

## **INNOVATIVE PCIB BRIDGE FOR A HIGHLY-SKEWED RAILROAD CROSSING**

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### **ABSTRACT**

*Palmer Engineering designed a cost-effective bridge to carry US 20 over a highly-skewed rail crossing in Ashtabula County, Ohio. The project was complicated by soils that are ill-suited for bridge foundations and cross roads intersecting US 20 near each end of the bridge. The solution spans the rail lines with prestressed concrete I-beams placed perpendicular to the abutments, instead of parallel to the roadway, greatly reducing the span length.*

**Keywords:** High Skew, Bridge, Prestressed Beams, Railroad

## PROJECT BACKGROUND

In Ashtabula County, Ohio, US 20 intersects a set of Norfolk Southern (NS) railroad tracks between the Village of North Kingsville and the City of Conneaut. The skew between the road and railroad at this location is approximately 72°. In the 1940's, a grade separation was constructed that consisted of a 23-span, multi-unit reinforced concrete beam/slab bridge, with numerous piers and crashwalls on NS right of way in close proximity to the tracks. Due to advancing age and deterioration, the bridge was in need of a major rehabilitation or replacement.

The primary challenge to replacement of the bridge was identifying a structure type that could cost effectively meet railroad design criteria. These criteria include making accommodations for two future tracks adjacent to the existing lines, increasing the vertical clearance from 22' to 23', and providing adequate horizontal clearance to substructure units. Meeting these requirements would have necessitated a main span of more than 320' in length and a roadway profile increase of approximately 9 feet. Due to the flat topography of the project site and local roads intersecting US 20 near each end of the bridge, any significant increase in the vertical profile would have been prohibitively expensive.

Given the anticipated cost of replacement, rehabilitation of the structure was initially expected to be the preferred improvement strategy, though it offered a limited-duration solution. However, the project team was able to successfully conceptualize, design, and construct an innovative single-span prestressed concrete I-beam structure that offered significant savings over a conventional bridge type.

## DESIGN

### BRIDGE LAYOUT

Numerous options were investigated during the preliminary engineering phase, including a single-span steel girder option with a 320' span, a precast concrete tunnel, and multi-span options using steel or post-tensioned concrete straddle bents. The preferred alternative was a single-span bridge with the beams oriented perpendicular to the centerline of the railroad tracks (See *Figure 1*). The portion of the superstructure that fell within the roadway limits would be supported on stub abutments behind MSE Walls, while outside these limits the bridge would be supported on piers. This layout called for 53 beams spaced at 9', with a span length of 100'. This span length provided adequate lateral clearance to the substructure units to avoid crashwall requirements.

To reduce construction costs, the design team opted to refine the original concept by using a splayed framing plan with a wider beam spacing at the acute corners. This strategy eliminated beam lines and minimized pier lengths. To realize additional cost savings, Palmer and ODOT agreed to place the deck only within the footprint of the roadway and sidewalks, leaving significant portions of the beams exposed. The final layout utilized 33 beams with the interior, most heavily loaded beams, placed perpendicular to the abutments and spaced at a constant

13' (See *Figure 2*). The span of these beams is 100'. The 10 beams on each side of these central beams are splayed, with spacings ranging from 6' to 16', and spans ranging from 100' to 141' (measured along the centerline of the beam). This layout further reduced the total pier length by 100'.

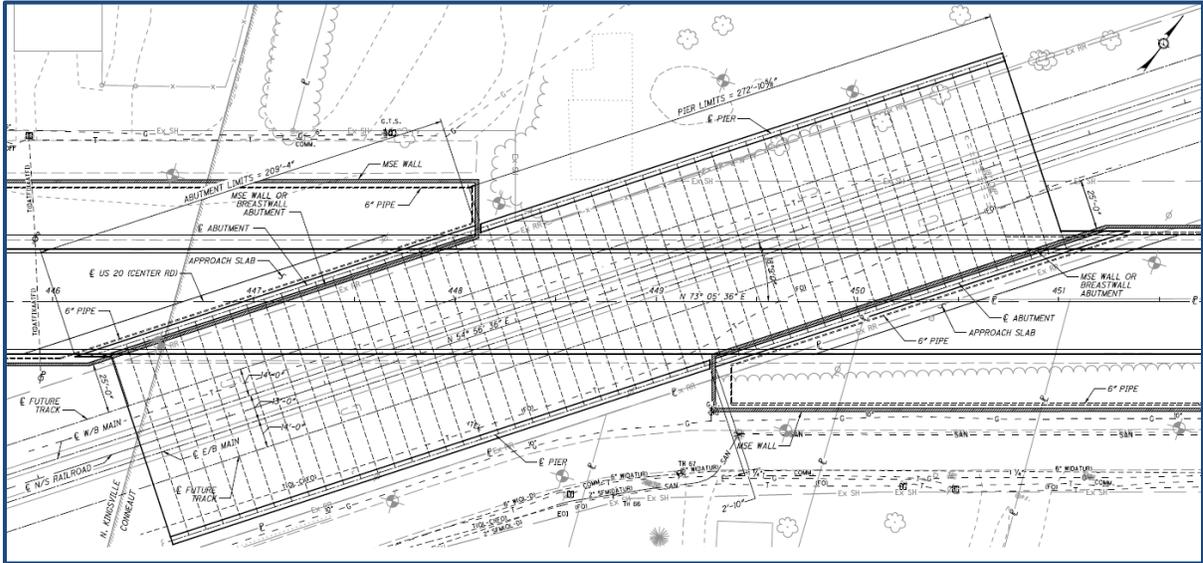


Figure 1: Conceptual Bridge Layout

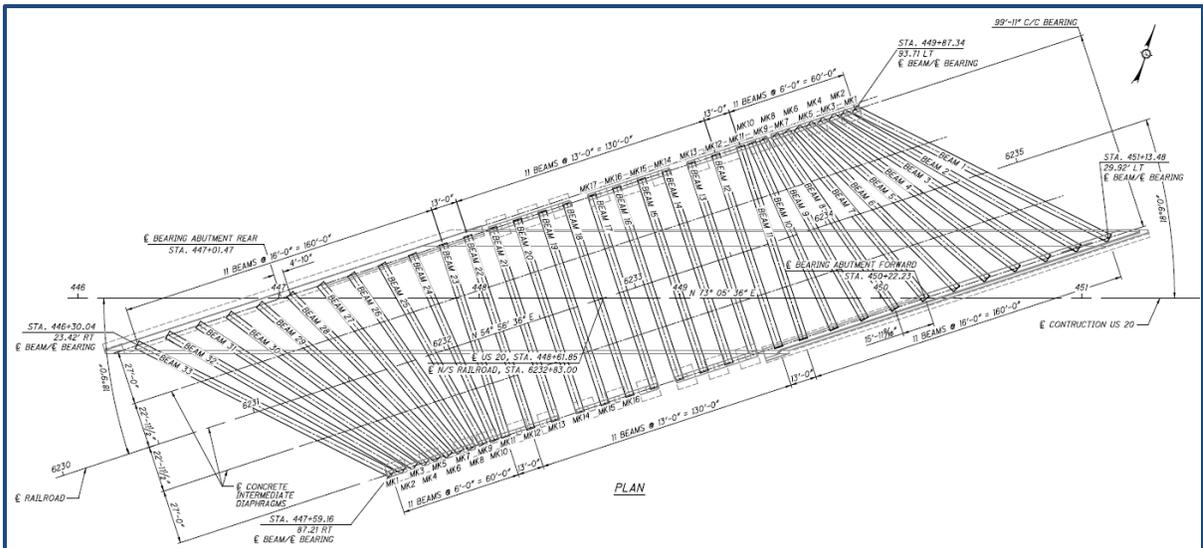


Figure 2: Final Bridge Layout

## BEAM TYPE SELECTION

The selection of the specific prestressed concrete beam section to be used for the bridge was also a major consideration during this phase of the project. Prestressed concrete was the preferred material of the project team for a number of reasons: Due to inconsistent performance, some ODOT districts have shifted away from weathering steel, leaving galvanizing as the preferred option for protection of steel girders in northeast Ohio, which has driven up costs. Additionally, the splayed alignment and unbalanced loading of the framing plan put a premium on the greater out-of-plane stiffness of prestressed concrete beams over steel girders for erection and final conditions. However, meeting the layout constraints with concrete would require a deeper beam section than steel, which would add significant cost due to the increased profile. Fortunately, during the structure type selection process, ODOT approved a new prestressed concrete I-beam standard drawing (PSIB-1-13)<sup>1</sup>, which included the AASHTO Wide Flange shapes for the first time. The wide flange shapes allowed for shallower beam sections for the chosen span length than a Modified AASHTO Type 4. After initial analysis, the WF60-49 was chosen as the optimal shape for both structural performance and roadway profile impacts. The cost analysis included discussions with precast beam manufacturers to determine the cost of fabrication, shipping, and setting of the wide flange beams. Although not taken into account in the cost analysis, the 49" wide top flange also proved to be an advantage in making the design span for the transverse deck more manageable in areas of 16' beam spacing.

## DETERMINATION OF LIVE LOAD EFFECTS

The complexity of the final bridge layout presented numerous challenges for design and detailing of the prestressed concrete beams. Due to the splayed configuration of the beams, the limitation of the deck to only the roadway and sidewalk footprint, and the presence of a sidewalk on one side only, each of the 33 beams had a different set of loading conditions.

In order to accurately determine Live Load distribution to the beams, as well as out-of-plane forces due to the complex layout, a model was created in CSi Bridge software (See *Figure 3*). The model utilized plates for the deck rigidly connected to frame elements for the beams. This convention was considered sufficient because the intent of the model was not to determine stresses in the members, but only to determine live load moments and shears in the beams. The model was also used to check for torsion of the beams, thermal expansion direction, deflections, and support rotations. These demands would then be used in a separate program for the beam designs. Crossframes were found to have limited impact on the distribution of the live loads; thus in order to increase conservatism, they were excluded from the final model.

CSi Bridge has an internal moving live load generator, which was used to account for the HL-93 loading. Both the truck with lane loads and the tandem with lane loads were considered. The bridge width of 59' (face to face of parapet including sidewalk) could accommodate up to 4 lanes. Loading scenarios with one to four lanes loaded were checked with the appropriate multiple presence factors. The dynamic loading allowance was input into the program based on AASHTO code requirements. Pedestrian loads were input as an area load.

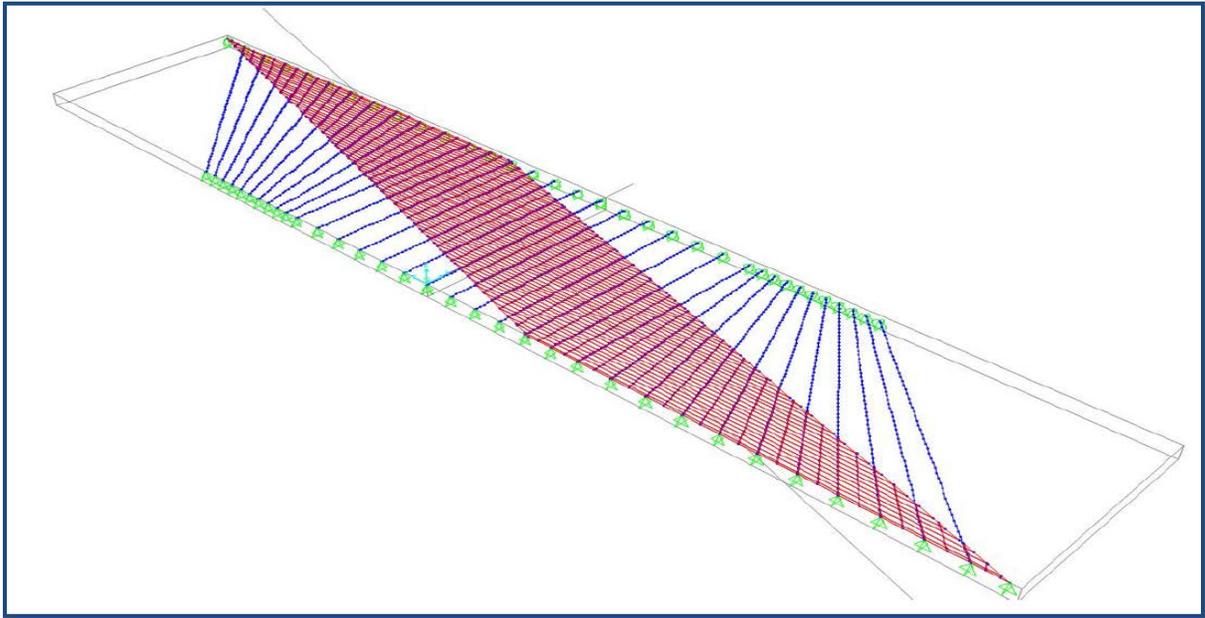


Figure 3: CSi Bridge Model

Although the tandem and truck loading produced similar results, the tandem loading controlled for both shear and moment on all the beams. The loading configuration that produced the maximum moment and shear for the main interior beams was four loaded lanes, with the multiple presence factor, and the pedestrian loading on the sidewalk. As expected, the center beam (Beam 17) had the highest live load moment, with moments decreasing in beams toward the edges (See *Figure 4*). The highest live load shears were found in locations where the deck extends over the beam supports (See *Figure 4*). Torsional forces were negligible, even without crossframes.

The model was verified with a GTStruDL model developed by a separate design engineer. The version of GTStruDL used does not have the ability to generate moving loads, so maximum loading scenarios were selected by observation and verified by the CSi Bridge model. Maximum moments and shears observed in the GTStruDL analysis varied by approximately 5% from the CSi Bridge model, with the CSi Bridge model predicting higher moments and shears.

## BEAM DESIGN

Beam	Moment (kip-ft)	Shear (kip)
1	169	36
2	414	74
3	652	85
4	853	104
5	1094	103
6	1306	111
7	1474	121
8	1655	122
9	1803	123
10	1899	119
11	1997	119
12	2112	105
13	2154	116
14	2176	103
15	2183	92
16	2196	84
17	2202	78

Figure 4: CSi Bridge - Maximum Live Load Results (including Impact)

Because the bridge is approximately symmetric, 17 beam designs were used to accurately capture the effects on all 33 beams. The beams were modeled in Bentley Conspan software, one of ODOT's approved prestressed beam design programs. Due to the inability to accurately capture the effects of a partially composite deck, and under the direction of ODOT, all beams were modeled assuming no composite action with the deck. However, for added reserve strength, the deck was detailed as composite.

In order to use the live load moments and shears determined in CSi Bridge, they had to be converted to "dead loads" for input into Conspan. Investigation of the live loads showed that the moment and shear distribution in the beams could be roughly approximated by placing a distributed load within the limits of the deck on each beam; however, the magnitude of this load differed for moment and shear. The distributed loads were calculated in a spreadsheet. Because these loads were input into Conspan as dead loads (DC2), load factors had to be manually adjusted for each applicable load case (Service I, Service III, and Strength I). This adjustment factor,  $\gamma_{adjusted}$ , was calculated as:

$$\gamma_{adjusted} = \frac{\gamma_{LL}}{\gamma_{DC}}$$

Five separate runs were needed per beam to capture the force effects:

- Dead Loads only – Used for camber and deflection calculations
- Service I – Max moment live loads applied with an adjustment factor of  $1.0/1.0 = 1.0$
- Service III – Max moment live loads applied with an adjustment factor of  $0.8/1.0 = 0.8$
- Strength I – Max moment live loads applied with an adjustment factor of  $1.75/1.25 = 1.4$
- Shear Strength I – Used for shear design. Max shear live loads applied with an adjustment factor of 1.4.

In an effort to simplify the design and manufacturing, two strand patterns were selected for use on all the beams (See *Figure 5*). The interior, parallel beams utilized 52 strands, including the use of draping and debonding. Specified release and final concrete stresses in these beams were 6.5 ksi and 10.0 ksi. The exterior, splayed beams utilized 39 strands, including the use of debonding. Specified release and final concrete stresses in these beams were 5.0 ksi and 7.5 ksi. The beam designs were controlled by either ultimate moment or bottom flange tension stress. Every beam had a unique shear reinforcing layout due to varying shear demands and deck limits.

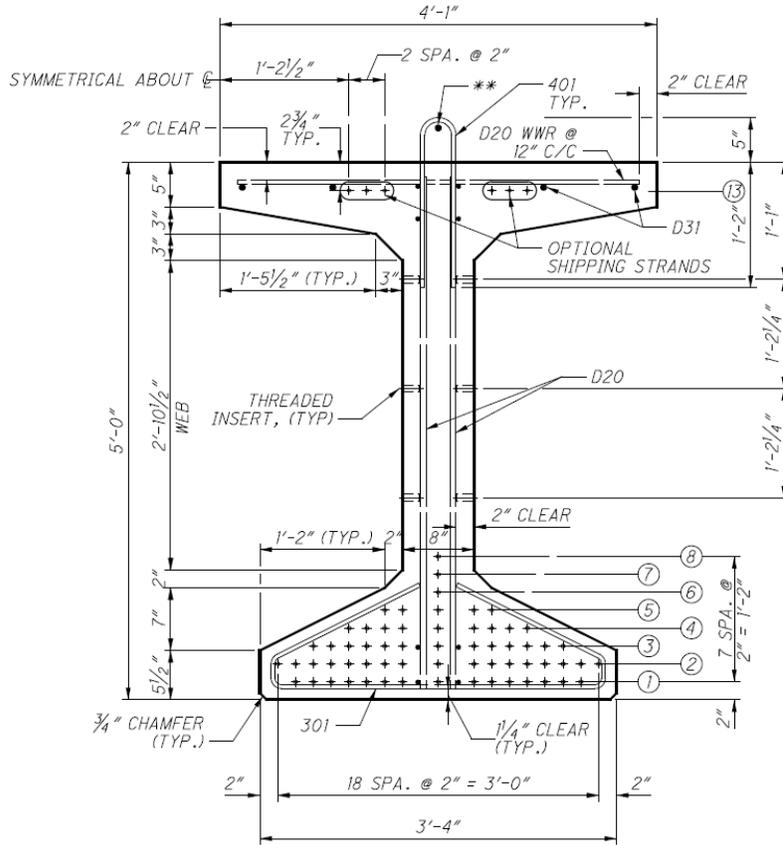


Figure 5: WF60-49 Beam Section<sup>1</sup>

### CAMBER AND DEFLECTION

Each beam was run as a line girder in Conspan, and the camber and deflection values calculated assume independent action of each beam. It was of special concern to the design team to ensure this assumption created no unforeseen complications during deck placement or for determining the effects of long-term creep. As noted, the CSi Bridge analysis included models with and without diaphragms. Dead load deflections from CSi Bridge were compared to those from Conspan and found to be similar. Additionally, the deflections and rotations were found to be essentially along the axis of each beam (i.e. negligible out-of-plane deflections).

Due to the complex alignment, a unique situation was created in determining the haunch thickness. The interior beams began at the rear abutment, crossed under the left fascia, intersected the crown of the roadway, and crossed under the right fascia before reaching the forward abutment. So, despite a camber of more than 2" after placement of dead loads, the haunch at midspan was around 8" for most of the interior beams. Thus, while negative haunch was not a concern, a smooth riding surface due to unanticipated deflections was a concern. To

address potential rideability issues, an extra 0.5” of sacrificial wearing surface was added to the deck, and a bid item for deck grinding for smooth riding was added to the contract.

Long-term upward creep of the lightly loaded exterior girders was an item of concern once the decision was made to only place the deck under the footprint of the roadway. However, visual observation of the layout shows that as the applied dead loads decrease, the length of the beams increases, thereby increasing the self-weight dead load. This situation essentially balances the total dead load on the exterior beams. This observation was verified by the calculations, which also showed that differential, long-term camber between two given beams was not predicted to be over 0.5”. The cast-in-place diaphragms should limit this differential value even further.

## CONSTRUCTION

The 33 beams were cast by Prestress Services in Lexington, Kentucky and shipped more than 300 miles to the site in Northeast Ohio. Beam erection was complicated by the permitted times the railroad allowed work over the tracks and the significant weight of the beams. The largest beam was almost 143’ long and weighed 157.6 kips. The beams were picked by a pair of 550 ton, all-terrain cranes, with one crane behind each abutment (See *Figure 6*).

The deck pour was completed with the finishing machine aligned perpendicular to the



*Figure 6: Beam Erection*

centerline of the roadway at the request of the contractor. Typical ODOT standards require the finishing machine to be placed along the skew. However, the contractor was allowed to place

the machine as requested, providing concrete was placed sufficiently ahead of the machine to preload the two beams ahead of the beam over which the finisher was positioned. This strategy required the use of a set retardant to maintain the workability of the concrete. No problems were encountered with this method, or with unanticipated deflections. Significant areas of the deck within the acute corners at each end of the bridge were finished using manual techniques.

## CONCLUSION

The bridge was constructed under a complete closure due to the complex nature of both the existing and proposed structures. It was re-opened to traffic within a single construction season, and fully completed soon thereafter. The completed bridge is an example of how innovation can be used to address a complex problem (see *Figures 7 through 10*).



*Figure 7: Aerial View of Bridge looking South*



*Figure 8: Aerial View of Bridge from above*



*Figure 9: View of Bridge looking West*



*Figure 10: View under Bridge looking West*

## **REFERENCES**

1. "Prestressed Concrete I-Beam Bridge Details," PSIB-1-13, Ohio Department of Transportation, Office of Structural Engineering. 2013.