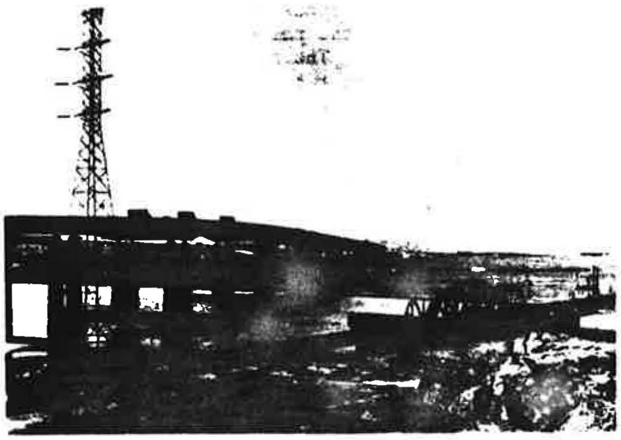
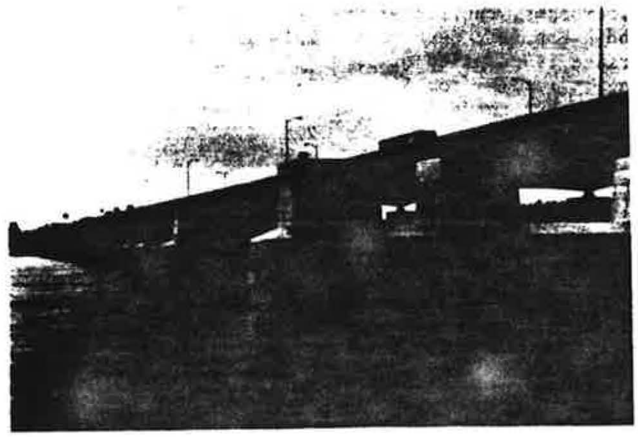


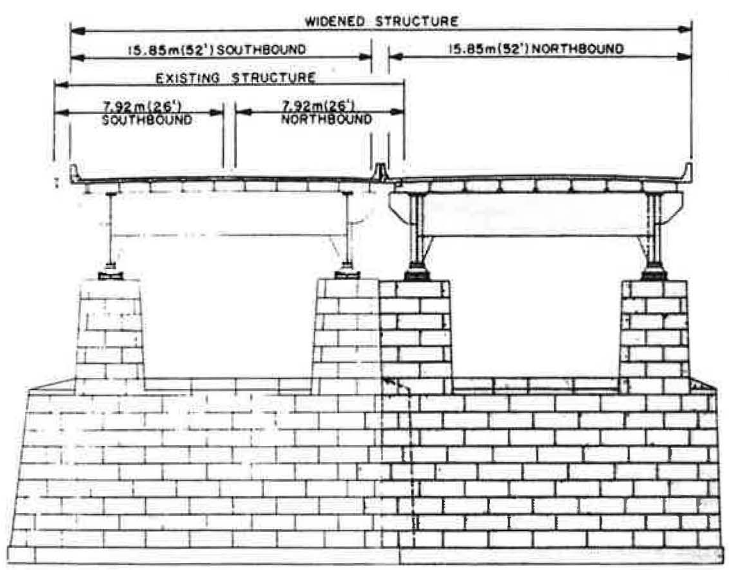
Figure 4.



Widening the John Harris Bridge, which carries I-83 over the Susquehanna River in Harrisburg, PA, is an example of a realignment and widening all on one side. See Figures 3 & 4. These two widening schemes and their associated structural design considerations will be discussed.

The New Jersey Turnpike Authority's General Engineering Consultant (2) studied several schemes for the proposed symmetrical widening before selecting the adopted scheme. This scheme proposed new pier shafts constructed adjacent to the existing piers with new longitudinal steel welded girders paralleling the existing girders. Floorbeams between the new and existing girders were to be constructed by splicing a segment onto modified existing cantilever brackets. New cantilever brackets were proposed on the outside of the new main girders. These floorbeams and brackets would then support the additional stringers and extended concrete deck and barrier parapet. All new steel members were to be of welded construction using ASTM 588 weathering steel with high strength bolted field connections.

Figure 3.



Given the structural scheme to be employed and the requirements for maintaining traffic, the superstructure design called for construction in two phases. The first phase consisted of modifying the median. The added width of roadway would later be very important when undertaking the shoulder widening. Space for temporary barriers and for the two lanes of traffic were at a premium. Moreover, of considerable importance was the lighting placed along the median when the outside lights were removed for the shoulder widening. The median lighting consisted of double bracket poles mounted at deck level and within the width of the median.

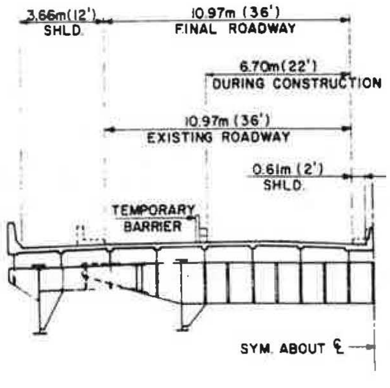
Structure Integration

For the shoulder widening, the integration of the new superstructure elements with the existing bridge required construction procedures that would insure the desired load distribution to both the new and existing main members.

The Hackensack River Bridge, constructed in 1951, is an all-riveted plate girder deck type bridge with floorbeams, cantilever brackets, a stringer system and a concrete deck slab. This 1707-m (5600-ft) bridge is composed of 35 simple approach spans varying in length from 32 m (105') to 52 m (170') and three continuous spans over the river measuring 68.6 m - 114.3 m - 68.6 m (225'-375'-225'). At the time of construction, this was a record main span for a plate girder bridge. The concrete deck provided a 11-m (36-ft) roadway curb to curb for each direction, thus accommodating two directional lanes of traffic with 3.66-m (12-ft) shoulder.

In 1956, the northern portion of the turnpike was converted to three lanes in each direction. The bridge was also converted to three lanes on the 11-m (36-ft) deck. Obviously, as traffic increased, there was no provision whatsoever for breakdowns, and the potential for a catastrophe was ever-present.

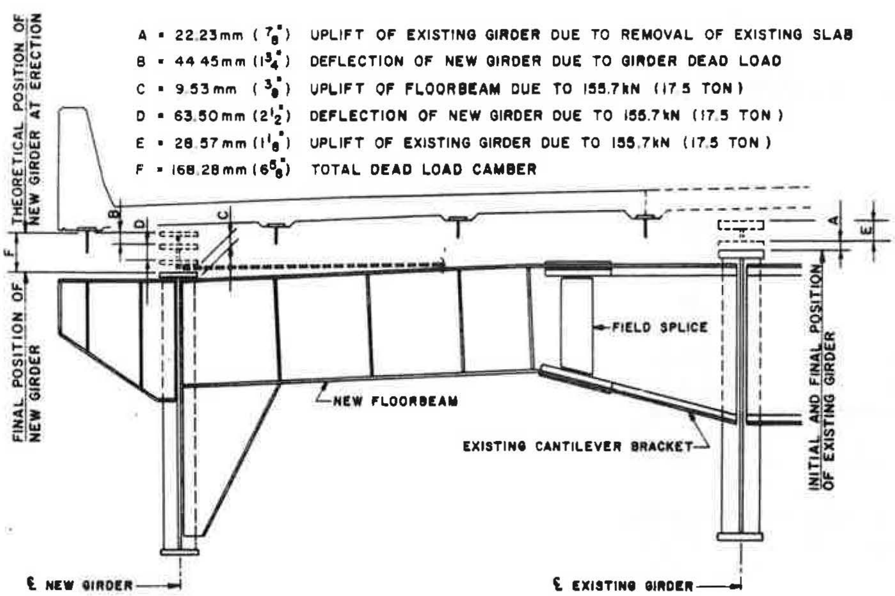
Figure 5.



The 1971 widening program focused on providing a 3.66-m (12-ft) shoulder on the right and a 0.61-m (2-ft) additional space adjacent to the high speed lane by replacing the raised median with a New Jersey-type barrier. The primary constraint for construction was that two lanes of traffic in each direction were to be maintained at all times. (See Figure 5). Alternates A & B were proposed in the contract plans for integrating the new members with the existing structural system.

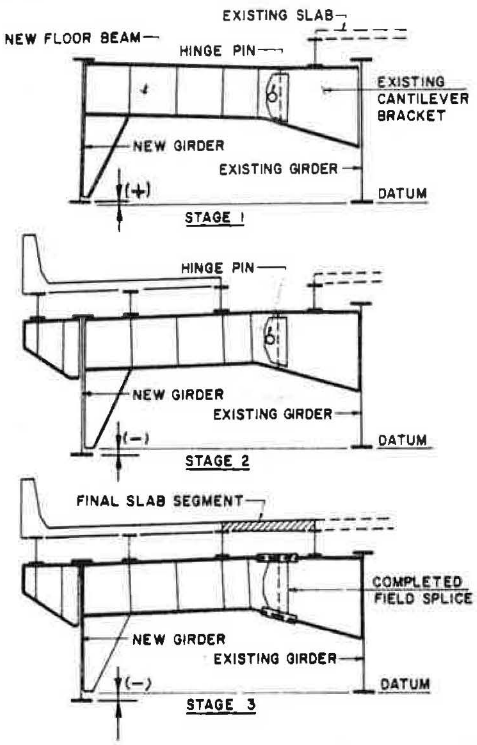
Under Alternate A, the plans called for preloading the new main girder by jacking against the new floorbeam to remove the dead load camber of the girder. The procedure called for a full strength splice between the new segment of floorbeam and the modified cantilever bracket. The floorbeam was to be erected in its final position with slotted guide holes for a temporary connection to the main girder. This temporary connection provided stability to the girder during erection and jacking. Once the girder was erected, the erector had merely to jack until the mating holes were in alignment. Figure 6 illustrates the magnitude of displacements at the center of the middle span of the three continuous spans. In this figure, (A) is the upward deflection of the existing main girder upon removal of steel and concrete beyond the adjacent stringer; (B) is the deflection of the new steel due to its dead load; (C) is the upward deflection of the floorbeam due to the jacking force [155688 N (17.5 tons);] (D) is the deflection of the new girder due to the jacking force; (E) is the upward deflection of the existing girder due to the jacking force. The summation of these equals the total dead load camber (F) of the new girder without increasing the loading on the existing girder.

Figure 6.



Under Alternate B, the plans called for placing a temporary pin at the splice between the new floorbeam segment and the modified existing bracket. (See Figure 7). In stage 1, the new girder and floorbeam are erected and the temporary pin connection is made. In the second stage, the stringers are erected and the deck slab and parapet are placed on the three outboard stringers, leaving a gap in the slab between the third and fourth stringers. This distributes the appropriate loading to the two girders. In stage three, the full section splice is completed and the last segment of slab is placed.

Figure 7.

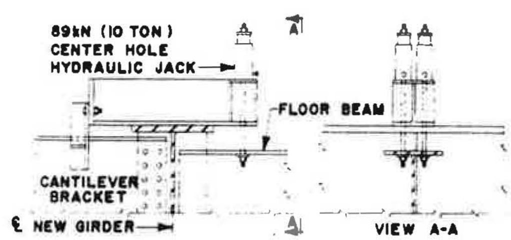


Alternate A has the advantage of having each trade complete its work at one time. In Alternate B, the ironworkers are interrupted and must return to complete the splice, erect diaphragms between the third and fourth stringers and erect the lateral bracing. Similarly, the concrete crew must return to form and place the last segment of slab.

The contract documents required the contractors to bid both alternates. The successful bidder bid Alternate A for less than Alternate B; thus Alternate A was employed. The contractor used a jacking set-up as shown on Figures 8 & 9. A simple yoke arrangement could have also been employed. This method of integrating a new member with an existing system, where proper distribution of load is essential, is not only simple, but direct and inexpensive.

Figure 8.

Figure 9.



<u>Fatigue Failures on the Hackensack Bridge</u>

During the initial stages of construction, cracks were discovered in the top flange of the cantilever brackets. The cracks started from the toe of a continuous fillet weld between a fill plate and the flange angle, with the crack propagating through the outstanding leg of the angle. These cracks were, we believe, just beginning to occur. It was considered unlikely that they would have been missed in a recent bridge inspection or by the painters who had also recently painted the structure. (See Figures 10 & 11).

The cause of these cracks became readily apparent when it was discovered that for several of the approach spans, the fill plate between the flange and tie plate had been inadvertently welded for its full length along the sides by an 8-mm (5/16-inch) fillet weld instead of by intermittent welds as was intended. At other locations where the intermittent welds had been used, these had failed and the stress transfer from flange to tie plate was through the rivets as intended.

At the locations where the continuous weld was employed, the welds were probably carrying the total load. This was the tension flange and the stresses were not excessive, but the stress ranges induced in the welds were obviously high enough to initiate fatigue failure. Where failures had occurred, these were repaired with splice angles (Figure 12). At all other locations, the weld was removed and the material ground smooth. The

Figure 10.

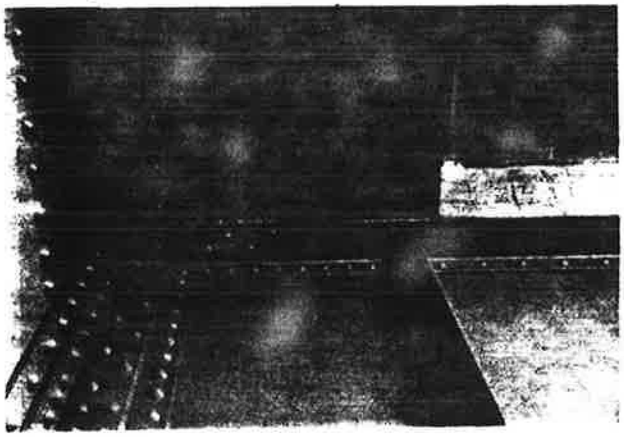


Figure 11.

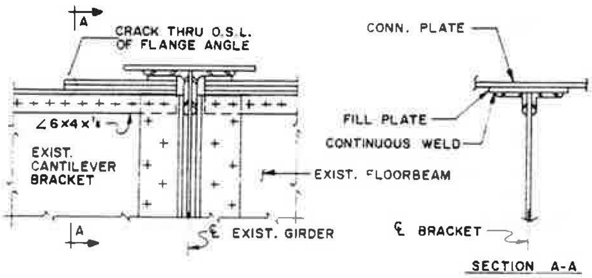
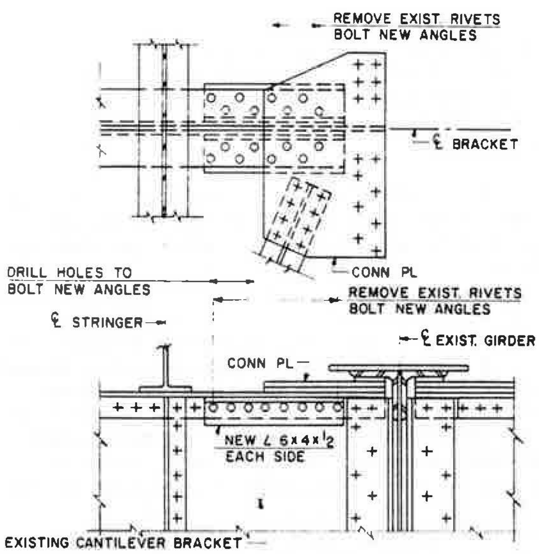


Figure 12.

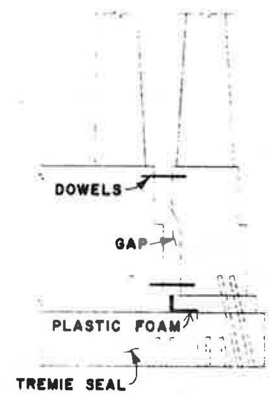


conversion of the cantilever bracket to a floorbeam materially reduced the tensile stresses at this location and no further remedies were considered.

Foundations for the Hackensack Bridge

Generally, all new pier shafts and foundations were constructed separately from the existing piers and foundations except for the river pier foundations. Because of space limitations and the desirability of a continuous footing, the new river pier foundations were integrated with the existing foundations. The existing pier was founded on 14 HP89 piles 15.25-m (50-ft) to 30.5-m (100-ft) long driven to rock. To preclude overloading the existing piles, the new foundations were constructed free of the existing footing, leaving a gap between the stems until the elastic shortening of the new piles induced by the DL of the new pier had occurred. (See Figures 13 & 14).

Figure 13.



The bottom and face of the new footing where it overlapped the existing tremie seal was placed on a plastic foam to prevent a load transfer. Dowels were placed between the two footings; however, these would not offer much resistance in bending but were required for the final pier integration. When the pier shaft was completed, the gap was filled with concrete, forming a composite foundation to sustain the superstructure dead and live loads.

Figure 15 shows the completed widening.

John Harris Bridge (I-83) over the Susquehanna River

The Safety Update for the John Harris Memorial Bridge carrying Interstate 83 over the Susquehanna River at Harrisburg, Pennsylvania will convert the existing four-lane two-direction crossing to a three-lane plus shoulder southbound crossing. An all-new separated superstructure, roughly a twin to the existing one, will accommodate northbound traffic lanes and shoulders. See Figure 3.

Although this method of widening does not normally present many complex structural problems, there are features worthy of note in this type of structure.

The existing John Harris Memorial Bridge was constructed in 1959. It includes an all-riveted deck plate girder portion of 19 spans. Lengths are

Figure 14.

Figure 15.

1 @ 43.2 m, 17 @ 51.82 m, 1 @ 45.3 m (1 @ 141'-8", 17 @ 170'-0", 1 @ 148'-7") for a total length of 969.4 m (3,180' - 2 3/4"). Of these 19 spans, four of them include hangers at a one-sixth point. This permits the advantages of continuity with a reasonable limit on the expansion dam movements.

Steel in the girders is High Strength Low Alloy steel with a minimum yield point of 345 MPa (50 ksi). Steel in the floorbeams, stringers, and bracing is carbon steel with a minimum yield point of 228 MPa (33 ksi).

The main bridge has two girders spaced 12.8 m (42 feet) apart. The girders are 2440 m (8 feet) deep at the center of each 51.82 m (170-foot) span. A circular arc for the bottom flange results in a depth of 3660 mm (12 feet) at each intermediate pier. Girder flanges are 203 x 203-mm (8 x 8-in) angles with multiple covers 508 mm (20 inches) wide.

In the typical region of the main bridge, there are 10 stringers spaced 1700 mm (5'-7") apart, except 1625 mm (5'-4") for the center space. The first interior roadway stringers are located only 480 mm (1'-7") inboard from the girders. All the stringers are rolled shapes---W21x68 sections except for W18x50 fascias. Stringer splices are located at floorbeams.

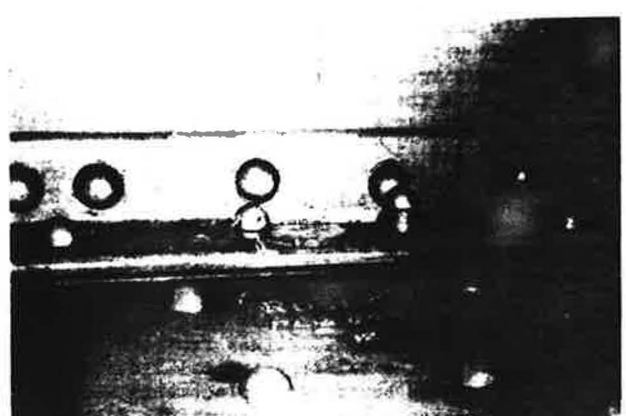
The typical floorbeam is riveted member with a web plate 1725 mm (68 inches) deep at the bridge centerline. Flanges are 152 x 152-mm (6 x 6-in) angles with 355-mm (14-inch) cover plates. The floorbeams at all the intermediate piers have 2340-mm (92-inch) webs, 152 x 152-mm (6 x 6-in) flange angles and 355-mm (14-inch) cover plates. All floorbeams have kneebraces to the bottom flanges of the girders.

In the typical section of the main bridge, floorbeams are spaced 8.64 m (28'-4") apart. Each has cantilever brackets on each side of the bridge 3125 mm (10'-3") long. These brackets support exterior roadway stringers and fascia stringers. A strap-plate over the top of the girder---and attached to the girder---transmits top flange tension from the bracket to the floorbeam.

The typical tension strap is cut from a steel plate 13 mm (1/2 inch) by 406 mm (16") by 1725 mm (5'-8"). The connection to the top flange of the bracket uses 14 rivets; to the girder, 8 rivets; to the floorbeam, 14 rivets. Four of the latter group also pass through the bottom flange of the first interior roadway stringer.

Prior to designing the widening details for the existing bridge, an inspection of the structural metalwork was conducted. It revealed a large number of broken and loose rivets in the connection of the first stringer inboard from the girder where the stringer is attached to the strap plate for the floorbeam out-rigger bracket and to the floorbeam top flange. The failed rivets were most often found at the floorbeams directly over the piers and in diminishing numbers the farther from a pier. See Figure 16.

Figure 16.



At the time of our inspection, we had been informed that several bridges in Pennsylvania had experienced floorbeam bracket tension strap plate cracking. We searched diligently for evidence of similar cracks, but found none. Our inspectors did find a number of defective rivets---predominantly in the stringer-to-strap-plate connection. The superstructure contains 117 floorbeams, so there are 234 strap plates and 936 rivets connecting stringers to strap plates. Of these, 124 were found loose or altogether missing. The failure rate---in excess of 13 percent---demanded an explanation and possibly remedial action.

The location of defective rivets was marked in the field on a set of 48 inspection sheets. A compilation revealed that the incidence of defective rivets favored the floorbeams directly over the piers, where approximately 40 percent of the defective rivets were found. Another 40 percent were found at the floorbeams immediately adjacent to the piers.

Severe rusting was observed at 29 locations on the line of the edge of the first interior roadway stringer, indicating relative motion with respect to the supporting strap plate. At 59 other such locations, light rust was observed. This makes a total of 88 out of 234 such locations, nearly 38 percent, at which evidence of distress existed. Floorbeams directly over piers had 32 percent of the rust locations. Another 40 percent were at floorbeams immediately adjacent to the piers.

Analytical and field studies of tie-plate stresses were conducted at about the time of these inspection findings. (3) Those studies resulted in a recommendation that designers should avoid connecting tie-plates to the girders, in bridges of this type. Our office, with the approval of PennDOT and FHWA, adopted that policy both for the new portion of the I-83 bridge and for modifications to the existing bridge.

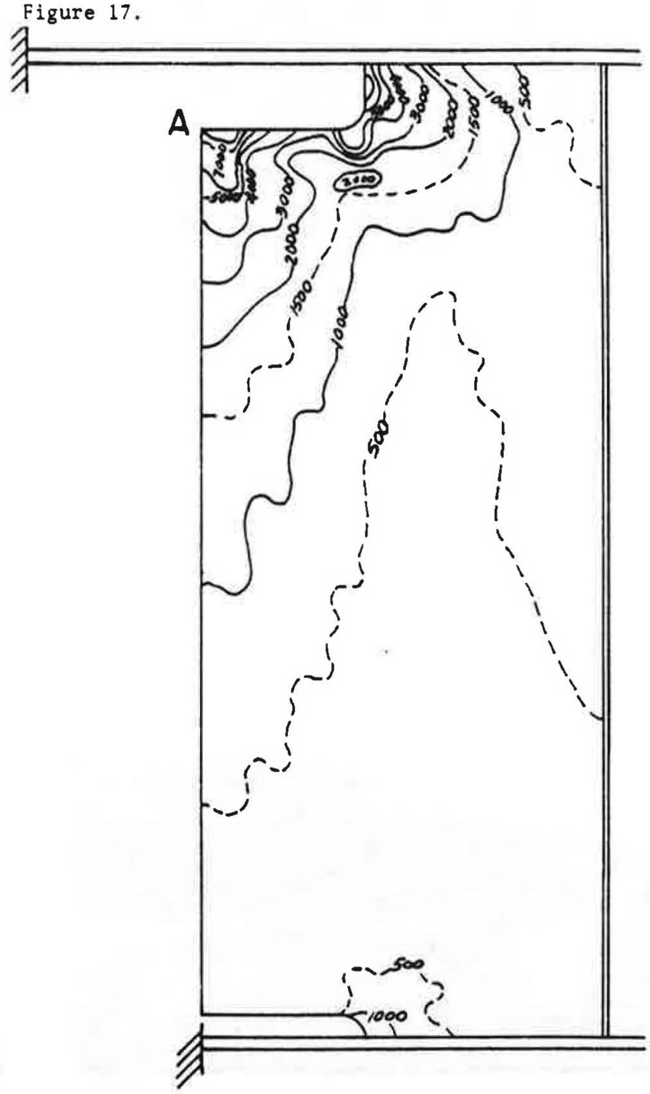
The plans for modification require removal of the existing rivets connecting the tie plates to the girder top flanges. The rivets will be replaced with high strength bolts which will not pass through the tie plates. Omission of existing fill plates leaves room for the bolt heads in many locations, so the existing tie plates may be reused. Elsewhere, special tie plate narrow enough to fit between the bolt heads in the existing pattern, with a welded-on stem (upstanding) to preserve the tension area, will be installed. Special surface protection will be provided to avoid corrosion on the resulting unconnected faces or in the resulting narrow spaces. The feature of special surface protection will apply both to the modified existing bridge and to the new bridge with strap plates unconnected where they pass over girder flanges.

The design of the tie plates was carried one step further. It was recognized that there existed the possibility of out-of-plane flexure of the web of the floorbeam near its connection to the girder due to the relative longitudinal motion of the girder top flange and the deck system.

An analysis, using a finite element procedure and the program RIPL1 (4) was undertaken. The mathematical model consisted of 704 plate elements and 68 beam elements, with a finer mesh in the region of greater interest. It was representative of the geometry of the existing typical floorbeam connection. Imposed boundary displacements were based on HS20 design vehicles at locations likely to be repeated 2 million times or more and, therefore, deemed significant fatigue loadings.

From the voluminous results obtained, maximum principal stresses were determined. The contours for those stresses have been plotted and are shown on Figure 17. Near point A, the level is 57 MPa (8.2 ksi). By extrapolation, it has been determined that for an HS20 vehicle located in the left-hand passing lane, at the near girder to floorbeam connection, the floorbeam web distortion stress level would be 69 MPa (10.0 ksi) which is well below that tolerable for fatigue at 2,000,000 cycles. Similarly, an HS20 vehicle located in the right-hand traffic lane (not the shoulder) would cause a floorbeam web distortion stress at the exterior girder to floorbeam connection of 54 MPa (7.9 ksi) which is well below that considered tolerable for fatigue at "over 2,000,000 cycles".

Figure 17.



Stress is shown in Pounds per Square Inch

It was concluded that cyclical web stresses generated in the floorbeam associated with elimination of the connection of the strap plates to the girders are low enough to create no difficulty in fatigue. However, it was requested that the details for the new structure be such that the displacement-induced secondary stresses (5) in the floorbeam web will not exceed the "threshold" 28 MPa (4.0 ksi), a value identified for the Lehigh Canal Bridge.

To accomplish this reduction, a study was made, again using the finite element procedure, in which the depth of the cope which clears the girder flange was varied. Three cope depths were used: 114 mm (4.5 inches), 165 mm (6.5 inches), and 203 mm (8 inches). As a result of this study, the 114-mm (4.5-inch) cope was modified, which is the least that will accommodate the girder flanges, to 165 mm (6.5 inches). This change reduced the "gage location" stress level from 45.6 MPa (6.61 ksi) to 22.9 MPa (3.32 ksi) which is well below the limiting "threshold".

The existing bridge does not permit this solution in any practicable way. Hence, it has been proposed to omit connectors from the two top outer locations in the connection angle and to place substitute bolts elsewhere in the pattern. Subsequently, PennDOT decided to apply these modifications to the bracket side of the girder as well as to the floorbeam side. They have also proposed to monitor the strains that occur in the floorbeam and bracket connection plates to ensure that the modified structure performs as intended.

Table of Stresses at the "Gage Location", MPa

Location	Web Thickness	Code Depth, mm 114	165	203
Girder G-3 at Piers	9.5 mm	45.6	22.9	9.9
Girder G-3 at Int. Fl. Bms.	11.1 mm	38.3	19.2	8.3
Girder G-4 at Piers	4.5 mm	36.1	18.2	7.9
Girder G-4 at Int. Fl. Bms.	11.1 mm	30.5	15.3	6.6

Note: 1 MPa = 145 lbf/in^2 = 0.145 ksi

Summary

Two illustrations have been presented of widening similar and conventional structures. Widening of the Hackensack River Bridge, because of alignment constraints, required symmetrical widening and the integration of new structure with existing. This widening was accomplished under traffic and although this complicated construction. the methods of integrating the new elements were simple and effective. The widening of the John Harris Bridge (I-83 over the Susquehanna River, now under construction, is being accomplished by adjacent construction. This does not involve any superstructure interaction between new and existing, or the same degree of impact on traffic. Several structural problems and their solutions common to this type of structure, were identified.

Acknowledgements

Gannett Fleming Corddry and Carpenter, Inc. was the design engineer and performed Technical Inspection for the Hackensack River Bridge project for the New Jersey Turnpike Authority. Howard Needles Tammen and Bergendoff was the General Engineering Consultant to the Authority, as well as designer of the original structure.

Widening design of the John Harris Bridge was by Gannett Fleming Corddry and Carpenter, Inc. for the Pennsylvania Department of Transportation.

References

1. John W. Fisher, et al: Transportation Research Record 607, pp. 56-62.
2. Howard Needles Tammen and Bergendoff, also the Engineers for the original structure.
3. Fisher, Yen, Daniels. "Fatigue Damage in the Lehigh Canal Bridge from Displacement-Induced Secondary Stresses". Transportation Research Record 607, pp 56-62.
4. Developed by Sandia Corporation.
5. At the gage location corresponding to that of a strain gage used on the Lehigh Canal Bridge.