THE GREEN RIVER BRIDGE ON THE WESTERN KENTUCKY TURNPIKE

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First of all, I would like to recognize the engineers who had a part in the design of the Green River Bridge on the Western Kentucky Turnpike. Most of the preliminary work in our office was done by Mr. Wallace Bennett and was closely coordinated with Mr. Robert Gillim of Brighton Engineers, as well as Mr. Smith and Mr. Vansant of the Department of Highways Bridge Office. The major part of the design was carried out by Carl Kroboth, Jr., and the checking of design calculations and details was in the hands of Mr. Jim Humphrey and Paul Patrick.

This bridge crosses the Green River from Muhlenberg to Ohio County and is located approximately one mile upstream from the bridge which carries U.S. 62 over Green River. The hydraulic computations indicated that an opening of 45,790 square feet below the 1937 high water elevation of 409.33 would be required. This was verified by the Louisville Office of the U.S. Geological Survey. The Green River is a navigable stream under control of the U.S. Corps of Engineers and since low steel on the existing bridge at Rockport provides a 35 ft. vertical clearance above high water, it was necessary to allow approximately the same vertical clearance above high water elevation. This requirement actually determined the location of the structure, since it was possible to take advantage of Jackson Bluff on the east end of the bridge for a termination point.

A number of span arrangements were considered for the twin bridges to satisfy the natural channel opening, the maximum opening required under high water and the economy of the entire structure. Preliminary estimates for the various span arrangements were computed and indicated a range in cost from $3,043,114.00 or $28.07 per square foot to $3,477,968.00 or $31.05 per square foot of roadway. The final span arrangement of 3-3 span continuous units was adopted, starting with two approach units of 160-200-160 foot spans and crossing the river with the center span of a 220-320-220 foot unit. Indications were that this span arrangement would be most economical. An all welded plate girder design was adopted with two girders to each bridge, placed 28 feet apart. Calculations indicated that depths of 8-15 feet for the approach spans would be needed and depths of 12-20 feet would be required for the main river spans. A36 steel was used for the webs and stringers, A373 steel with an allowable stress of 18,000 psi for flange plates of approach spans and A441 high strength low-alloy steel with an allowable stress of 22,000 to 27,000 psi, depending on the plate thickness, for flange plates for the main river spans. The diaphragms were designed as frames similar to the diaphragms used for the Kentucky River bridges on I-64 at Frankfort. Three longitudinal I-beam stringers, continuous over one or two diaphragms were placed parallel to the girders, equally spaced between them to support the deck slab, which has a roadway width of 30 feet with a 1-foot 6-inch safety curb on either side. All field connections were designed for the use
of high strength bolts. After final design and plans were completed and bids taken, a tabulation of the successful bids showed that the bridge would be built for approximately $2,888,184.00 plus or minus some minor construction changes at 0.67 per cent below our preliminary estimate. Quantities for the entire superstructure are as follows:

<table>
<thead>
<tr>
<th>Material</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>A36 steel</td>
<td>2,573,346 pounds</td>
</tr>
<tr>
<td>A373 steel</td>
<td>2,912,392 pounds</td>
</tr>
<tr>
<td>A441 steel</td>
<td>1,026,968 pounds</td>
</tr>
<tr>
<td>A7 steel</td>
<td>303,810 pounds</td>
</tr>
<tr>
<td>High strength bolts</td>
<td>85,814 pounds</td>
</tr>
<tr>
<td>Incidentals</td>
<td>66,868 pounds</td>
</tr>
<tr>
<td>Total</td>
<td>6,969,198 pounds</td>
</tr>
</tbody>
</table>

or approximately 3,500 tons

Class “A” Concrete ........................................ 3,712 cu. yards
Reinforcement ............................................. 717,282 pounds

Fabrication of the superstructure is well under way. The following photographs were taken during January at the Hammond, Indiana plant of Allied Structural Steel Company and show a number of girder sections ready for shipment as well as others being in the process of fabrication. Erection is scheduled to be started in April.

Subsurface investigations indicated that rock would be encountered approximately 80 feet below the natural ground line on the west side of the river. The logical method for the support of the west abutment and piers 1 through 6 was to place them on piles. Our first design was based on using 14-inch Bp @ 73 lbs. with an allowable load of 60 tons per pile. Since it is permissible to allow steel piles to carry a greater load than 6,000 psi, if substantiated by test loading. It was later decided to perform two pile test loads and possible use the same number of 12-inch Bp @ 73 lbs.

Seldom does the Design Engineer have the opportunity to participate in the construction phase and experience the difficulties that are incurred at times during construction. Since we also have the construction supervision contract, I am going to elaborate on some of the experiences which we encountered, such as the actual pile test loading and the cofferdams. The pile test loads were to be made by driving a test pile and two adjacent tie down piles to refusal. All three of these piles were placed so as to be in the correct position for incorporation into the finished structure. A 24 WF beam was placed across the two tie down piles and over the load test pile. This beam was attached to the tie down piles by means of channel yokes welded to the piles and coming over the top flange of the beam. A 150 ton hydraulic jack was placed between the load test pile and the 24-inch cross beam which applied the load to the load test pile. The jack was equipped with a calibrated dial and indicated the exact load at all times. The increment of settlement or deflection was measured to .001 of an inch by means of two deflection gage assemblies which were mounted on an independent arm attached to the test pile with the dial pin contacting the stationary supports driven into the ground a sufficient distance away from the piles so as not to be effected by any possible movement of the piles. Actually only one such deflection gage assembly would be necessary. However, it was decided to use two assemblies as a precautionary measure in case one of them should be accidentally disturbed. No load was applied for 72 hours after driving the piles. The first increment of load was 48 tons. 2 Deflection readings were taken just prior and immediately after the application of the load and every one-half hour thereafter until the deflection since the last reading was .02 inch or less. One hour after this reading the second increment of load in the amount of 24 tons was applied and the same procedure followed. The third increment of 24 tons was applied two hours after the deflection was
less than .02 inch. The final increment of 24 tons was applied three hours after
deflection became less than .02 inch between successive readings. The total load
of 120 tons was left in place for 60 hours and readings were recorded every hour.
At the termination of this 60 hour period, the first increment of load amounting to
24 tons was removed, the second of 24 tons was removed one hour later, the third
of 24 tons two hours after the second and the final of 48 tons three hours after
the third increment. Settlement readings were made before and after each load
removal and at 1/2 hour intervals between load removal with a final net settlement
reading three hours after the removal of the final load. The load test indicated
that 12 WF @ 53 pounds would be satisfactory which resulted in a saving of
approximately $57,500.00. The cost of the two pile tests loads were $4,000.00.

The rest of the test piles could now be driven and the piling ordered prepared.
All in all, approximately 28,000 lin. ft. of piles were driven for piers 1
through 6. A 50 C double action air driven hammer was used.

The four river piers were designed to be founded on rock. This necessitated
the use of cofferdams approximately 80 feet deep on the west bank and approximately
40 feet deep on the east bank of the river. Two different types of coffer-
dams were designed. For the west side of the river, semi circular cells were
designed with a strut in the center. The whalers consisted of 12 WF beams rolled
circular to fit the inside of the cellular arrangement of sheet piling. The vertical
spacing of the whalers and struts varied from 8 feet C to C at the top to 8 feet
six inches at the bottom of the cofferdam. No particular difficulties were encoun-
tered in driving and excavating the cofferdam for the south pier on the west bank.
Only one blow was experienced which was due to boulders lodged under the
sheet piling. This blow occurred after considerable amount of excavation inside
the cofferdam had been removed and the pressure from the outside forcing the
boulders into the cofferdam. The sheet piles were then re-driven and excavation
continued without any further mishap. The reinforced footing or distribution
block 22 ft. x 45 1/2 ft. x 14 ft. high which necessitated the pouring of approxi-
ately 519 cu. yds. of concrete in a continuous pour required approximately 20
hours. It was keyed approximately 4 ft. into rock. Construction of the pier
column to an elevation above the sheet piling was finished without any trouble, as
was the backfilling and removal of whalers and struts as backfilling operations
progressed. However, it was soon evident that it would possibly be more diffi-
cult to pull the sheet piles than to drive them. The sheet piles had to be re-
forced at the pin connection by welding additional plates to the sheets and a
stiff leg had to be employed to pull them the first six feet.

It is difficult to understand why so little trouble was encountered in the con-
struction of this pier, yet considerable difficulties had to be overcome in the con-
struction of the adjacent companion pier a few feet to the north. The coffer-
dam for the companion pier was started immediately. The sheet piling was again
driven to refusal. Excavation of the cofferdam was started and it soon became
apparent that there was a slight tilting movement in the cofferdam, tilting towards
the river. This situation was corrected by removing some of the surcharge on the
land side and anchoring the cofferdam to one finished pier. Excavation then was
continued to approximately 40 feet above rock when the first of ten blows occurred.
An attempt to redrive the sheets as proved successful during the excavation oper-
ations of the cofferdam for the south pier, proved fruitless. It was then decided to
drive a blister at the spot where the blow occurred. After this blister was driven,
the cofferdam was again pumped dry and excavation continued. After removal of
an additional 5-8 feet of material out of the cofferdam, the second blow occurred,
again in the north cell but closer to the center of the cofferdam. An additional
blister was driven extended from the first blister almost across the entire land side
of the cofferdam and again dewatering and excavation was resumed. It wasn't
long, however, before the third blow occurred. Again the cofferdam had to be
flooded and over 2,000 sacks of grout were deposited along the bottom of the
sheet piles. Shortly after excavation operations were resumed, an additional blow
after load ing to the third load ing of the pre­
pared project were killed. As before, the work was continued.

Each time it was possible to excavate a few feet more but a fifth, sixth, seventh, eighth and ninth blow occurred. By this time, nine whaler rings had been placed and excavation was within approximately 18 feet of rock. At this stage the tenth and last blow happened and this was by far the worst. Fortunately there were no workmen in the cofferdams nor the rig left on the trestle, since the night shift was not due to work that night. This blow brought approximately 10 additional feet of silt into the cofferdam. In the process, the pile bent closest to the cofferdam on which the trestle was supported was torn loose and two of the 50-foot timber piles were sucked into the ground and to date have not been found.

Since time for completion of this pier was growing short, due to anticipated high water during the last part of February, a meeting was held which was attended by representatives of the contractor, Brighton Engineers, the Department of Highways Bridge Division and Construction Division and ourselves. At this meeting it was decided to permit the contractor to drive a new rectangular cofferdam inside of the north cell and pour the footing against the new sheet piling. This rectangular cofferdam was of the same width and approximately two feet shorter than half of the length of the planned footing. It was hoped that after this new cofferdam was in place it would be possible to block off the south cell, excavate same in the usual manner and then pour the balance of the footing against the footing in the north cell. It was evident at this time that there also was trouble in the south cell and the contractor was permitted to follow the same procedure for the placement of the footing in the south cell. This resulted in two separate footings instead of one continuous footing. Since rock was encountered nine feet above anticipated rock elevations, it was possible to revise footing pressure calculations which showed that the foundation pressure for the two separate footings was not increased over the allowable pressure of the original design. The pier is now poured to an elevation above high water and no additional difficulties are expected. Just what caused the trouble we will perhaps find out when the sheet piling is pulled. The cofferdam for the two piers on the east bank were conventional rectangular cofferdams and since they were only approximately 40 feet deep no difficulties were encountered.