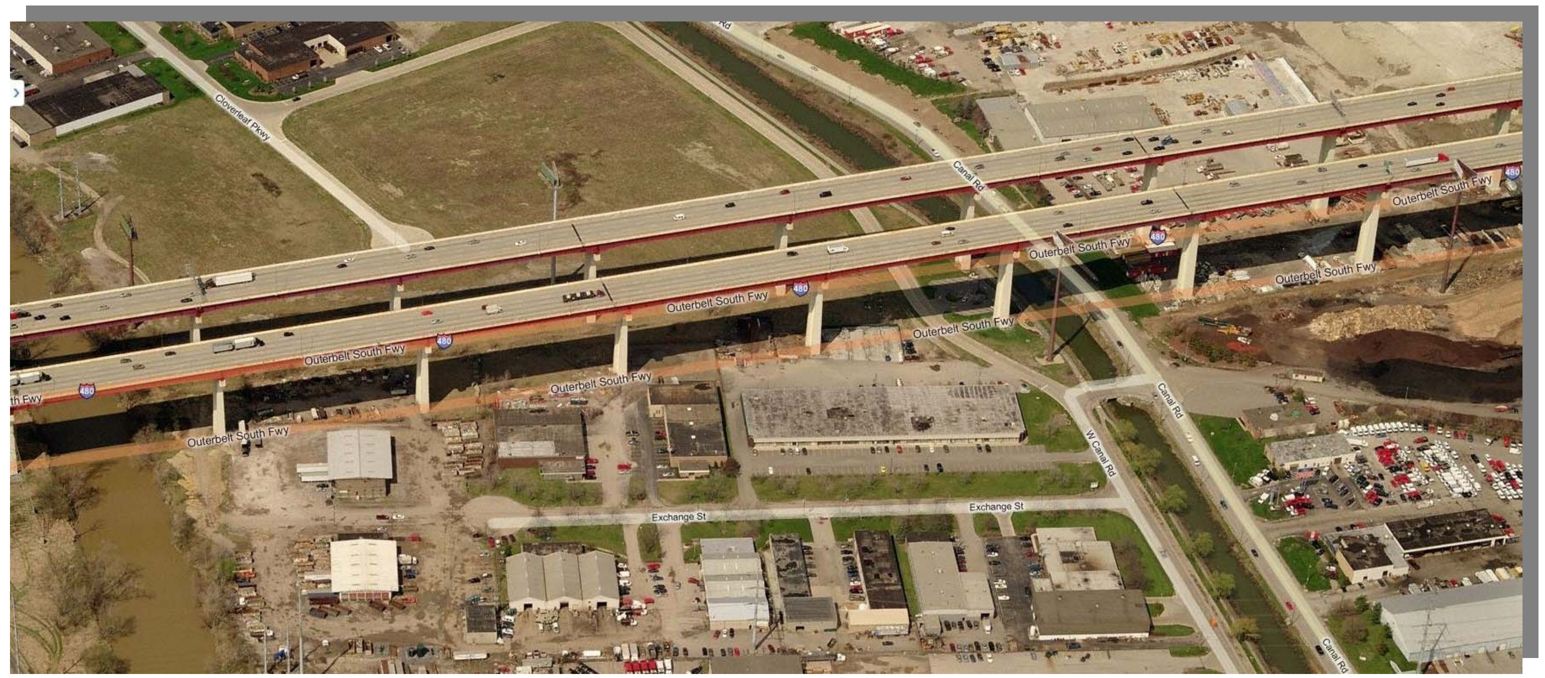
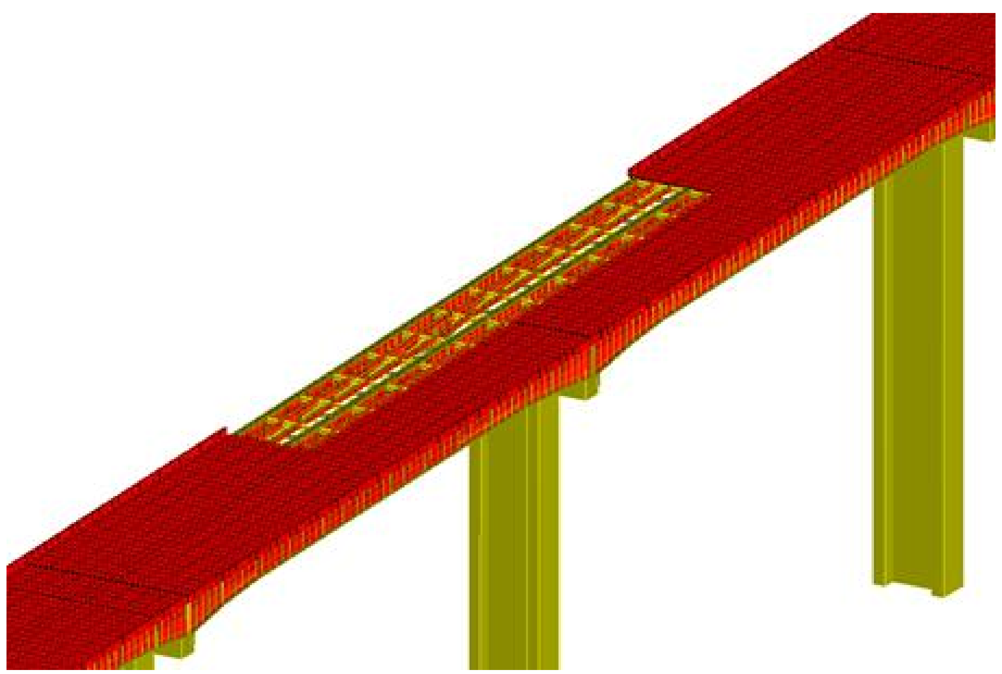
3-D Finite Element Modeling of CUY-480-18.42 Deck Replacement

Volume I

SFN # 1812521 & 1812548

PID 90591

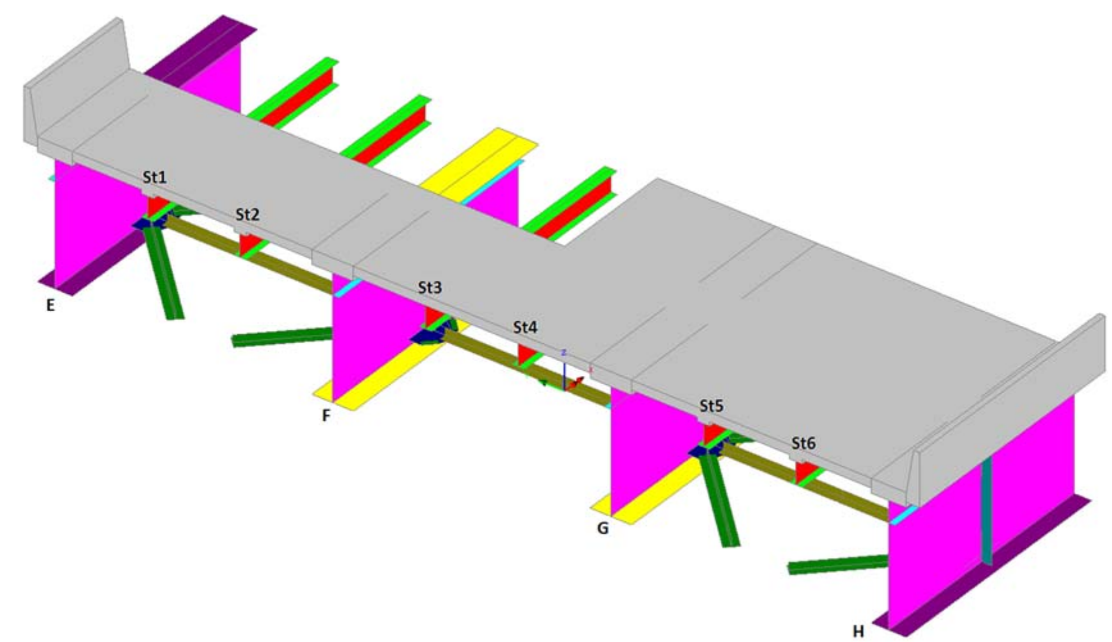




ODOT District 12
5500 Transportation Blvd.
Garfield Heights, Ohio 44125

E.L. ROBINSON ENGINEERING

August 6, 2012



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July 31, 2012

Myron Pakush, P.E.
Deputy Director
ODOT – District 12
5500 Transportation Blvd.
Garfield Heights, OH 44125

RE: CUY-480-18.42
PID 90591
Deck Replacement Study

Attention Mr. Pakush:

The enclosed report addresses the out-of-plane distortions that occur as a result of replacing the decks for the CUY-480-18.42 L\R twin structures when utilizing part-width construction methods. P.E. Stamps have been affixed below for this report. The engineering analyses efforts that are specifically represented by the P.E. stamps are listed as follows:

- 3-D Finite Element Modeling (FEM) of the CUY-480-18.42 R Bridge for Deck Replacement analyses.
- Superstructure analysis and code checking during deck removal and existing conditions per AASHTO Standard Specifications, 17th Edition 2002.
- Using 3-D FEM to determine superstructure deflections, out-of-plane movements, and possible impacts to fatigue prone bridge components.
- Deck replacement analyses and construction recommendations.
- Retrofit design to control out-of-plane movements and related stresses.

If additional information is desired regarding the discussions provided in this report, please contact E.L. Robinson Engineering.

Respectfully,

Richard L Engel

Richard Engel, P.E.
Vice President

ENGINEERS SEAL:

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Volume I – Contents

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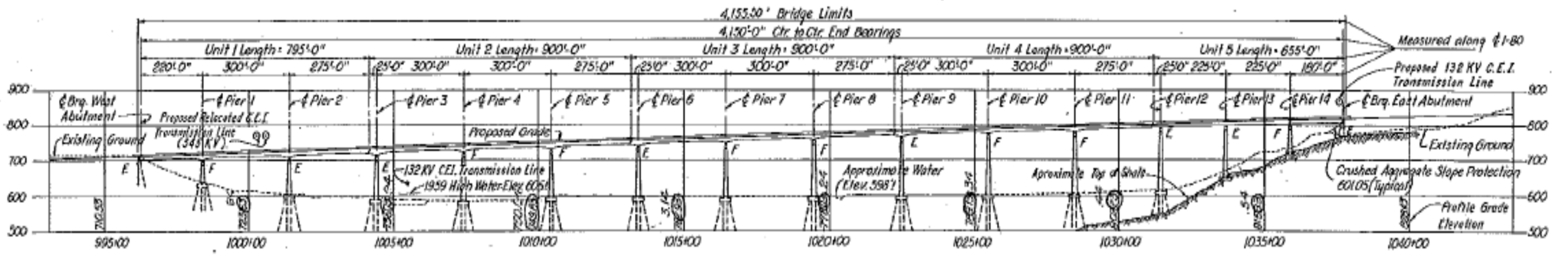
PART I
Development of a 3-D Finite Element Model for the CUY-480-1842 Superstructure



I. Introduction

This study investigated the part-width replacement of the reinforced concrete decks for the twin CUY-480-1842 L/R structures. Each structure currently carries four lanes of traffic. The existing structures each have four main girders spaced at 22'-4". There are two intermediate stringers located between the main girders.

The scope of work for this study requires that during replacement of the decks, a minimum of three 12 foot lanes of traffic be maintained in both directions (eastbound and westbound). The existing width of one of the bridges cannot accommodate six lanes of traffic; therefore, a full closure of one bridge at a time is not acceptable. Part-width deck replacement procedures will cause differential deflection between adjacent main girders and stringers when the existing deck is removed in part-width segments. ELR was directed to determine if the components of the existing steel superstructure would perform satisfactorily when part-width deck replacement procedures are utilized. A component of the evaluation includes establishing if and where retrofit details must be designed and constructed to control out-of-plane distortions during phase construction procedures. Some of the major scope items included:

- *Development of a 3-D Finite Element Model for the CUY-480-1842 Superstructure(s).*
- *Study the CUY-480-1842 R structure, noting that both of the structures are somewhat similar.*
- *Study part-width deck replacement using the MOT Alternative Scheme termed (5+1).*
- *Develop Sub-Model(s) for Out-of-Plane Distortion only for areas of high stress identified in the full model.*

The existing 4150 feet long I-480 EB and WB structures utilize fifteen spans to carry a relatively high volume of South Freeway traffic over the Cuyahoga River Valley. These structures are an important component of the interstate network traversing the City of Cleveland, which makes maintenance of traffic a paramount issue during the rehabilitation work. Within the limits of the river valley are the Cuyahoga River, Erie Canal, CSX and RTA railroad tracks, Cleveland Metro Park bike path and Canal Road.

The existing structures were designed using the AASHTO Allowable Stress Design Method in accordance with the 1965 AASHTO Standard Specifications, interim specifications and the Ohio "Supplement". The applied Design Loading was the HS 20-44 loading and the Interstate Alternate Loading. The construction of the twin bridges was completed in 1975. The superstructure is divided into five units with four hinges having intermediate steel finger deck joints at each hinge to accommodate thermal movement. Elastomeric compression seal joints are provided at the abutments. The lengths of the five (5) superstructure units are as follows: Unit 1 is 795'; Units 2-4 are 900' and Unit 5 is 655'. The sub-stringers are supported on a steel floorbeam system. The original superstructure design philosophy assumed that the reinforced concrete deck and stringers would perform as non-composite members. The steel material used for the main girders is ASTM A588 and the remaining steel members consist of A36 steel. The existing reinforced concrete substructure units are T-type piers varying in height from 75 feet to 185 feet and stub abutments which are located at the top of the spill thru slopes. The substructures are supported on either steel H-piles or spread footings on shale. The current BARS (LFD) load rating for the bridge is HS19.4, which is governed by the moment capacity at one of the piers.

The existing structural steel experienced fatigue cracks dating back to the shipment of the girders to the construction site. In 1973, after cracks were discovered in the main steel girders during erection, ODOT commissioned Prof. John Fisher of Lehigh University to perform a fatigue evaluation of the bridge to determine the cause of the cracks. Prof. Fisher's findings concluded that the cause of the cracks was fatigue related to and caused by the relative out-of-plane distortion caused by cyclical loading of the girders which occurred during shipping on railroad flatbed cars.

The existing reinforced concrete decks are approximately 35 years old. An overlay was placed on the decks in 1989. Due to the configuration of the superstructure's transverse floorbeam/crossframe members, it is necessary to avoid eccentric loads to the main girders from the concrete deck dead load and traffic live load. The original contract plans did not allow part-width deck construction procedures to be used. The plans stated that *"Deck concrete shall be placed symmetrically about longitudinal centerline of the deck. No longitudinal construction joints shall be permitted*".

The 2010 Bridge Inspection Report performed by HDR Engineering, Inc. and Northwest Consultants, Inc. states that the overall condition for both bridges is "satisfactory" (General Appraisal is 6). The deck for both bridges is rated as being in "fair" condition (condition rating is 5). As indicated in the inspection reports, the girders are in good condition. The stringers, which are welded to the top flange of the top chord of the floorbeam, have experienced some cracking at the connections. The structural steel was last painted in 2001.

Additional project background information can be found in Appendix C. The cost estimates for the deck replacement work, provided in Appendix A, were prepared using planning level information. The cost estimates were prepared to aid in making preliminary design decisions.

Part I of Volume I of this study provides a summary of the procedures utilized to develop and perform a 3D finite element evaluation of the existing structures. The existing structures have experienced and are prone to high localized out-of-plane stresses at the crossframe to web connections. The feasibility of developing an acceptable deck replacement design and sequence of work has been evaluated and the results are presented in this report.

Part I contains multilevel 3-D modeling evaluations developed for the right bridge. ELR calculated the deflections and out-of-plane deformations caused by part-width construction alternatives and determined possible impacts to the fatigue prone details. Selected computational results are presented in Volume II of this report. The out-of-plane distortion causes higher stresses in the positive moment regions than in the negative moment regions near the pier support. Part I includes the evaluation of retrofit details with the objective of controlling the distortion-induced stresses.

As a result of the work accomplished during the Part I study phase, ODOT District 12 personnel were in a position to provide direction for the work to be performed during the Part II study phase. Part II of Volume I contains results of the study which focused on establishing a preferred deck replacement removal sequence along with recommended deck removal and replacement dimensions. The Part II analyses were performed to determine the specific construction procedures to be used to replace the existing decks when utilizing part-width construction methods, while satisfying maintenance of traffic obligations. ODOT has a strong desire to replace each deck in one construction season. This part-width construction work must be accomplished without jeopardizing the structural integrity of the superstructure. The recommended design procedures must include construction constraints necessary to ensure that out-of-plane stresses caused by the deck removal distortions are not any higher than the highest operating stress level presently occurring in the existing girders.

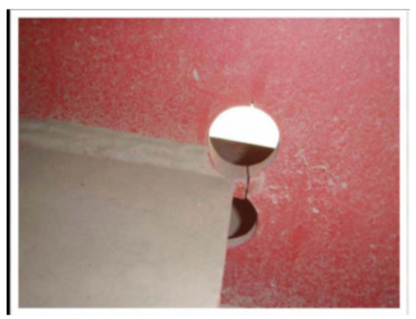
Volume II contains the calculations performed for the analyses portion of this study.

Physical Condition of Existing Superstructure

The Bridge Inspection Reports performed by HDR Engineering, Inc. and Northwest Consultants, Inc. (inspections dated 9/20-10/1/2010, 2009 and 10/13-10/16/2008) were reviewed.

The 2010Inspection Report, for both bridges indicates that the overall condition is described as "satisfactory" (General Appraisal rating of 6).

The girders were found to be in "Fair" condition. The inspection report indicated the presence of two 2" diameter holes drilled in the web on each side of the lower lateral bracing gusset plate. These holes were used to arrest crack growth that is common at these locations throughout the superstructure. The inspection report recommended monitoring some of the high stress areas where the holes were overcut.



Holes measuring ½", 1", 1 ½", or 2" have been drilled in the web to relieve stresses at various locations.

The stringers sit on top of the floorbeams and are welded to the top flange of the top chord of the floorbeams. The stringers were reported to be in good condition. Several of these stringer/floorbeam connections have developed cracks (see photo showing crack in weld of stringer 11 to the floor beam in span 3 of the right bridge). These cracks have not propagated into the stringer flanges.

The floorbeams and its connections were found to be in good condition.

The superstructure was painted in 2001. The paint coating has numerous areas where the primer is visible; however, there is no evidence of corrosion. Overall the protective coating system is in good condition (between 1 and 5% of the painted area needs to be repainted).

The bridge decks are in fair condition (5 rating). There are several areas where the concrete has spalled and exposed the reinforcing steel. The underside of the deck contains numerous hairline cracks, narrow transverse cracks, and map cracking.

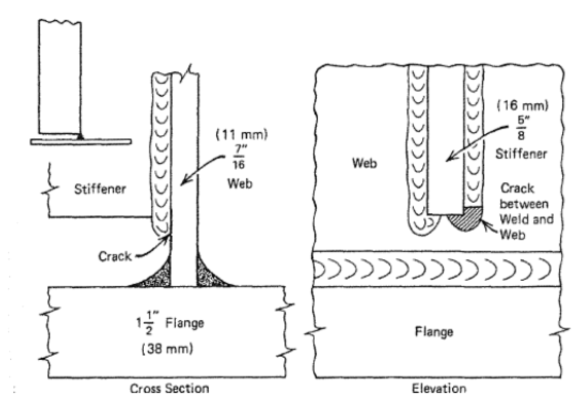
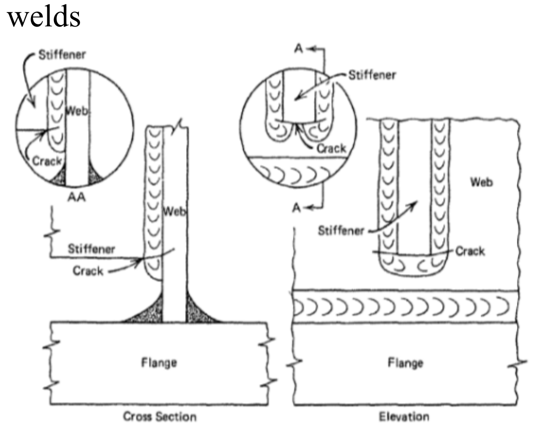
The ***"Bearings"*** section of the Supplemental Report of the 2009 Inspection Results, which accompanied the 2009 Annual Inspection of the "Valley View" Bridges, indicates that the bearings show no sign of movement. This conclusion is verified by observing that the paint is not cracked over the junctions between the gears of the rollers and the racks above and below the rollers.

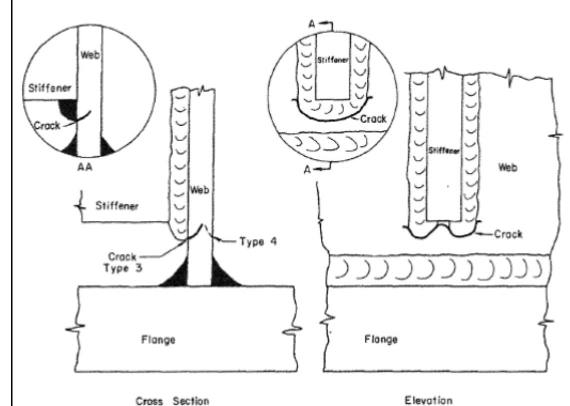
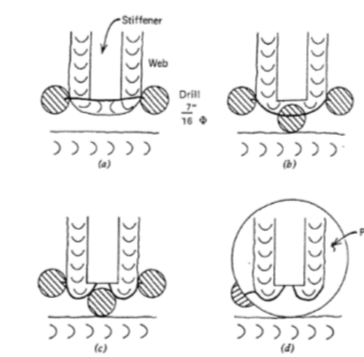
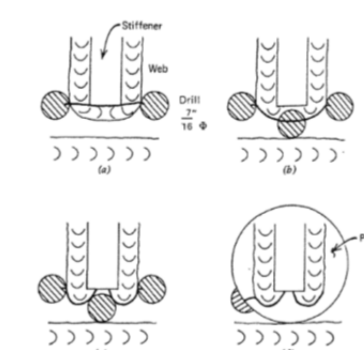
History of Out-of-Plane Distortion Induced Cracking

As a result of the steel girder cracks which occurred during erection, Professor John Fisher was contacted in 1973 to perform a fatigue evaluation of the superstructure.

As a part of the 1973 investigations, cores/coupons were obtained at several crack locations and fractographic examinations of the crack surfaces were performed.

All the cracks were found to be fatigue related and were determined to be caused by the relative out-of-plane distortion and bending of the short web length between the end of the stiffeners and the web-to-flange fillet weld. The cracks were primarily parallel to the longitudinal direction of the girder and to the bending stresses. The relative movement was caused by the cyclic swaying motion of the girders while in transit to the construction site and/or wind-induced motion during storage on the ground. The following exhibits summarize the types of cracks found and the repairs that were performed as summarized from Fisher (1984).

<table>
<tr><th>Crack Location</th><th>Crack Characteristics</th><th>Repairs</th></tr>
<tr><td>Between the stiffener weld and web.

</td><td>This type of crack was parallel to the primary bending of the girders and constituted 80% to 90% of cracks noted in the girders. These cracks were not through cracks and noted on the near side of the web. These cracks were caused by cyclical applied loads.</td><td>No repairs were performed for this type of crack. It was determined that these cracks would not propagate any further.</td></tr>
<tr><td>Across the welds

</td><td>This type of cracks were noted at the end of several stiffeners and in some instances extended completely across the weld and penetrated into the web.</td><td>The cracks that propagated into the weld near the stiffener end were removed by grinding out the crack. The ground-out area was examined with dye-penetrant to confirm the removal of the crack tip.</td></tr>
</table>

<table>
<tr><th>Crack Location</th><th>Crack Characteristics</th><th>Repairs</th></tr>
<tr><td>At fillet weld toes

</td><td>These types of cracks were noted at the weld toes at the end of stiffeners adjacent to the tension flange. The sample cores indicated that the crack had propagated into the web at the end of the fillet weld and turned and moved up the web after a short distance into the web.</td><td>Repairs were done by drilling 7/16” diameter holes at the end of each crack.

</td></tr>
<tr><td>Web surface opposite side of the stiffeners</td><td>These types of cracks were noted in the web surface opposite the stiffener. The cracks originated on the web surface and did not join the crack propagating into the web from the other surface at the end of the transverse stiffener.</td><td>Repairs were done by drilling 7/16” diameter holes at the end of each crack.

</td></tr>
</table>

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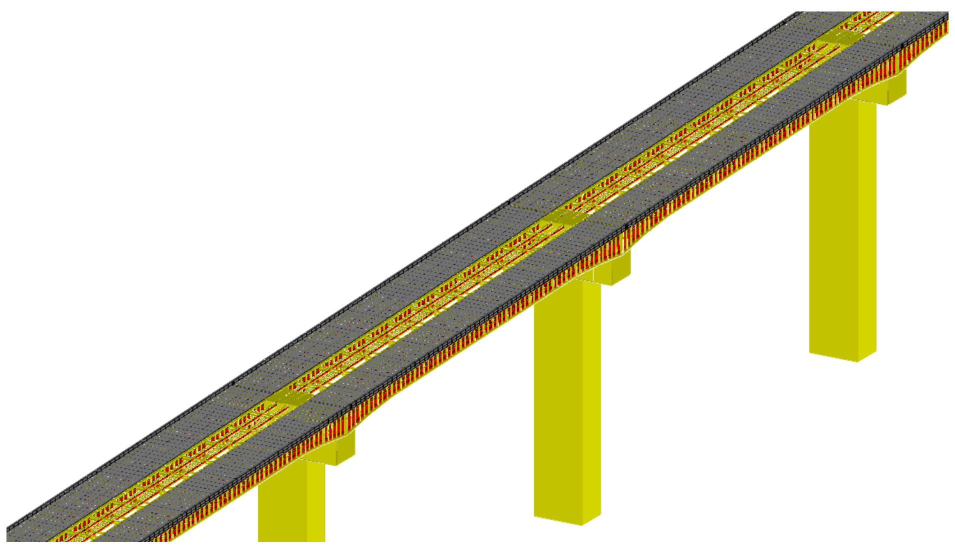
Maintenance of Traffic Schemes

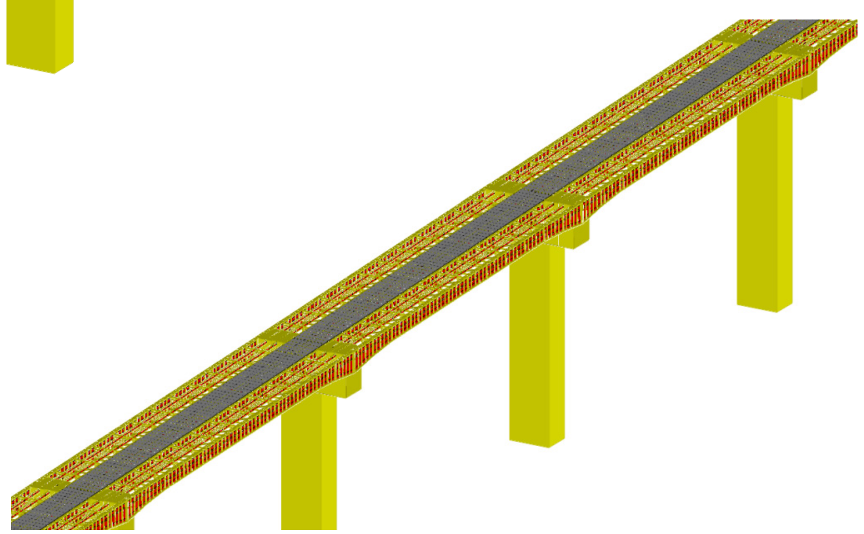
Following the creation of the 3-D model, two maintenance of traffic schemes were evaluated for the part-width replacement of the existing deck. Alternative 1 was designated as (5+2) and Alternative 2 as (5+1).

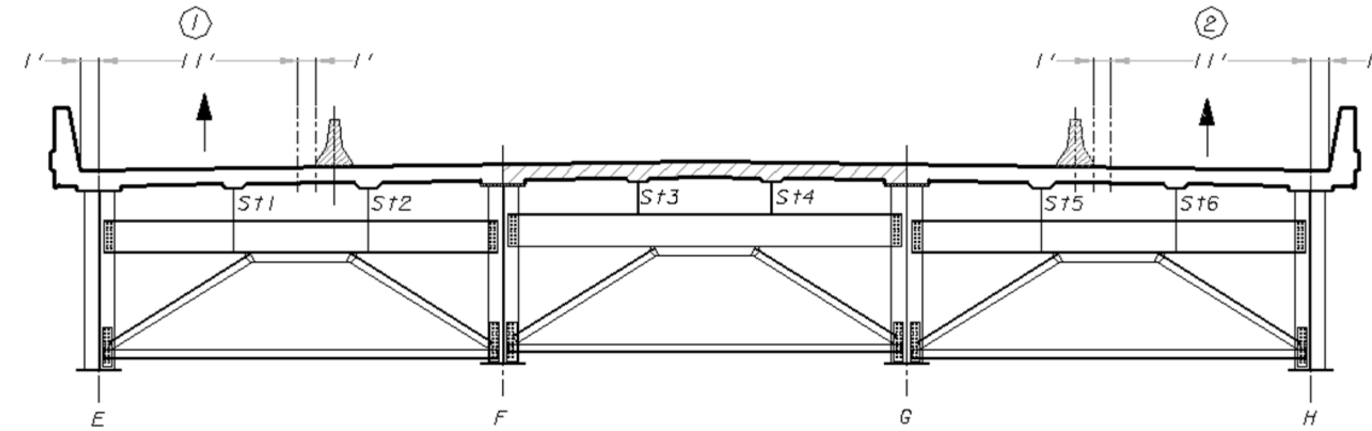
Alternative 1 (5+2)

For this alternative, five lanes of traffic are placed on the left bridge while 2 lanes of traffic are placed on the right bridge. The right bridge deck removal and replacement is carried out in two phases as shown. In phase 1, two lanes of traffic are placed on the right bridge while constructing a composite deck in the middle bay. In phase 2, traffic lane is placed in the middle of the deck while constructing both outer bay composite decks.

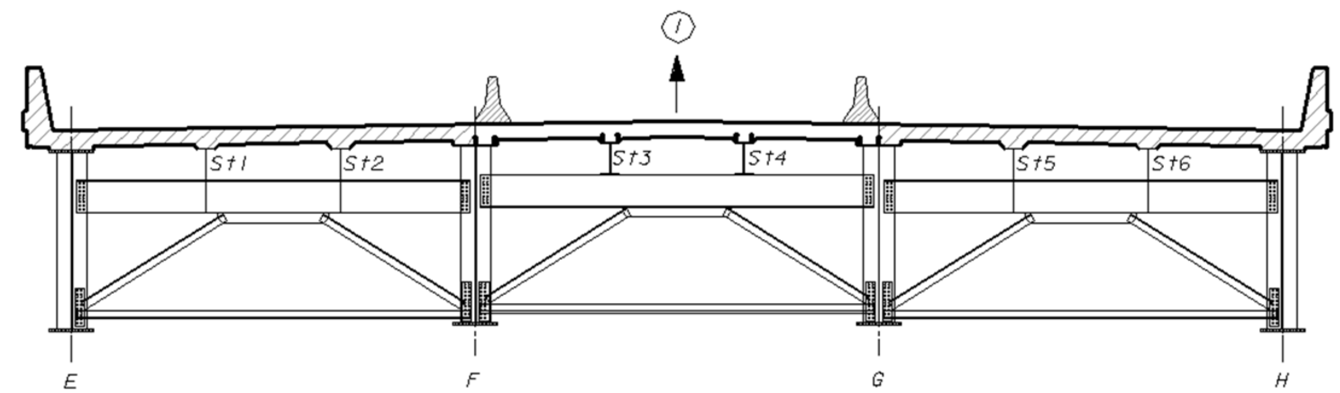
Phase 2 has a composite deck in the middle which is constructed in phase 1 of this alternative as shown below.







Alt. 1 - Phase 1

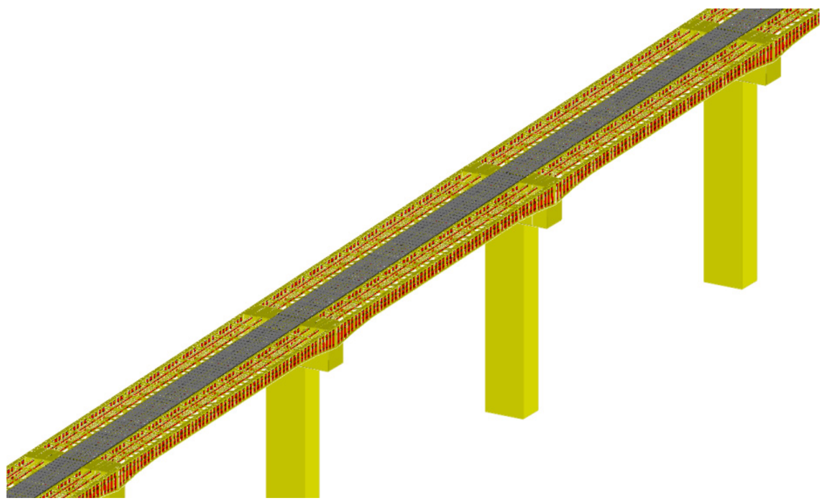


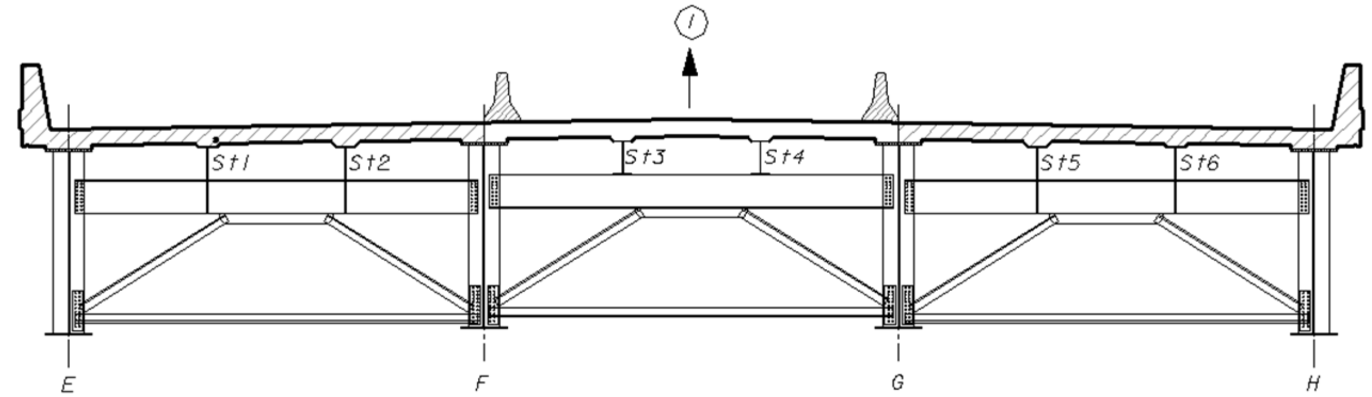
Alt. 1 - Phase 2

Alternative 2 (5+1)

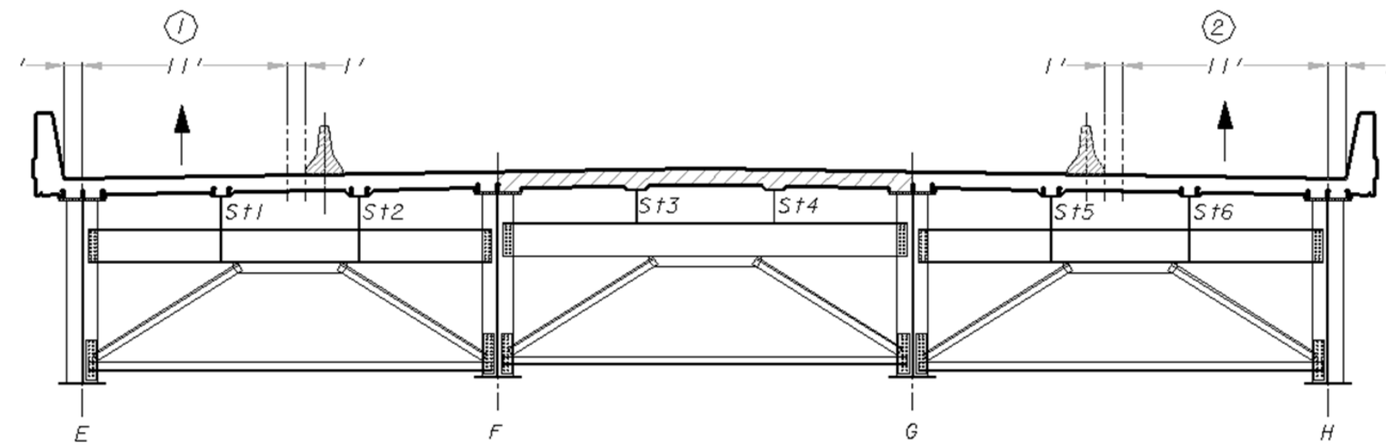
For this alternative, five lanes of traffic are placed on the left bridge while the deck is replaced on the right bridge. This alternative is also carried out in two phases as shown. In phase 1, one lane of traffic is placed in the middle bay of the right bridge while the outside bays are removed and replaced with a composite deck. In phase 2, one lane of traffic is placed in each on the newly constructed portions of the deck while the middle bay is replaced.

Alternative 2 (5+1) is the scheme that is studied in more detail in Part 1 based on direction provided by ODOT at the Nov. 4, 2011 meeting.

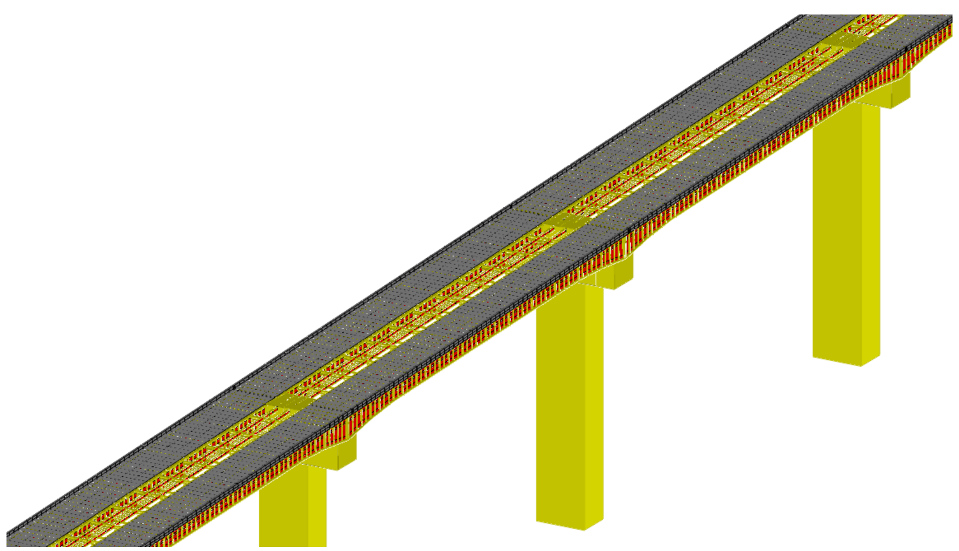




Alt. 2 - Phase 1



Alt. 2 - Phase 2



II. 3-D Finite Element Modeling

ELR reviewed the original design plans, the rehabilitation plans, and the latest inspection report prior to beginning development of the 3-D FE Model. The CAD model was generated through the use of the framing plan, horizontal curves, vertical curves, deck dimensions, girder locations, pier positions, and abutment geometry from the existing plans.

The 3-D FE modeling was accomplished as follows:

1. Build and analyze the entire right bridge for the existing conditions and MOT phases using LARSA 4D. The resulting model was defined as a coarse-model.
2. Create sub-models of regions: near the pier and mid-span using the LUSAS program.
3. Obtain the forces and moments at the boundary nodal locations from the coarse-model.
4. Apply boundary conditions to the sub-models. Analyze sub-models and ensure compatibility of the deformed shape of the girders/stringer and stringers between the sub-models and the coarse-models.

Because the CUY-480-18.42 bridges are relatively large structures, out-of-plane analyses could not be performed utilizing only a course-model, thereby, requiring a coarse-model and sub-model evaluation procedure.

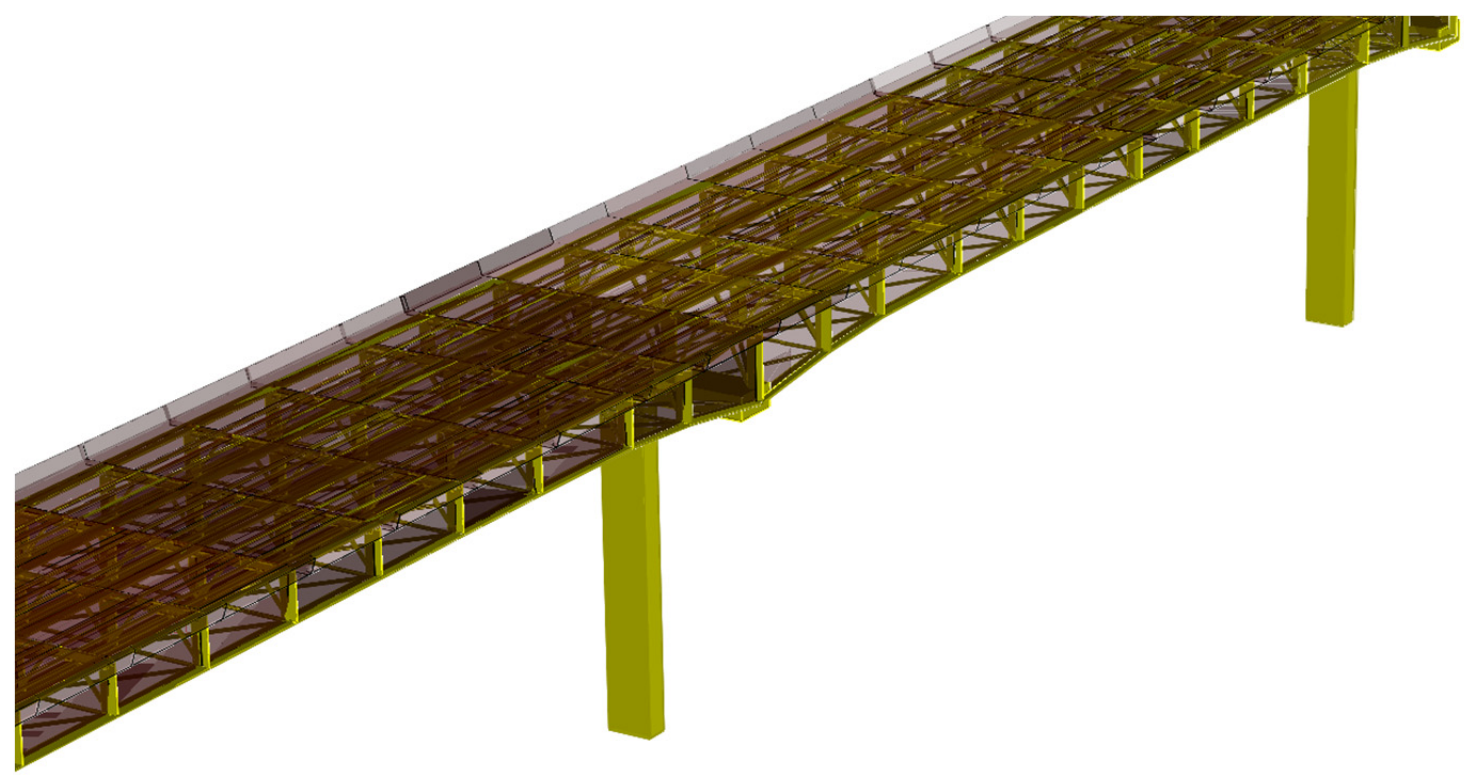
The LARSA 4D computer program (version 7.05.35) was used for the 3-D Finite Element Modeling of the eastbound structure (Right Bridge). In the 3-D coarse model, flanges of girders, floorbeams and stringers were modeled using line/beam elements while the webs were modeled using plate/shell elements. The cross frames and lateral bracing were modeled using line/beam elements.

Typically, the effects of the configuration and the stiffness of substructures on the behavior of the superstructure are insignificant and can be safely neglected in the superstructure analysis. As noted in the 2009 inspection report, the bearings show no sign of movement (the paint over the junctions between the gears of the rollers and the racks above and below the rollers is not cracked). The same inspection report also indicated that noticeable sway (back and forth) of the tall piers is observable when a person is standing or sitting on the piers while a truck passes overhead.

Due to the complex size of the bridge, the modeling of the live loads presented several computational challenges.

Model	Influence Surface/Line	DOF	Lanes	Number of Moving Load cases	Computation Time (Run Time)	Disk Space
Coarse Model - Existing Conditions	Surface	289350	5	14800	16 days	500 (GB)
Coarse Model - Existing Conditions	Surface	289350	2	3552	5 days	201 (GB)
Coarse Model - Existing Conditions	Line	289350	5	5914	5 days	200 (GB)
Coarse Model - Existing Conditions	Line	289350	2	2372	2 days	100 (GB)
Coarse Model of Unit 2 ONLY - Existing Conditions	Surface	44622	5	3225	5 Hours	24 (GB)
Coarse Model of Unit 2 ONLY - Alt 2 – Phase 1	Surface	34352	1	1280	2 Hours	9 (GB)
Coarse Model of Alt 2 – Phase 1	Surface (Short)	226602	1	860	14 Hours	20 (GB)
Coarse Model of Alt 2 – Phase 2	Surface (Short)	283140	2	1720	24 Hours	45 (GB)

The above summary documents the unusually long computational time periods needed to evaluate each loading condition. These relatively long computational time periods, which result from the large size of the 3-D model, limited the study of the presence of live loads in only spans 10, 11 and 12. The detailed evaluation of out-of-plane distortion was focused in span 11.



No. of Nodes = 48,226

No. of Shells = 36,563

No. of Beam Elements = 41,602

Development of the Coarse Model

This section provides the background for the development of the coarse model. The 3-D FE model consists of five units matching the units shown in the construction plans. Units 2, 3, and 4 are identical to each other. Each of the five units were built in separate files which were then combined as a single LARSA model file.

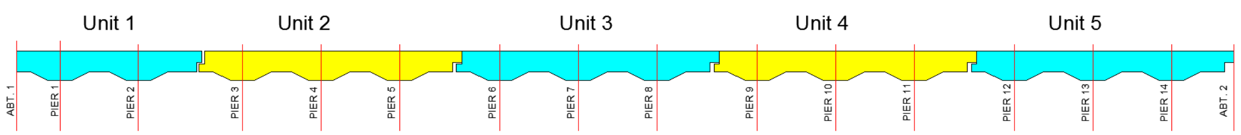


Table 1 Units, Spans, and Spans Lengths

Unit	Spans	Length (feet)
Unit 1	1,2,3	220, 300,300
Unit 2	4,5,6	300,300,300
Unit 3	7,8,9	300,300,300
Unit 4	10,11,12	300,300,300
Unit 5	13,14,15	225,225,180

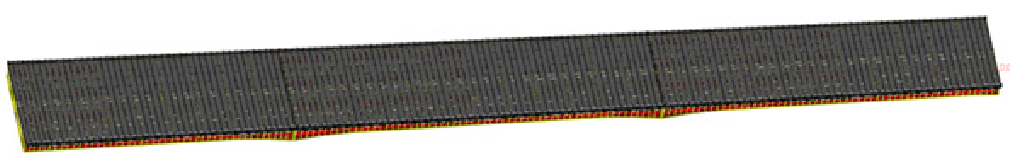


Figure 1 *Unit 1*

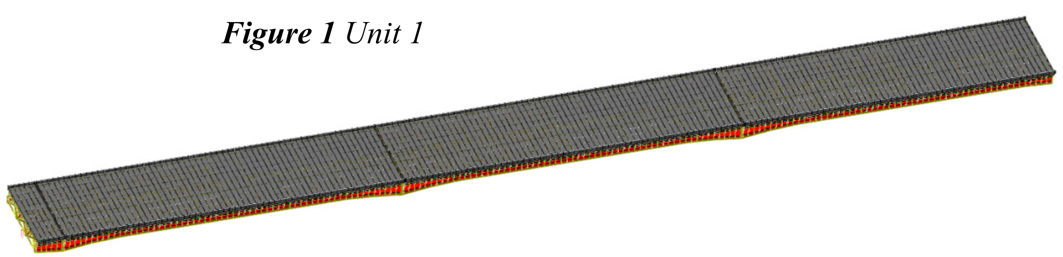


Figure 2 *Units 2, 3 and 4*

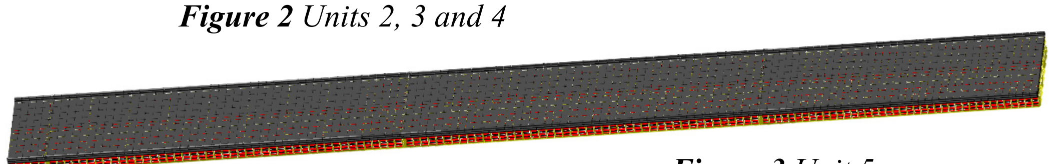


Figure 3 *Unit 5*

Modeling of Girders and Cross Frames

The webs of the girders and the stringers were modeled using 4-node shell elements while the flanges were modeled using beam elements. The cross frame chords were modeled using beam elements. Bearing stiffeners, intermediate stiffeners, and longitudinal stiffeners were modeled using beam elements.

Figures 4 and 5 show detailed views of one cross frame as modeled in LARSA. The 3-D viewing option was used in Figure 5 to show the graphical representation of the flanges and stiffeners.

The deck slab and barriers were modeled using 4-node shell elements. The pier columns, pier cap, and pier bearing were modeled using beam elements as shown in Figure 6.

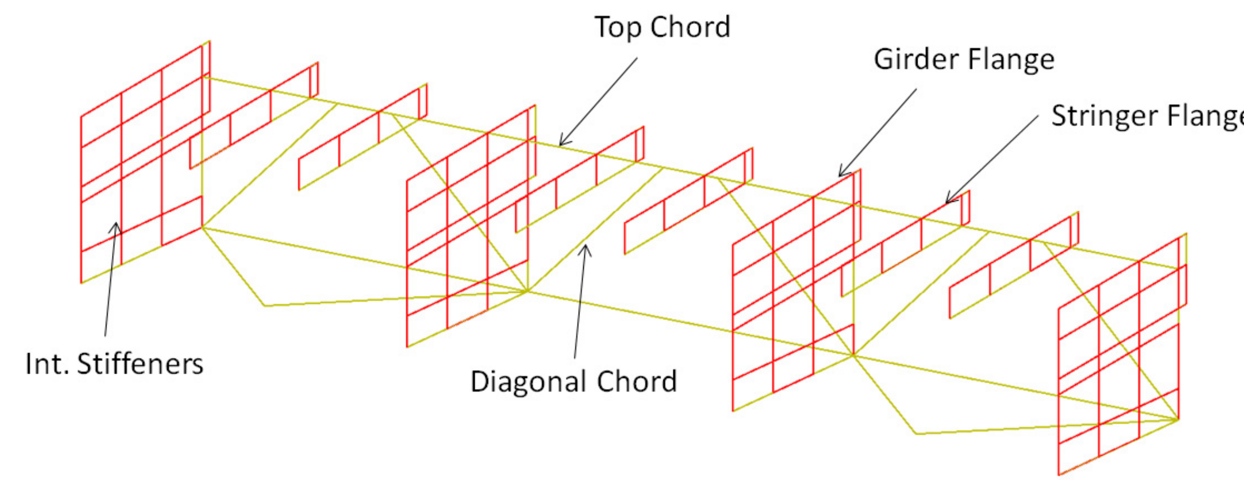


Figure 4 *Modeling of Girders, Stringers and Cross frames*

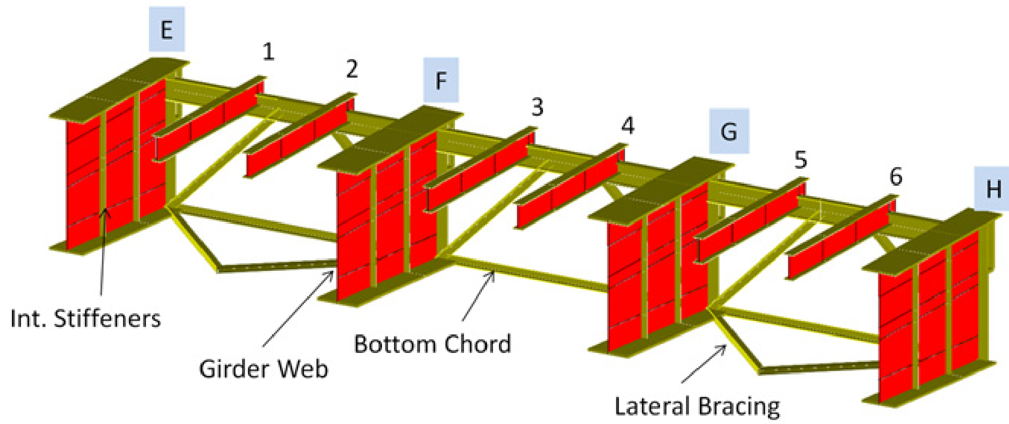


Figure 5 *3-D view of Girders and Cross Frames*

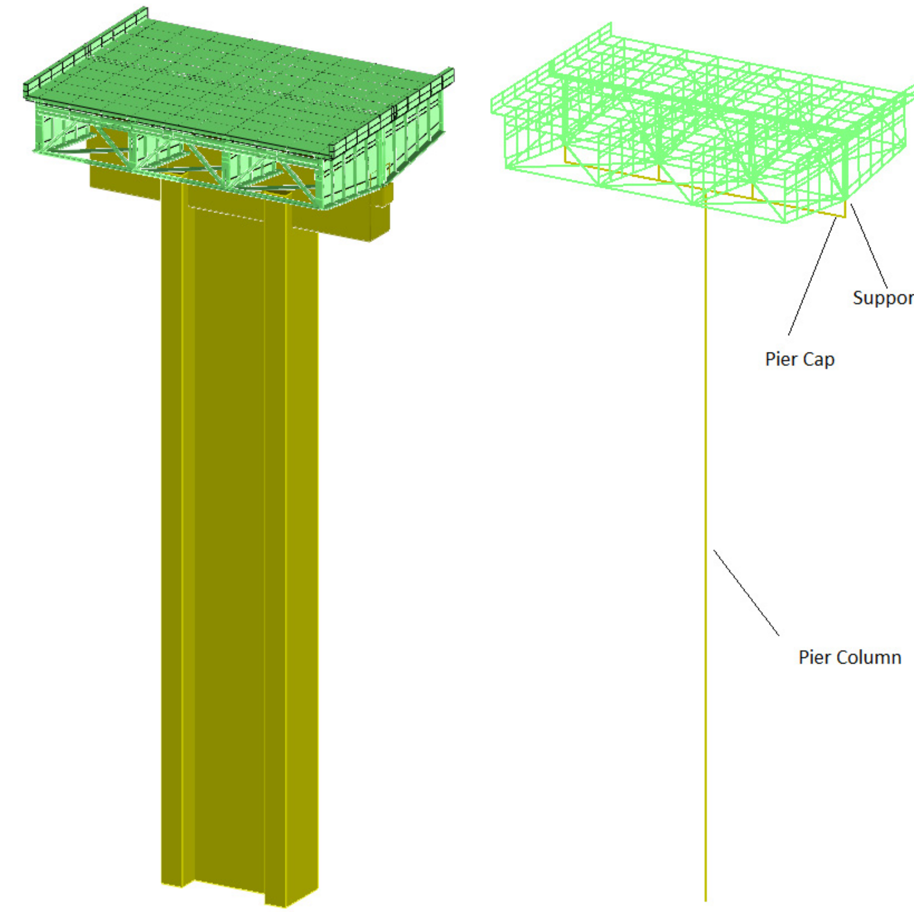


Figure 6 *Modeling of Pier Column and Cap*

A fixed end boundary condition was specified at the bottom of the relatively tall pier columns. The actual fixity is located below the pier footing. Soil-structure interaction modeling was not performed because the actual location of the fixed end boundary condition does not change the performance of the model for these tall piers.

The master-slave connection option available in the LARSA program was used to model the hinges as shown in Figure 7. Forces (Fx, Fy and Fz) are allowed to transfer but Moments (Mx, My and Mz) are not transferred.

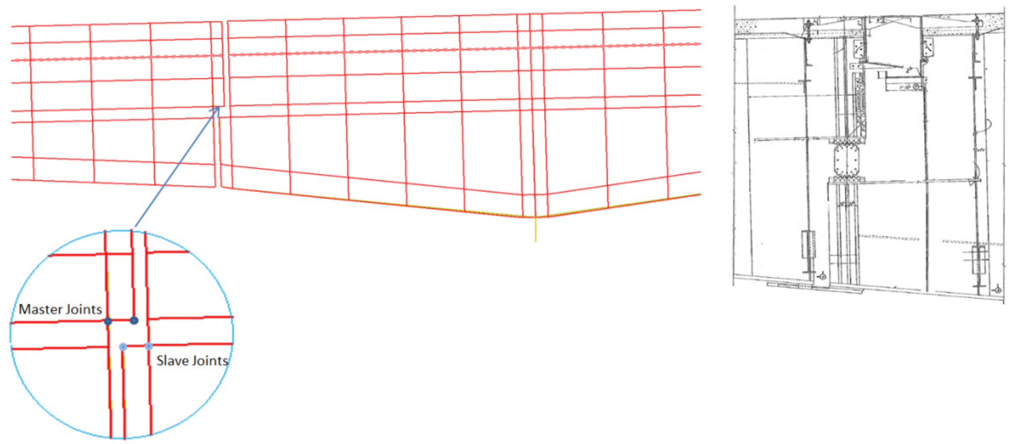


Figure 7 *Modeling of the Hinges*

The height of the bearings was also modeled. The moments were released and forces are transferred to the pier cap as shown in Figure 8.

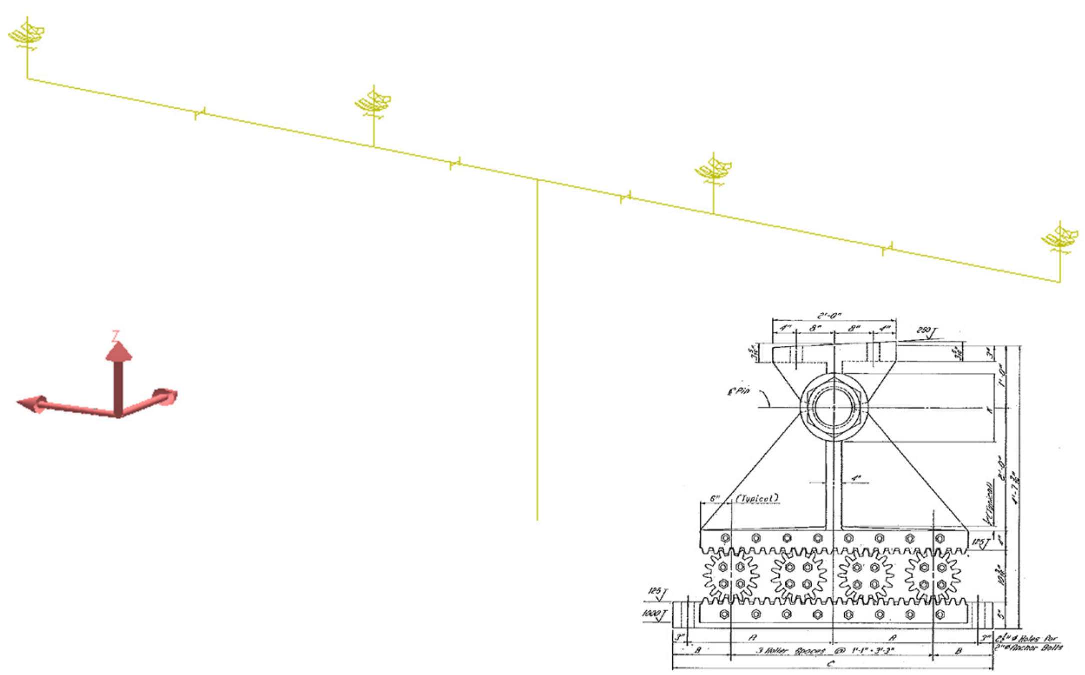


Figure 8 *Modeling of the Bearings*

Moment Release of Diagonal and Lateral Bracing

The moments at both ends of the diagonal and lateral bracing were released. Figure 9 shows the locations of the released moments at the connections between the diagonal members. The diagonals for this design, which consist of a non-skewed and tangent condition, are treated as truss ended or truss members, not as flexural members. This is typical when developing models of this nature (see P 58 - NSBA/AASHTO's Guidelines for the Analysis of Steel Girder Bridges). This cross frame modeling approach was decided at an ODOT Central Office- Office of Structures meeting requested by ELR.

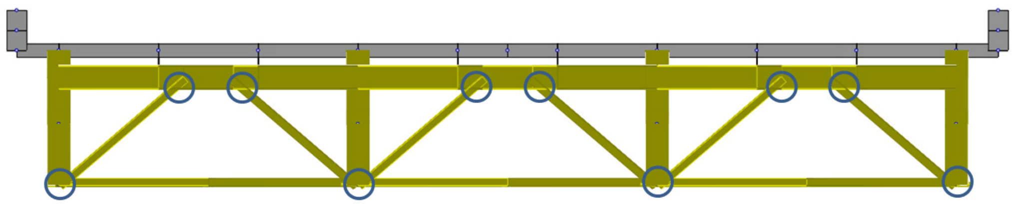


Figure 9*CrossframeDetail (Released Moments)*

Modeling of Non-composite and Unintended Composite Action

The existing superstructure was originally designed assuming that the reinforced concrete deck would perform as a non-composite superstructure component. The most accurate finite element approach for modeling the assumed non-composite behavior is to perform a contact analyses. This procedure takes into account the friction developed between the reinforced concrete deck and the steel girder. However, this modeling approach is very complex and requires significant computer resources.

Non-composite bridges generally exhibit composite action under service loads due to the chemical bond and the friction between the two different materials. This composite action can range from fully composite to completely non-composite, depending on the actual deck to stringer details, the years of service, and the loading conditions.

In our sub-models, both composite and non-composite behaviors were evaluated. Linear coupled degrees of freedom (master/slave options) were used for modeling this behavior.

For non-composite behavior, it is assumed that the corresponding deck and girder contact surface nodes will displace the same in the vertical, longitudinal and transverse directions, but independently in the three rotational directions (rotation around vertical, longitudinal and transverse directions). The deck bottom surface and the girder top flanges are meshed with a series of coincident nodes. Each pair of coincident nodes are linked together for the transverse (Y), longitudinal (X) and vertical (Z) degrees of freedom (DOF), but are "untied" for the rotation around X, Y and Z DOF so that the model can behave non-compositely.
For composite behavior (top flange is restrained by the deck), it was assumed that the corresponding deck and girder contact surface nodes would displace and rotate the same in the vertical, longitudinal and transverse directions. The deck bottom surface and the girder top flanges are meshed with a series of coincident nodes. Each pair of coincident nodes are linked together for the transverse (Y), longitudinal (X) and vertical (Z) DOF for rotation and translation.

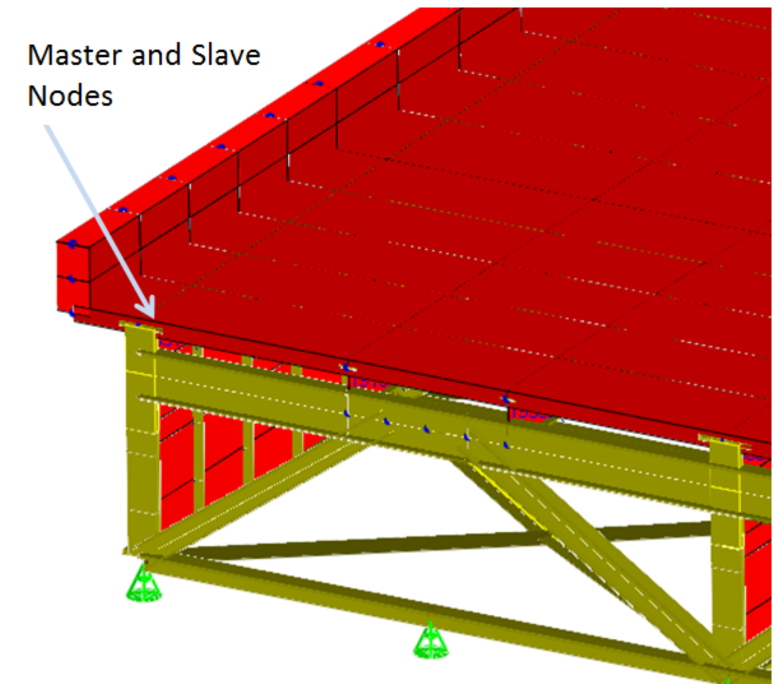


Figure 10 *Modeling of Non-composite Action*

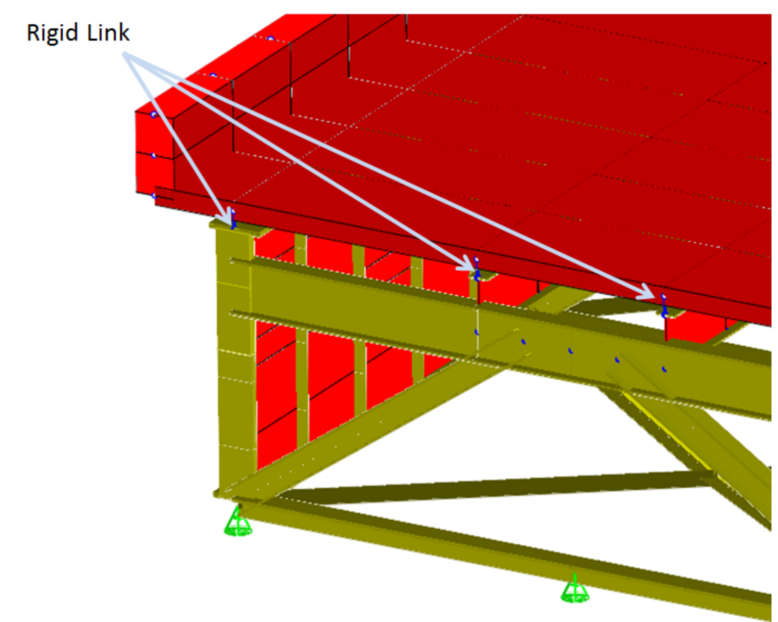


Figure 11*Modeling of Composite Action*

Geometry Grouping

LARSA 4D allows breaking the model into groups based on geometry thereby making the development of the model more manageable and easier to review. In LARSA 4D, groups can be any selected geometric objects in the structure such as points, beams, or plates.

The structure groups allow for ease of assigning material properties and help with managing the entire model for results.

In the FE model, there are five main groups for each of the units. Under each unit, groups of parameters were defined. The chart in Figure 12 shows an exploded view of the Unit 1 folder.

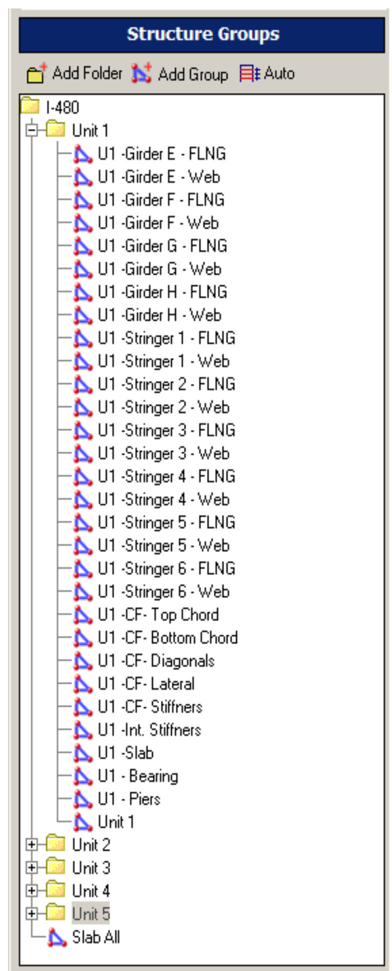
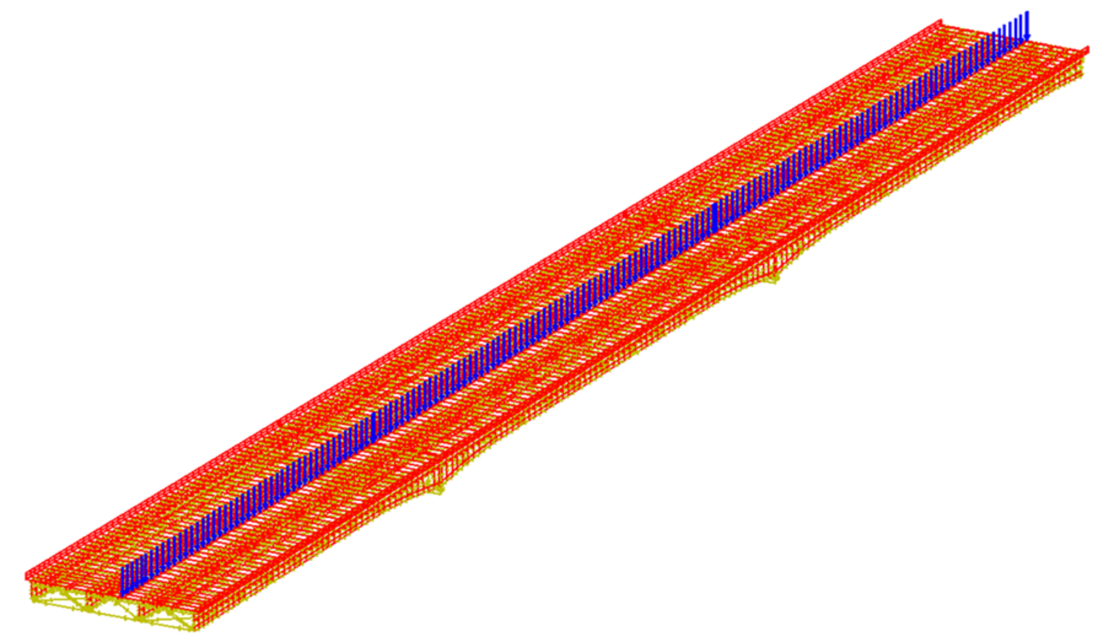


***Figure 12** Groups*

Loads

Dead Loads

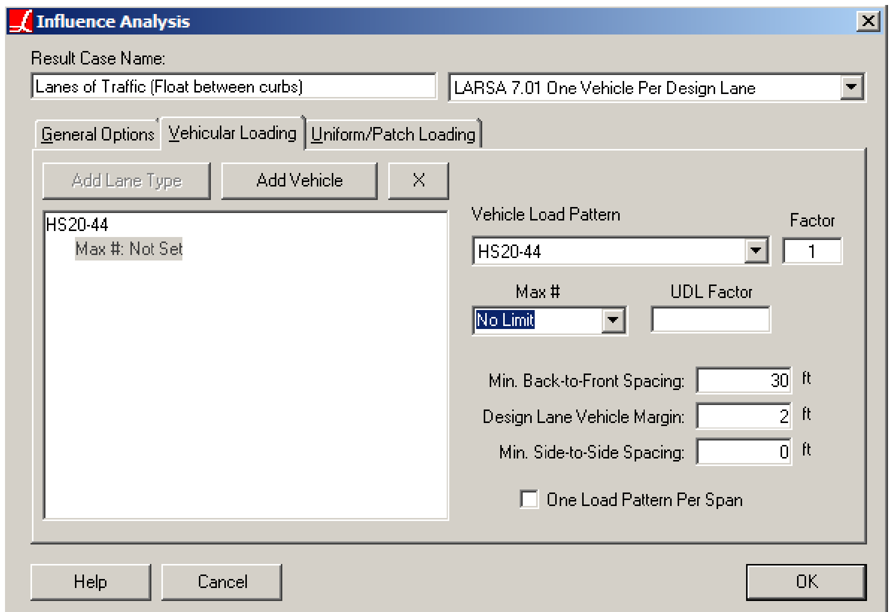
The temporary barrier dead load used for the MOT was applied to the model. The barrier was modeled as shown in the image of the superstructure provided below.



The self-weight of the structure is computed internally by the SELFWEIGHT load case available in the computer program.

Live Loads

The HS20- Truck, the HS20- Lane, and a Train of HS20 Trucks were used to apply live load to the computer model.



The HS20-44 truck and lane loads are defined in the LARSA program.

In accordance with Section 6B.7.2 of the AASHTO Manual for Bridge Evaluation (2nd Edition), for spans over 200 feet in length, a train of HS20 trucks should be considered. Trucks are spaced with 30 feet clear distance between vehicles to simulate a train of vehicles in one lane. A number of analyses were made to establish the truck spacing that will generate the maximum live load stresses.

Influence Surface Analyses

The common approach for influence line analysis is to lump each axle or wheel of the vehicle on the centerlines of the members in the lane path, unless: 1) the lane is specified with offsets that take it away from the member centerline, 2) a transverse offset is specified in the influence result case options, or 3) the load pattern has transverse offsets specified on the wheels. In cases 2 and 3, a second set of influence coefficients, based on a one-unit torque, is used to compute the moment induced by the eccentricity as described in LARSA-4D documentation. Load distributions across multiple girders are accomplished either through rigid cross-beams connecting girders, or by loading multiple lanes simultaneously (see Figure 13).

With the influence surface method, load distribution is accomplished automatically. Plate decks by their nature will spread load from the point of contact with the wheel throughout the deck, and to any connected elements.

The first step in defining the live load analysis is to define the traffic lanes. Lanes can be defined by selecting a series of lines/plates along the traffic path. The traffic lanes are adjusted to fit the actual traffic paths, as shown in Figure 13.

When using the standard solution method, LARSA 4D places as many lanes as will fit on the surface, according to the width of the surface specified in the input geometry, simultaneously maximizing the live load effect according to any multiple presence factors.

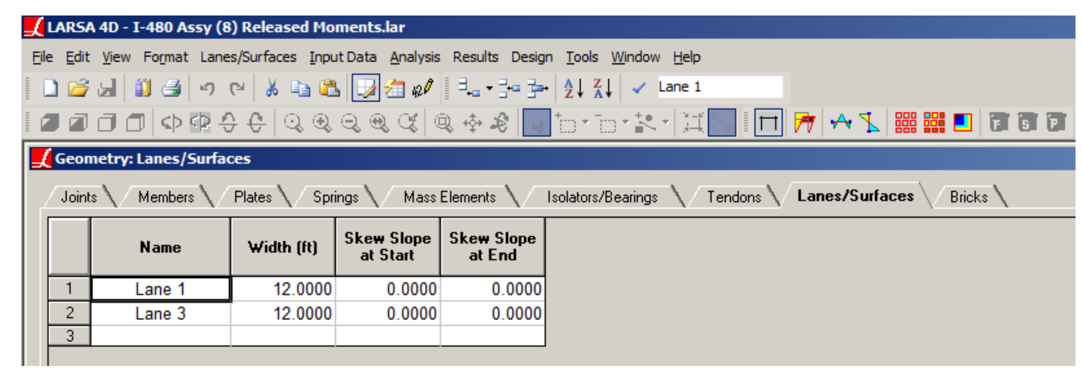


Figure 13 *Lane Definition (1)*

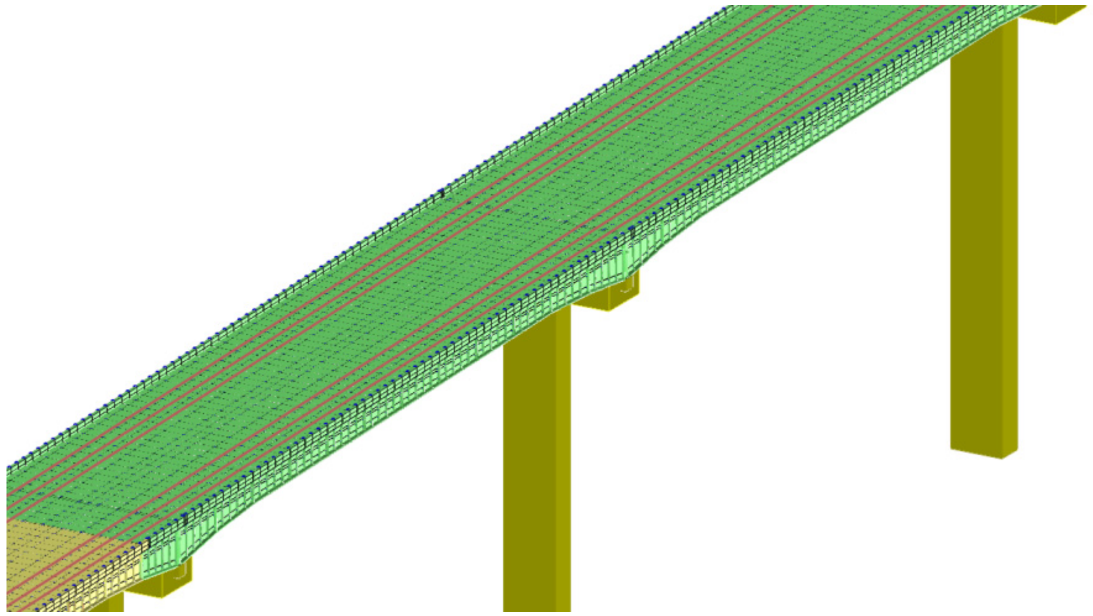
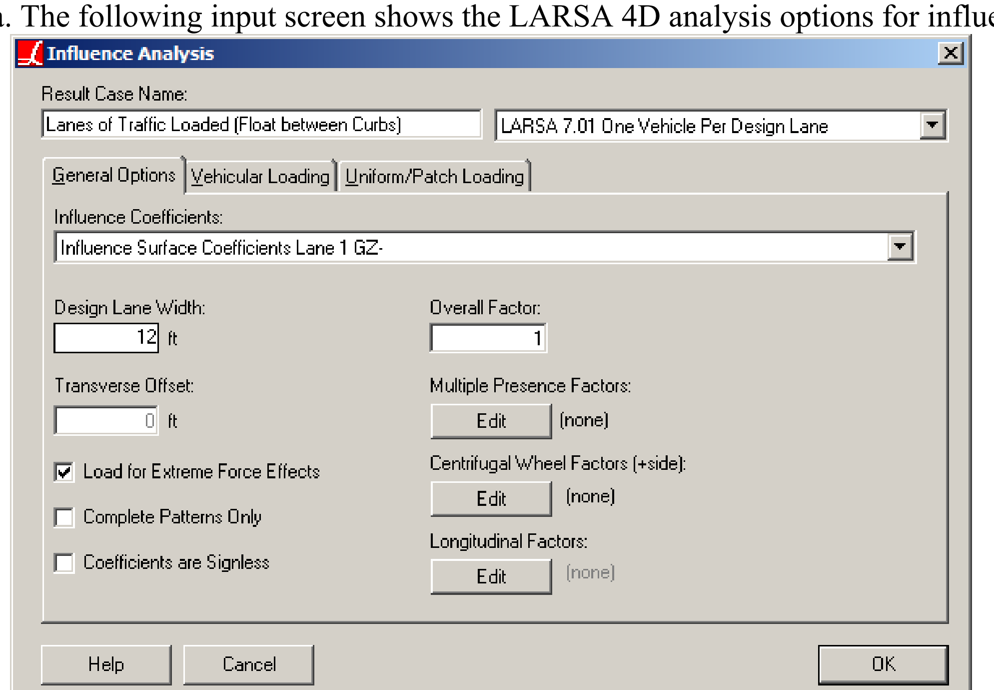


Figure 14 *Depiction of Lanes*

The second step in the analyses is to define the influence loads. For each lane, a moving load case is defined in the LARSA program

The resulting load cases are created based on the moving load analyses by utilizing the influence line/surface data. The following input screen shows the LARSA 4D analysis options for influence analyses.



Methodology for the Sub-Model Development

The LARSA 4-D model of the entire bridge is defined in this report as a coarse model. A finer mesh (sub-model) was used for evaluating out-of-plane displacement in the superstructure components. Sub-models were prepared using LUSAS, because this program has advanced mesh generation features.

The maximum Dead Load (DL) moment was found to be at Pier 2, followed by the moment at the Pier 10 location. For analyses of out-of-plane displacement or distortion, regions in Span 11 were selected because this span does not have any hinges.

Sub-models capturing the relevant three-dimensional out-of-plane displacements are prepared for the following:

- Phase 1 of MOT Scheme Alt 2 (5+1). By initial evaluation, Phase 2 was found to be susceptible to lesser out-of-plane distortions
- Existing conditions (existing deck slab – considered both behaviors: non-composite and composite)

The sub-models prepared were for interior and exterior girders in both negative and positive moment regions of Span 11 as shown in Figure 15.

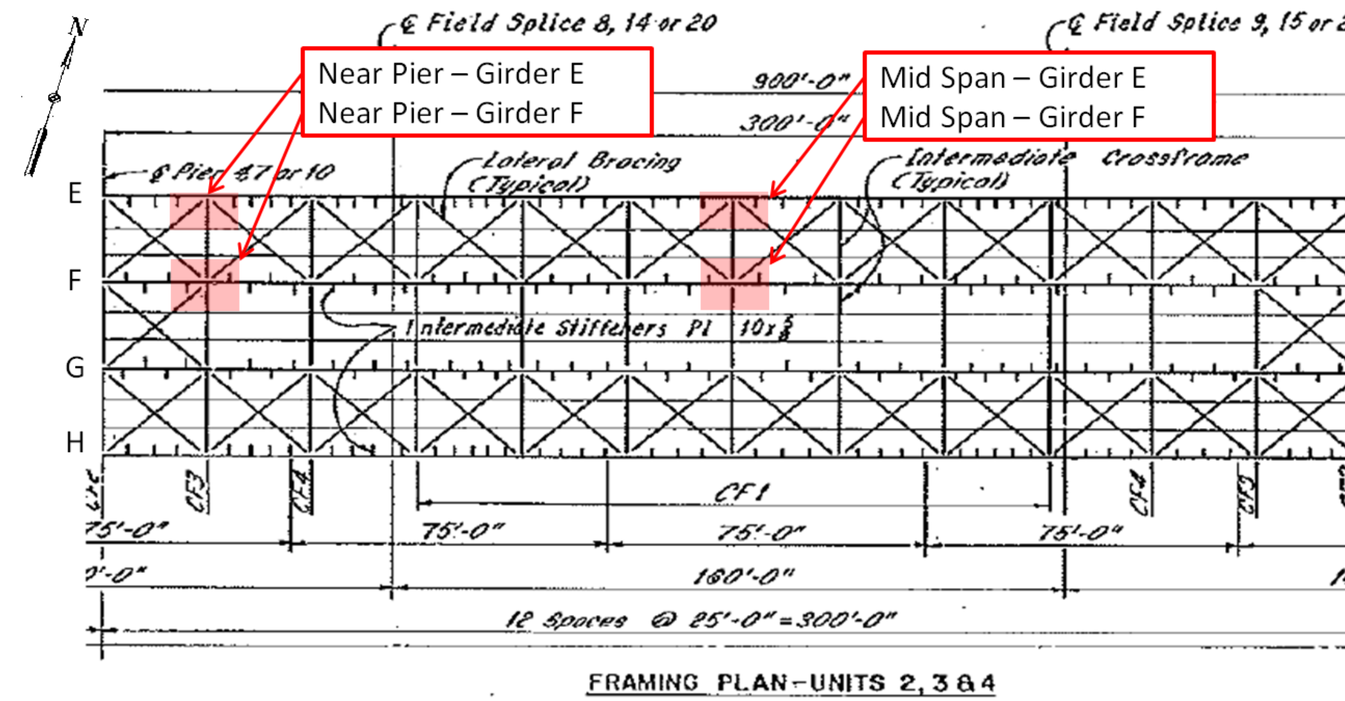
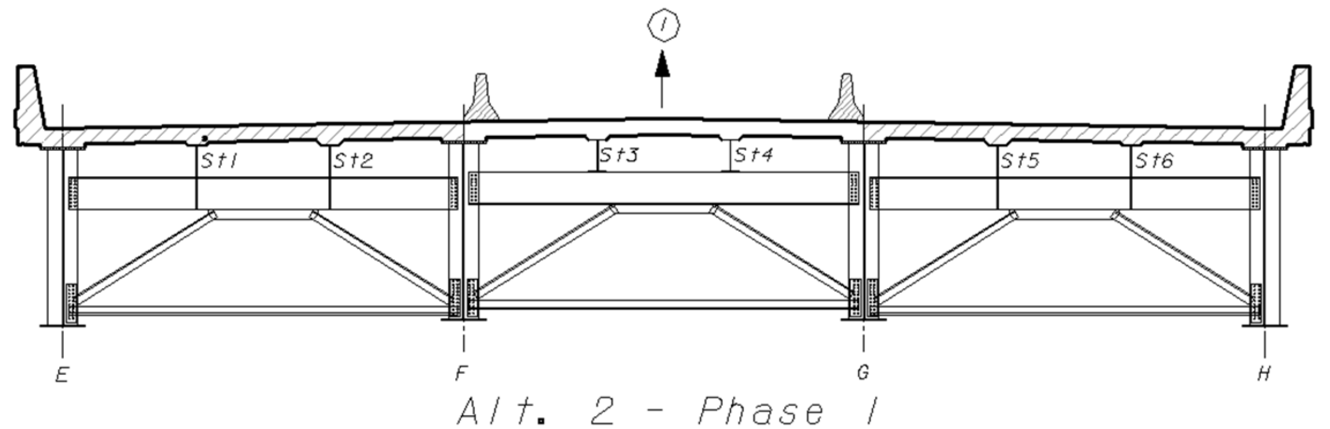
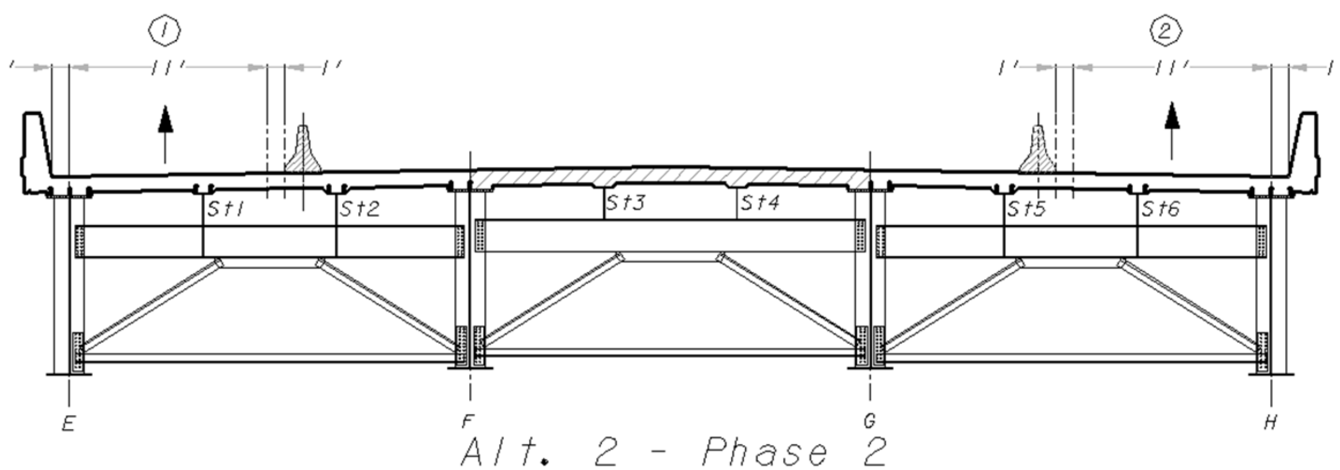


Figure 15 *Mid-span and Near Pier Sub-Models*

Modeling of Girders/Stringers and Floorbeams





The cut length for sub-models was 20 feet with half of the length (10 ft.) on each side of the connection plate.

All the plates (flanges, webs, stiffeners) of interior and exterior girders are modeled by 4-node thick shell elements as shown on this page.

Each of the sub-models has approximately 27000 4-node shell elements (361422 structural degrees of freedom for each of the sub-models)

Figures 16 & 17 identify plate names for plate thickness assignments of the various members.

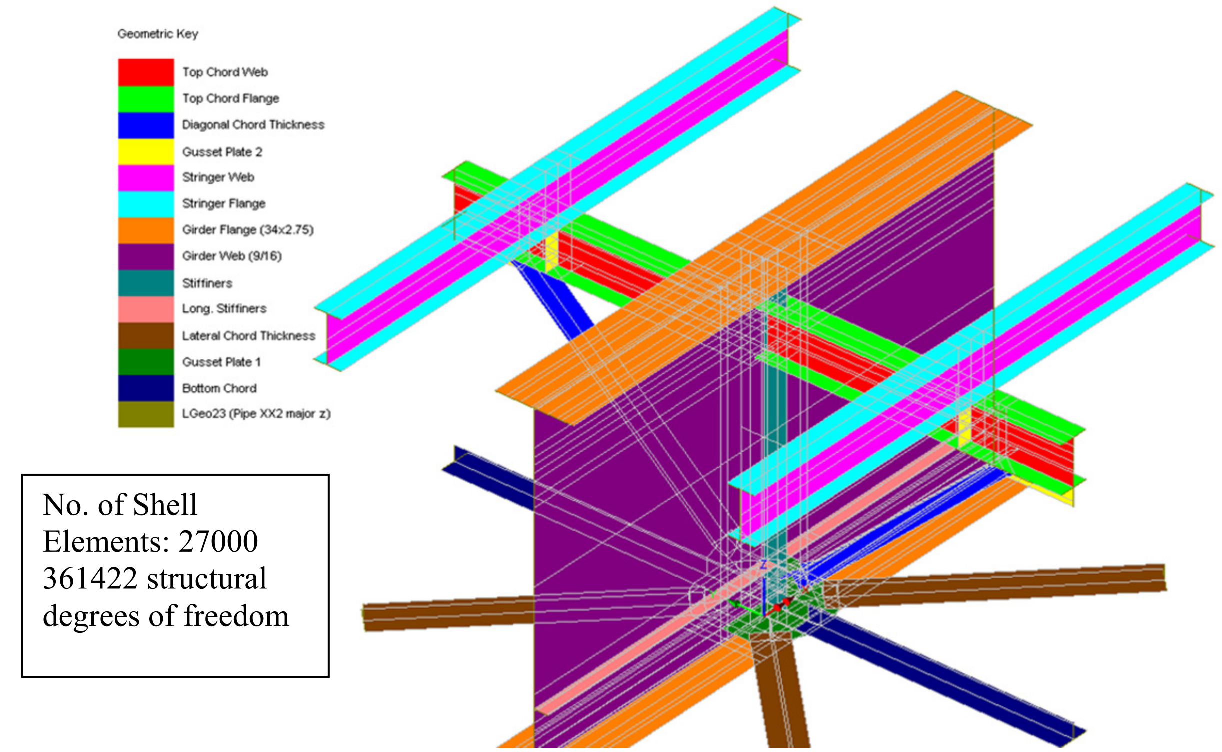


***Figure 16** Properties Assignment of the Interior Sub-Model*

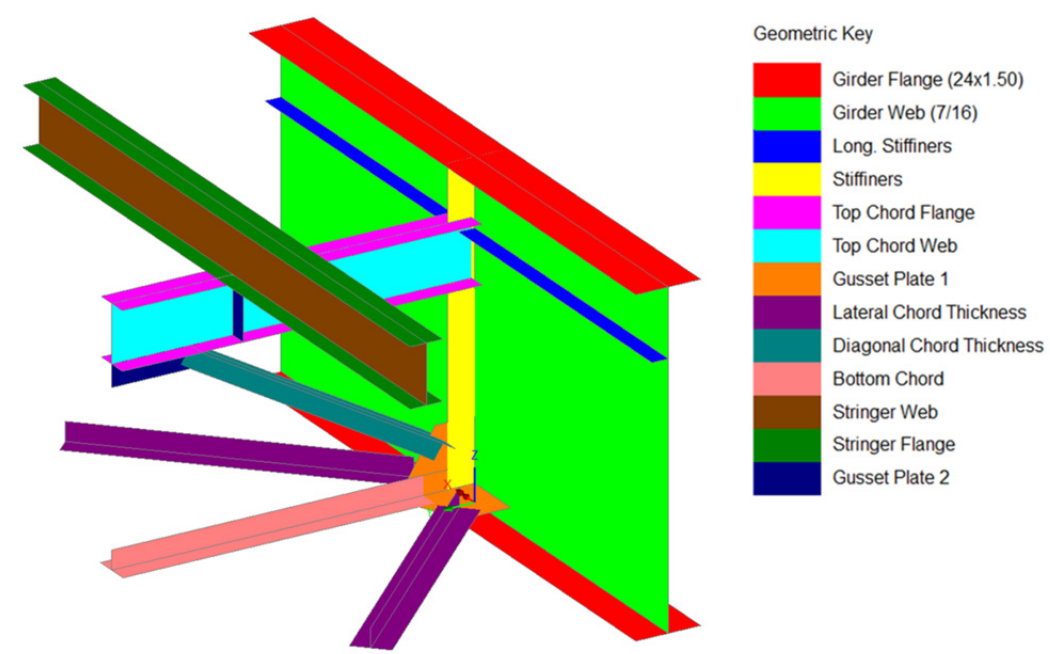


***Figure 17** Properties Assignment of the Exterior Girder Sub-Model*

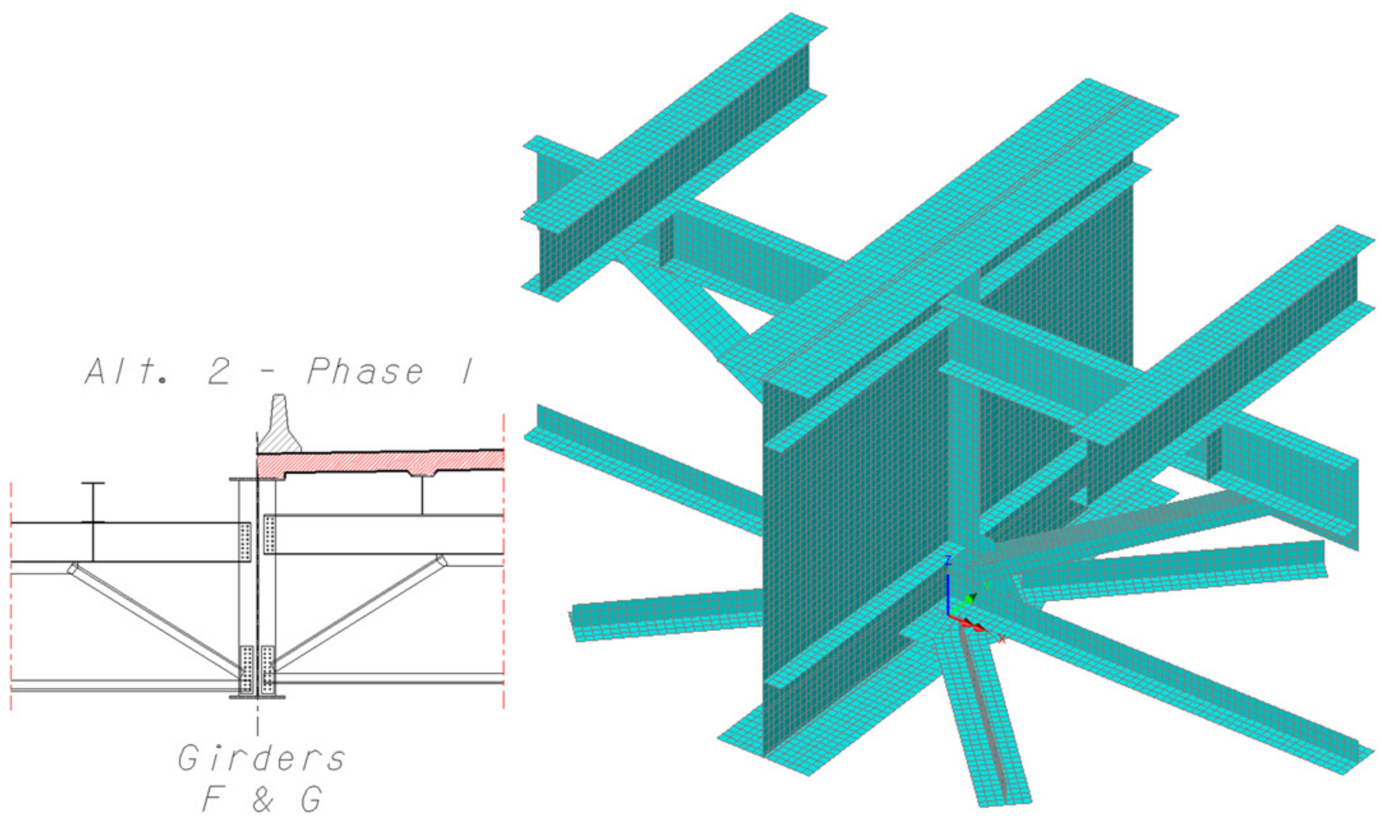


Figure 18 *FE model of Interior Girder Sub-Model*

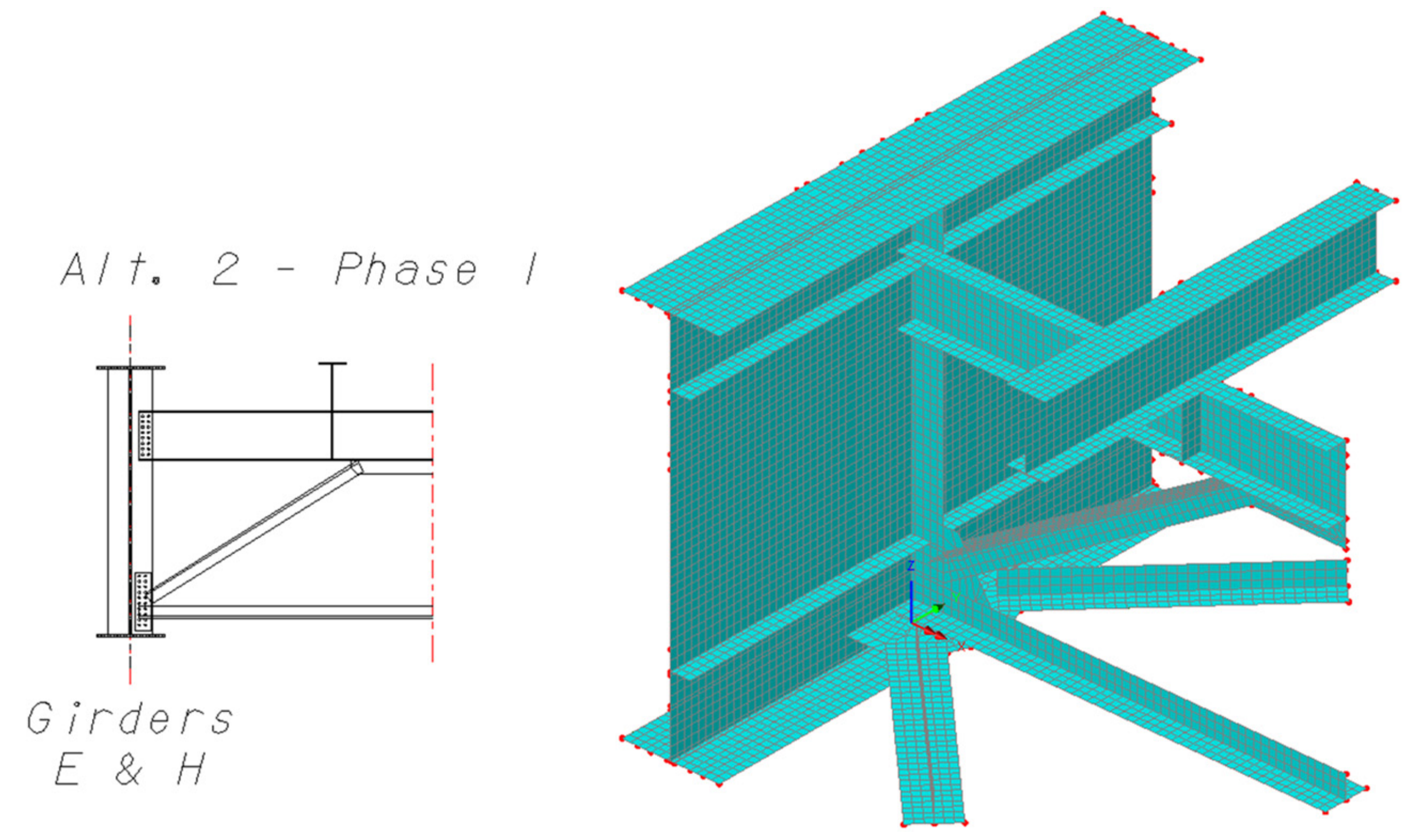
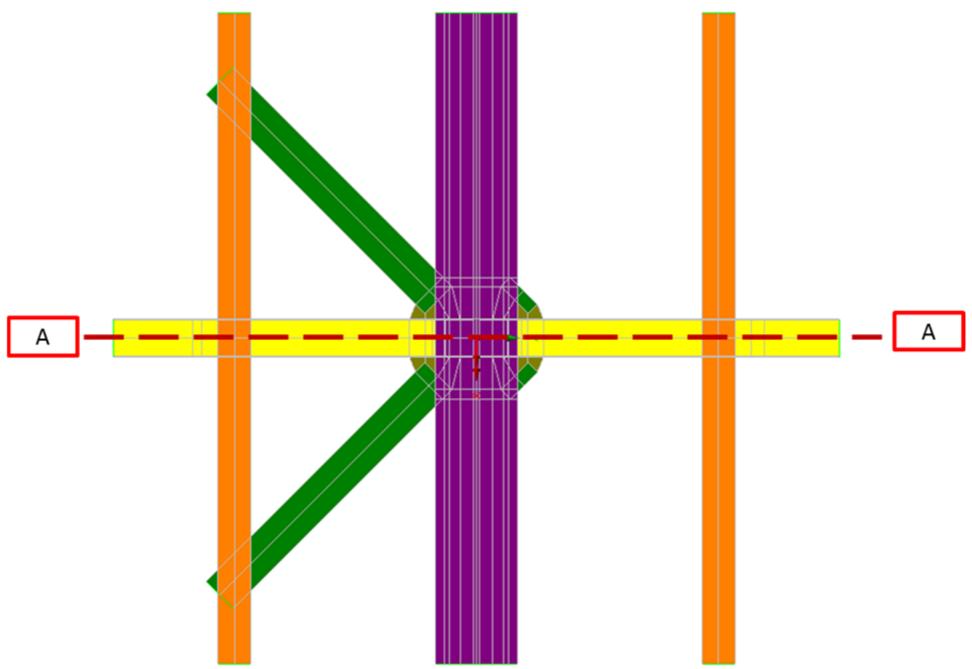


Figure 19 *FE model of Exterior Girder Sub-Model*

Boundary Conditions

Figures 20 and 21 depict the boundary conditions used in the sub-models. Tx = 0 indicates that translation is prevented in the X-direction. Rx = 0 refers to prevention of rotation in the x-direction. At the boundary, all conditions of statics were satisfied. The boundary conditions (Moments and Forces) for the sub-models were obtained from the full model of the structure (LARSA 4-D Model – Entire Structure – Alternative 2).



A-A is a line of Symmetry

Figure 20 *Boundary Conditions of the Interior Girder Sub-Model*

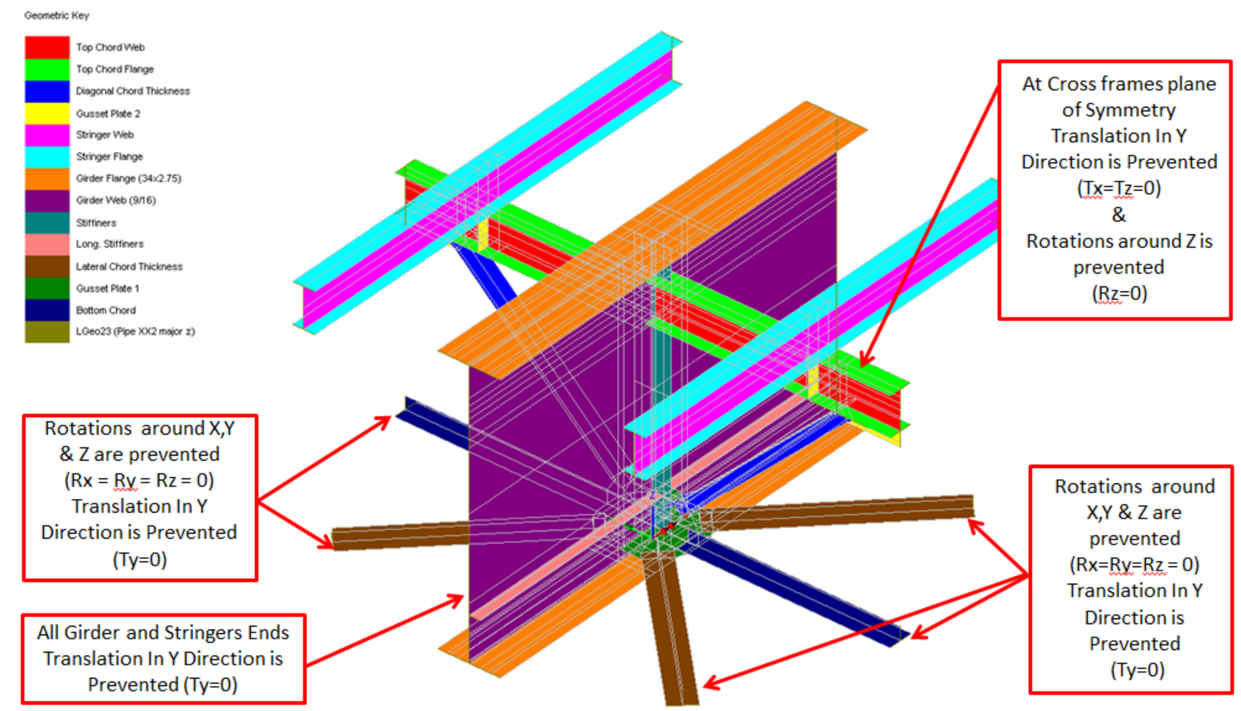


Figure 21 *Boundary Conditions of the Interior Girder Sub-Model*

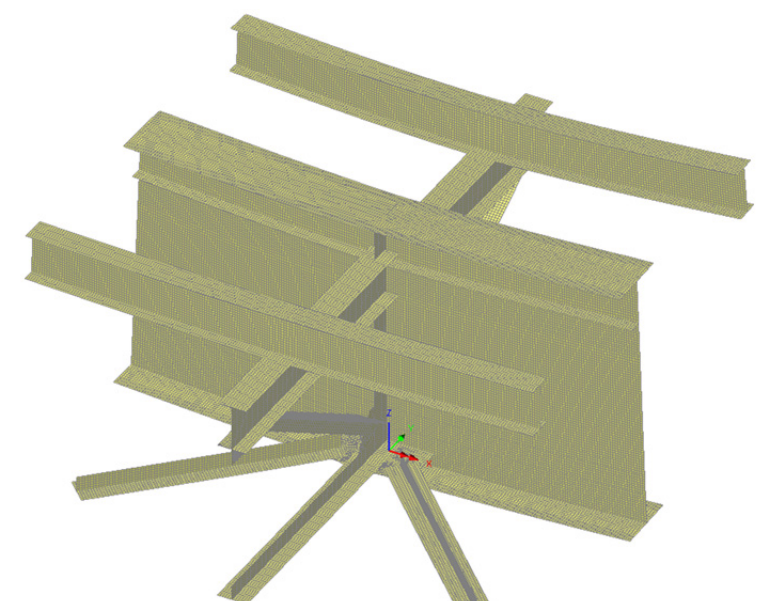


Figure 22 *Deformed Shape of the InteriorGirder Model*

III. Superstructure Analysis and Code Checking

The existing girders and stringers were checked with existing loading conditions and Alternate 2 MOT. The following checks were performed according to AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002. The results indicate all moment and shear stresses due to phase construction to be acceptable. The complete results are presented in Volume II of this report.

Shear Capacity Check

According to AASHTO Standard Specifications, shear is assumed to be carried by the steel web. The strength of web is determined by elastic methods.

The maximum shear in the web cannot exceed the plastic shear force, V_p, given in 10.48.8.1 of AASHTO Standard Specifications as follows

$$V_p = 0.58 F_y D t_w \qquad (10\text{-}115)$$

The spacing of intermediate transverse stiffeners is based on the shear capacity, V_u, defined in Article 10.48.8.1 of AASHTO Standard Specifications as follows:

$$V_u = V_p\left[C + \frac{0.87(1-C)}{\sqrt{1+(d_o/D)^2}}\right] \qquad (10\text{-}114)$$

V_u in the above equation is equal to the shear buckling capacity. The constant, C, is equal to the ratio of the shear buckling stress to the shear yield stress and is specified in Article 10.48.8.1 of the Standard Specifications. For transversely stiffened webs, C is calculated using a shear-buckling coefficient k equal to $5+5/(d_o/D)^2$, where d_o is equal to the transverse stiffener spacing. The maximum spacing of intermediate transverse stiffeners is limited to the web depth D. The maximum spacing of the first stiffener in an end panel is limited to 0.5D.

'V_u' is the absolute value of the shear capacity of the web (kips) based on the transverse stiffener spacing as shown on the plans.

Girder Section Capacity

The composite section capacity is calculated according to AASHTO's Standard Specifications Section 10.50.

- Positive Moment Sections
  - Noncompact sections

The bending stresses due to appropriate loadings shall not exceed:

- the yielding stress of the tension flange (F_y)
- $F_y R_b$ of in the compression flange, where

$$R_b = 1 - 0.002\left(\frac{D_c t_w}{A_{fc}}\right)\left[\frac{D_c}{t_w} - \lambda\Big/\sqrt{\frac{M_r}{S_{xc}}}\right] \le 1.0 \qquad (10\text{-}103b)$$

Here, A_{fc} shall be taken as the effective combined transformed area of the top flange and concrete deck that yields, D_c is calculated with accordance to article 10.50b, f_b is equal to factored bending stress in the compression flange not exceeding F_y. λ is defined as follows:

λ = 15,400 for sections where $D_c \le D/2$
= 12,500 for sections where $D_c > D/2$, D_c is the depth of the web of the steel girder in compression

According to Section 10.50.1.2.2, when girders are not provided with temporary supports during the placing of the dead load, the sum of the stresses produced by $1.3D_s$ acting on the steel girder alone with $1.3(D_c + 5(L+I)/3)$ acting on the composite girder shall not exceed yield stress at any point, where D_c and D_s are the moments caused by the dead load acting on the steel girder and composite girder, respectively.

- Negative Moment Sections
  - Noncompact sections

The girder maximum strength, M_u is defined as follows:
The bending stresses due to appropriate loadings shall not exceed:

- the yielding stress of the tension flange (F_y)
- $F_{cr} R_b$ of in the compression flange, where

$$F_{cr} = \left(4{,}400\frac{t}{b}\right)^2 \le F_y$$

Where b and t are the compression flange width and thickness, respectively.

$$R_b = 1 - 0.002\left(\frac{D_c t_w}{A_{fc}}\right)\left[\frac{D_c}{t_w} - \lambda\Big/\sqrt{\frac{M_r}{S_{xc}}}\right] \le 1.0$$

Here, f_b is equal to factored bending stress in the compression flange not exceeding F_y. λ is defined as above.

Intermediate Cross Frames& Floor Beams

AASHTO Standard Specifications for Highway Bridges, 17th Edition-2002 was used to check the adequacy of the cross frame sections. Each member was checked for the axial capacity and the combined axial load and bending effect, Sections 10.54.1 and 10.54.2 respectively. Equation 10-150 is used to calculate the maximum axial strength, while equations 10-155 and 10-156 set limits for the combined axial loads and bending moments.

$$P_u = 0.85 A_s F_{cr} \qquad \text{AASHTO 10-150}$$

$$\frac{P}{0.85 A_s F_{cr}} + \frac{M\ C}{M_u \left(1 - \frac{P}{A_s F_e}\right)} \le 1.0 \qquad \text{AASHTO 10-155}$$

$$\frac{P}{0.85 A_s F_y} + \frac{M}{M_p} \le 1.0 \qquad \text{AASHTO 10-156}$$

.

$$F_{cr} = F_y \left[1 - \frac{F_y}{4\pi^2 E}\left(\frac{KL_c}{r}\right)^2\right] \ for\ \frac{KL_c}{r} \le \sqrt{\frac{2\pi^2 E}{F_y}} \qquad (10\text{-}151)$$

$$F_{cr} = \frac{\pi^2 E}{\left(\frac{KL_c}{r}\right)^2} \ for\ \frac{KL_c}{r} \ge \sqrt{\frac{2\pi^2 E}{F_y}} \qquad (10\text{-}154)$$

Constructability

AASHTO's Standard Specifications Constructability Section 10.61 states that the moment and shear capacities of a girder shall meet the requirements to control local buckling of the web and compression flange, and to prevent lateral torsional buckling of the cross section under the non-composite dead load prior to hardening of the deck slab. A load factor of $\gamma = 1.3$ shall be used in calculating the applied moments and shears.

The requirements are as follows:

- **Web Bend Buckling**

According to AASHTO Standard Specifications Section 10.61.1, the maximum factored non-composite dead load compressive bending stress in the web shall not exceed the value given below:

$$f_b \le \frac{26{,}200{,}000\ \alpha k}{\left(\frac{D}{t_w}\right)^2} \le F_{yw} \qquad (10\text{-}173)$$

Where

F_{yw} : minimum yield strength of the web

D_c : depth of the web of the steel girder in compression

D : web depth

t_w: thickness of the web

Here the buckling coefficient k is taken to be 9 (D/Dc) for members without longitudinal stiffeners. When longitudinal stiffeners are present the buckling coefficient k is calculated as:

$$for\ \frac{d_s}{D_c} \ge 0.4 \qquad k = 5.17\left(\frac{D}{d_s}\right)^2 \ge 9\left(\frac{D}{D_c}\right)^2$$

$$for\ \frac{d_s}{D_c} \ge 0.4 \qquad k = 11.64\left(\frac{D}{D_c - d_s}\right)^2$$

In the case when both edges of the web are in compression, k should be taken as 7.2 for members with or without longitudinal stiffeners. $\alpha = 1.3$ for members without a longitudinal stiffener and 1 for members with longitudinal stiffer (Equation 10-173).

- **Web shear buckling**

According to AASHTO Standard Specifications Section 10.61.2, the sum of the factored noncomposite and composite dead-load shears shall not exceed the shear buckling capacity of the web:

$V_u = C\ V_p$ (10-113)

The maximum shear in the web cannot exceed the plastic shear force, Vp, given in 10.48.8.1 of AASHTO Standard Specifications as follows

$V_p = 0.58\ F_y\ D\ t_w$ (10-115)

The spacing of intermediate transverse stiffeners is based on the shear capacity, Vu, defined in Article 10.48.8.1 of AASHTO Standard Specifications as follows:

$$V_u = V_p\left[C + \frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_0}{D}\right)^2}}\right] \qquad (10\text{-}114)$$

Vu in the above equation is equal to the shear buckling capacity. The constant, C, is equal to the ratio of the shear buckling stress to the shear yield stress and is specified in Article 10.48.8.1 of the Standard Specifications. For transversely stiffened webs, C is calculated using a shear-buckling coefficient k as follows:

$$for\,\frac{D}{t_w} < \frac{6{,}000\sqrt{k}}{\sqrt{F_y}} \qquad C = 1.0$$

$$for\,\frac{6{,}000\sqrt{k}}{\sqrt{F_y}} \le \frac{D}{t_w} < \frac{7{,}500\sqrt{k}}{\sqrt{F_y}} \qquad C = \frac{6{,}000\sqrt{k}}{\left(\frac{D}{t_w}\right)F_y} \qquad (10\text{-}116)$$

$$for\,\frac{D}{t_w} > \frac{7{,}500\sqrt{k}}{\sqrt{F_y}} \qquad C = \frac{4.5\,x10^7 k}{\left(\frac{D}{t_w}\right)^2 F_y}(10\text{-}117)$$

where $k = 5+5/(d_o/D)^2$; d_o is equal to the transverse stiffener spacing. The maximum spacing of intermediate transverse stiffeners is limited to the web depth D. The maximum spacing of the first stiffener in an end panel is limited to $0.5D.d_0$ = distance between transverse stiffeners

Vu is the absolute value of the shear capacity of the web (kips) based on the transverse stiffener spacing as shown on the plans.

- **Lateral-Torsional Buckling of the Cross Section**

According to AASHTO Standard Specifications Section 10.61.3 the maximum factored non-composite dead-load moment shall not exceed the values of Mu calculated as partially braced member according to the following equation:

$M_u = M_rR_b$ (10-103a)
$R_b = 1$ for longitudinally stiffened girders

if $\frac{D}{t_w} \le 5{,}460\sqrt{\frac{k}{f_b}}$

$$for\,\frac{d_s}{D_c} \ge 0.4\; k = 5.17\left(\frac{D}{d_s}\right)^2 \ge 9\left(\frac{D}{D_c}\right)^2$$

$$for\;\frac{d_s}{D_c} \ge 0.4\; k = 11.64\left(\frac{D}{D_c - d_s}\right)^2$$

Where
d_s = the distance from centerline of a plate longitudinal stiffener
f_b = factored bending stress in the compression flange

In the case when both edges of the web are in compression k should be taken as 7.2
Otherwise

$$R_b = 1 - 0.002\left(\frac{D_c t_w}{A_{fc}}\right)\left[\frac{D_c}{t_w} - \frac{\lambda}{\sqrt{\frac{M_r}{S_{xc}}}}\right] \le 1.0 \quad (10\text{-}103b)$$ for girders with or without longitudinal stiffeners

Where:
D_c = depth of the web in compression (in)
t_w= thickness of the web (in)
A_{fc} = are of compression flange (in^2)
M_r = lateral torsional buckling moment (lb.-in)
S_{xc} = section modulus with respect to compression flange (in^3)
λ= 15,400 for sections where $D_c \le D/2$
= 12,500 for sections where $D_c > D/2$

The moment capacity, M_r should be less than yielding moment M_y at all times, and should be less than the lateral torsional buckling moment as follows:
For

$$\frac{D_c}{t_w} \le \frac{\lambda}{\sqrt{F_y}}$$

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$M_r = 91x10^6 C_b \left(\frac{I_{yc}}{L_b}\right) \sqrt{0.772 \frac{J}{I_{yc}} + 9.87 \left(\frac{d}{L_b}\right)^2} \leq M_y$ (10-103c)

$\frac{D_c}{t_w} > \frac{\lambda}{\sqrt{F_y}}$

$for L_b \leq L_p$

$M_r = M_y$ (10-103d)

$for L_r \geq L_b > L_p$

$M_r = C_b F_y S_{xc}\left[1 - 0.5\left(\frac{L_b - L_p}{L_r - L_p}\right)\right]; \quad L_r = \left(\frac{572\,x10^{\,6} I_{yc} d}{F_y S_{xc}}\right)^{1/2}$ (10-103e,f)

$for L_b > L_p$

$M_r = C_b \frac{F_y S_{xc}}{2}\left(\frac{L_r}{L_b}\right)^2$ (10-103g)

Where

L_b = unbraced length of the compression flange (in)

$L_p = 9{,}500r`/(F_y)^{1/2}$

r` = radius of gyration of compression flange about vertical axis in the plane of the web, (in^4).

d = depth of girder (in)

$J = [(bt^3)_c+(bt^3)_t+Dt_w^3]/3$ where b and t represent the flange width and thickness of the compression and tension flange, respectively, (in^4).

$C_b = 1.75+1.05(M1/M2)+0.3(M1/M2)^2 \leq 2.3$ where M1 is the smaller and M2 is the larger end moment in the unbraced segment of the beam.

C_b = 1.0 for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the largest of the segment end moments

- **Compression Flange Local Buckling**

According to Article 10.61.4, in positive moment regions, the ratio of the top compression flange width to thickness shall not exceed the following formula:

$\frac{b}{t} = \frac{4{,}400}{\sqrt{f_{dl}}} \leq 24$ (10-174)

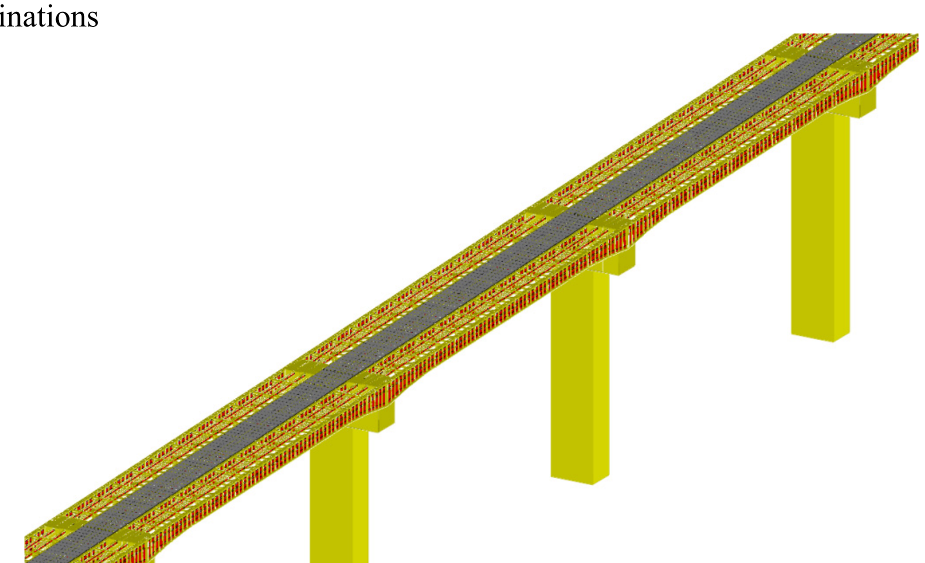
Where f_{dl} is the top flange compressive stress due to the factored non-composite dead load divided over R_b, but not exceeding F_y. R_b is defined in the lateral torsional buckling section.

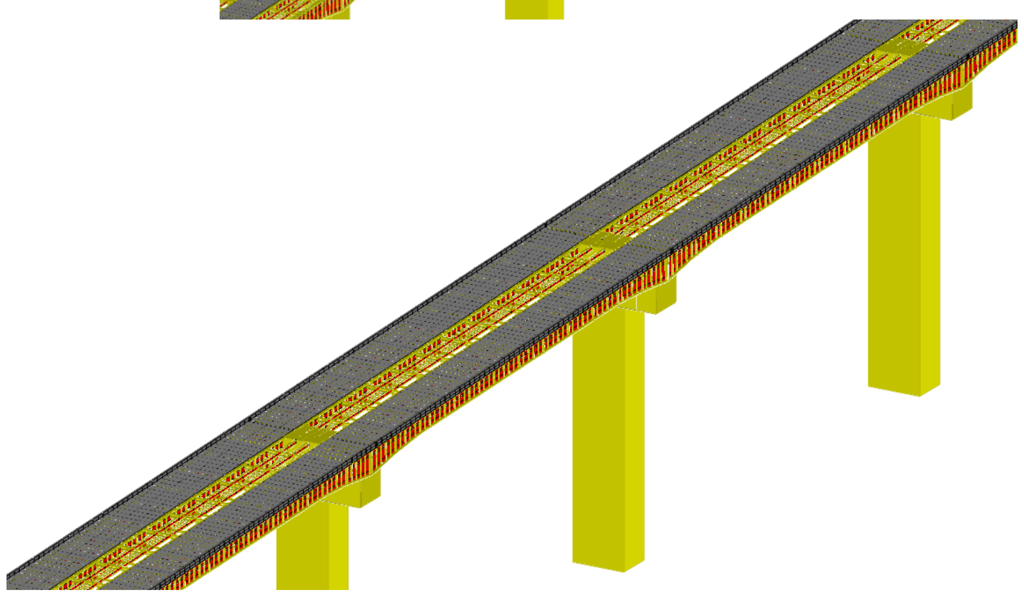
Selected Results from 3-D Analysis

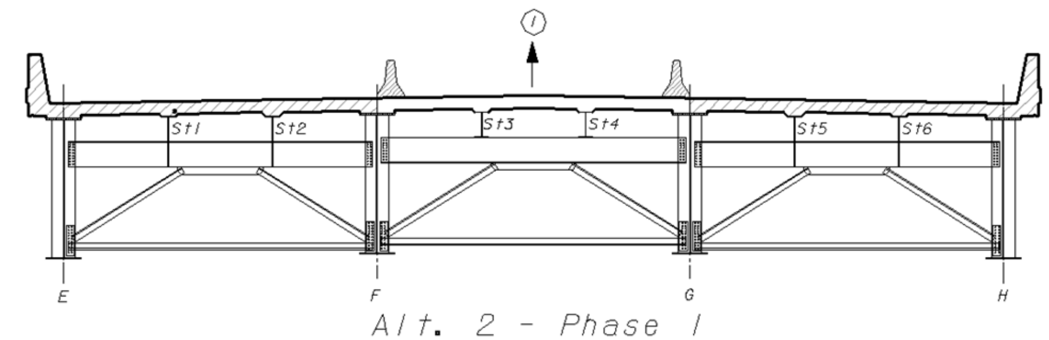
Preliminary findings indicated large out-of-planes stresses due to both MOT alternates. Alternate 1 MOT yielded significantly higher stresses, so it was determined after consultation with ODOT that further analyses efforts be limited to Alternate 2 MOT.

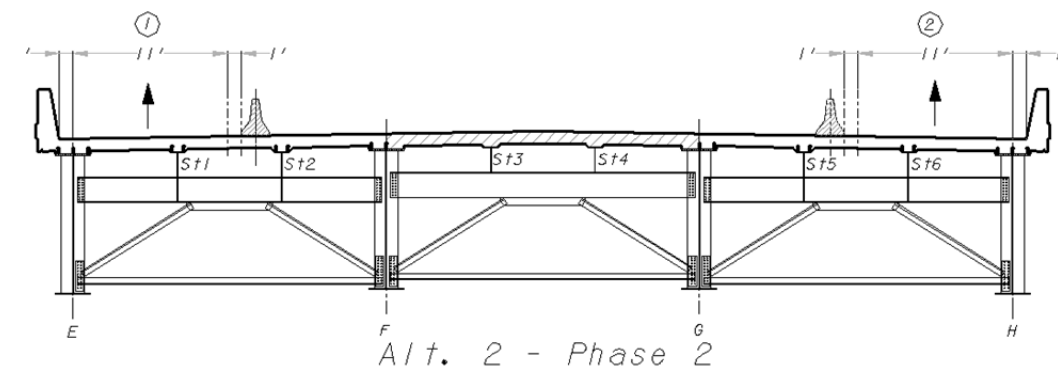
The Alternative 2 (5+1) has two phases. Phase 2 has a composite deck on the outsides as shown here. Results from the analyses are presented:

- Unfactored Shear and Moments
- Bending (Flexure) Stresses for Girders F and Stringer No. 3 for various AASHTO Load Combinations









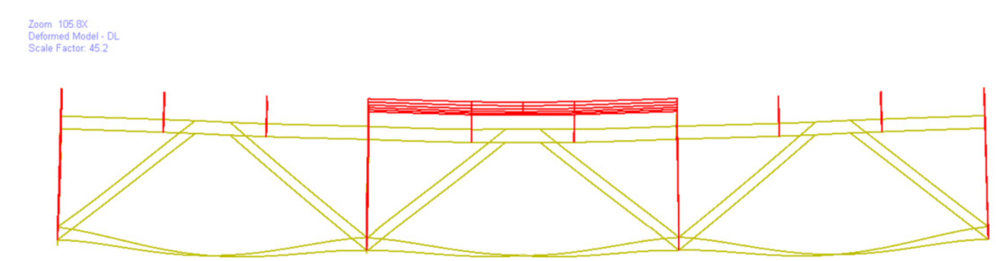


Figure 23*Alt 2- Phase 1 - Deformed Shape - Mid Span 11*

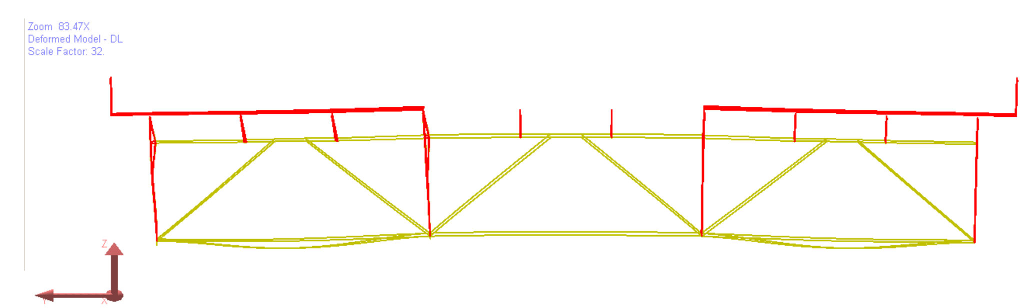


Figure 24 *Alt 2- Phase 2 - Deformed Shape - Mid Span 11*

Table 2 summarizes load combinations considered in the analyses.

Table 2 *Factors for load combinations (LFD) used in the LARSA Model*

Load Combination	AASHTO GROUP	DL	Live Load	W
1	I	1.3	1.30 [5/3 HS20 Truck Plus impact (30 %)]	0
2	I	1.3	1.30 [5/3 HS20 Moment Plus impact (30 %)]	0
3	I	1.3	1.30 [5/3 HS20 Shear Plus impact (30 %)]	0
4	I	1.3	1.30 [5/3 HS20 Train Plus impact (30 %)]	0
5	II	1.3	0	1
6	III	1.3	1.30 [1.0 HS20 Truck Plus impact (30 %)]	0.3
7	III	1.3	1.30 [1.0 HS20 Moment Plus impact (30 %)]	0.3
8	III	1.3	1.30 [1.0 HS20 Shear Plus impact (30 %)]	0.3
9	III	1.3	1.30 [1.0 HS20 Train Plus impact (30 %)]	0.3
10			Fatigue Truck Plus impact (10 %)	

Table 3 *Unfactored Shear and Moments (Mid Span 11)*

	DL Moment (Kips.ft)		DL Shear (Kips)		Dead Load Deflection (in)		Live Load Max Deflection (in)		(DL+LL) Max Deflection (in)	
	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
Current (mid span 11)	4291	2996	-21.7	-19.4	1.93	1.85	3.3	3.0	5.23	4.85
MOT Phase 1 (mid span 11)	2309	1695	-9.44	-4.11	0.79	0.57	1.8	1.7	2.59	2.27

Table 3 shows that no appreciable moments or shears occur during MOT phasing. The deflections of the interior and exterior girders in relationship to each other are consistent.

Table 4 summarizes stresses for various AASHTO load combinations. The maximum stresses computed are about 23 ksi in Girder F and about 11 ksi in Stringer 3. This is well below the allowable stress of 27 ksi for the main girder steel and 20 ksi for the stringers.

***Table 4** Max and Min Bending Factored stresses on Girder F (interior) and Stringer 3 (MOT Alternative 2 – Phase 1)*

Combination No.	AASHTO Group	Girder F Sxx (min) (ksi)	Girder F Sxx(max) (ksi)	Stringer 3 Sxx (min) (ksi)	Stringer 3 Sxx(max) (ksi)
1	I	-13.1	6.7	-8.5	7.5
2	I	-14.5	7.3	-8.1	7.4
3	I	-14.6	7.3	-8.4	8.0
5	II	-22.7	10.1	-10.6	9.1
6	III	-10.2	5.9	-5.9	4.6
7	III	-13.5	6.7	-7.2	6.3
8	III	-13.7	6.8	-7.5	6.7
9	III	-12.8	6.3	-7.5	6.4
10		-14.8	7.8	-8.8	7.3

***Table 5** Factored Live Load Bending Stresses – Girder E (Exterior)*

		Bending Stress(ksi) Near Pier		Bending Stress (ksi) MidSpan	
		Max -ve	Max +ve	Max -ve	Max +ve
Existing	At Top Flange	-0.55	0.03	-0.11	0.13
	At Bottom Flange	-0.26	0.81	-4.09	1.61
MOT Ph1	MOT 1 - At Top Flange	-0.1	2.12	-3.26	2.12
	MOT 1 - At Bottom Flange	-4.31	0.78	-3.78	6.81
MOT Ph2	At Top Flange	-0.33	0.83	-0.96	0.74
	At Bottom Flange	-2.22	1.98	-1.59	6.31

***Table 6** Factored Live Load Bending Stresses – Girder F (Interior)*

		Bending Stress(ksi) Near Pier		Bending Stress (ksi) MidSpan	
		Max -ve	Max +ve	Max -ve	Max +ve
Existing	At Top Flange	-0.11	1.12	-0.99	1.61
	At Bottom Flange	-2.34	0.81	-4.56	8.13
MOT Ph1	MOT 1 - At Top Flange	-0.07	3.33	-2.82	2.36
	MOT 1 - At Bottom Flange	-4.65	1.74	-3.28	8.27
MOT Ph2	At Top Flange	-0.33	0.76	-0.96	0.13
	At Bottom Flange	-3.49	0.74	-1.56	5.69

Notes
In Tables 5 and 6, the term "Near Pier" refers to girder area near the location of Pier 10 and the term "Midspan" refers to middle of span 11. MOT Ph1 & 2 refers to MOT Alternative 2 Phase 1 & 2 respectively.
Max –ve and +vein the Tables 5 and 6 refer to maximum and minimum live load envelopes. For stringer and girder locations – refer to page 16 of this report.

Summary of Code Checking

The existing girders and stringers were checked with both the existing loading condition and Alternate 2 MOT. The following checks were performed according to AASHTO Standard Specifications. The results indicate all moment and shear stresses due to phase construction to be acceptable. The results are presented in Part I of Volume II in tabular format for Girders E, F and Stringers 1, 2 & 3 (see page 16 for the girder and stringer location). The results include girder capacity (moment and shear) and constructability checks for following:

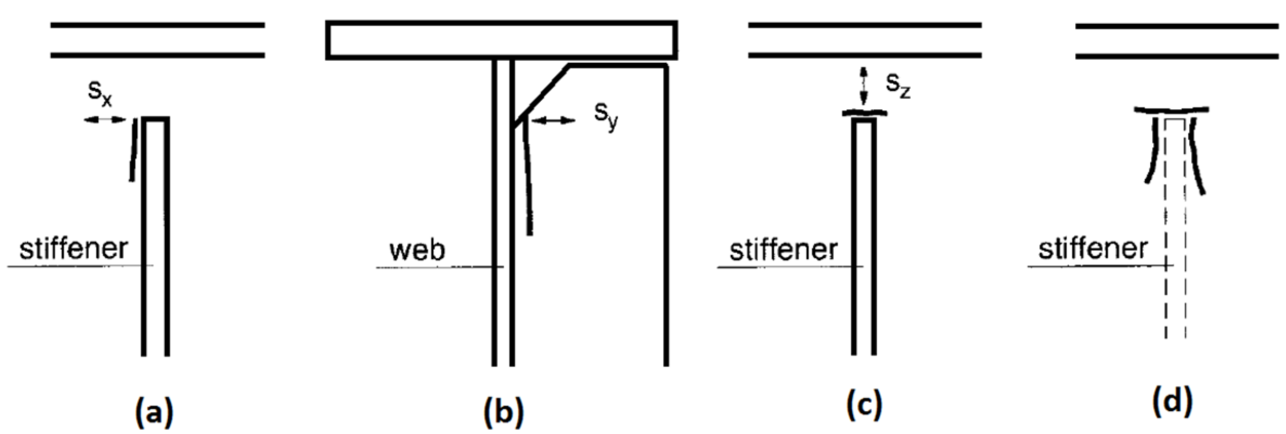
- Existing Deck (Maximum and Minimum LL Envelopes)
- Alternative 2 – Phase 1 MOT (Maximum and Minimum LL Envelopes)
- Alternative 2 – Phase 2 MOT (Maximum and Minimum LL Envelopes)

The detailed computations worksheets are also included in the Appendix of Volume II.

IV. Out of Plane Distortion

Out-of-plane distortion will cause stresses in the localized web gap region. In this section, results from out-of-plane distortion modeling at the following locations are presented: Near pier 10 (Negative Moment Region) and middle of span 11 (Positive Moment). A comparison between composite (top flange restrained by the deck) and non-composite structural performance was evaluated only for the existing design conditions.

The figures below show the cracks that may develop due to the various components of out-of-plane distortion induced axial stresses. The high stresses in the X direction cause the initiation of vertical cracks as shown in Figure (a). The high stresses in the Y direction may cause failure of the stiffeners web welds ending in stiffener detachment. Similarly, high stresses in the Z direction may cause horizontal cracks. The maximum tension component of the principal stress (S1) is a critical force for initiating cracks.



Sx = Stress in X-direction i.e. along the length of the girder (Refer to Figure a)
Sy = Stress in the Y-Direction i.e. perpendicular to the length of the girder (Refer to Figure b)
Sz = Stress in the vertical direction i.e. along the depth of the girder (Refer to Figure c)
S1 = Maximum principal stress (See Figure d)

The table shown below contains a summary of out-of-plane distortion induced stresses and maximum deflections for interior and exterior girders at near pier 10 (Negative Moment Region) and middle of span 11 (Positive Moment). The following cases are presented: Composite and non-composite structural performance and Phase I of MOT Scheme (5+1).

	Stress (ksi) Interior				Stress (ksi) Exterior				Max Deflection (in)	
	Sx	Sy	Sz	S1	Sx	Sy	Sz	S1	Interior	Exterior
MOT (5+1) - Phase I										
Mid Span	80	26	25	100	25	17	25	38	0.054	0.03
Near Pier	50	23	20	51	30	17	20	35	0.02	0.04
EXISTING DECK - NON COMPOSITE DECK										
Mid Span	54	20	21	55	#	#	#	#	0.0216	#
Near Pier	20	12	14	22	49	18	34	66	0.01	0.024
EXISTING DECK MODELED AS COMPOSITE (BENCHMARK)										
Mid Span	45	7	11	46	26	4	11	27	0.024	0.0132

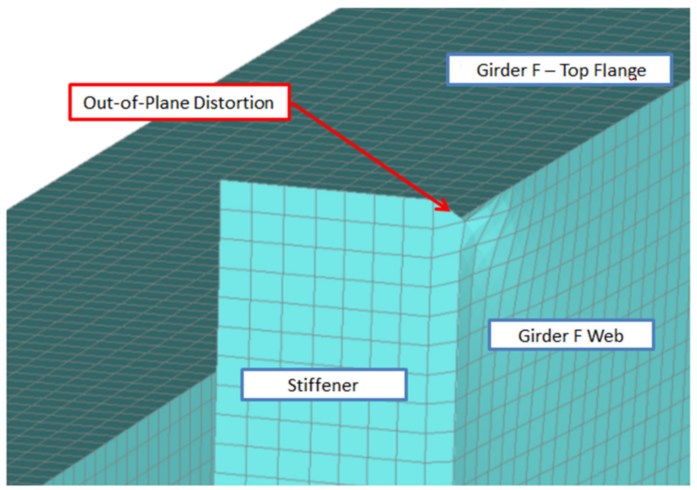
Note: *# Based upon preliminary analysis, the interior girder controlled the design. Therefore only the interior girder was analyzed*

As stated in Fisher (1985), these out-of-plane stresses are caused by bridge members moving in three dimensions. The lateral movement, y-axis, is termed as out-of-plane displacement and is caused by lateral bracing or transverse beams.

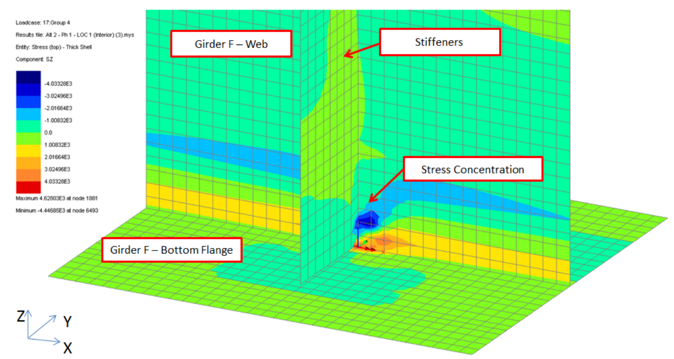
Volume II of this report includes the following information for interior and exterior girders in both the negative and positive moment regions of span 11:

- Out-of-plane distortion contours at the top and bottom
- Out-of-plane stresses (Sx, Sy, Sz and S1)

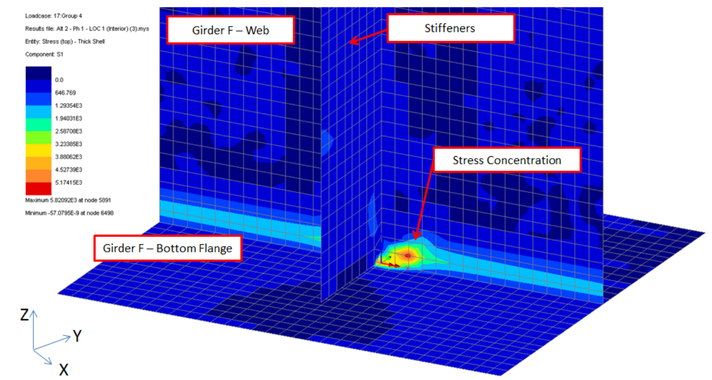
The figures on this page show the typical stress distribution due to out-of-plane distortions caused by the forces in the floorbeams and crossframe members. Out-of-plane distortion behavior is depicted as follows: Distortion or deformed shape is shown in figure (a), the stress concentrations at the connection plates are provided in figures (b) and (c). The out-of-plane stresses are plotted and shown in figure (d), four components (Sx, Sy, Sz, S1) are shown.



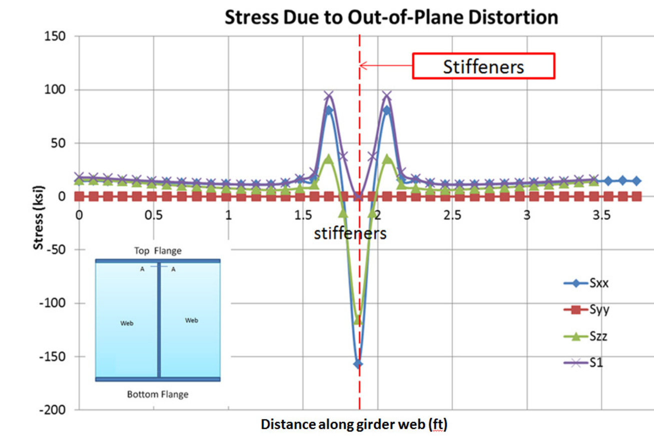
(a)



(b)



(c)



(d)

Figure 25 *Typical Stress Distribution due to out-of-plane Distortion*

The modeling of existing conditions provided a benchmark for comparison with the phase construction stress levels. The non-composite assumption yielded overstresses that the current bridge does not reveal. The existing, condition were also modeled considering composite action which yielded stresses below the yield strength of the material, thereby reflecting more accurately the anticipated performance of the existing structure. Therefore, the existing structure modeled as composite serves as the final benchmark for comparison with the phase construction model.

Figures 26 through 30 show the FE mesh and boundary conditions for the aforementioned benchmark (Positive Moment Region – Existing Conditions). Figures 31 through 46 show out-of-plane stress contours (Sxx, Syy, Szz and S1) for top and bottom flanges of Girders E & F.

Results in the Positive Moment Region – Existing Conditions (Composite)

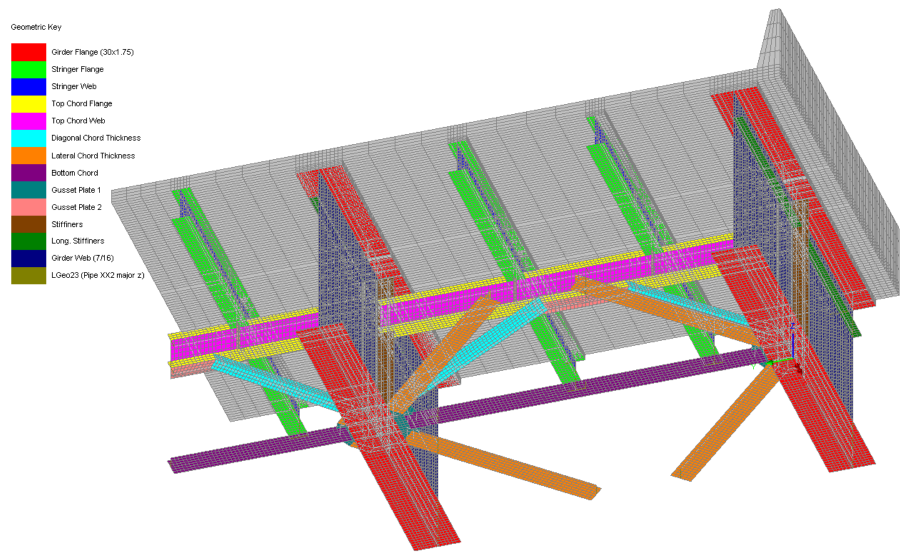


Figure 26 *Composite FE model for Girder E and F*

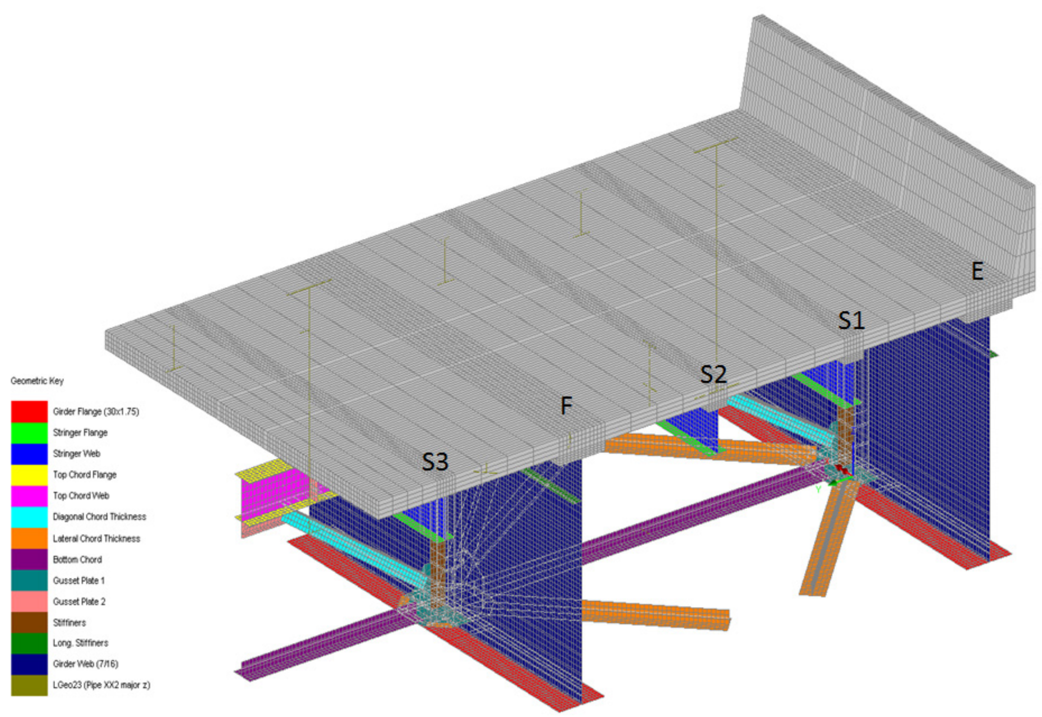


Figure 27 *Composite FE Model for Girder E and F*

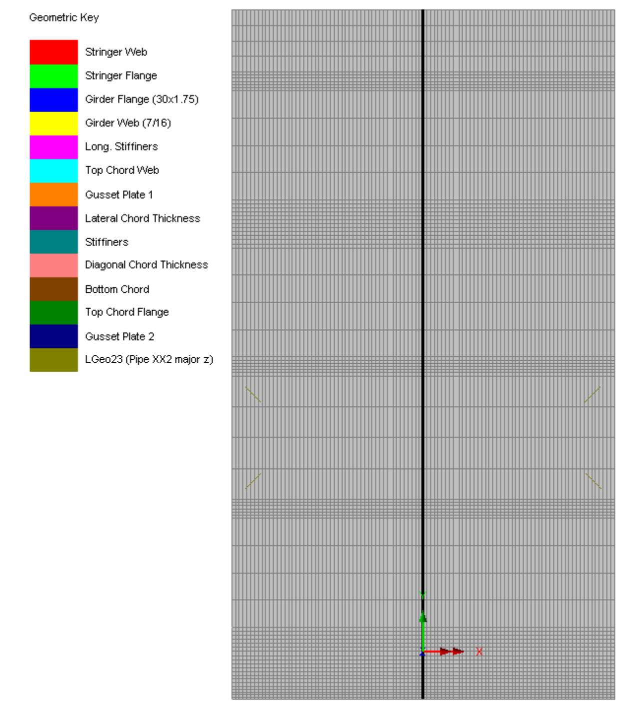


Figure 28 *BC – Plane of Symmetry*

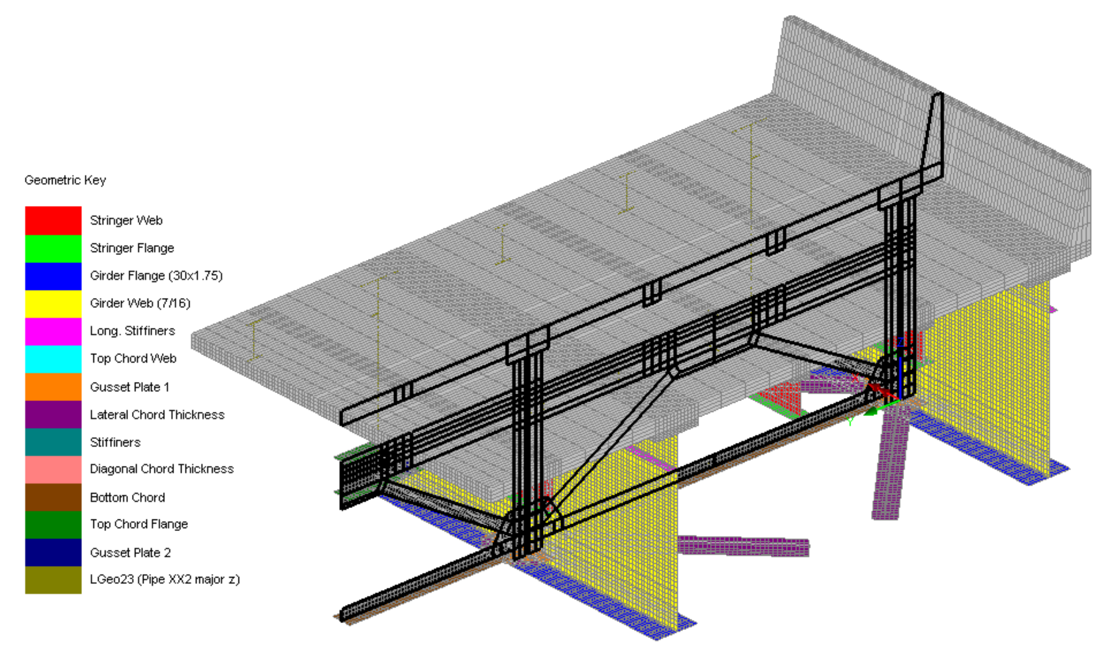


Figure 29 *BC – Plane of Symmetry in the Composite FE model*

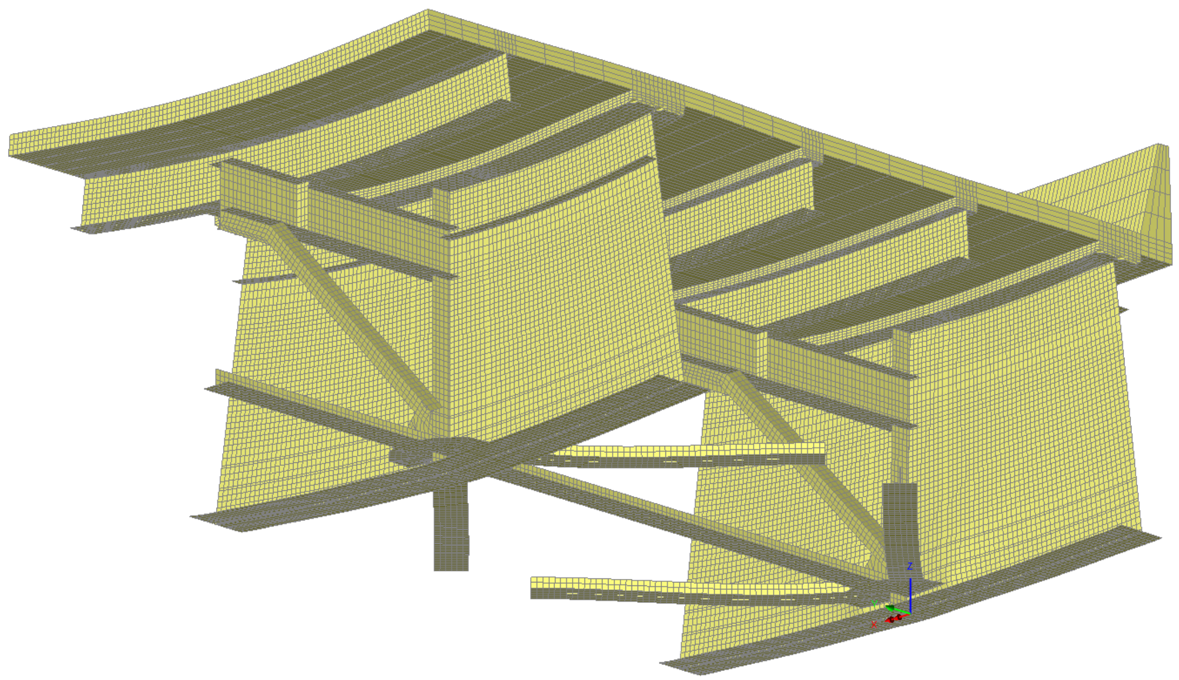


Figure 30 *Results – Deformed Mesh for the Composite Model*

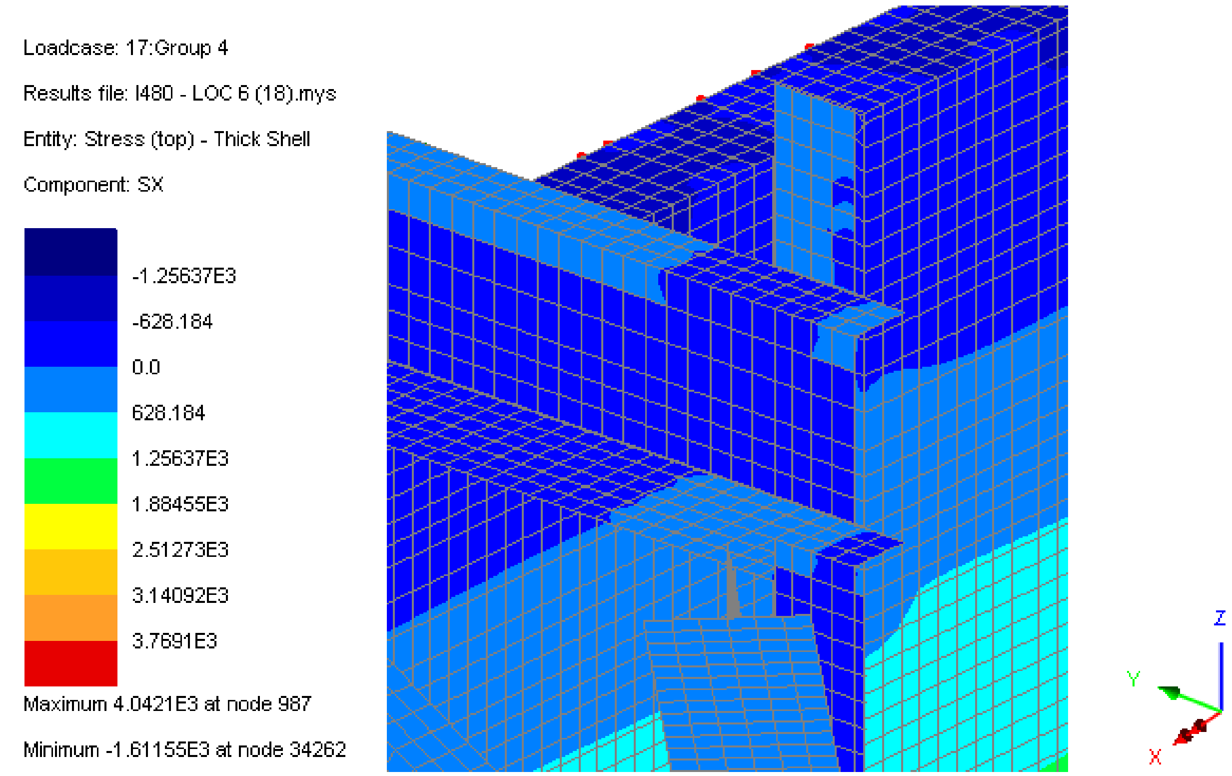


Figure 31 *Positive Moment – Girder E – Top Flange Sxx*

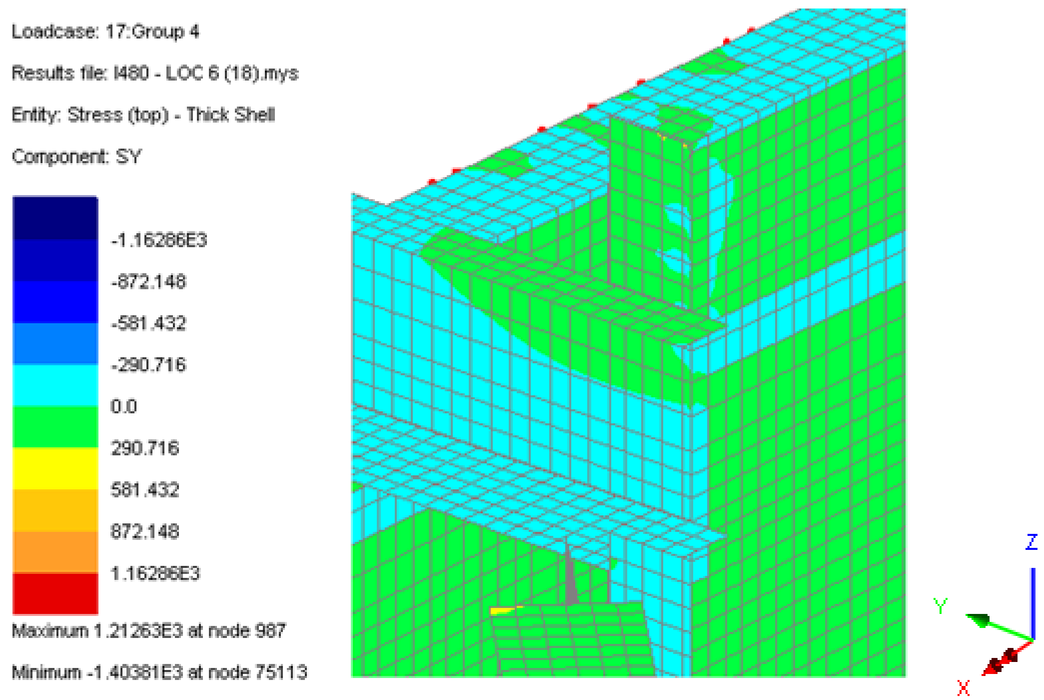


Figure 32 *Positive Moment – Girder E – Top Flange Syy*

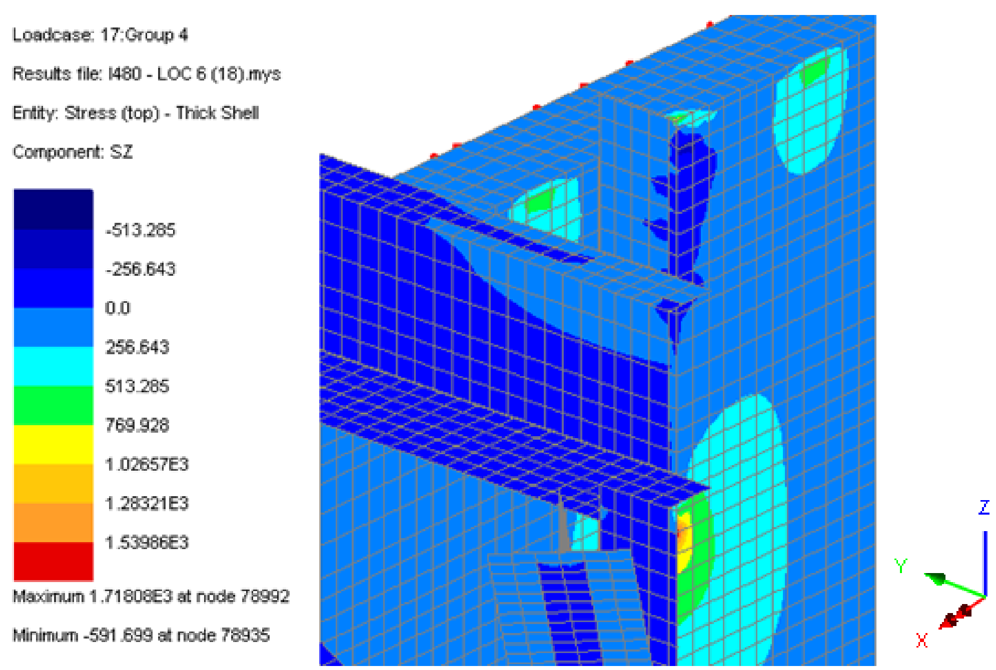


Figure 33 *Positive Moment – Girder E – Top Flange Szz*

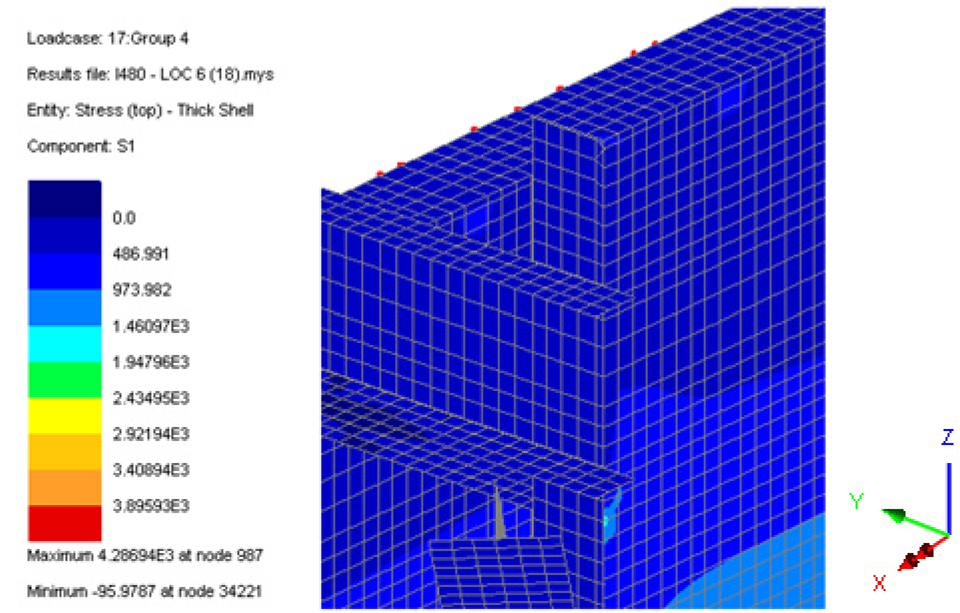


Figure 34 *Positive Moment – Girder E – Top Flange S1*

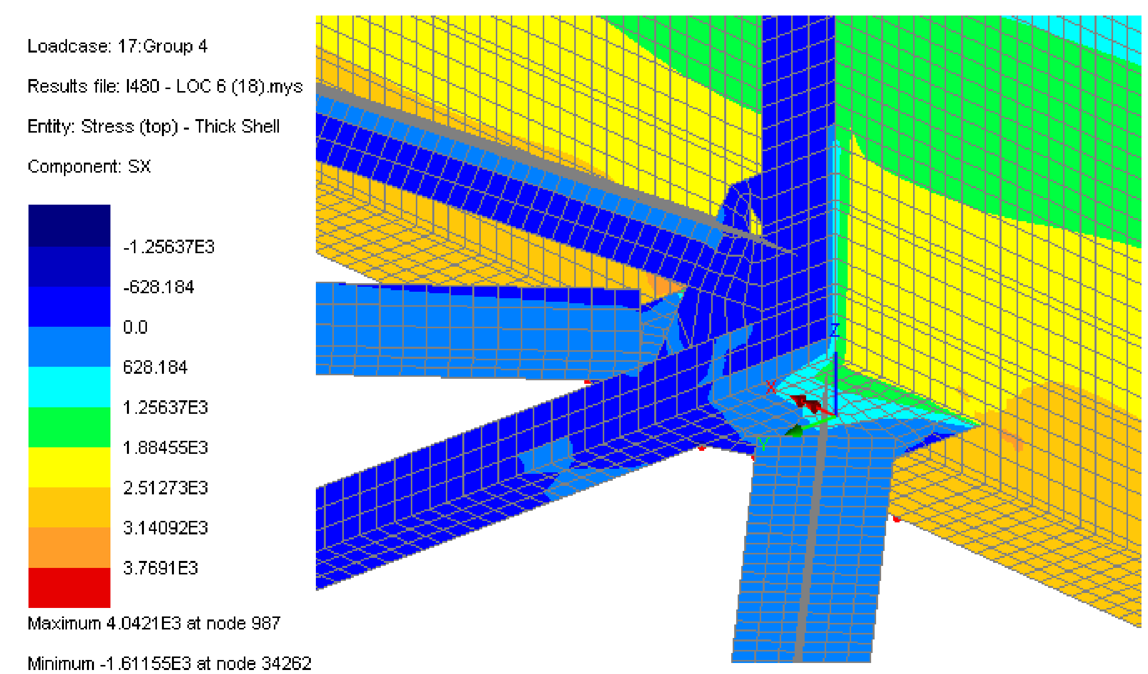


Figure 35 *Positive Moment – Girder E – Bottom Flange Sxx*

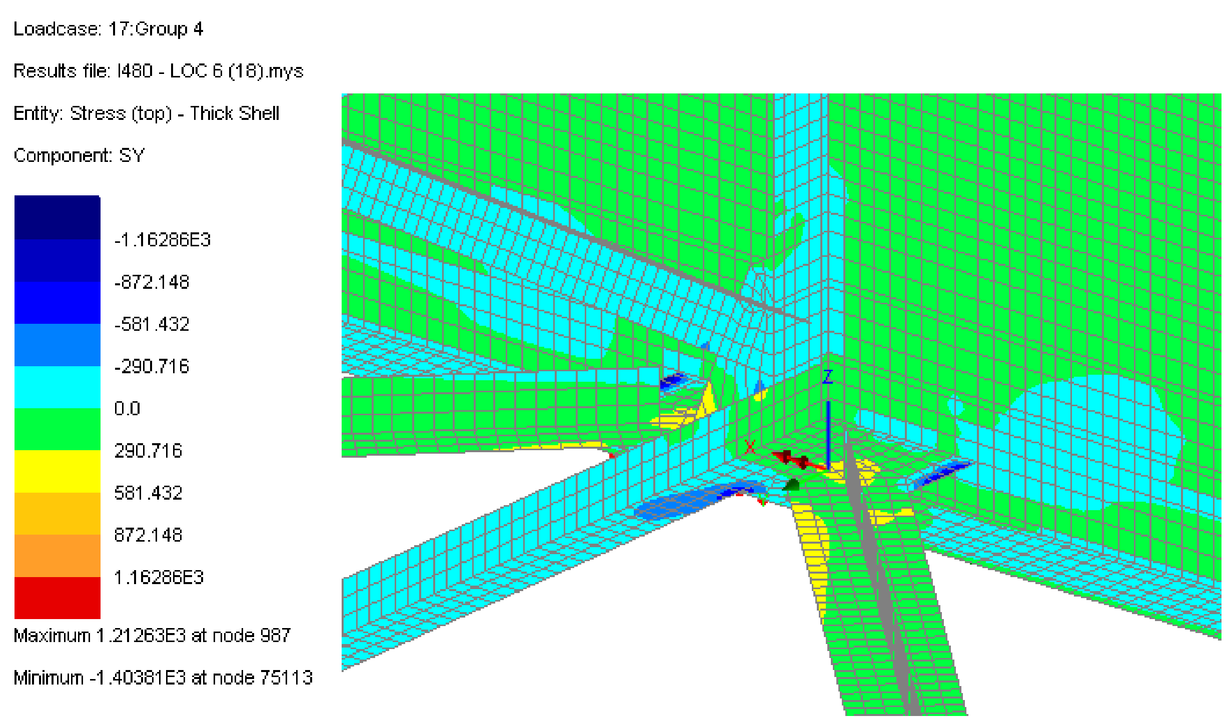


Figure 36 *Positive Moment – Girder E – Bottom Flange Syy*

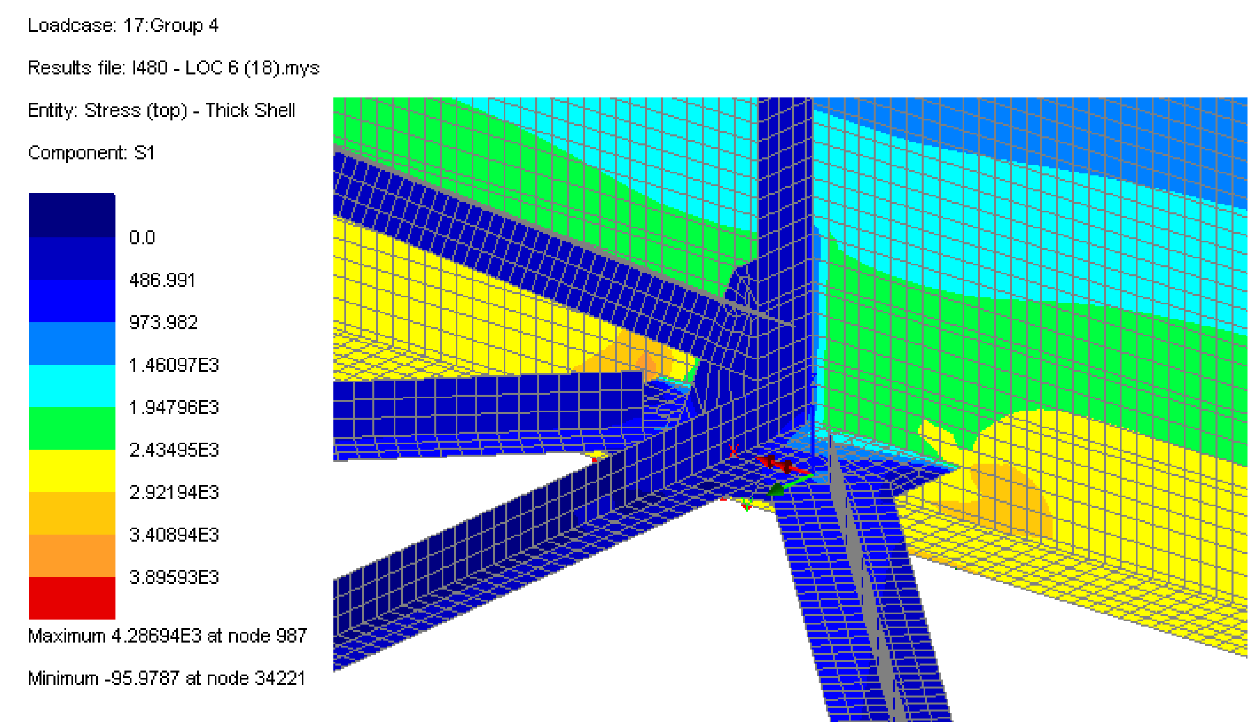


Figure 38 *Positive Moment – Girder E – Bottom Flange S1*

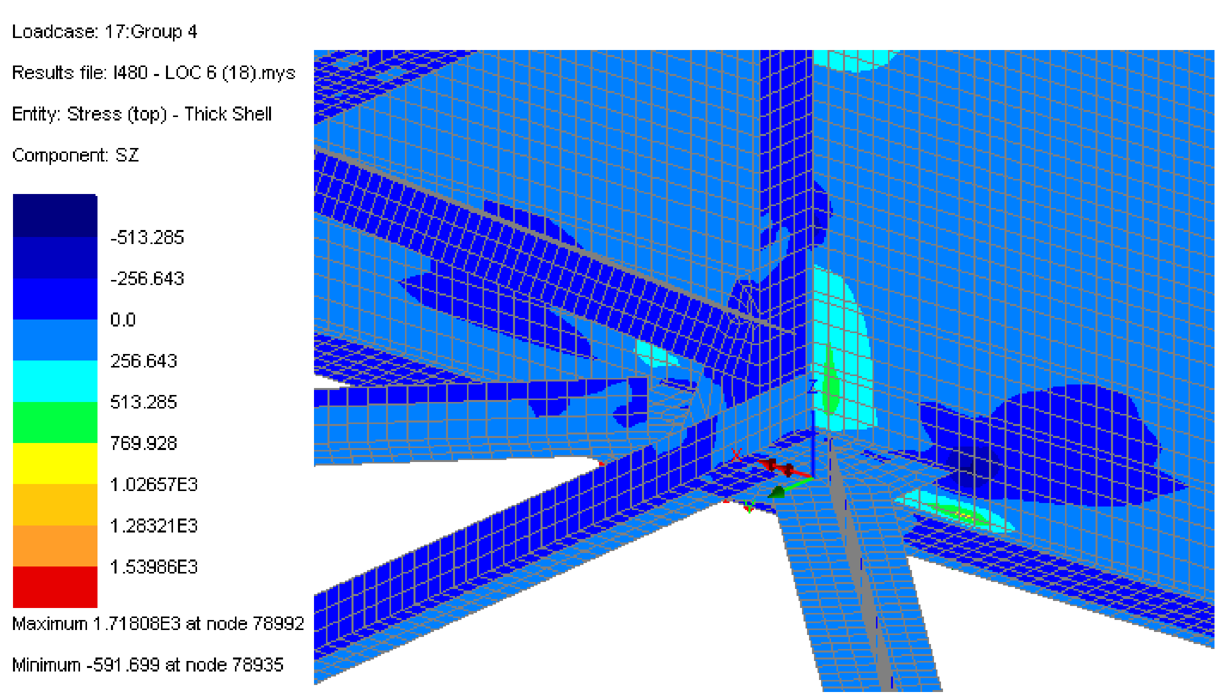


Figure 37 *Positive Moment – Girder E – Bottom Flange Szz*

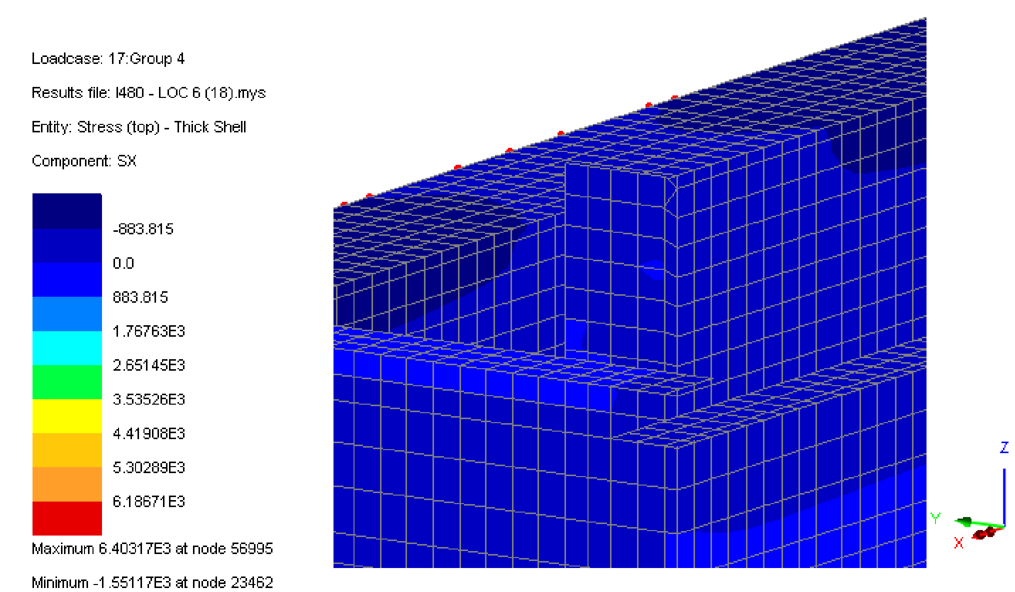


Figure 39 *Positive Moment – Girder F – Top Flange Sxx*

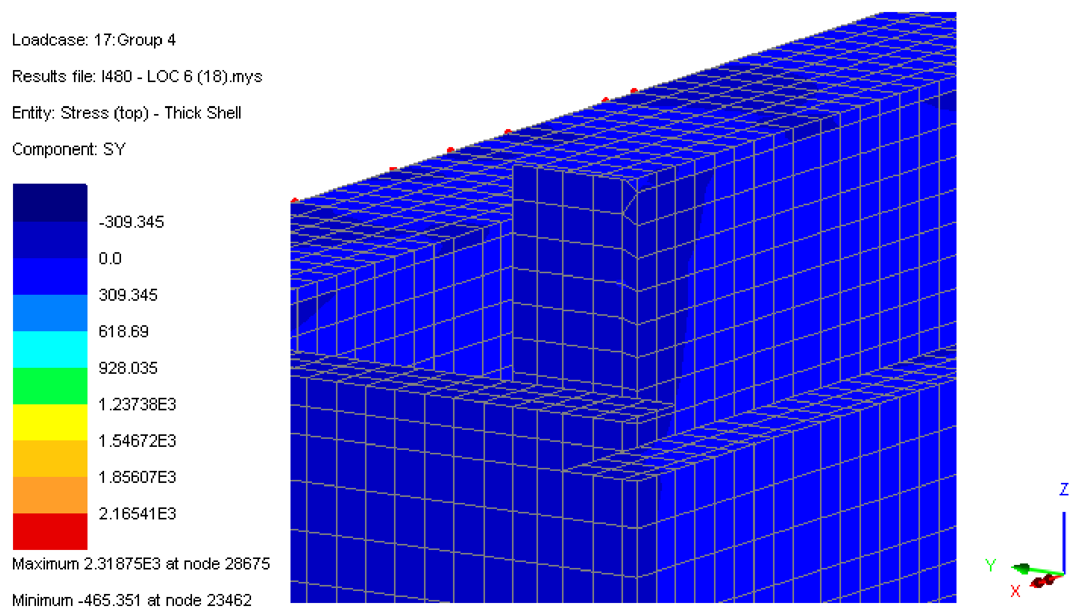


Figure 40 *Positive Moment – Girder F – Top Flange Syy*

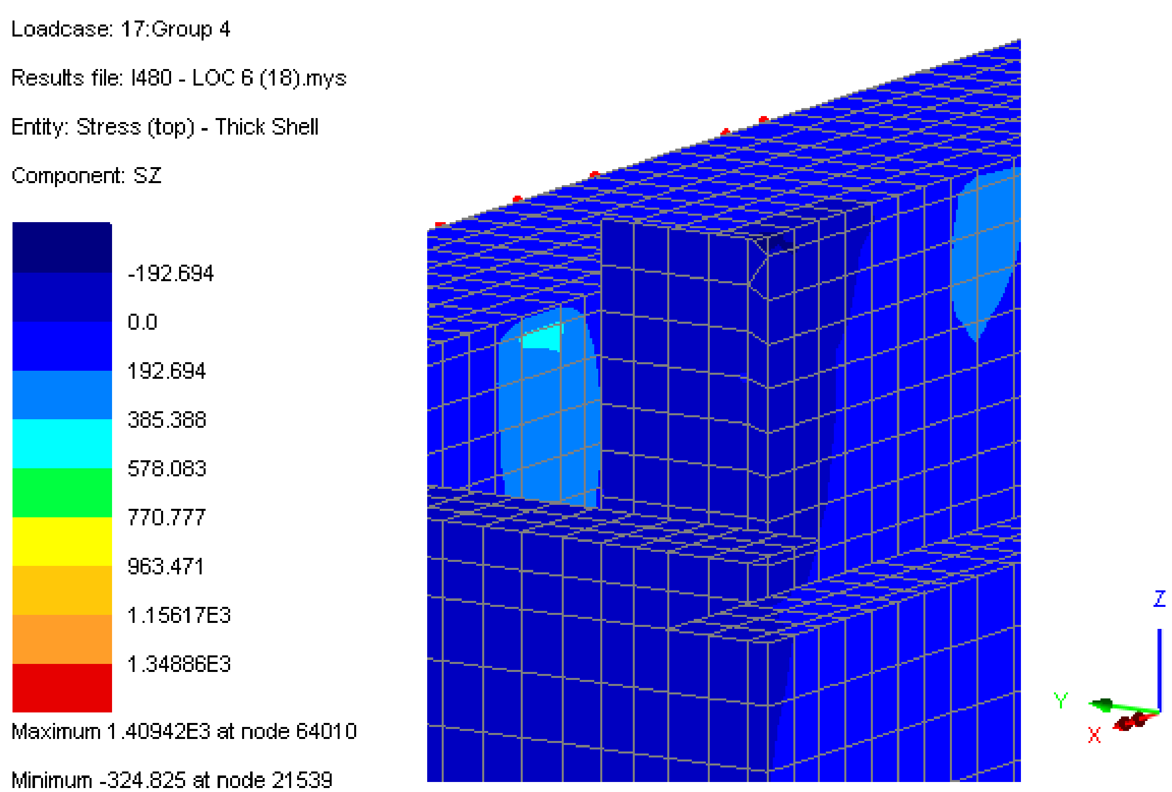


Figure 41 *Positive Moment – Girder F– Top Flange Szz*

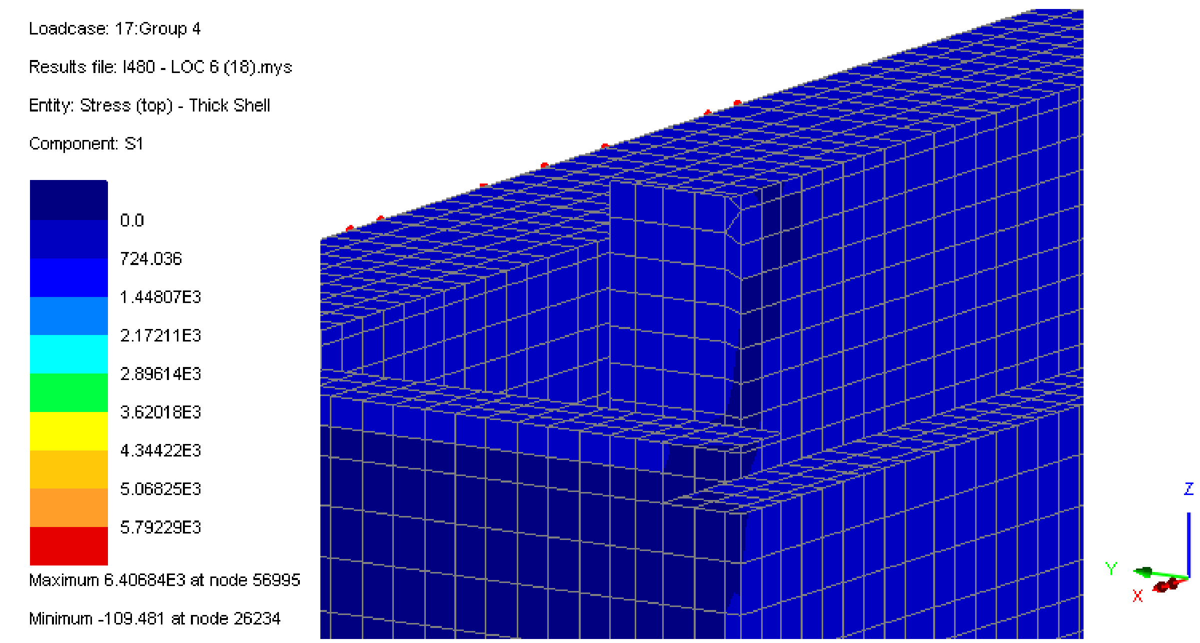


Figure 42 *Positive Moment – Girder F – Top Flange S1*

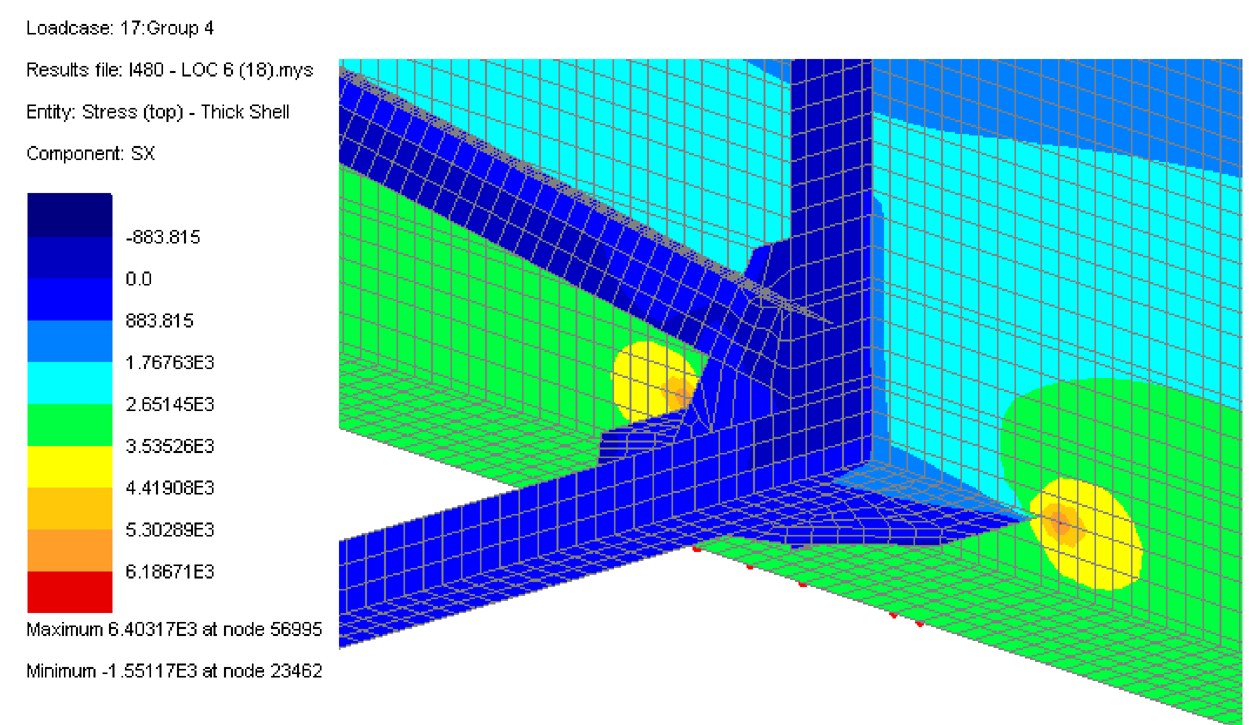


Figure 43 *Positive Moment – Girder F – Bottom Flange Sxx*

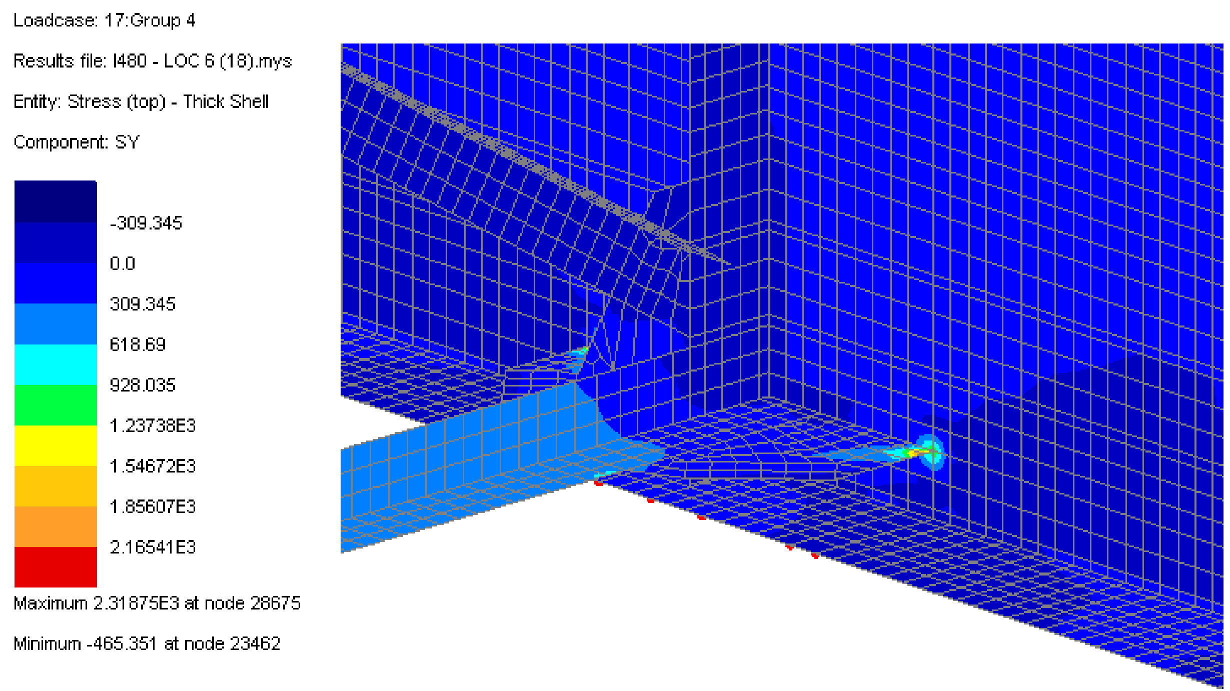


Figure 44 *Positive Moment – Girder F – Bottom Flange Syy*

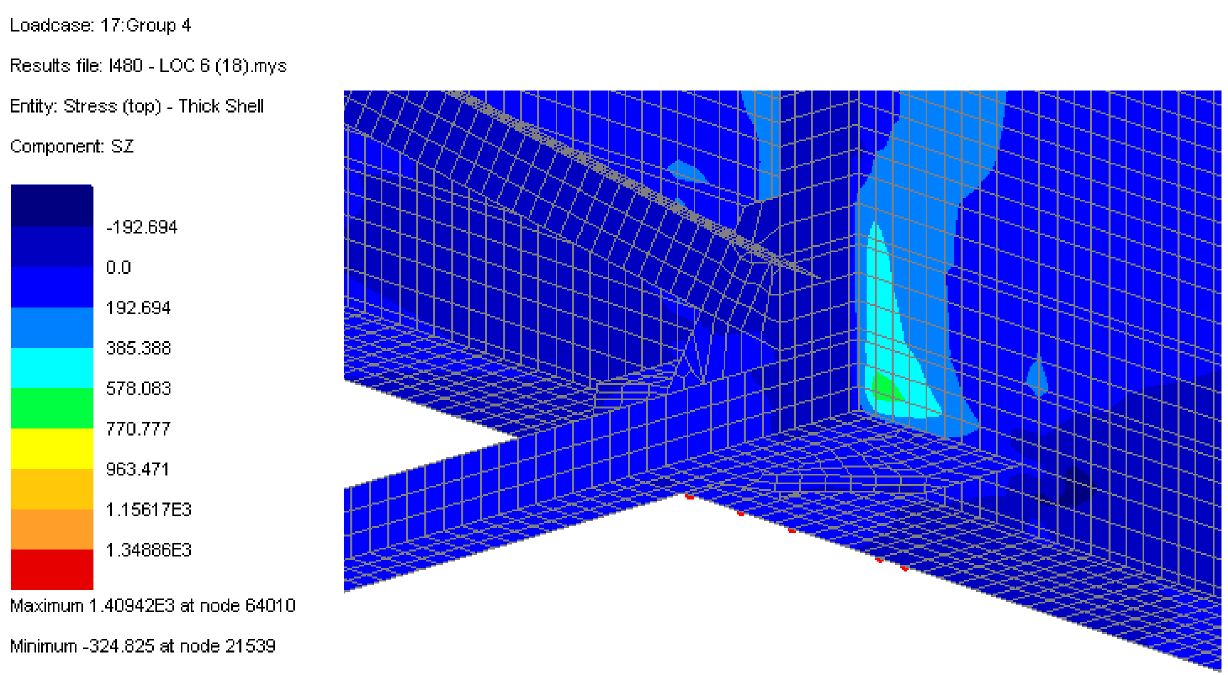


Figure 45 *Positive Moment – Girder E – Bottom Flange Szz*

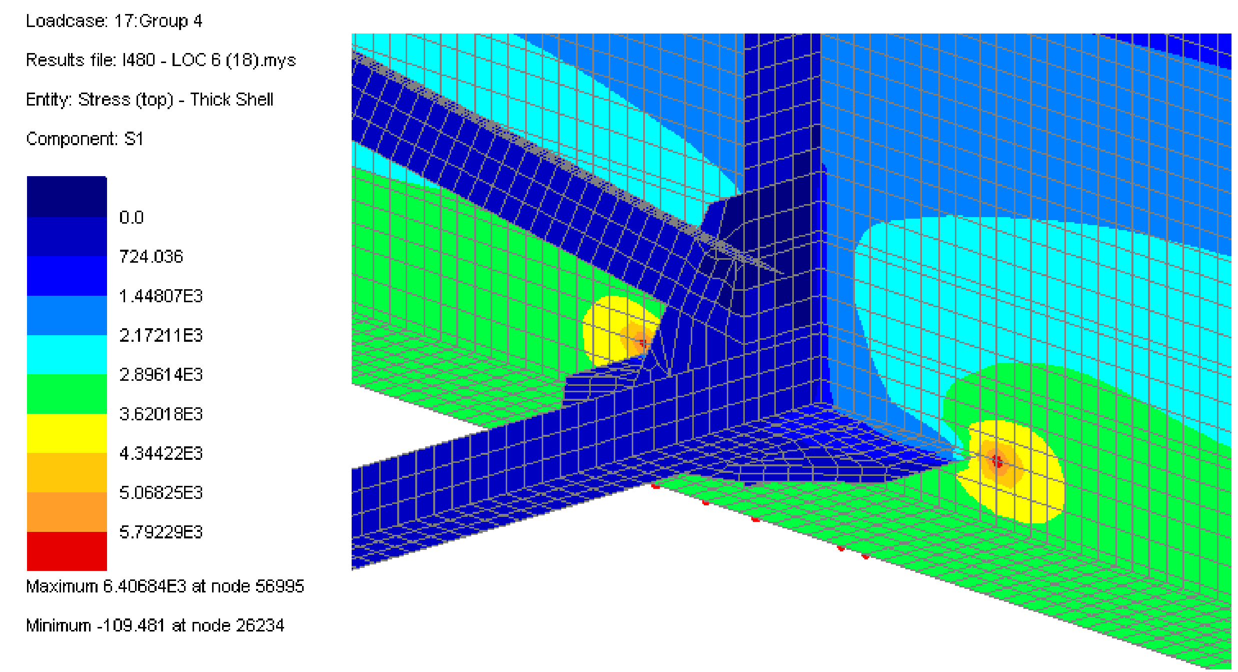
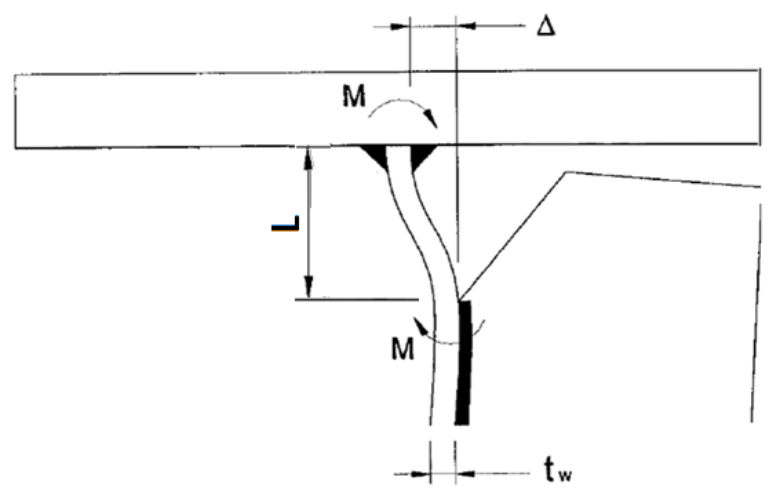


Figure 46 *Positive Moment – Girder F – Bottom Flange S1*

Fisher's s-Δ Expression

The web gap stress calculation due to the out-of-plane distortion, which is shown in the figure below, was established by Fisher (1998). The expression indicates that the bending effect due to out-of-plane distortion causes a stress that increases proportionally with Young's modulus (E), out-of-plane distortion (Δ), and the web thickness. The stress(s) is inversely proportional to the square of web gap length. In the expression, a fixed end moment is computed based on the distortion for the stress computation.

The Table 7 shows a summary of the calculated stress(s) using Fisher's formula. The out-of-plane distortion (Δ) was calculated using the sub-models. The computation is a means to perform hand calculations which indicate even small displacement will yield large stresses, which is the case for the existing structure.



$$\sigma = \frac{My_c}{I} = \frac{6EI\Delta}{L^2}\left(\frac{t_w}{2}\right)\left(\frac{1}{I}\right) = \frac{3E\Delta t_w}{L^2}$$

σ = web gap bending stress (ksi)
M= web gap bending Moment (Kips-in) $= \frac{6EI\Delta}{L^2}$ (fixed end beam moment)
y_c=distance from neutral axis to extreme fiber (in.)
I= Moment of Inertia (in.^{4})
E= Young's Modulus (ksi)
L= Web gap Length (in.)
Δ= out-of-plane displacement (in.)
t_w= web thickness (in.)

***Table** 7 Summary of out-of-plane distortion (Δ) and the Out-of-Plane bending stress (calculated using Fisher's Formula)*

Girder/Location	Web Gap Length (L)	Out of Plane Displacement (Δ) - inches	Thickness of Web (t_w)	E (ksi)	Stresses Using Fisher's Formula
Existing Deck (Considering Composite Action)					
Ext Girder E or H - Midspan 11	1	0.0132	0.4375	29000	>> Yield Strength
Int Girder F or G - Midspan 11	1	0.024	0.4375	29000	>> Yield Strength
Existing Deck (Non Composite)					
Ext Girder E or H - Pier 10	1	0.024	0.5625	29000	>> Yield Strength
Int Girder F or G - Midspan 11	1	0.0216	0.4375	29000	>> Yield Strength
Int Girder F or G - Pier 10	1	0.01	0.5625	29000	>> Yield Strength
MOT ALT 2 - Phase I (without retrofit)					
Ext Girder E or H - Midspan 11	1	0.03	0.4375	29000	>> Yield Strength
Ext Girder E or H - Pier 10	1	0.04	0.5625	29000	>> Yield Strength
Int Girder F or G - Midspan 11	1	0.054	0.4375	29000	>> Yield Strength
Int Girder F or G - Pier 10	1	0.025	0.5625	29000	>> Yield Strength

As indicated above, the out-of-plane bending stresses well exceed the material yield strength for all conditions. It should be noted that if the out-of-plane displacement doubles or triples as shown in the above table. It is safe to assume based on the simplified expression that the stresses will also double or triple for phase construction. Therefore, retrofit details are required to prevent displacement.

V. Retrofit Options to Control Out of Plane Distortion

As stated in Fisher (1985), three techniques can be used to control out-of-plane distortions:

(1) Drill holes at each end of the high stress areas.
(2) Remove a segment of the connection plate near the stress area to lengthen the web gap.
(3) Bolt the connection plate to the tension flange in the bridge's negative moment areas.

Partial Removal of Connection Plate

Option 2 was studied by removing approximately 6 inches of connection plate (top and bottom) as shown below. As presented in Section VI of this report, Option 2 caused an increase in out-of-plane distortion induced stresses.

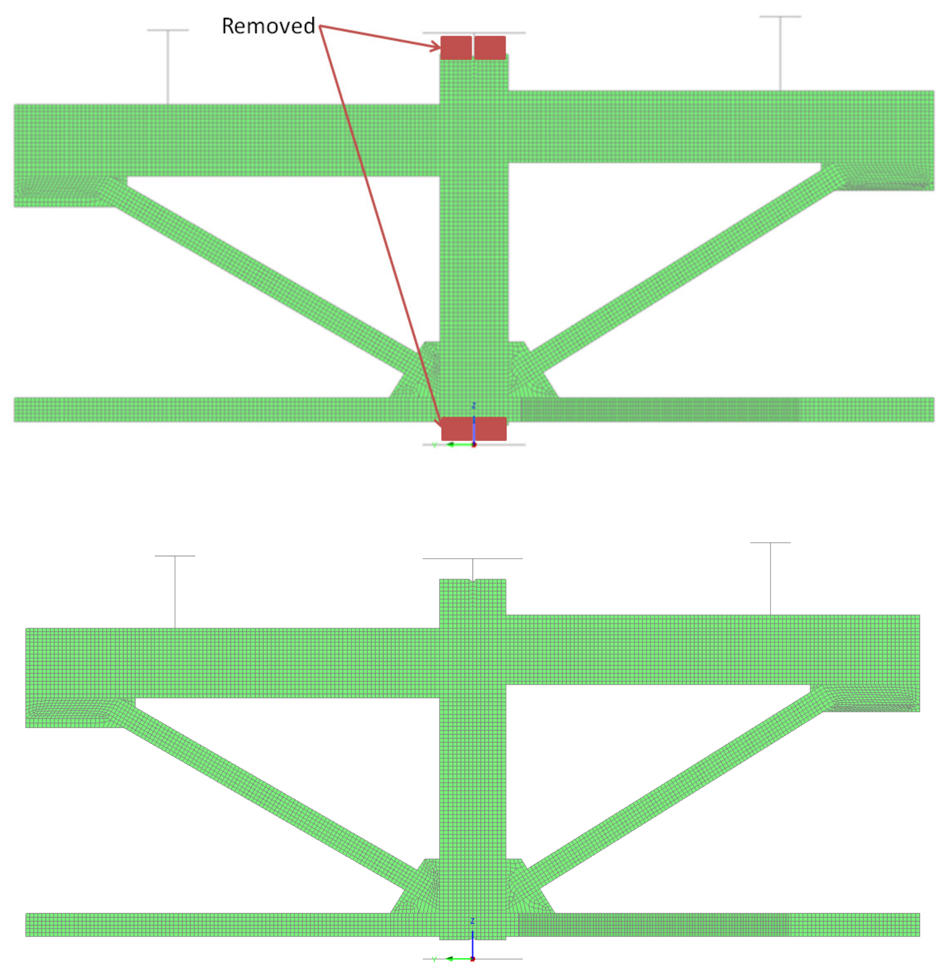
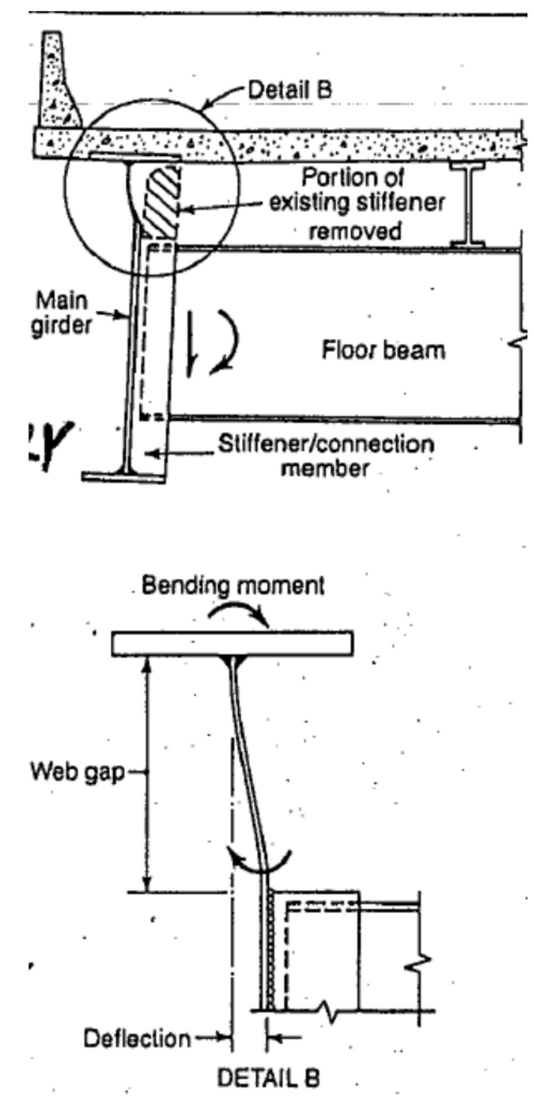


Figure 47 *Positive Moment - Interior – FE model after shorten the CF stiffeners*



If a piece of the stiffener nearest the location of potential cracks is removed, the web gap length is thereby increased and bending stresses in the web are reduced. In some cases this step plus holes drilled at each crack's ends will prevent further crack growth.

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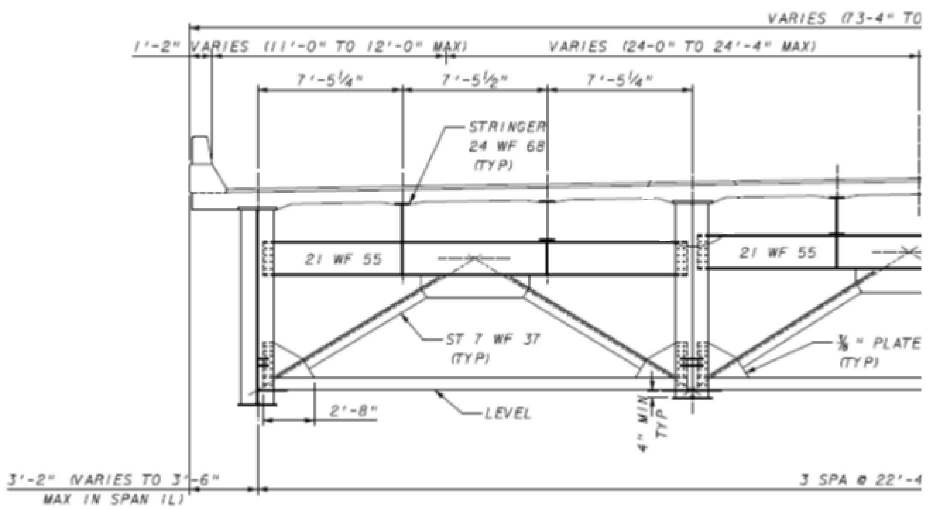
Rigid Connection Retrofit

A bolted connection of the stiffener to the flange is assumed to limit out-of-plane rotation (stresses). Stresses were calculated to assess the effectiveness of this type of connection. Various retrofit options are available to make this connection. Figures 48 – 50 show views of FE mesh used for the modeling of a rigid connection retrofit.



TYPICAL SECTION

CF-1, CF-3, CF-4

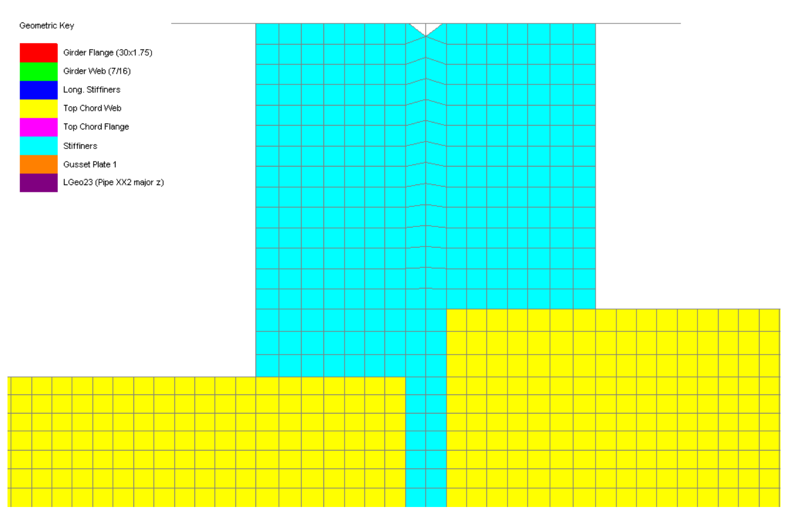


Figure 48 *FE Model - Stiffeners Connected to Girder Top Flange*

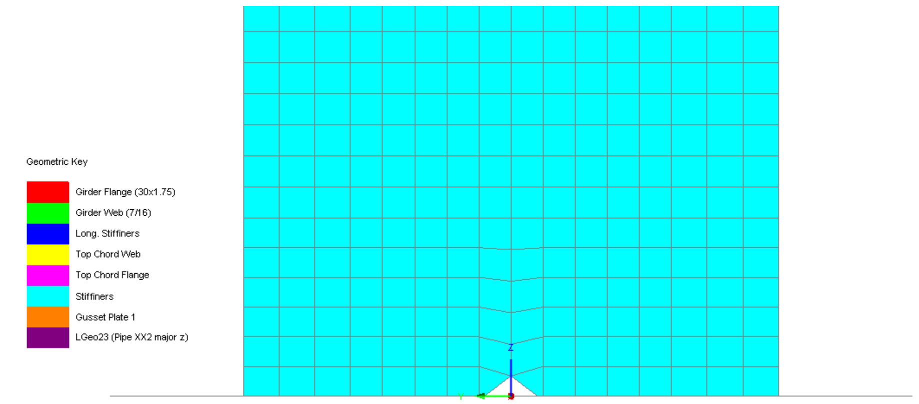


Figure 49 *FE Model - Stiffeners Connected to Girder Bottom Flange*

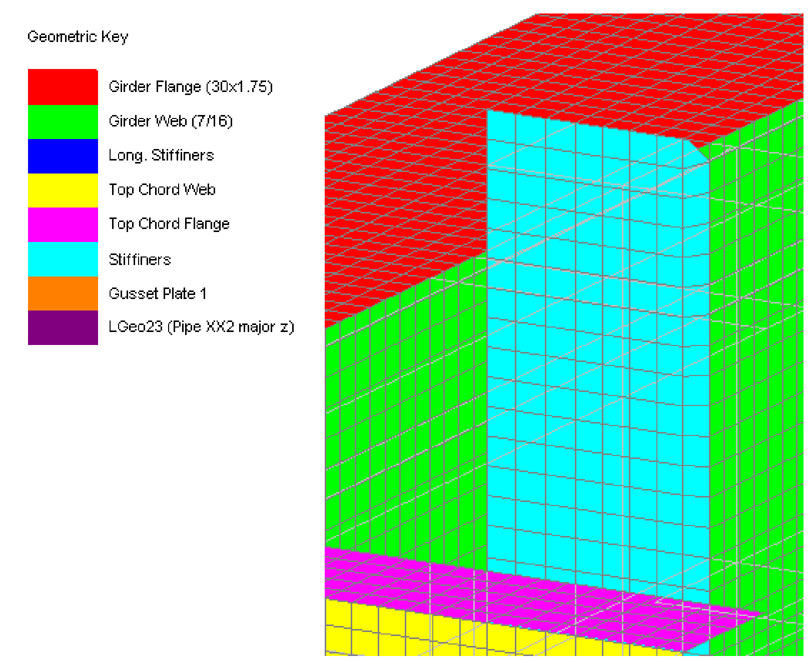


Figure 50 *FE Model - Stiffeners Connected to Girder Top Flange*

VI. Effectiveness of Finite Element Modeling in Predicting Out-of-Plane Distortion Induced Stresses in Bridges

When the adjacent girders deflect unequally under traffic loading, the end of the transverse structural member is forced to rotate, pulling the unstiffened portion of the girder web out-of-plane, creating high secondary stresses at the connection plate end and leading to possible conditions for fatigue cracking. Unlike load-induced fatigue, procedures for prediction of distortion-induced stresses are not in bridge design specifications. Procedures for determination of secondary stresses are not specified in the design or rating process.

The LARSA 4D computer program (version 7.05.35) was used for the 3-D Finite Element Modeling of the eastbound structure (Right Bridge). In the 3-D Model, flanges of girders, floorbeams and stringers were modeled using line/beam elements while the webs were modeled using plate/shell elements. The elements of cross frames and lateral bracing were modeled using line/beam elements. The 3-D FE model was developed using LARSA 4D to create a model that is referred to as a coarse-model. In order to compute out-of-plane distortions, sub-models were prepared using the LUSAS program. This process involved obtaining the forces and moments at the boundary nodal locations from the coarse-model and then applying boundary conditions to the sub-models. The next step was to analyze sub-models and ensure compatibility of the deformed shape of the girders/stringer and stringers between the sub-models and coarse-models. This course-model/sub-model process is required because a single model would result in an unusually large computer model to compute the out-of-plane stresses.

A literature search indicates that the applied procedure as described above is logical, rational and appropriate for this structural performance evaluation. We acknowledge that generally it is preferred to utilize field instrumentation to make measurements so that a calibration process can be used for understanding this relatively large finite element model However, it is not practical to expect to be able to measure the out-of plane strains. Our literature search indicates that statistically we can estimate that the computed strains can be expected to be within approximately 10% of actual field strain values.

VII. Summary

Multilevel 3-D Finite Element modeling designs were developed for the right bridge. The models were used to evaluate:

- The deflections and out of plane movements caused by part-width construction scenarios and
- Possible impacts to fatigue prone bridge components. (Since retrofits were found to be required as a result of this portion of the study, fatigue life of the existing details was not addressed).

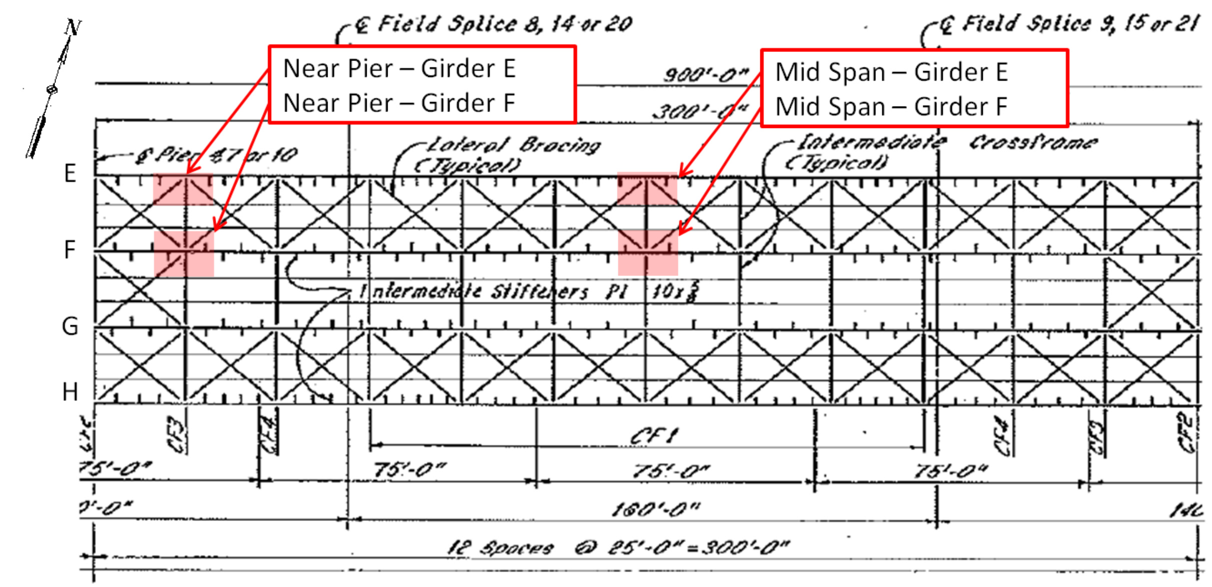
Field observations indicate that the existing deck and steel beams are in full contact and that composite action typically occurs even without a positive connection. Therefore, the stresses determined from modeling the deck and girders as a composite structure were used as the baseline stresses. To ensure a safe deck replacement protocol, it is recommended that the baseline stresses, assuming composite action (only for lateral restraint), should not be exceeded for any deck replacement scheme or MOT sequence.

The overall characteristics of out-of-plane distortion and induced stresses are higher in the positive moment regions, because relatively large differential girder deflections are present. In the negative moment regions near the pier support, the stresses were determined to become smaller.

Retrofit details were evaluated with the objective of controlling distortion-induced stresses. The results from those analyses are presented below.

For the partial removal of the connection plate in the positive region, the available length is less than required 12 inches. According to NCHRP 336, to efficiently release the restrained web, a minimum cut-short dimension of 12 inches or 20 times of the web thickness, whichever is larger, is recommended. Based upon the analysis, available dimensions were found to be inadequate for web gap stress release.

Of the two retrofit options to control out-of-plane distortion, only the rigid plate connection alternative was able to reduce the out-of-plane stresses. This option uses a bolted angle to provide rigid load paths for transmitting forces from the transverse members into the longitudinal girders.



FRAMING PLAN-UNITS 2, 3 & 4

	Stress (ksi) Interior				Stress (ksi) Exterior				Max Deflection (in)	
	Sx	Sy	Sz	S1	Sx	Sy	Sz	S1	Interior	Exterior
MOT (5+1) - Phase I										
Mid Span	80	26	25	100	25	17	25	38	0.054	0.03
Near Pier	50	23	20	51	30	17	20	35	0.02	0.04
EXISTING DECK - NON COMPOSITE DECK										
Mid Span	54	20	21	55	#	#	#	#	0.0216	#
Near Pier	20	12	14	22	49	18	34	66	0.01	0.024
EXISTING DECK MODELED AS COMPOSITE (BENCHMARK)										
Mid Span	45	7	11	46	26	4	11	27	0.024	0.0132
MOT (5+1) - PHASE I -- RETROFIT - PARTIAL REMOVAL OF CONNECTION PLATE										
Mid Span	80	8	288	298	#	#	#	#	0.72	#
Near Pier	20	7	14	22	#	#	#	#	0.12	#
MOT (5+1) - PHASE I - RETROFIT - RIGID CONNECTION PLATE										
Mid Span	44	10	9.7	45	24	8	5	25	0.036	0.012
Near Pier	4	6	4	13	8	6	6	11	0.002	0.036

Note: *# Based upon preliminary analysis, the interior girder controlled the design. Therefore only the interior girder was analyzed.*

Out-of-plane distortion causes very high stresses in the localized region around the web gap. The Figure below shows the cracks due to the various components of out-of-plane distortion induced axial stresses. The high stresses in the X direction cause the initiation of vertical cracks as shown in Figure 51(a). The high stresses in the Y direction may cause failure of the stiffeners web welds leading to stiffener detachment. Relatively high stresses in the Z direction may also cause horizontal cracks. The maximum tension component of the principal stress (S1) is also a critical measure for potential crack initiation.

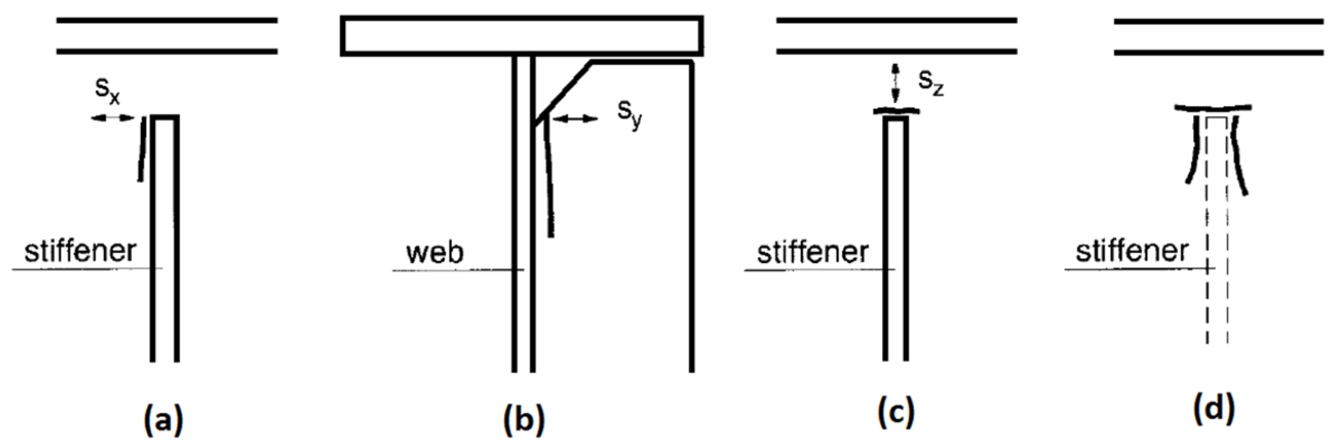


Figure 51 *Cracks due to different axial stresses.*

*Comparison of Pre-Retrofit and Post-Retrofit Factored **Out-of-plane Stresses** at Mid-span (Alt 2.Phase 1).*

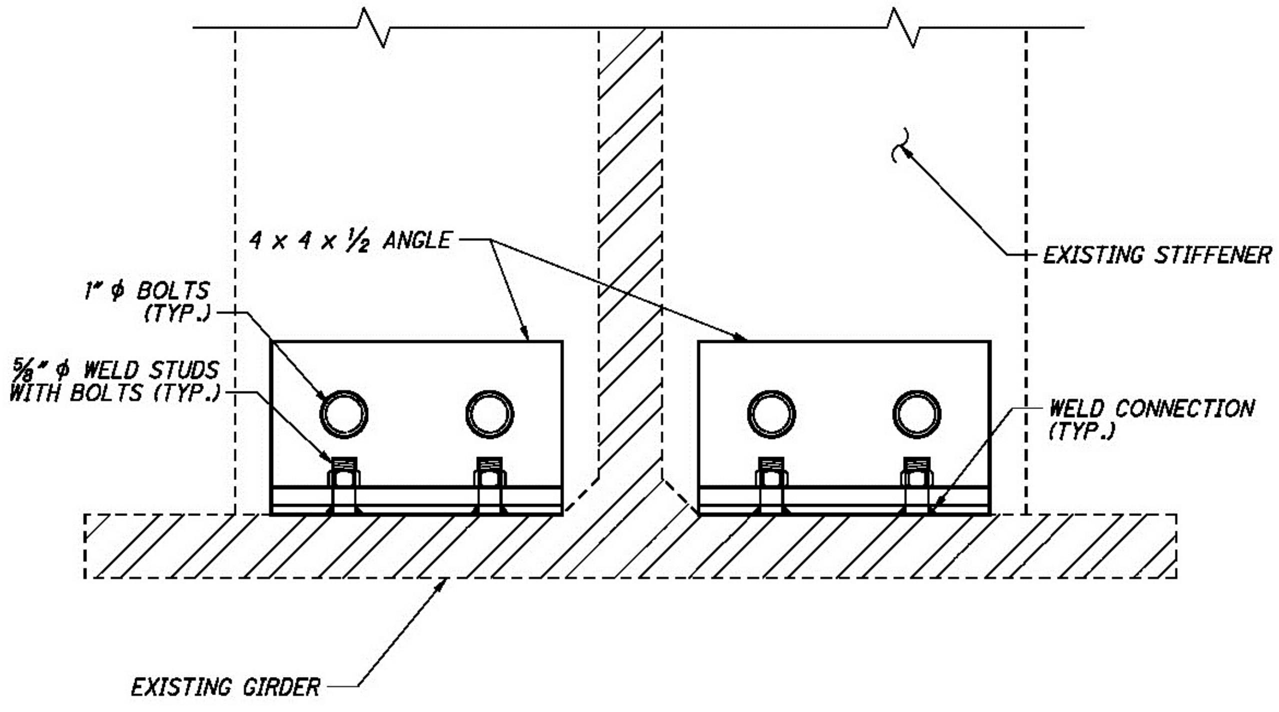
		Positive Moment				Negative Moment				Stress Range			
		Sx (ksi)	Sy (ksi)	Sz (ksi)	S1 (ksi)	Sx (ksi)	Sy (ksi)	Sz (ksi)	S1 (ksi)	Sx (ksi)	Sy (ksi)	Sz (ksi)	S1 (ksi)
Before Adding Rigid Connection	Top Flange	80	26	25	100	33	7	70	71	47	19	45	31
	Bottom Flange	63	10	45	100	17	4	47	48	46	6	2	54
Rigid	Top Flange	0	0	0	0	3	8	4	4	3	8	4	4
	Bottom Flange	44	10	9.7	45	0	0	0	0	44	10	9.7	45

CONSIDERATION OF INSTRUMENTATION FOR VERIFICATION OF THE 3-D MODEL

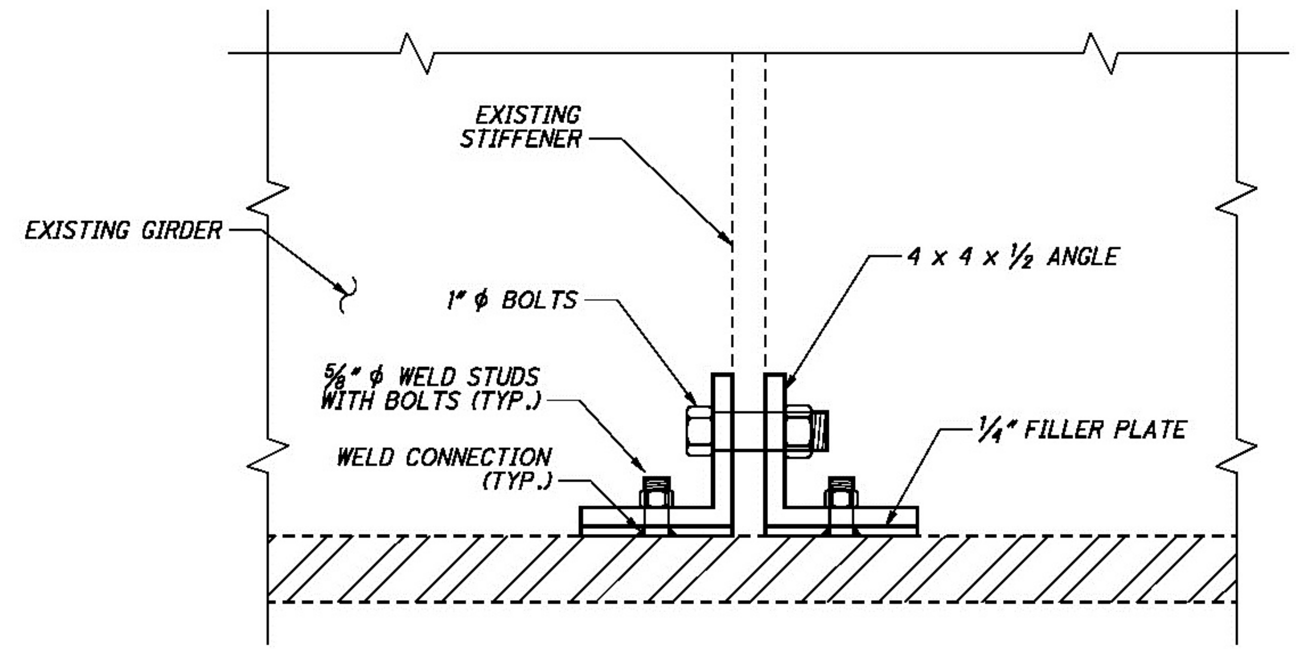
Retrofit options to control out-of-plane distortions were evaluated using 3-D Finite Element Computer Models. Consideration was given to instrumenting the girders for the purpose of validating the 3-D models. The instrumentation on the girders can be used to provide data for refinment of the computer model.

The use of instrumentation was discussed with Professor Dennis R. Mertz, Ph.D.,P.E.. He indicated that the comparison of FE-calculated distortion-induced stresses with field-measured stresses is not so simple or even informative. He reflected from his past experience where he had difficulty trying to measure the out-of-plane stresses in in-service bridges. He stated that, "It is very difficult to place gages and measure these stresses since the web gaps are so small, the strain gages are relatively large and the stress gradients in the gap are large also. The stresses are a maximum at the weld toe yet the center of the gage will be relatively far from the toe. In the end, the measured stresses are really extrapolated stresses not capturing the stress-concentration effect."

He also stated "as our analytical techniques have matured, I think that it is no longer necessary to try to compare or calibrate FE results with field-measured results. In both cases, I believe that trends can be observed but the actual magnitude of the stresses is not readily obtainable. In my opinion, field measuring out-of-plane distortion-induced stresses is a waste of resources and a distraction."



TYPICAL RETROFIT DETAIL



TYPICAL RETROFIT ELEVATION

PART II
HALF-WIDTH DECK REMOVAL STUDY

Introduction

Part I of this study consisted of the development of multilevel 3-D finite element (FE) models used to evaluate deck replacement options. The existing superstructure was modeled and analyzed to establish the present operating condition (maximum stresses) as a benchmark for the performance of the existing structure during the replacement of the deck. The existing girders, stringers and crossframes were evaluated by applying the existing live load and dead load condition. Code checks were performed according to the AASHTO Standard Specifications. The results of the analyses indicated that all primary (in-plane) stresses due to moments and shears were acceptable.

The 3-D FE models provided predictions for superstructure deflections, out-of-plane movements, and possible impacts to fatigue prone bridge components, caused by removing dead load and live load during part-width construction operations. The overall characteristics of out-of-plane distortion and induced stresses were the highest in the positive moment regions because relatively large differential girder deflections are present at the mid-span. At the negative moment regions near the pier supports, the differential deflections were much lower in magnitude.

The 3-D FE models were used to evaluate proposed retrofit details, with the objective of controlling distortion-induced stresses. Of the two retrofit options used to control out-of-plane distortion, only the rigid plate connection alternative was able to reduce the out-of-plane stresses. The rigid plate connection option uses a bolted angle that provides rigid load paths for transmitting forces from the transverse members into the longitudinal girders.

The Part I study results indicate that using part-width deck replacement construction methods with retrofit details was most likely feasible. Therefore, considering maintenance of traffic preferences, ODOT decided to evaluate various lengths of half-width deck removal segments as the main task to be performed and documented in Part II of Volume I. Using the FE models, the deflections and out-of-plane movements caused by the half-width construction loading conditions were computed and evaluated. The stress related impacts to fatigue prone bridge components were studied.

In 1989, retrofits were applied to the stiffener-floorbeam connections in conjunction with the placement of a concrete overlay on the original decks. The retrofits included additional welding at the top flange in the positive moment regions and the removal of 12 inches of the stiffeners in the negative moment regions. The crossframe locations at the piers were not retrofit. Analyses of the negative moment retrofit locations, using the 3-D model, indicated that the retrofits applied to the superstructure did not help to improve or modify the performance of crossframes, as shown by the results provided in Table 8 (page 44). Note that the presence of the deck provides restraint to the top flange of the stringers which is an important contribution to the resistance against out-of-plane movements.

Several half-width deck removal segment lengths were evaluated for the purpose of establishing a preferred design. The preferred design should permit the replacement of the deck for one of the twin structures in one construction season. Multilevel 3-D finite element models were used to evaluate the girder and beam live load and dead load deflections and out-of-plane movements caused by the part-width construction removal of the deck. After evaluating numerous deck removal alternatives, removing a 300 foot long half-width segment centered over a pier was found to be the best construction procedure. Maximum out-of-plane stresses and out-of-plane deflections for a 300 foot long half-width deck removal at various locations along the length of the bridge are presented in Table 9 (page 49).

A project cost estimate and construction schedule are provided in Appendix A. The planning level construction schedule has been provided for the purpose of predicting if it is reasonable to expect a contractor to be able to construct the bridge decks in half-width 300 feet segments in one construction season.

Reconnaissance Information Obtained for Verification of As-Built Conditions

The 3-D FE model prepared to evaluate the originally designed bridge was modified to include the 1989 retrofit details. The 1989 negative moment rehabilitation plans were obtained from the District 12 plan archive files. After reviewing the plans, ELR personnel performed a limited field review of the superstructure during a site visit on May 30, 2012. The purpose of the site visit was to confirm that the retrofits shown in the 1989 plans were performed as detailed and that the retrofits were performing as intended. The two 2" diameter holes drilled at each end of the cracks at the bottom of the transverse stiffeners, where the gusset plate is welded to provide wind bracing, were performing satisfactorily. All fatigue retrofits on the girders that were inspected appear to be functioning as intended, as was confirmed in the latest bridge inspection reports.

The retrofit removal details for the top 12" of the transverse stiffeners in the negative moment region were visually identified, but were not measured to verify the exact dimensions. The web removal details matched the proportions and details shown in the 1989 retrofit plans. The latest bridge inspection report states that there are some overcuts/nicks in the web where the crack arrest holes are drilled; overcuts were not located in the areas of the girders that were inspected.

ELR personnel had discussions with current and past ODOT personnel (Jim Barnhart, George Maki, David Leake, Bonnie Teeuwen, Mike Malloy, and Scott Slack) for the purpose of gaining knowledge of the work that has previously been performed on these superstructures. District 12 has had a history of dealing with several problems related to full depth girder cracks on multi-girder bridges as a result of the details similar to those used for the CUY-480 superstructure. Rather than reacting to cracks after the fact, ODOT promoted the policy of retrofitting known problem details before they could result in undesirable full depth cracks.

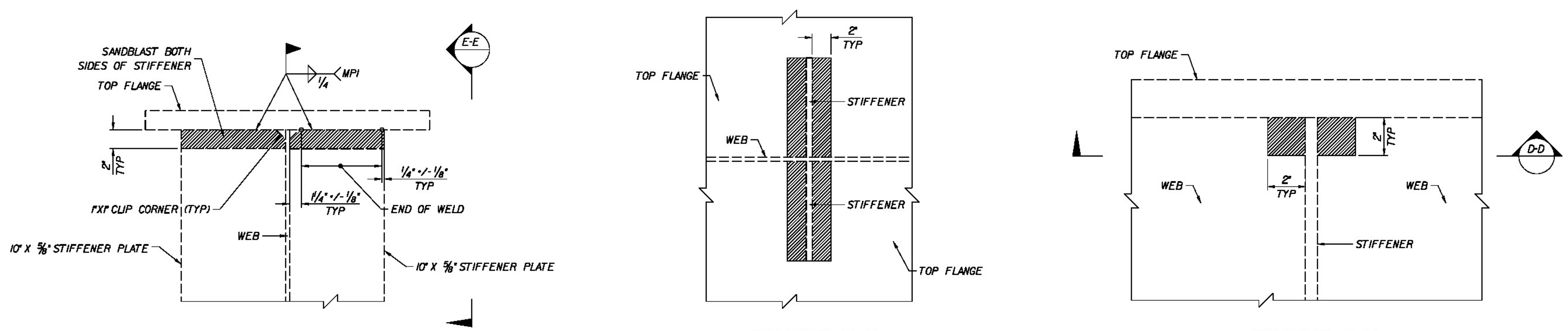
District 12 was proactive in trying to minimize future cracking problems in their bridges by retrofitting structural members that were known to cause problems due to out-of-plane fatigue cracking. This decision was driven primarily by the problems with the I-77 Kingsbury Run Bridge as well as other bridges where fatigue cracking had led to full depth girder cracking. The floorbeam attachment retrofit details used on the

CUY-480 superstructure were developed sometime before the details were developed for the lower lateral retrofits. The primary focus was on the twin girder/floorbeam connections because of non-redundancy issues with twin girder bridges. The initial goal was to minimize the occurrence of cracks due to out-of-plane bending at connections. When a significant crack occurred, emergency contracts were used to provide the repair details. By being proactive, District 12 was able to minimize the necessary number of emergency repair contracts. Even though the CUY-480 Bridge was not a non-redundant twin girder bridge, the structural steel had a history of undesirable fatigue cracking, therefore, since these twin bridges are very significant structures, proactive retrofits were utilized to help minimize the development of future cracking issues.

Superstructure Retrofits Performed in 1989

When the concrete overlay was placed in 1989, ODOT performed two types of retrofits to the stiffener-floorbeam connections:

- At the positive moment regions, where the top flange is in compression, additional welding was provided.
- At the negative moment regions, but not at the pier locations, the stiffeners were retrofitted by cutting out 12 inches of the stiffeners.

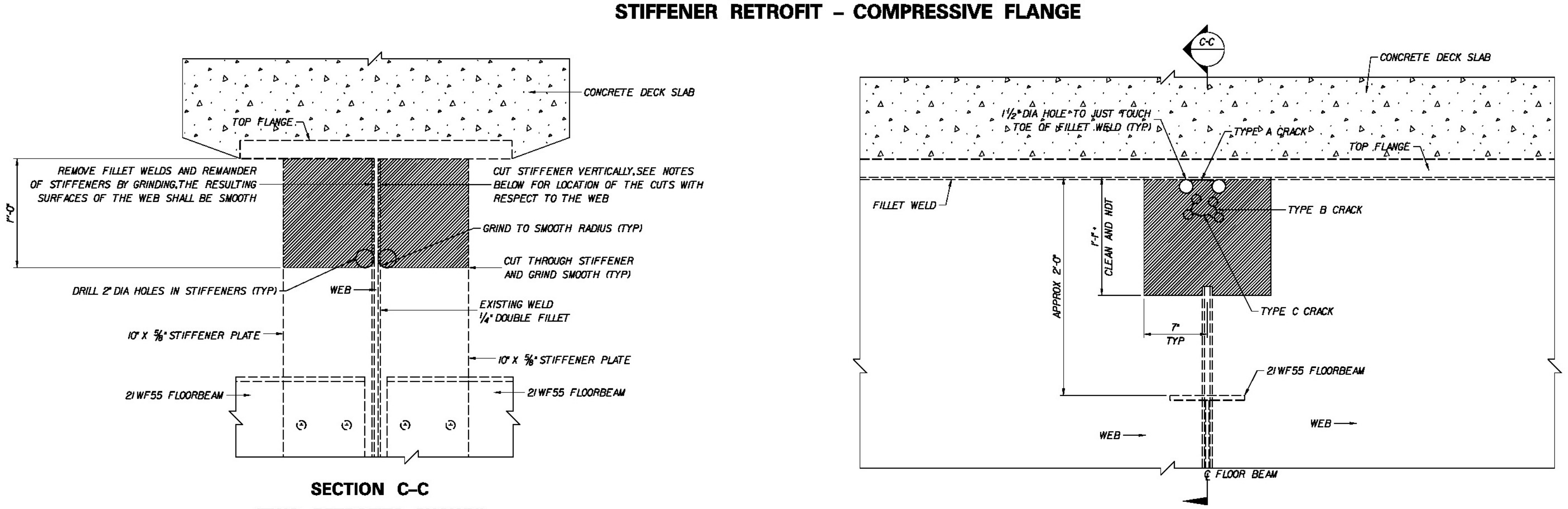


WELDING STIFFENER RETROFIT
1-RETROFIT SHOWN
MPI- MAGNETIC PARTICLE INSPECTION

SECTION D-D

SECTION E-E

STIFFENER RETROFIT - COMPRESSIVE FLANGE



SECTION C-C
(TWO RETROFITS SHOWN)
PORTIONS OF STIFFENERS AND FILLETS TO BE REMOVED

ELEVATION VIEW
INTERMEDIATE STIFFENER END RETROFIT
AREA TO BE SANDBLASTED

STIFFENER RETROFIT - NEGATIVE TENSION FLANGE

The 1989 retrofits consisted of the removal of 12 inches of the stiffener. This retrofit detail was evaluated for its ability to modify the out-of-plane distortion induced stresses. The interior girder was analyzed near Pier 10 in Span 11.The LARSA 4-D model of the entire bridge (coarse model) was utilized to obtain the forces and moments at the boundary nodal locations. As documented in Part I of this Volume, a finer mesh was used for evaluating out-of-plane displacements in the superstructure components.

Table 8 *Comparison of Pre and Post 1989 Retrofits; Factored Out-of-plane Stresses in Span 11 near Pier 10*

	Stress (ksi) Interior		
	Sx	Sz	S1
COMPARISON OF PRE & POST 1989 RETROFITS - NEAR PIER LOCATION			
Pre 1989	6.7	2.9	6.9
Post 1989	5.2	3	6.8

As shown in the table above, the 1989 retrofits were only moderately successful in reducing the out-of-plane distortion induced stresses.

Maintenance of Traffic

Maintenance of traffic is a key component of the CUY-480-18.42 project. The urban-interstate features of this project location along with the constraints associated with removing the existing deck make the design of a desirable maintenance of traffic plan a challenging engineering exercise. The design of a maintenance of traffic scheme that is safe, efficient, and cost effective is a paramount feature of this deck replacement project.

The ODOT Permitted Lane Closure Map/Schedule stipulates that four lanes of traffic in each direction shall be maintained on I-480. There are periods where traffic can be reduced to three lanes in each direction, but those times are for a short durations during nighttime and weekend periods. A reduction to two lanes is permitted, but only during nighttime periods. Due to the nature of the project's construction, these lane reductions are not feasible to perform long-term construction activities, but may be beneficial for delivering materials to the construction site. The scope of services for the project states that for maintenance of traffic, it is desired to maintain three lanes of traffic for both directions. This was used as the minimum number of lanes for each maintenance of traffic scheme analyzed.

Currently, there are four eastbound lanes on I-480 that taper to three lanes just west of I-77. I-480 continues as three eastbound lanes under I-77 and becomes four lanes on the CUY-480-18.42 bridge when I-77 merges into I-480. I-77 traffic to I-480 eastbound consists of two lanes that were formed by two southbound lanes merging with 1 northbound lane. The right lane of I-480 eastbound merges with the left lane of the I-77 ramp traffic. I-480 eastbound continues as four lanes east of the bridge and has a diverge lane to the E 98th Street/Transportation Boulevard interchange.

I-480 westbound is four lanes east of the CUY-480-18.42 bridge with a merge lane from the E 98th Street/Transportation Boulevard interchange. I-480 continues as four lanes across the bridge and has a 3-2 split where three lanes go to I-480 westbound and two lanes to I-77. The I-77 ramp has a 2-1split with two lanes going to I-77 northbound and one lane to I-77 southbound.

It is anticipated that standard construction drawings can be used for lane reductions on I-480 from four lanes to three. Additional signing can be utilized to provide motorists guidance on the lane reductions, shifts, and/or closures. A contra flow MOT scheme is a potential solution. This will require significant advance signing to notify the motorists of the contraflow configuration and the loss of access to ramps from the crossover contraflow lanes. It may be necessary to utilize diagrammatic signing for the contraflow maintenance of traffic scheme.

The existing structures are typically 69.5' wide toe to toe of existing parapets. It was assumed that 12' lanes, one foot minimum barrier offsets, two foot portable concrete barrier (PCB), and a one foot minimum offset behind the PCB would be utilized during maintenance of traffic. Different maintenance of traffic schemes have been investigated to determine the scheme that maintains the required number of lanes in a safe and cost efficient manner. The proposed deck will remain at 69.5' toe to toe of proposed parapets, except for the western end of the eastbound deck which widens to 85'. To facilitate construction, the use of stay in place forms is anticipated. For this project, there is a preference to locate the deck construction joint over a girder.

Part-Width Construction- No Crossover:

If traffic was not crossed over the median and part-width construction with three lanes of traffic in each direction was utilized, then 41' of width in each phase, and 82' total width would be needed. The project does not involve widening the structures, so it was determined that part-width construction with three lanes in each direction is not feasible.

Total Traffic Crossover Option:

The total traffic crossover option with three lanes in each direction would require 78 feet of width. The project does not involve widening the structures, so it has been determined that crossover construction with three lanes in each direction is not desirable. A total traffic crossover using the 69.5' width would limit the lanes to 10.5' each with one foot barrier offsets. The other obstacles to providing a total traffic crossover are as follows: A total traffic crossover option would allow for uninterrupted access for the bridge construction, but provide difficulties with maintaining ramp access and crossover geometrics. On the western end of the bridge, if traffic is crossed over to the westbound structure, then access from I-77 SB and NB to I-480 eastbound would be difficult. This is due to having to cross over from the eastbound side of I-480 to the westbound side of I-480 in the short distance between the Brecksville Road overpass bridge and the CUY-480-18.42 abutment, which is only about 600' +/-. Speed reductions may be necessary to facilitate these geometrics. It could be determined that this ramp movement would need to be detoured if the geometrics could not be worked out. When traffic is crossed over on the westbound structure, access to the eastbound I-480 to E. 98th Street/Transportation Boulevard ramp would be difficult. The existing off-ramp is in close proximity to the CUY-480-18.42 abutment so a temporary ramp and/or pavement may be required to provide access for this movement or this movement may need to be detoured if geometrics cannot be worked out. When traffic is crossed over to the eastbound structure, the E. 98th/Transportation Boulevard to I-480 westbound and the I-480 westbound to I-77 NB and SB ramp movements would be difficult due to their proximity to the CUY-480-18.42 abutments and the Brecksville Road overpass.

Since the part-width and total crossover options do not safely maintain three lanes in each direction on the existing structures, it was determined that a contra flow style maintenance of traffic scheme should be utilized. The contra scheme will be able to maintain more lanes of traffic while providing access to the ramps in a safe and efficient manner. Ramp traffic will most likely be shifted, but will utilize the existing ramp pavement and shoulders. A contraflow maintenance of traffic scheme is the recommended option for redecking the structure while providing the minimum number of three lanes in each direction.

Contraflow Crossover to Westbound Bridge:

This option could be phased so that there are three westbound and two eastbound lanes on the existing westbound bridge. The existing eastbound bridge would provide for two eastbound lanes (one lane from I-480 eastbound merging with two lanes from I-77). The eastbound bridge will be constructed part-width (in halves) in two phases by utilizing the existing deck and then the newly constructed deck. This option will allow three lanes of I-480 westbound traffic and provide four lanes for I-480 eastbound traffic (two lanes crossed over and two lanes on the eastbound bridge).

Contraflow Crossover to Eastbound Bridge:

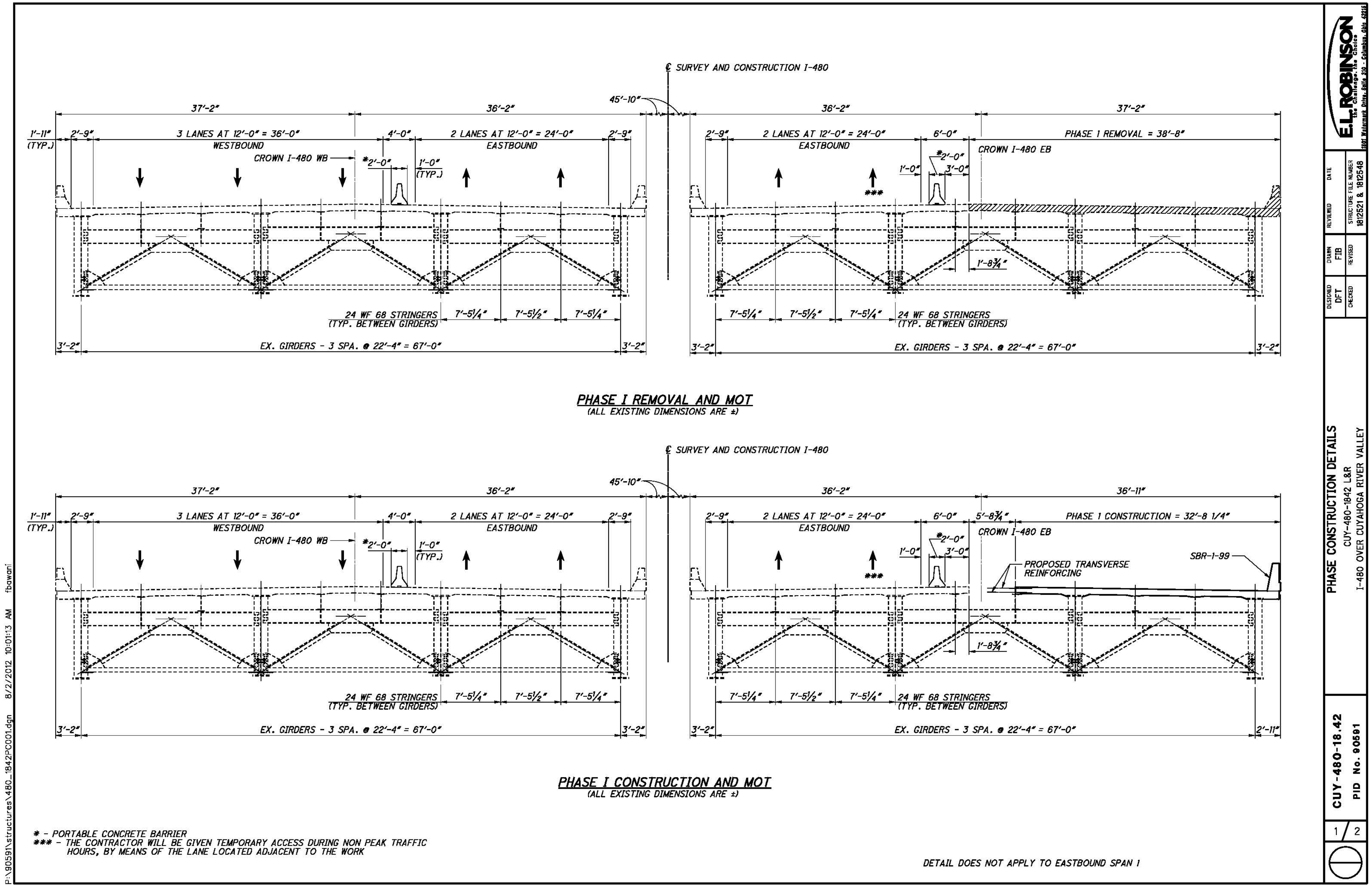
Once the eastbound bridge is completed, 2 lanes of I-480 westbound traffic will now be crossed over to the eastbound structure and the westbound structure will be completed part width in two phases similar to the eastbound structure. There can be 3 eastbound lanes and two westbound lanes on the eastbound bridge, with 2 westbound lanes on westbound bridge. In both phases, westbound I-480 traffic will be able to access the I-77 ramps.

Contraflow Summary:

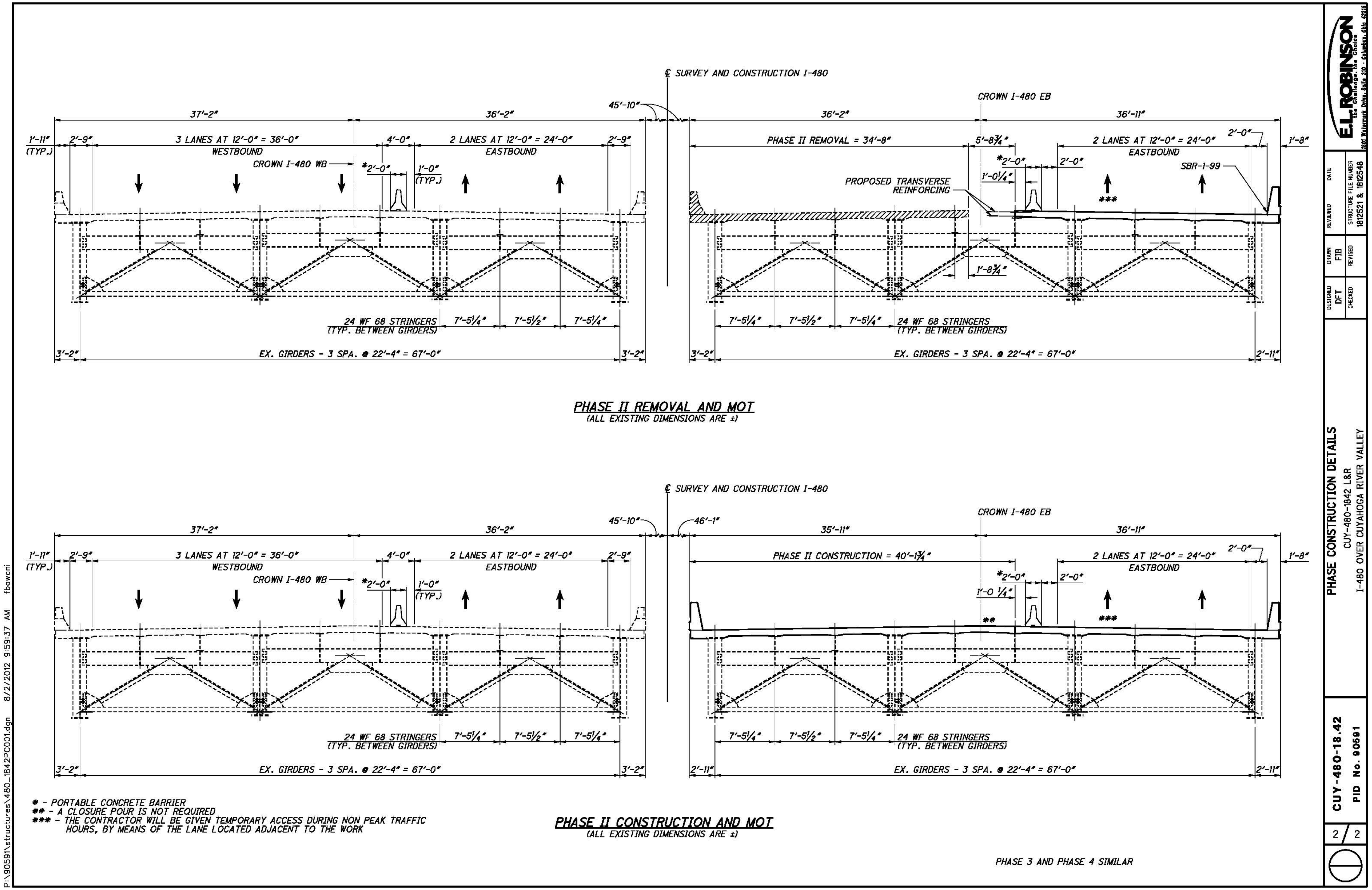
The contraflow maintenance of traffic option reduces capacity by one lane in each direction during different construction seasons. There will be four lanes in one direction and three lanes in the opposite direction per phase. All crossovers and maintenance of traffic zones provide a minimum of two lanes. This is extremely advantageous if a breakdown were to occur. For these reasons, the contraflow crossover scheme as described above is the recommended configuration.

If the CUY-77.9.50 project was under construction at the same time as the CUY-480-18.42 project, there may be potential for conflicts with the physical MOT zones. Coordination between the projects would be required, mainly, for Ramp E-N, I-480 westbound to I-77 northbound and Ramp N-E, I-77 southbound to I-480 eastbound, both 2 lane ramps. In certain phases of the CUY-77-9.50 MOT, these ramps are restricted to one lane. If these projects were constructed at the same time, it may be desirable to keep these ramps as one lane ramps where it would benefit both projects.

The next two pages in the report provide the deck replacement phase construction work dimensioned in section views of the superstructure.



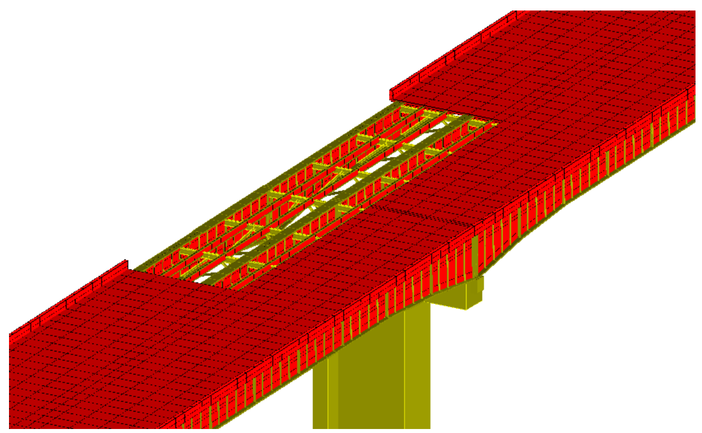
E.L. ROBINSON
ENGINEERING


℄ SURVEY AND CONSTRUCTION I-480
CROWN I-480 EB
37'-2"
36'-2"
45'-10"
36'-2"
36'-11"
1'-11" (TYP.)
2'-9"
3 LANES AT 12'-0" = 36'-0"
WESTBOUND
4'-0"
2 LANES AT 12'-0" = 24'-0"
EASTBOUND
CROWN I-480 WB
*2'-0"
1'-0" (TYP.)
PHASE II REMOVAL = 34'-8"
5'-8¾"
2'-0"
1'-8"
PROPOSED TRANSVERSE REINFORCING
1'-0¼"
SBR-1-99
***
1'-8¾"
24 WF 68 STRINGERS (TYP. BETWEEN GIRDERS)
7'-5¼"
7'-5½"
3'-2"
EX. GIRDERS - 3 SPA. @ 22'-4" = 67'-0"
2'-11"
PHASE II REMOVAL AND MOT
(ALL EXISTING DIMENSIONS ARE ±)
46'-1"
35'-11"
PHASE II CONSTRUCTION = 40'-1¾"
1'-0 ¼"
**
PHASE II CONSTRUCTION AND MOT
(ALL EXISTING DIMENSIONS ARE ±)
* - PORTABLE CONCRETE BARRIER
** - A CLOSURE POUR IS NOT REQUIRED
*** - THE CONTRACTOR WILL BE GIVEN TEMPORARY ACCESS DURING NON PEAK TRAFFIC HOURS, BY MEANS OF THE LANE LOCATED ADJACENT TO THE WORK
PHASE 3 AND PHASE 4 SIMILAR
P:\90591\structures\480_1842PC001.dgn 8/2/2012 9:59:37 AM fbawani
E.L. ROBINSON
DESIGNED DFT
DRAWN FJB
REVIEWED
DATE
STRUCTURE FILE NUMBER 1812521 & 1812548
PHASE CONSTRUCTION DETAILS
CUY-480-1842 L&R
I-480 OVER CUYAHOGA RIVER VALLEY
CUY-480-18.42
PID No. 90591
2/2

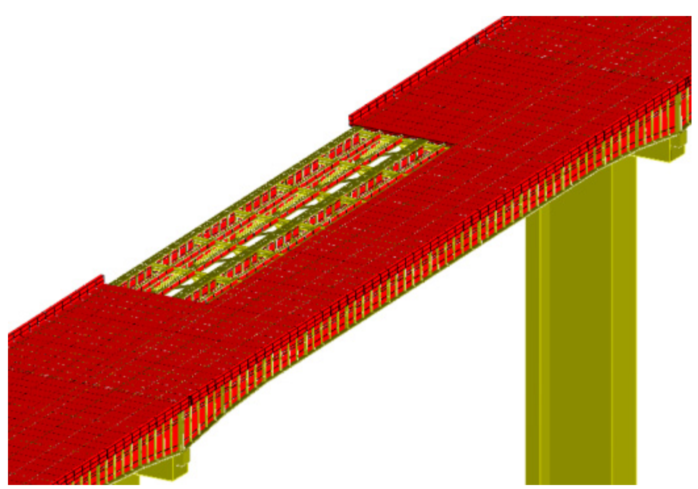
Deck Removal Segments

The goal of this study is to develop a plan to replace the deck for one of the twin structures in one construction season, and then replace the adjacent superstructure deck in the following construction season. The following half-width deck removal segments were evaluated for out-of-plane distortion induced stresses. The results of these out-of-plane distortion evaluations were used to formulate a one construction season deck replacement plan. The following options were evaluated:

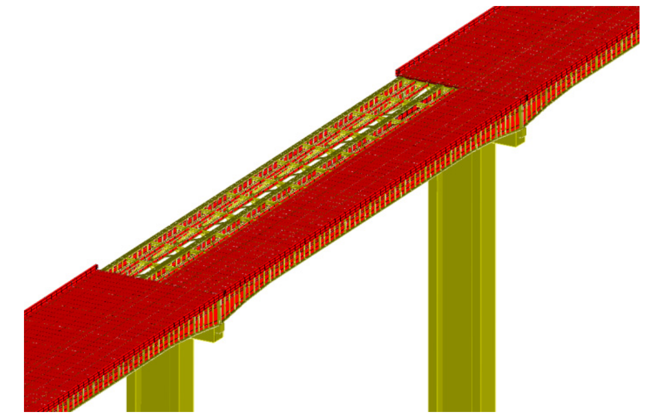
1. 150 feet deck removal centered over a pier, 75 feet removed on each side of the pier
2. 150 feet deck removal centered at mid-span.
3. 300 feet deck removal from centerline of pier to centerline of pier
4. 300 feet deck removal centered over a pier, 150 feet removed on each side of the pier.
5. 600 feet deck removal centered over a pier, 300 feet removed on each side of the pier



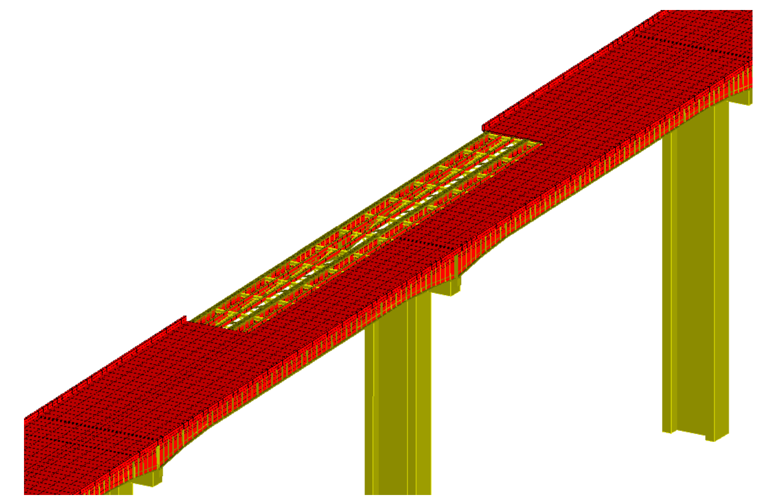
1) 150 feet Deck Removal – 75 feet on each side of pier



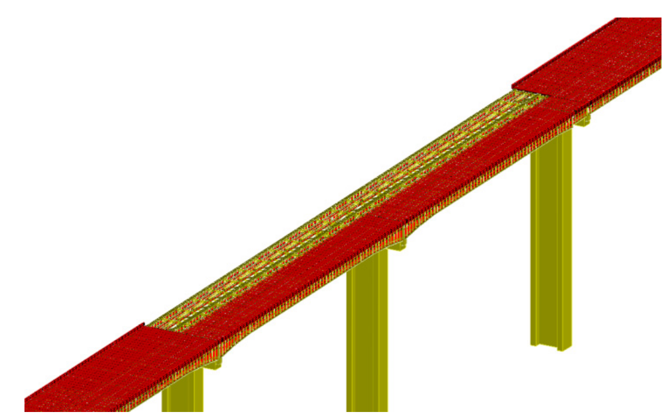
2) 150 feet Deck Removal at mid-span



3) 300 feet Deck Removal – Pier to pier



4) 300 feet Deck Removal – 150 feet on each side of pier

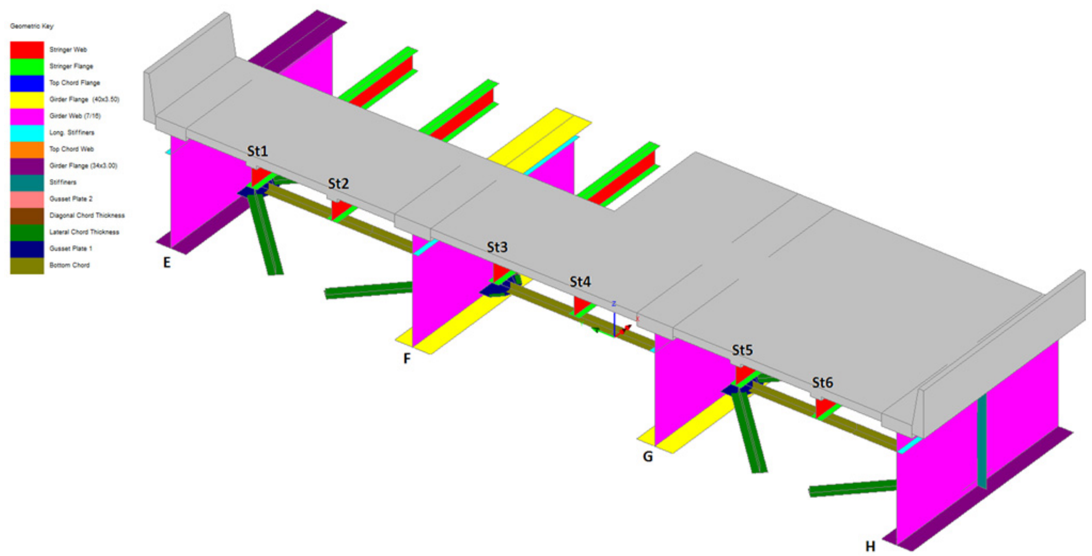


5) 600 feet Deck Removal

Out of Plane Distortion Induced Stresses for the Half-Deck Removal Segments

Multilevel 3-D finite element models were developed for the right bridge. The models were used to evaluate the girder and beam deflections and out-of-plane movements caused by the part-width construction removal of live load and dead loads. The impacts to fatigue prone details were also evaluated.

As stated in Part I of this report, the LARSA 4-D model of the entire bridge is considered to be a coarse model. Sub-models were prepared using the LUSAS program because the LUSAS program has the ability to provide advanced mesh generation features which use a finer mesh for evaluating the out-of-plane displacement in the superstructure components. These sub-models are necessary to compute the out-of-plane distortions and related stresses to a reasonable desired accuracy.



Properties Assignment in the Sub-Model

Sub-models capturing the relevant three-dimensional out of plane displacements were prepared. Relative stress level results were computed for the following span removal segments:

1. 150 feet deck removal, 75 feet on each side of the pier
2. 150 feet deck removal centered at mid-span.
3. 300 feet deck removal, centerline of pier to centerline of pier
4. 300 feet deck removal, 150 feet on each side of the pier
5. 600 feet deck removal, centered over a pier

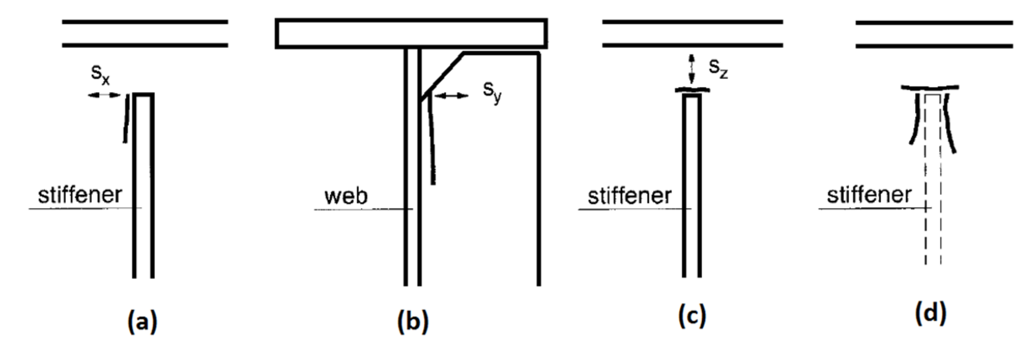
Table 9 (page 49) provides a comparison of the out of plane distortion induced stresses for the half-deck removal segments listed above.

When the 300 feet long deck segment is removed, centerline of pier to pier, there is a significant increase in the out of plane stresses, especially for the exterior girder. When a 300 feet long segment **centered** at a pier is removed, the resulting distortion induced stresses are acceptable because the stresses are less than the 46 ksi benchmark stress; therefore retrofits are not necessary when this removal option is used. Based on the results of these analyses, additional evaluation work was performed for the purpose of understanding the deck removal option where 300 feet of deck is removed in segments centered over a pier.

Table 9 *Comparison of out-of-plane distortion induced stresses (factored) for half-deck removal segments*

	Stress (ksi) Interior				Stress (ksi) Exterior			
	Sx	Sy	Sz	S1	Sx	Sy	Sz	S1
(1)	At Near Pier - Deck Removal (150 ft) - 75 ft on each side of the Pier							
After Deck Removal	2	10	15	17	2	5	2	7
Existing w/ 1989 Retrofit	5.2	1	3	6.8	#	#	#	#
(2)	At Mid Span - Deck Removal (150 ft - mid span)							
After Deck Removal	43	4	16	44	40	2	8	41
Existing	45	7	11	46	26	3	11	27
(3)	At Mid Span (300 ft Deck Removal) - CL Pier to CL Pier							
After Deck Removal	43	1	14	44	46	4	18	47
Existing	45	7	11	46	26	3	11	27
(4)	At Mid Span (300 ft Deck Removal - 150 ft on each side of the pier)							
After Deck Removal	30	2	6	31	24	1	6	25
Existing w/ 1989 Retrofit	45	7	11	46	26	3	11	27
(5)	600 ft Deck Removal - MIDSPAN of SPAN 11							
After Deck Removal	48	3	10	49	49	4	15	49
	600 ft Deck Removal - MIDSPAN of SPAN 12							
After Deck Removal	50	11	7	51	46	12	9	47
Existing	45	7	11	46	26	3	11	27

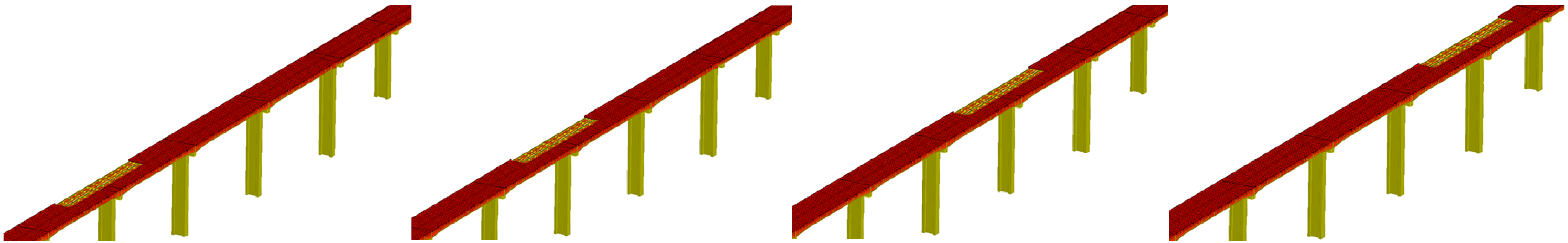
\# Based on preliminary analyses, the interior girder controlled. Therefore, only the interior girder was analyzed.



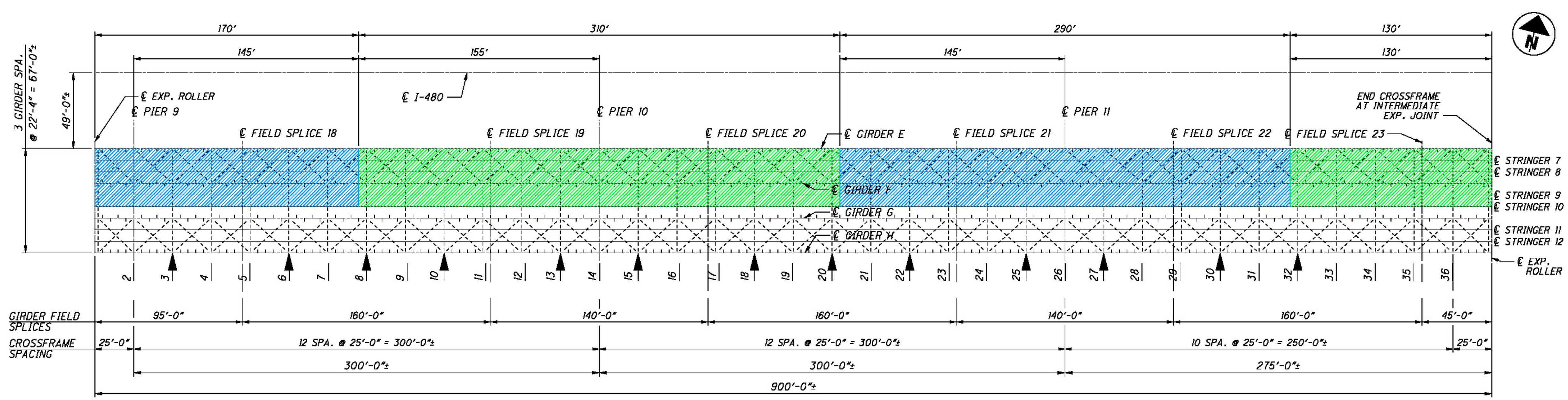
Sx = Stress in x-direction i.e. along the length of the girder (Refer to Figure a)
Sy = Stress in the y-direction i.e. perpendicular to the length of the girder (Refer to Figure b)
Sz = Stress in the vertical direction i.e. along the depth of the girder (Refer to Figure c)
S1 = Maximum principal stress (Refer to Figure d)

Evaluation of the Removal of the 300 feet long Half-Width Deck Segment

Since the out-of-plane stresses are at acceptable levels when the removal of a 300 feet long half-deck segment is centered at a pier, retrofits are not required for this option. Multilevel 3-D finite element modes were developed for the sequence of the deck removal segments in UNIT 4 as shown below.



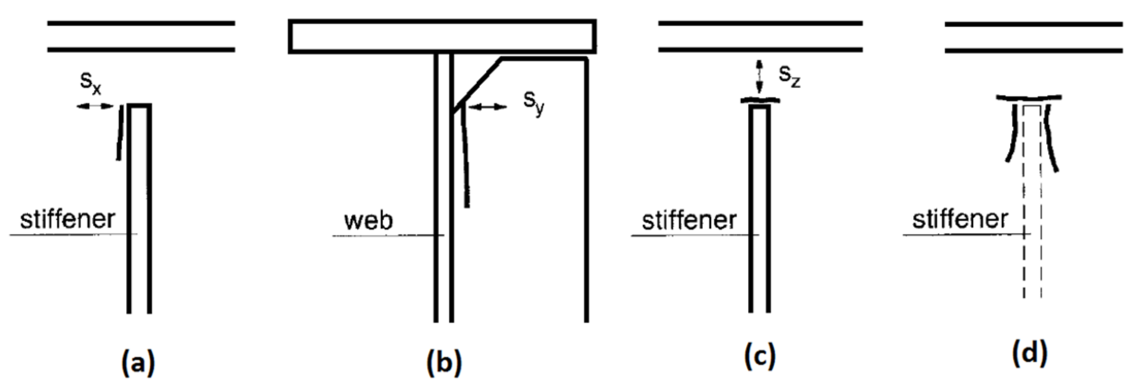
The figure below provides a location key for the results of the analyses. The change in color of the segments represents the limits of the each deck removal segment.



LOCATION NUMBERING FOR SUB-MODELS

***Table 10** Summary of the out-of-plane distortion induced stresses (factored loads) and for the unfactored deflections for a 300 feet half-deck removal segment centered at a pier:*

		Stress (ksi) Interior				Stress (ksi) Exterior				Out-of-Plane Distorsion	
	Location	Sx (ksi)	Sy (ksi)	Sz (ksi)	S1 (ksi)	Sx (ksi)	Sy (ksi)	Sz (ksi)	S1 (ksi)	Interior (in)	Exterior (in)
SPAN 10	03	1	3	3.4	3.6	6.1	5.0	3.7	6.4	0.00756	0.00402
	06	8.2	5	22	24	10.5	4.0	19	20	0.01800	0.02400
	08	30	6	9.5	31	22	4.2	3	23	0.04200	0.04200
	10	30	6	29	31	22	4.9	18.5	22	0.04800	0.03000
	13	2.8	1	1.5	3.1	0.25	3.0	5.3	5.6	0.00520	0.00630
SPAN 11	15	3.3	3	4.7	4.7	15.1	2.0	2	15.3	0.00468	0.00180
	18	19	1	19	20	13.9	1.0	13.5	16.8	0.01920	0.00300
	20	30	2	6	31	24	1.0	6	24	0.04800	0.01800
	22	29	2	20	29	22	1.0	11	23	0.00960	0.01680
	25	4.3	5	8.2	8.5	7.8	7.0	3.8	8.9	0.00348	0.00144
SPAN 12	27	9.6	2	5.3	9.8	6.0	5.0	4.8	6.5	0.00370	0.00200
	30	41	6	31.5	42	35.5	3.0	26	37	0.06600	0.01440
	32	41	5	17.5	42	20.5	3.0	17	31	0.02400	0.03600



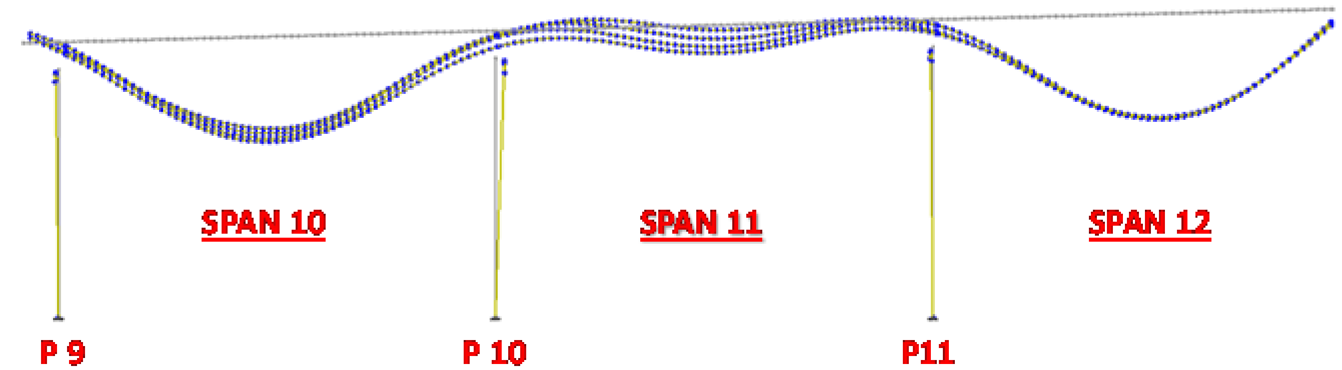
Sx = Stress in x-direction i.e. along the length of the girder (Refer to Figure a)
Sy = Stress in the y-direction i.e. perpendicular to the length of the girder (Refer to Figure b)
Sz = Stress in the vertical direction i.e. along the depth of the girder (Refer to Figure c)
S1 = Maximum principal stress (Refer to Figure d)

As shown in Table 10 (page 51), the computed maximum out-of-plane stresses are below the 46 ksi existing condition threshold value that was established in the Part I evaluation of the superstructure. Therefore, the half-width deck replacement option can be accomplished using the removal of 300 feet segments centered at a pier without the use of any additional retrofits.

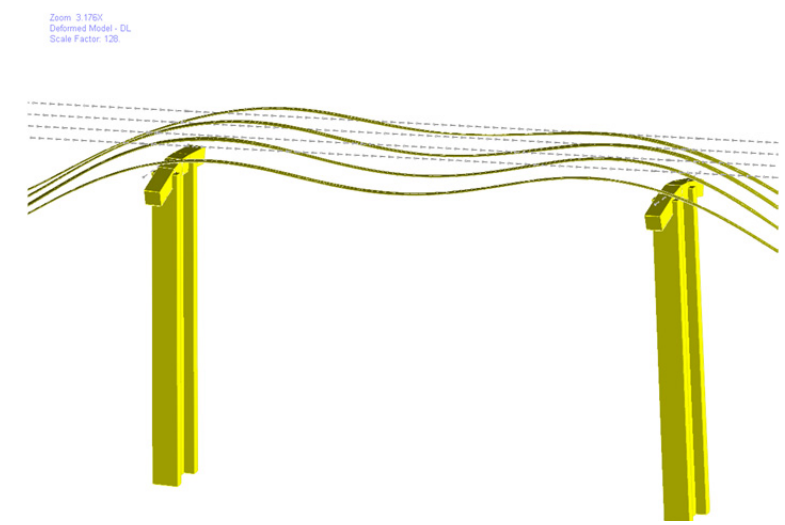
The analyses for this evaluation are provided in detail in Volume II.

Dead load & Live Load Deflections

Dead load and live load deflections for Girders E, F, G, & H are presented for Spans 10 & 11 when a 300 feet long half-width deck segment is removed with 150 feet removed on each side of Pier 10. The deflections of the pier and pier cap are shown in Figures (a) & (b). Deflections were calculated for the purpose of verifying that the 3-D FE model provides results that are consistent with common sense engineering expectations. As shown below, the differential deflection between adjacent girders F & G is approximately one (1) inch when the half-width deck is removed. The live load differential deflection is approximately 1 inch between girders F & G. The live load differential deflections, which may include vibrations, should be mitigated by lowering the traffic speed during deck pours and also closing the adjacent lane until sufficient concrete set has been achieved.



(a) Girder deflected shapes for spans 10, 11 & 12 including piers during half-deck removal of 300 feet centered at pier 10



(b) Pier 10 & 11 including pier cap deflections during deck

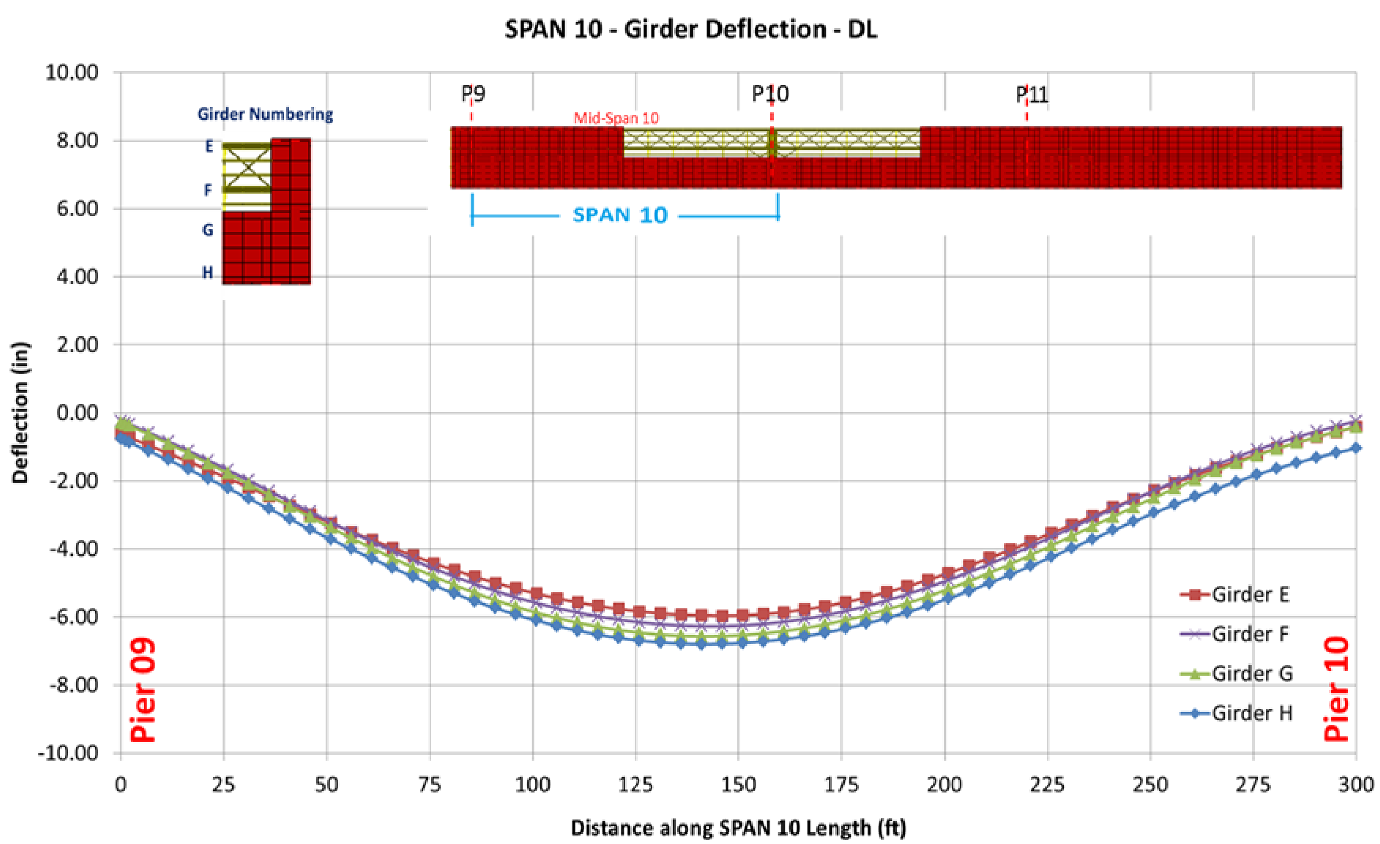


Table 11 LL Deflections at Mid Span 10

Span 10	
Girder	Live Load Deflection at Mid Span (inches)
E	1.32
F	2.5
G	3.46
H	3.98

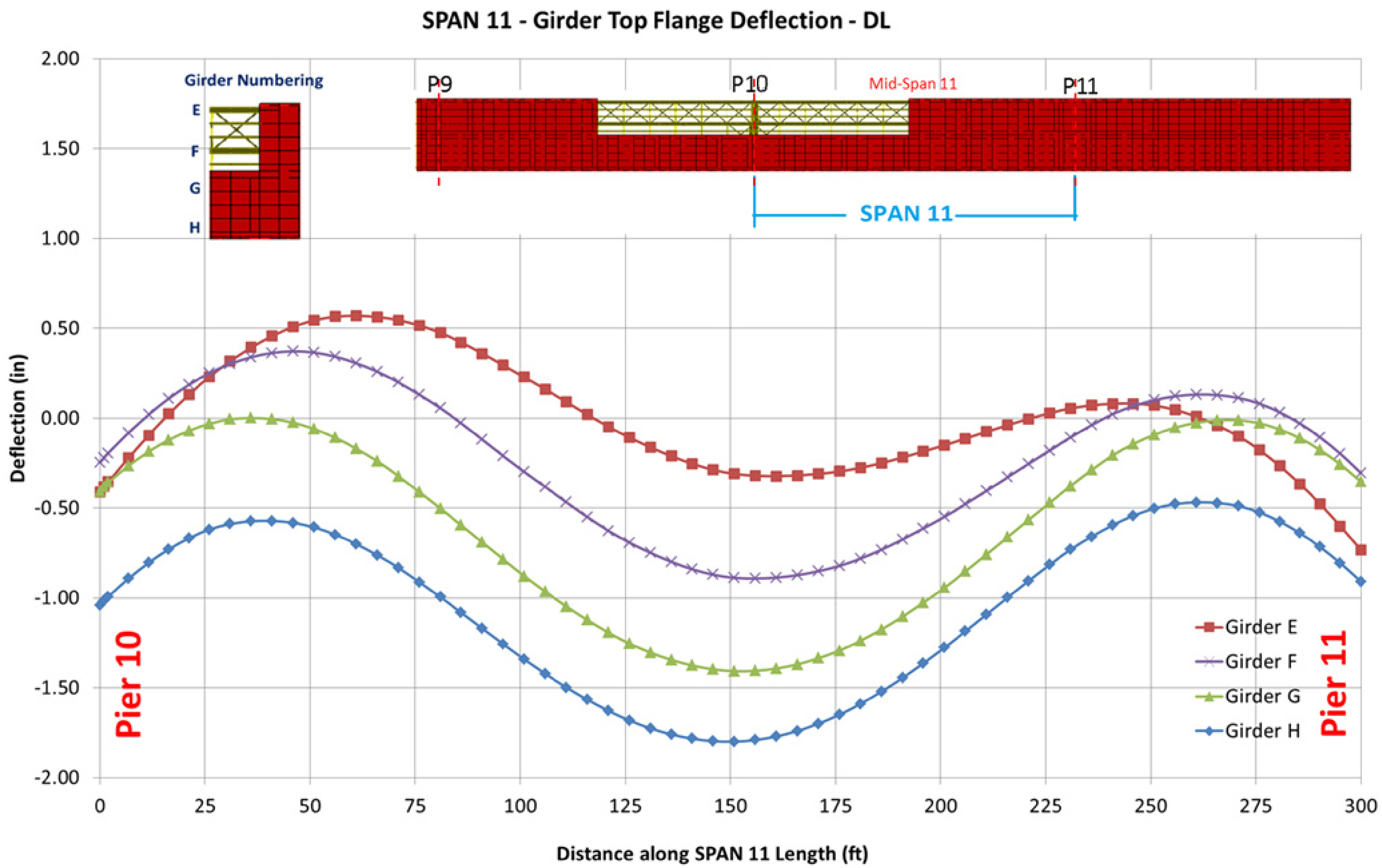


Table 12 LL Deflections at Mid Span 11

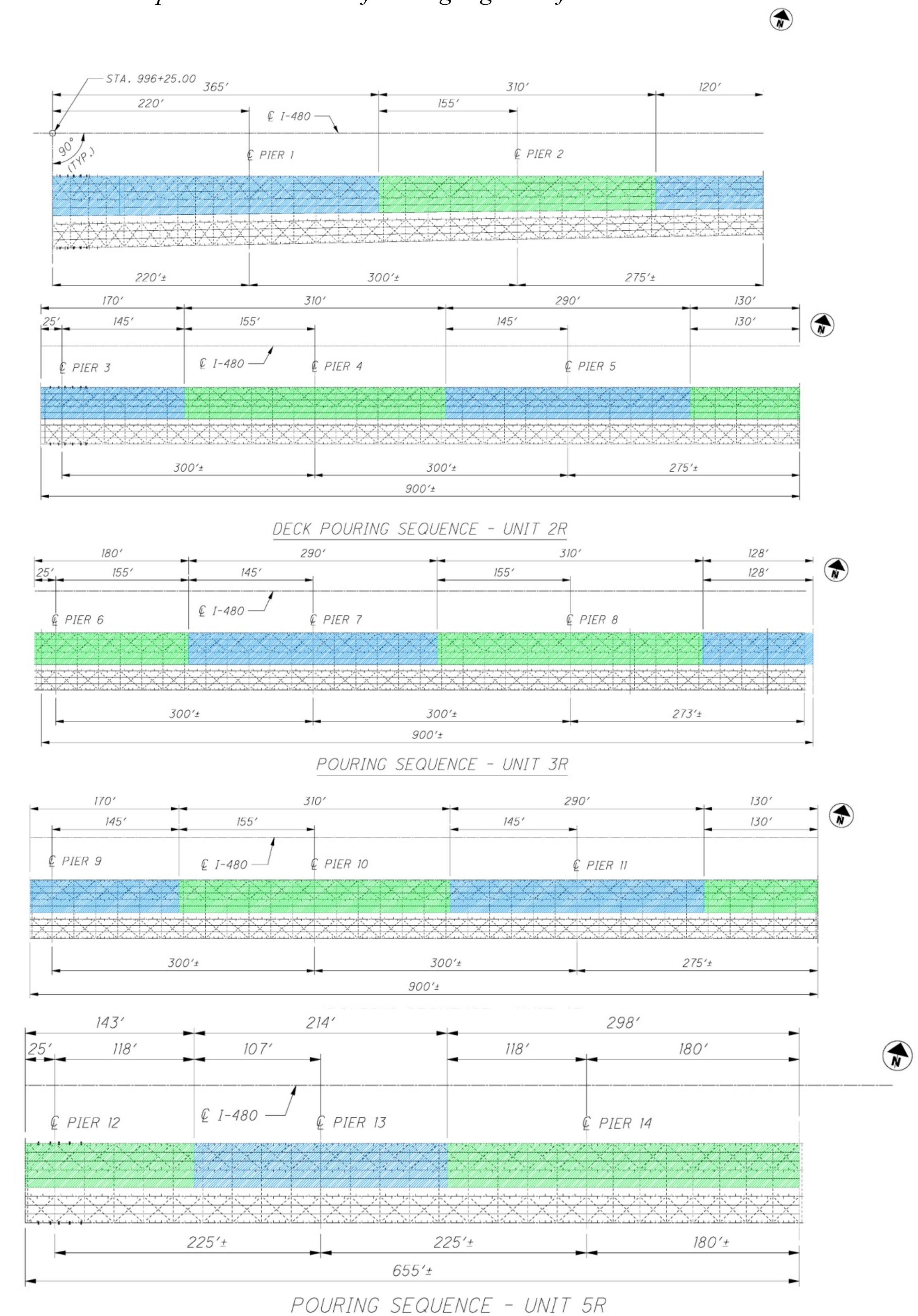
Span 11	
Girder	Live Load Deflection at Mid Span (inches)
E	0.92
F	1.75
G	2.44
H	3.51

Discussions with Contractors

Information regarding the replacement of the decks for the CUY-480 twin structures was solicited from three contractors The Ruhlin Company, The Great Lakes Construction Company and The Kokosing Construction Company. Below is a list of generalized statements received from these contractors:

1. In order to replace the deck for one of the twin superstructures within in a 9 month construction season time frame, consensus is that at least 300 feet long deck removal segments must be permitted (see figure on this page). It is a very aggressive schedule to complete two 300 feet long removal/replacements in 8 days while working two 10 hour shifts.
2. Allowing the closure of an additional lane to place concrete at night would be very beneficial (10 hour time frame).
3. The cost of providing retrofits may offset the advantage gained with the larger 600 feet deck section removal and replacement option. It is a very aggressive schedule to complete two 300 feet long removal/replacements every 2 weeks
4. The 300 feet option seems possible but would require 2 operations, double shifts and would be expensive.
5. A large crane or cranes may be needed below to furnish rebar. Possibly in prefabbed mats.
6. Could slip forming be used for the barriers?
7. Precast barriers should be considered.
8. Note: time is needed in the schedule to construct the barrier.
9. All access efforts to the site will be a challenge.
10. Are the expansion joints being replaced? If so that will impact schedule as well.
11. Consensus was that the Contractor would platform the entire bridge for access for the steel repairs, for safety and for containment of debris removal. Since the repairs need to be completed prior to removal and replacement of the deck, it may be necessary to platform both bridges. This would be very expensive.
12. Our steel retrofit expert studied the retrofit detail provided by ELR and he estimates each retrofit detail will cost in the $2500-$3000 range (per each for labor and materials, included is a foreman's pickup, welder, and compressor). This cost doesn't include any major access money since we don't know exactly what the conditions would be (i.e. would we be using a snooper truck, man lift from underneath, installing a full under deck system, etc...)
13. For a retrofit detail in the negative moment region the ELR detail shows 7/8 inch welded threaded studs on the underside of the top flange. If they want them "shot" on like a shear stud that is not going to happen, as you may have the same problem we always run into on the horizontal studs, but much worse. We would have to stick weld them or use a smaller diameter threaded stud. The maximum diameter would be 5/8 inch.

Deck pours based on 300 feet long segments for Unit 1R thru 5R.



Conclusions

The objective of this study was to determine a plan that would best permit the construction of new decks for the CUY-480-1842 twin structures while avoiding undesirable out-of plane induced stresses. ODOT has placed the following desired constraints on the design and construction of the new decks:

1. Do not generate any out-of-plane stresses in the superstructure that are higher than 46 ksi, which has been established as the baseline upper stress limits value (as computed by the 3-D model).
2. Maintenance of traffic barriers should not be present during snow removal operations. Any work that disrupts the flow of traffic should be avoided during the winter months.
3. The deck for one of the twin superstructures must be removed and replaced in one construction season.
4. Attempt to avoid the use of mechanical splices when constructing the new deck.
5. Address the use of retrofit details.

After evaluating numerous deck removal alternatives, removing a 300 feet long half-width segment centered over a pier was found to be the most conservative construction procedure. By centering the 300 feet deck removal and replacement work over a pier, the out-of-plane stresses in the web were found to be at an acceptable level, therefore, retrofits are not necessary for controlling stresses caused by deck replacement work when using this design alternative. The restraint provided by the reinforced concrete deck to the top flange of the stringers is considered to be a key component of the satisfactory stress levels found to be present during the 300 feet removal alternative. For removal and replacement of one 300 feet long half-width segment, the prediction from contractors that were interviewed, is that this work can be accomplished in approximately a two week time interval. Work will begin at the center pier (Pier 8) and simultaneously progress both up-station and back-station.

If there is a desire to remove the deck in 600 feet long half-width sections, connection plate to bottom flange retrofits will be necessary. The minimum number of locations that should be retrofitted are based on stresses computed by the 3-D FE modeling and are shown on pages 56 through 60. There are 6 retrofit locations per unit in Units 1 through 4, and 5 retrofit locations in Unit 5. These retrofit locations represent the minimum locations that need to be retrofitted for removal of 600 feet segments. According to the contractors interviewed, there does not appear to be a clear indication that utilizing 600 feet removal limits will shorten the duration of time necessary to complete one of the twin superstructure decks.

In the preparation of the cost estimate, we have assumed that retrofits will be provided at an average of 3 crossframe stations per span. For each crossframe station, 6 retrofits are required (one retrofit for each exterior girder and 2 retrofits for each of the two interior girders). Therefore, the number of individual retrofits is computed as (6 per station) * (3 stations per span) * (15 spans) * (2 structures) = 540 total retrofits. This retrofit work is estimated to cost approximately $3,000,000. A complete project construction cost estimate and construction schedule are provided in Appendix A.

The following items should be considered when preparing the construction contract plans for this project:

1. Perform an in-depth inspection.
2. Develop deck replacement plans based on limiting the deck removal and replacement to 300 feet segments for a half-width phase construction.
3. Replace a portion of the abutment backwalls.
4. Provide new deck joints at the abutments.
5. Rehabilitate the existing finger joints.
6. Provide details to repair structural steel and reinforced concrete components deemed necessary based on inspection. This could include partial painting, structural steel repairs, and patching and sealing of existing concrete substructures.
7. The final design for these superstructures should be load rated. The ODOT Office of Structural Engineering provided existing BARS analysis files which analyzed one interior girder and one stringer for the left and right structures. The bridges were analyzed as non-composite structures and all the loads were distributed uniformly to a girder or stringer. In the existing analysis files, the dead load consisted of a 7.5" concrete deck and a 2.75" super plasticized dense concrete wearing surface. ELR performed a rating of the superstructures with the modifications as proposed in this report. The files were modified to include a composite deck. All the loads were distributed uniformly to a girder or stringer. The dead load used consisted of an 8.5" concrete deck, 20 psf for SIP forms and 42" single slope barriers. The modified files were run using the BARS-PC Release 5.5 using the Load Factor Method. The computed rating was found to be greater than the HS20 loading at the Inventory Level.

Recommendations

The final recommendation gleaned from the information in this report is to remove 300 feet of deck in half-width segments centered over a pier, at each construction interval during the construction of the reinforced concrete deck. Our recommendation is based on a desire to limit the out-of-plane stresses at the crossframe to web connections. The procedures for performing this work are detailed within the contents of this report.

If the 300 feet half-width removal limits, which are the preferred removal limits, cannot be accomplished in one construction season, an acceptable design alternative is to remove up to 600 feet of half-width deck provided the appropriate retrofits have been installed prior to deck removal.

The fatigue life of the existing bridge has not been discussed in this study which focuses on evaluating deck replacement alternatives. Fatigue life is a concern, although predictions have not specifically been addressed within the contents of this report. We believe that it would be appropriate to recommend that consideration be given to providing the bottom flange retrofits at approximately three crossframe stations in each span. The position of the crossframes to be retrofitted should be near the center of each span in the superstructure. Engineering judgment is necessary to establish criteria for predicting the locations of where retrofits, for

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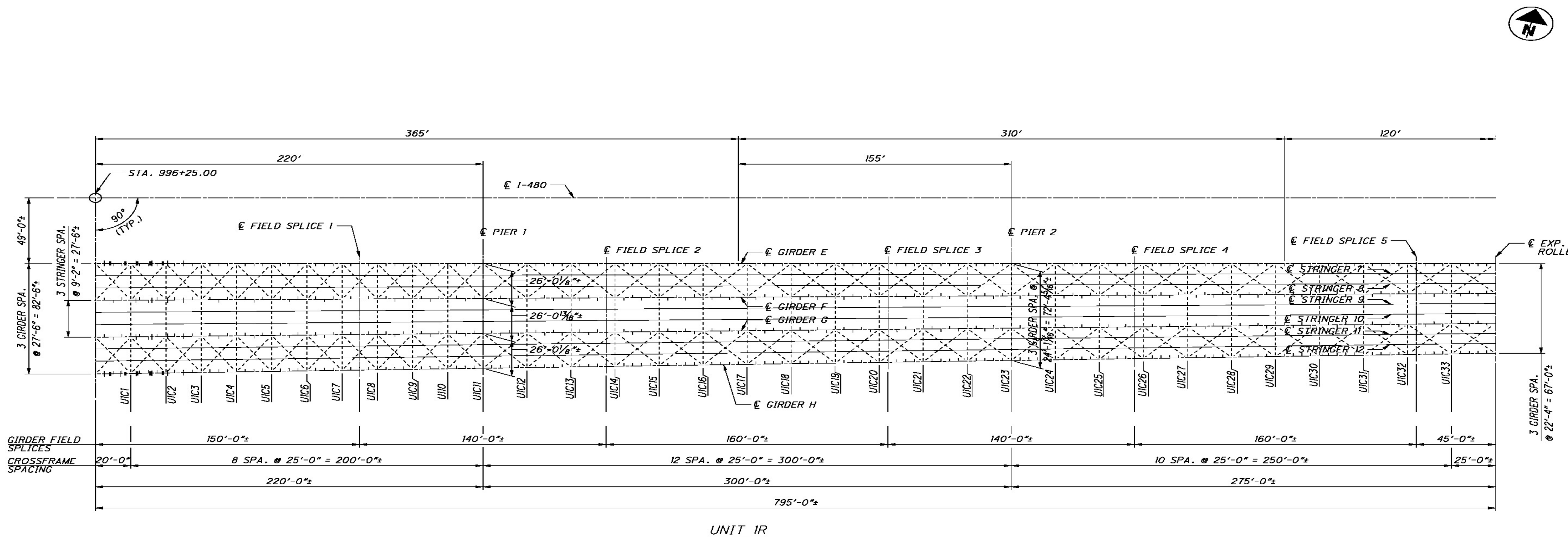
fatigue life concerns, are considered to be appropriate. Professor Dennis R. Mertz, Ph.D., P.E. informed us that, "Estimating the remaining life of distortion-induced fatigue details is foolhardy at best. Even fatigue-life estimates for load-induced fatigue details can be misleading. Due to the uncertainties involved and the probabilistic nature of the fatigue limit state as defined by AASHTO, estimated fatigue lives are lower bounds. If the remaining life of bridges with lower fatigue category details is estimated, many times a negative life for successfully performing in-service bridges results. If this is the case for load-induced fatigue, the problem of accuracy only magnifies with distortion-induced fatigue. I do not recommend estimating the remaining fatigue life of distortion-induced fatigue details. The proper retrofit detail will extend the life indefinitely if the web gap is sufficiently stiffened."



CROSS FRAMES RETROFIT LOCATIONS - UNIT 1
U1C6
U1C17
U1C28
U1C29
U1C30
U1C31

DESIGN AGENCY
E.L. ROBINSON

REVIEWED DFT DATE 02/20/2012
STRUCTURE FILE NUMBER 1812521 & 1812548
DRAWN JEG
REVISED
DESIGNED FA
CHECKED

RETROFIT LOCATIONS FOR 600 FT POURS
CUY-480-1842 L&R
I-480 OVER CUYAHOGA RIVER VALLEY

CUY-480-18.42
PID No.

1 / 5

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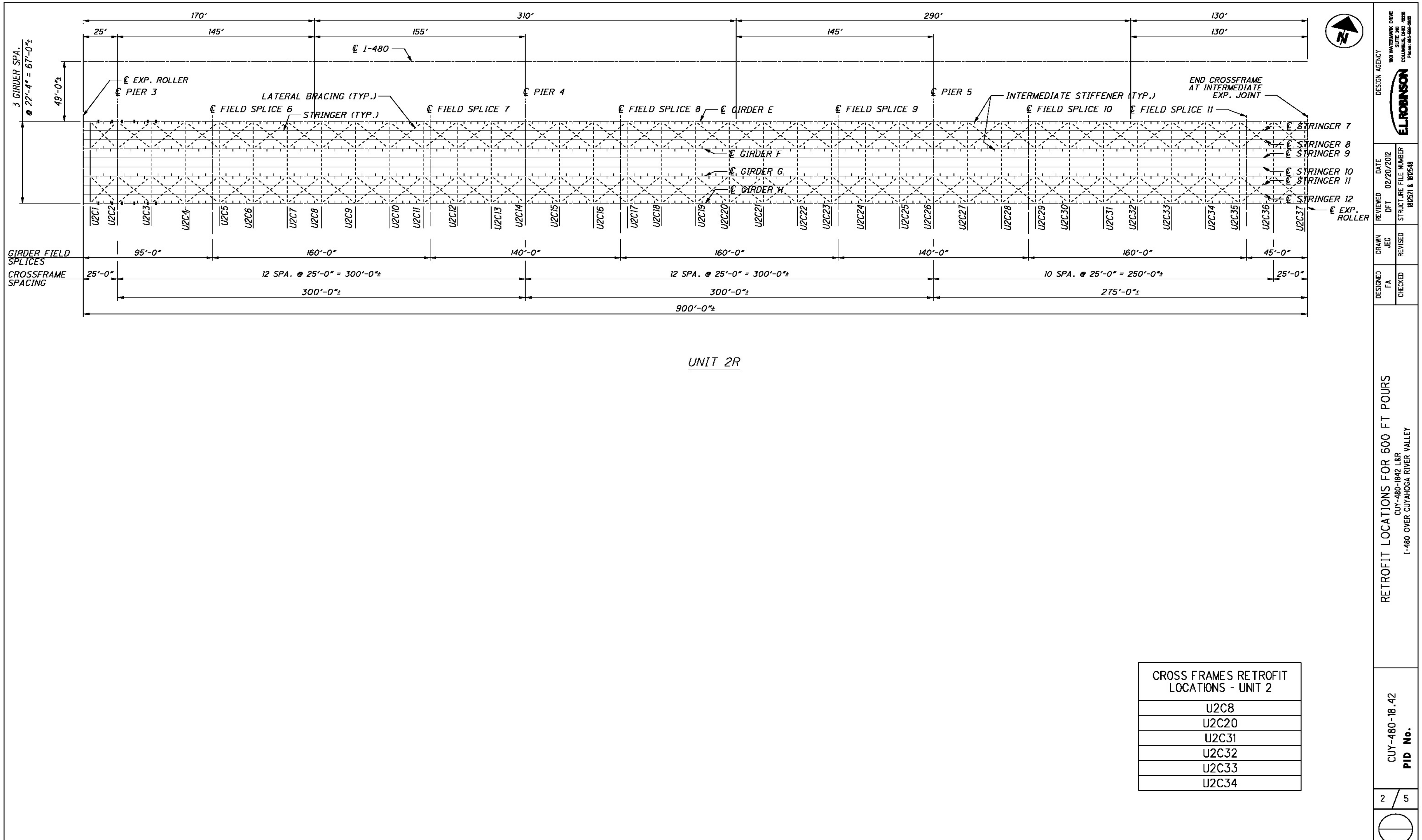
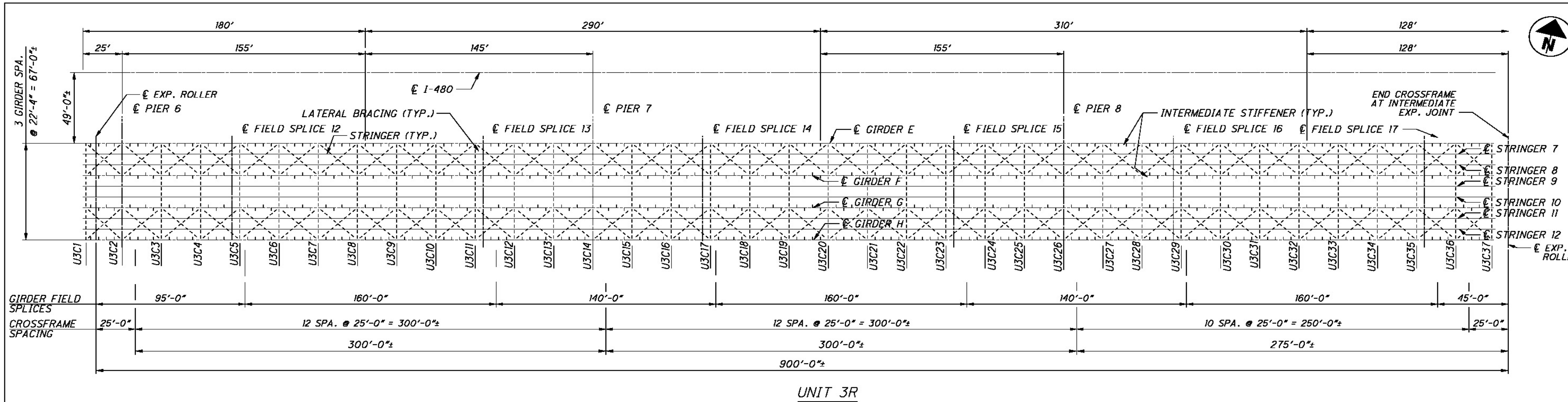


CROSS FRAMES RETROFIT LOCATIONS - UNIT 2
U2C8
U2C20
U2C31
U2C32
U2C33
U2C34



CROSS FRAMES RETROFIT LOCATIONS - UNIT 3
U3C8
U3C20
U3C31
U3C32
U3C33
U3C34

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REVIEWED DFT DATE 02/20/2012
STRUCTURE FILE NUMBER 1812521 & 1812548
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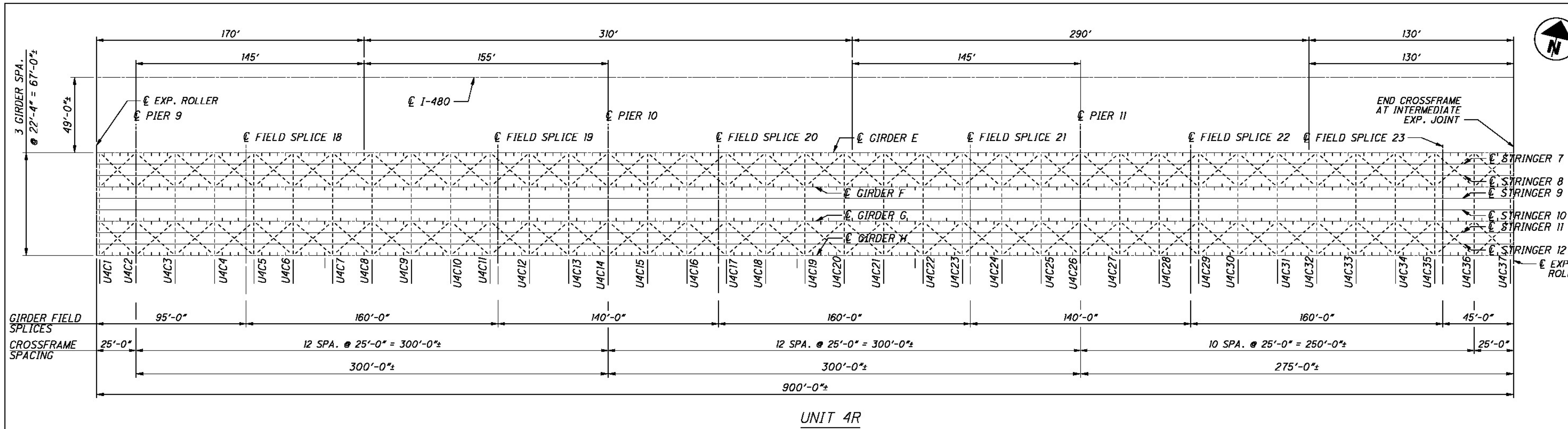
RETROFIT LOCATIONS FOR 600 FT POURS
CUY-480-1842 L&R
I-480 OVER CUYAHOGA RIVER VALLEY

CUY-480-18.42
PID No.

3 / 5



CROSS FRAMES RETROFIT LOCATIONS - UNIT 4
U4C8
U4C20
U4C31
U4C32
U4C33
U4C34

RETROFIT LOCATIONS FOR 600 FT POURS
CUY-480-1842 L&R
I-480 OVER CUYAHOGA RIVER VALLEY

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CUY-480-18.42
PID No.

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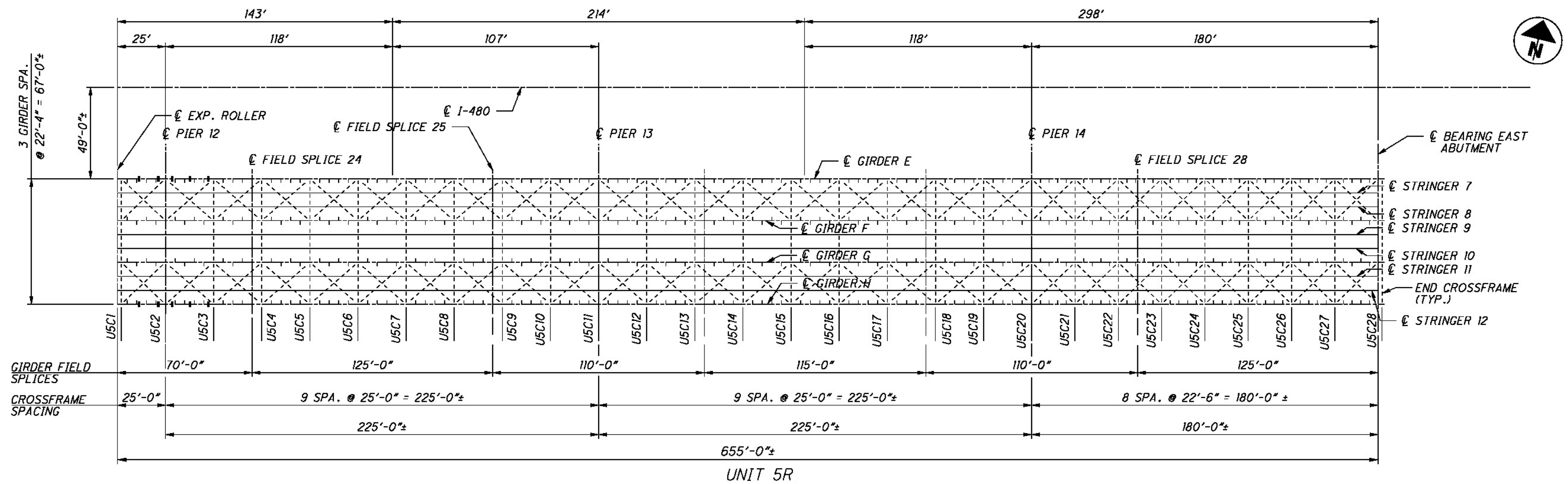

CROSS FRAMES RETROFIT LOCATIONS - UNIT 5
U5C6
U5C7
U5C15
U5C16
U5C24

DESIGN AGENCY
E.L. ROBINSON

DESIGNED	DRAWN	REVIEWED	DATE
FA	JEG	DFT	02/20/2012
CHECKED	REVISED	STRUCTURE FILE NUMBER 1812521 & 1812548	

RETROFIT LOCATIONS FOR 600 FT POURS
CUY-480-1842 L&R
I-480 OVER CUYAHOGA RIVER VALLEY

CUY-480-18.42
PID No.

5 / 5

E.L. ROBINSON ENGINEERING

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Fisher et al. (1998) – A Fatigue Primer for Structural Engineers. National Steel Bridge Alliance

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Physical Condition Report (In-Depth Inspection) of Valley View Bridges over the Cuyahoga River (2008)

Annual Inspection of the Valley View Bridge (2009)

2010 Routine Inspection Report of I-480 Valley View Bridge Over the Cuyahoga River

LARSA 4D Version 7.05.35 LARSA, Inc. 68 S Service Road Suite 100 Melville, New York 11747

LUSAS Version 14.7-1 LUSAS 66 High Street Kingston upon Thames Surrey United Kingdom

"Behavior and Rehabilitation of Distortion-Induced Fatigue Cracks in Bridge Girders",2001, D'Andrea, M., M.Sc. Thesis, Dept. of Civil Eng., University of Alberta. Edmonton, Alberta, Canada.

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AASHTO Standard Specifications for Highway Bridges, 17th Editions - 2002

APPENDIX A

Estimated Construction Cost & Schedule

E.L. Robinson Engineering of Ohio

Estimate 90591

Estimated Cost:$60,008,910.40

Contingency: 29.60%

Estimated Total: $77,771,547.88

CUY-480-1842L/R (I.R. 480 E.B./W.B. VALLEY VIEW BRIDGES)

Base Date: 06/21/12

Spec Year: 10

Unit System: E

Work Type: BRIDGE REHABILITATION

Highway Type:

Urban/Rural Type: URBAN CLASS

Season:

County: CUYAHOGA

Midpoint of Latitude:

Midpoint of Longitude:

District: 12

Federal/State Project Number: 90591

Prepared by E.L. Robinson

Estimate: 90591 E.L. Robinson Engineering of Ohio

Line #	Item Number / Description / *Supplemental Description*	Quantity	Units	Unit Price	Extension
Group 0002: Structures Over 20 Foot Span					
0006	202E11305 PORTIONS OF STRUCTURE REMOVED, AS PER PLAN	68,017.000	SY	$225.00000	$15,303,825.00
0007	202E22901 APPROACH SLAB REMOVED, AS PER PLAN	578.000	SY	$31.50000	$18,207.00
0009	509E10001 EPOXY COATED REINFORCING STEEL, AS PER PLAN	5,810,100.000	LB	$1.10000	$6,391,110.00
0010	512E10101 SEALING OF CONCRETE SURFACES (EPOXY-URETHANE), AS PER PLAN	19,753.000	SY	$14.00000	$276,542.00
0011	513E17001 STRUCTURAL STEEL MEMBERS, MODULAR EXPANSION JOINT, LEVEL UF, AS PER PLAN	325.000	FT	$1,558.00000	$506,350.00
0012	513E20001 WELDED STUD SHEAR CONNECTORS, AS PER PLAN	265,600.000	EACH	$2.30000	$610,880.00
0013	514E00051 SURFACE PREPARATION OF EXISTING STRUCTURAL STEEL, AS PER PLAN	6,625.000	SF	$9.44043	$62,542.85
0014	514E00057 FIELD PAINTING OF EXISTING STRUCTURAL STEEL, PRIME COAT, AS PER PLAN	6,625.000	SF	$2.55866	$16,951.12
0015	514E00061 FIELD PAINTING STRUCTURAL STEEL, INTERMEDIATE COAT, AS PER PLAN	6,625.000	SF	$1.85249	$12,272.75
0016	514E00067 FIELD PAINTING STRUCTURAL STEEL, FINISH COAT, AS PER PLAN	6,625.000	SF	$3.15475	$20,900.22
0017	519E11101 PATCHING CONCRETE STRUCTURE, AS PER PLAN	5,982.000	SF	$75.00000	$448,650.00
0018	530E00400 SPECIAL - STRUCTURE, MISC.: *10% CONTINGENCY*	1.000	EACH	$5,406,184.46000	$5,406,184.46
0019	601E20001 CRUSHED AGGREGATE SLOPE PROTECTION, AS PER PLAN	477.000	SY	$33.54423	$16,000.60
0020	607E39901 VANDAL PROTECTION FENCE, 6' STRAIGHT, COATED FABRIC, AS PER PLAN	16,620.000	FT	$25.00000	$415,500.00
0027	898E10201 QC/QA CONCRETE, CLASS QSC2, SUPERSTRUCTURE (DECK), AS PER PLAN	18,468.000	CY	$950.00000	$17,544,600.00
0028	898E10709 QC/QA CONCRETE, CLASS QSC2, SUPERSTRUCTURE (APPROACH SLAB), (T=17"), AS PER PLAN	1,083.000	SY	$232.19912	$251,471.65
0029	898E11001 QC/QA CONCRETE, CLASS QSC2, SUPERSTRUCTURE (PARAPET), AS PER PLAN	2,660.000	CY	$599.60000	$1,594,936.00
				Total for Group 0002:	$48,896,923.65
Group 0003: Roadway					
0005	103E06000 PREMIUM FOR CONTRACT PERFORMANCE BOND, PAYMENT BOND AND MAINTENANCE BOND	1.000	LS	$270,309.22000	$270,309.22
0008	203E98500 ROADWAY, MISC.: *LUMP*	1.000	LS	$250,000.00000	$250,000.00
0021	614E18002 MAINTAINING TRAFFIC, MISC.: *MOT*	1.000	LS	$5,000,000.00000	$5,000,000.00
0022	619E16020 FIELD OFFICE, TYPE C	36.000	MNTH	$1,843.56414	$66,368.31
0023	623E10000 CONSTRUCTION LAYOUT STAKES	1.000	LS	$270,309.22000	$270,309.22
0024	624E10001 MOBILIZATION, AS PER PLAN *PROJECT ACCESS*	1.000	LS	$5,000,000.00000	$5,000,000.00
0025	832E15000 STORM WATER POLLUTION PREVENTION PLAN	1.000	LS	$5,000.00000	$5,000.00

Estimate: 90591 E.L. Robinson Engineering of Ohio

Line #	Item Number / Description / *Supplemental Description*	Quantity	Units	Unit Price	Extension
0026	832E30000 EROSION CONTROL	1.000	EACH	$250,000.00000	$250,000.00

Total for Group 0003:$11,111,986.75

E.L. Robinson Engineering of Ohio

Estimate 90591

Estimated Cost:$2,362,500.00

Contingency: 29.60%

Estimated Total: $3,061,800.00

CUY-480-1842L/R (I.R. 480 E.B./W.B. VALLEY VIEW BRIDGES - Connection Plate Retrofit)

Base Date: 06/21/12

Spec Year: 10

Unit System: E

Work Type: BRIDGE REHABILITATION

Highway Type:

Urban/Rural Type: URBAN CLASS

Season:

County: CUYAHOGA

Midpoint of Latitude:

Midpoint of Longitude:

District: 12

Federal/State Project Number: 90591

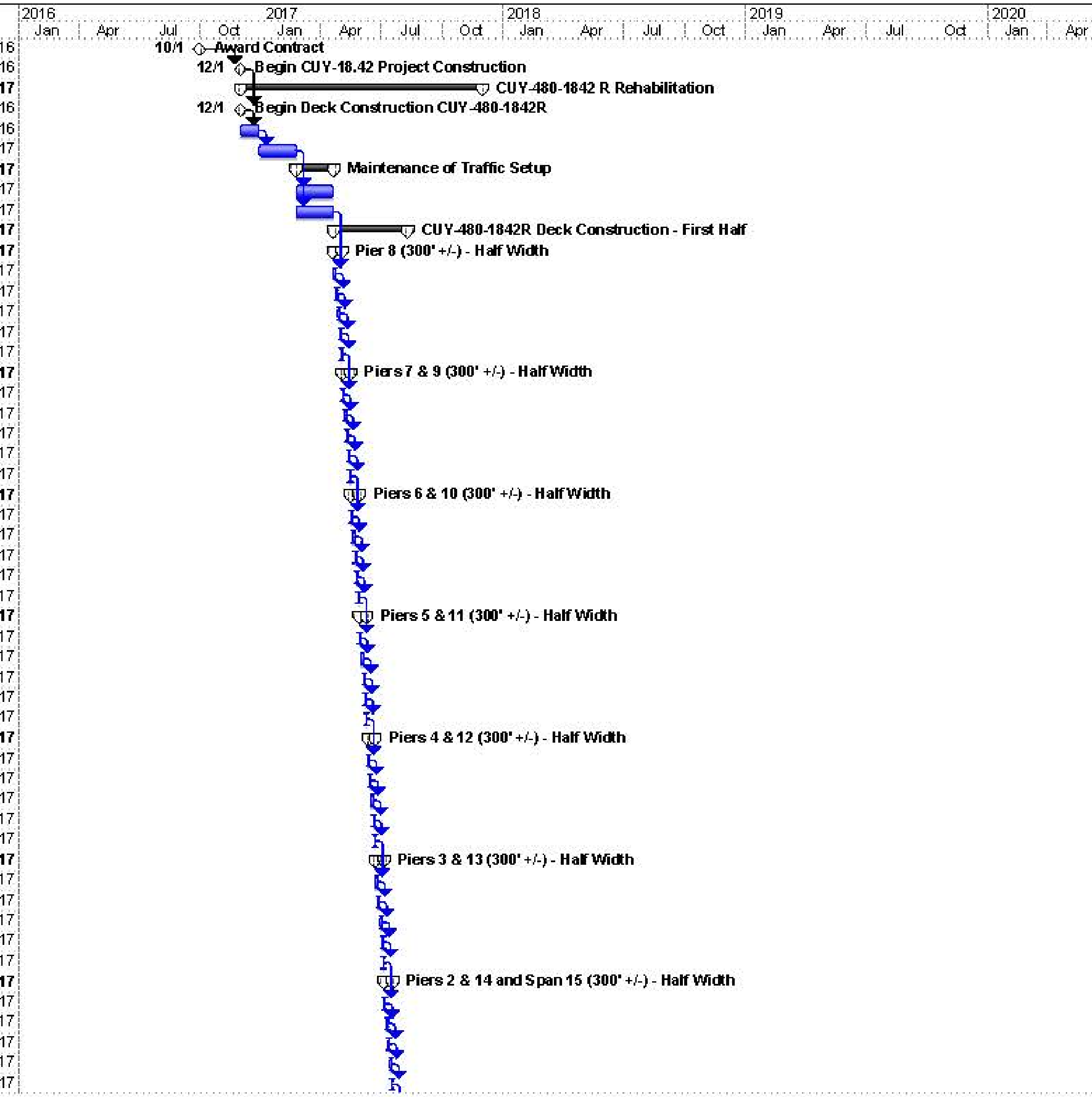
Prepared by E.L. Robinson

Estimate: 90591 E.L. Robinson Engineering of Ohio

Line #	Item Number / Description / *Supplemental Description*	Quantity	Units	Unit Price	Extension
Group 1503: Connection Plate Retrofits					
0011	513E95020 STRUCTURAL STEEL, MISC.: *CONNECTION PLATE RETROFITS*	1.000	LS	$1,362,500.00000	$1,362,500.00
0012	513E95020 STRUCTURAL STEEL, MISC.: *ACCESS*	1.000	LS	$1,000,000.00000	$1,000,000.00

Total for Group 1503:$2,362,500.00

ID	Task Name	Duration	Start	Finish
1	Award Contract	0 days	Sat 10/1/16	Sat 10/1/16
2	Begin CUY-18.42 Project Construction	0 days	Thu 12/1/16	Thu 12/1/16
3	**CUY-480-1842 R Rehabilitation**	**260 days**	**Thu 12/1/16**	**Thu 11/30/17**
4	Begin Deck Construction CUY-480-1842R	0 days	Thu 12/1/16	Thu 12/1/16
5	Mobilization	20 days	Fri 12/2/16	Thu 12/29/16
6	Install Falsework/Protective Netting	40 days	Fri 12/30/16	Thu 2/23/17
7	**Maintenance of Traffic Setup**	**40 days**	**Fri 2/24/17**	**Thu 4/20/17**
8	Install Signing	40 days	Fri 2/24/17	Thu 4/20/17
9	Install PCB	40 days	Fri 2/24/17	Thu 4/20/17
10	**CUY-480-1842R Deck Construction - First Half**	**80 days**	**Fri 4/21/17**	**Thu 8/10/17**
11	**Pier 8 (300' +/-) - Half Width**	**9 days**	**Fri 4/21/17**	**Wed 5/3/17**
12	Demo Slab	2 days	Fri 4/21/17	Mon 4/24/17
13	Stay In Place Forms	2.5 days	Tue 4/25/17	Thu 4/27/17
14	Rebar	2 days	Thu 4/27/17	Mon 5/1/17
15	Setup Bidwell	1.5 days	Mon 5/1/17	Tue 5/2/17
16	Pour Deck	1 day	Wed 5/3/17	Wed 5/3/17
17	**Piers 7 & 9 (300' +/-) - Half Width**	**9 days**	**Thu 5/4/17**	**Tue 5/16/17**
18	Demo Slab	2 days	Thu 5/4/17	Fri 5/5/17
19	Stay In Place Forms	2.5 days	Mon 5/8/17	Wed 5/10/17
20	Rebar	2 days	Wed 5/10/17	Fri 5/12/17
21	Setup Bidwell	1.5 days	Fri 5/12/17	Mon 5/15/17
22	Pour Deck	1 day	Tue 5/16/17	Tue 5/16/17
23	**Piers 6 & 10 (300' +/-) - Half Width**	**9 days**	**Wed 5/17/17**	**Mon 5/29/17**
24	Demo Slab	2 days	Wed 5/17/17	Thu 5/18/17
25	Stay In Place Forms	2.5 days	Fri 5/19/17	Tue 5/23/17
26	Rebar	2 days	Tue 5/23/17	Thu 5/25/17
27	Setup Bidwell	1.5 days	Thu 5/25/17	Fri 5/26/17
28	Pour Deck	1 day	Mon 5/29/17	Mon 5/29/17
29	**Piers 5 & 11 (300' +/-) - Half Width**	**9 days**	**Tue 5/30/17**	**Fri 6/9/17**
30	Demo Slab	2 days	Tue 5/30/17	Wed 5/31/17
31	Stay In Place Forms	2.5 days	Thu 6/1/17	Mon 6/5/17
32	Rebar	2 days	Mon 6/5/17	Wed 6/7/17
33	Setup Bidwell	1.5 days	Wed 6/7/17	Thu 6/8/17
34	Pour Deck	1 day	Fri 6/9/17	Fri 6/9/17
35	**Piers 4 & 12 (300' +/-) - Half Width**	**9 days**	**Mon 6/12/17**	**Thu 6/22/17**
36	Demo Slab	2 days	Mon 6/12/17	Tue 6/13/17
37	Stay In Place Forms	2.5 days	Wed 6/14/17	Fri 6/16/17
38	Rebar	2 days	Fri 6/16/17	Tue 6/20/17
39	Setup Bidwell	1.5 days	Tue 6/20/17	Wed 6/21/17
40	Pour Deck	1 day	Thu 6/22/17	Thu 6/22/17
41	**Piers 3 & 13 (300' +/-) - Half Width**	**9 days**	**Fri 6/23/17**	**Wed 7/5/17**
42	Demo Slab	2 days	Fri 6/23/17	Mon 6/26/17
43	Stay In Place Forms	2.5 days	Tue 6/27/17	Thu 6/29/17
44	Rebar	2 days	Thu 6/29/17	Mon 7/3/17
45	Setup Bidwell	1.5 days	Mon 7/3/17	Tue 7/4/17
46	Pour Deck	1 day	Wed 7/5/17	Wed 7/5/17
47	**Piers 2 & 14 and Span 15 (300' +/-) - Half Width**	**9 days**	**Thu 7/6/17**	**Tue 7/18/17**
48	Demo Slab	2 days	Thu 7/6/17	Fri 7/7/17
49	Stay In Place Forms	2.5 days	Mon 7/10/17	Wed 7/12/17
50	Rebar	2 days	Wed 7/12/17	Fri 7/14/17
51	Setup Bidwell	1.5 days	Fri 7/14/17	Mon 7/17/17
52	Pour Deck	1 day	Tue 7/18/17	Tue 7/18/17



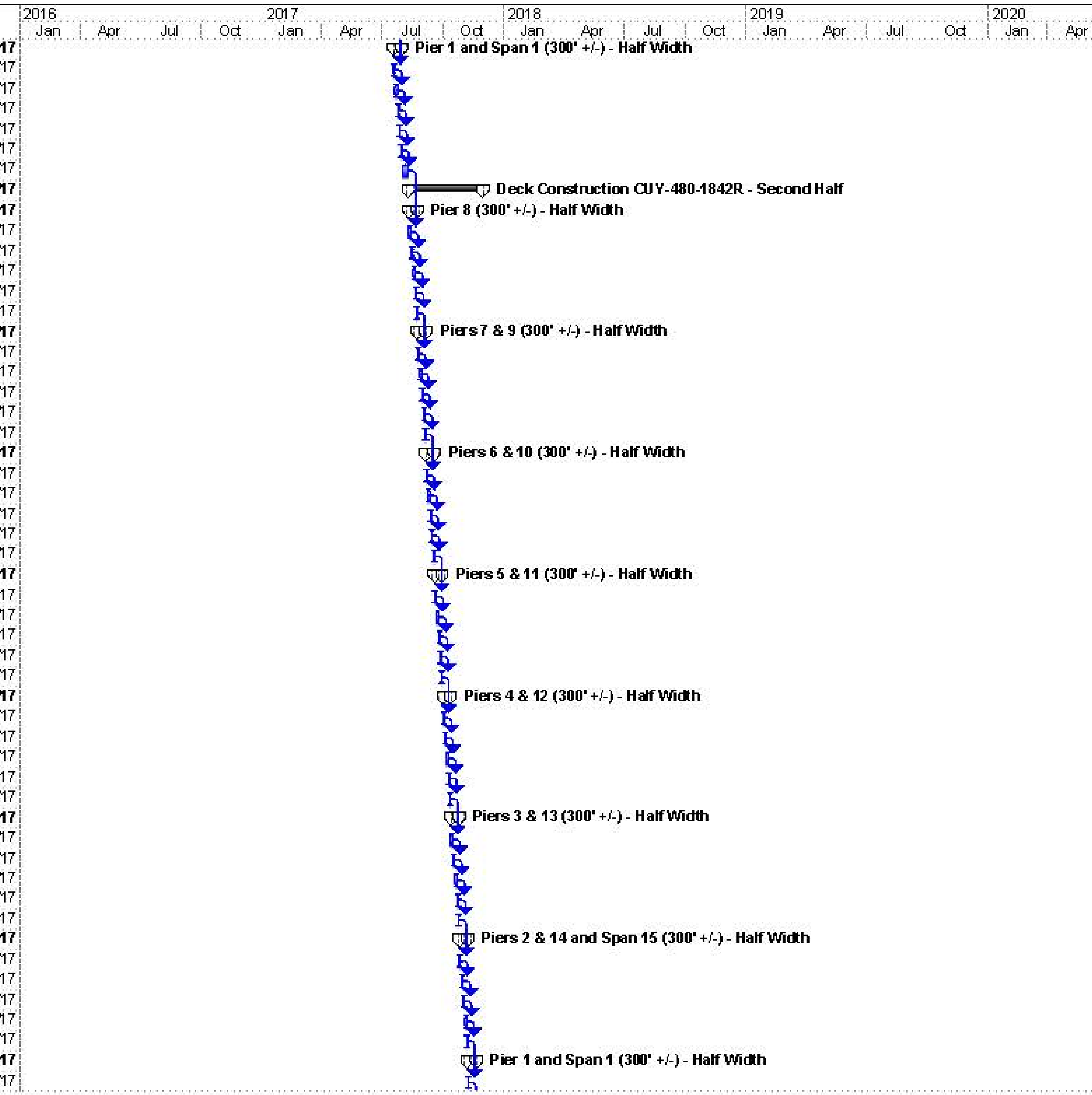
Project: CUY-480-18.42 Construction :
Date: Mon 8/6/12

Task | Split | Progress | Milestone | Summary | Project Summary | External Tasks | External Milestone | Deadline

E.L. ROBINSON ENGINEERING

CUY-480-18.42
PID 90591

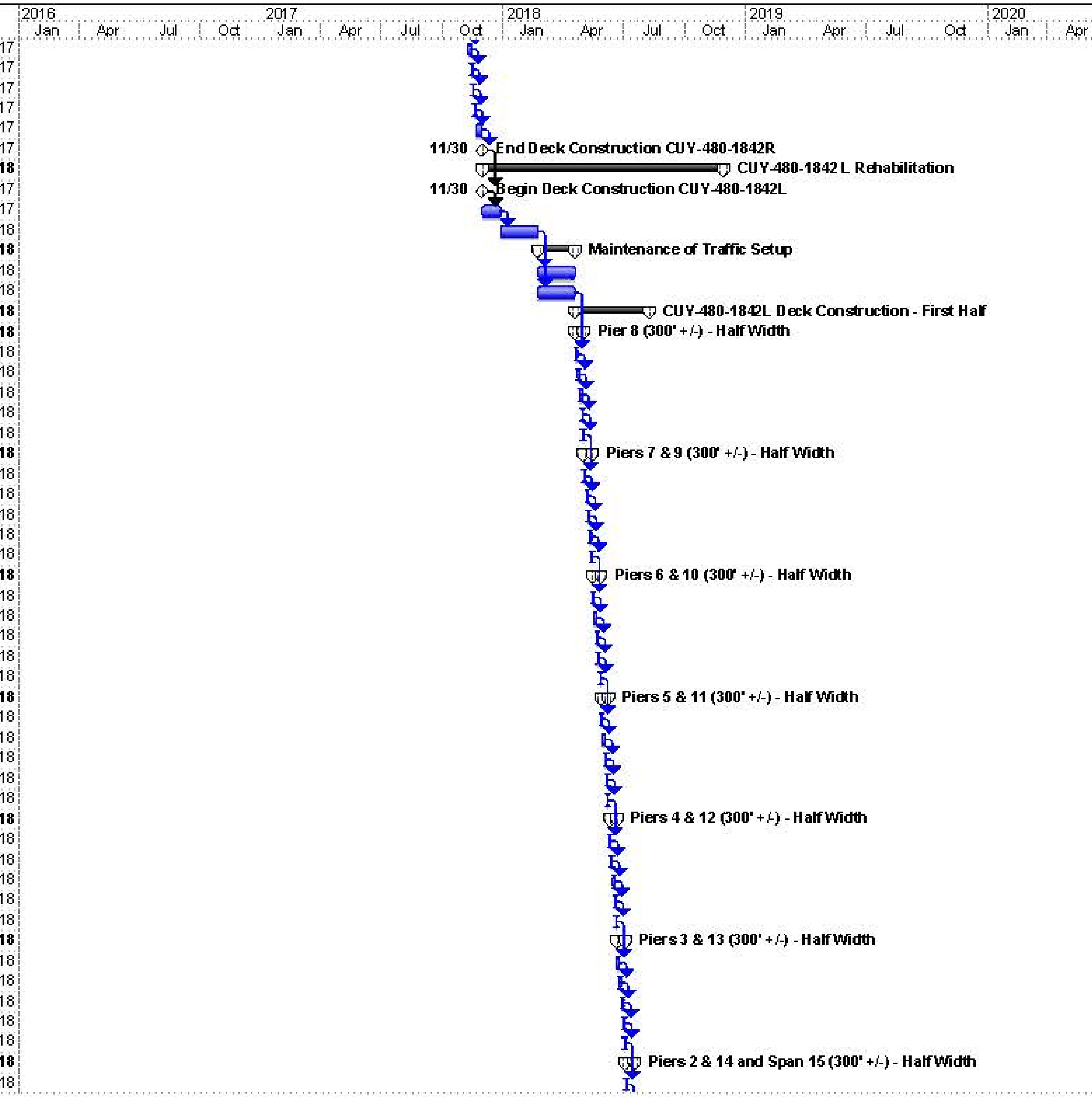
ID	Task Name	Duration	Start	Finish
53	**Pier 1 and Span 1 (300' +/-) - Half Width**	**9 days**	**Wed 7/19/17**	**Mon 7/31/17**
54	Demo Slab	2 days	Wed 7/19/17	Thu 7/20/17
55	Stay In Place Forms	2.5 days	Fri 7/21/17	Tue 7/25/17
56	Rebar	2 days	Tue 7/25/17	Thu 7/27/17
57	Setup Bidwell	1.5 days	Thu 7/27/17	Fri 7/28/17
58	Pour Deck	1 day	Mon 7/31/17	Mon 7/31/17
59	Finalize CUY-480-1842R including Parapet- First Half	8 days	Tue 8/1/17	Thu 8/10/17
60	**Deck Construction CUY-480-1842R - Second Half**	**80 days**	**Fri 8/11/17**	**Thu 11/30/17**
61	**Pier 8 (300' +/-) - Half Width**	**9 days**	**Fri 8/11/17**	**Wed 8/23/17**
62	Demo Slab	2 days	Fri 8/11/17	Mon 8/14/17
63	Stay In Place Forms	2.5 days	Tue 8/15/17	Thu 8/17/17
64	Rebar	2 days	Thu 8/17/17	Mon 8/21/17
65	Setup Bidwell	1.5 days	Mon 8/21/17	Tue 8/22/17
66	Pour Deck	1 day	Wed 8/23/17	Wed 8/23/17
67	**Piers 7 & 9 (300' +/-) - Half Width**	**9 days**	**Thu 8/24/17**	**Tue 9/5/17**
68	Demo Slab	2 days	Thu 8/24/17	Fri 8/25/17
69	Stay In Place Forms	2.5 days	Mon 8/28/17	Wed 8/30/17
70	Rebar	2 days	Wed 8/30/17	Fri 9/1/17
71	Setup Bidwell	1.5 days	Fri 9/1/17	Mon 9/4/17
72	Pour Deck	1 day	Tue 9/5/17	Tue 9/5/17
73	**Piers 6 & 10 (300' +/-) - Half Width**	**9 days**	**Wed 9/6/17**	**Mon 9/18/17**
74	Demo Slab	2 days	Wed 9/6/17	Thu 9/7/17
75	Stay In Place Forms	2.5 days	Fri 9/8/17	Tue 9/12/17
76	Rebar	2 days	Tue 9/12/17	Thu 9/14/17
77	Setup Bidwell	1.5 days	Thu 9/14/17	Fri 9/15/17
78	Pour Deck	1 day	Mon 9/18/17	Mon 9/18/17
79	**Piers 5 & 11 (300' +/-) - Half Width**	**9 days**	**Tue 9/19/17**	**Fri 9/29/17**
80	Demo Slab	2 days	Tue 9/19/17	Wed 9/20/17
81	Stay In Place Forms	2.5 days	Thu 9/21/17	Mon 9/25/17
82	Rebar	2 days	Mon 9/25/17	Wed 9/27/17
83	Setup Bidwell	1.5 days	Wed 9/27/17	Thu 9/28/17
84	Pour Deck	1 day	Fri 9/29/17	Fri 9/29/17
85	**Piers 4 & 12 (300' +/-) - Half Width**	**9 days**	**Mon 10/2/17**	**Thu 10/12/17**
86	Demo Slab	2 days	Mon 10/2/17	Tue 10/3/17
87	Stay In Place Forms	2.5 days	Wed 10/4/17	Fri 10/6/17
88	Rebar	2 days	Fri 10/6/17	Tue 10/10/17
89	Setup Bidwell	1.5 days	Tue 10/10/17	Wed 10/11/17
90	Pour Deck	1 day	Thu 10/12/17	Thu 10/12/17
91	**Piers 3 & 13 (300' +/-) - Half Width**	**9 days**	**Fri 10/13/17**	**Wed 10/25/17**
92	Demo Slab	2 days	Fri 10/13/17	Mon 10/16/17
93	Stay In Place Forms	2.5 days	Tue 10/17/17	Thu 10/19/17
94	Rebar	2 days	Thu 10/19/17	Mon 10/23/17
95	Setup Bidwell	1.5 days	Mon 10/23/17	Tue 10/24/17
96	Pour Deck	1 day	Wed 10/25/17	Wed 10/25/17
97	**Piers 2 & 14 and Span 15 (300' +/-) - Half Width**	**9 days**	**Thu 10/26/17**	**Tue 11/7/17**
98	Demo Slab	2 days	Thu 10/26/17	Fri 10/27/17
99	Stay In Place Forms	2.5 days	Mon 10/30/17	Wed 11/1/17
100	Rebar	2 days	Wed 11/1/17	Fri 11/3/17
101	Setup Bidwell	1.5 days	Fri 11/3/17	Mon 11/6/17
102	Pour Deck	1 day	Tue 11/7/17	Tue 11/7/17
103	**Pier 1 and Span 1 (300' +/-) - Half Width**	**9 days**	**Wed 11/8/17**	**Mon 11/20/17**
104	Demo Slab	2 days	Wed 11/8/17	Thu 11/9/17



Project: CUY-480-18.42 Construction :
Date: Mon 8/6/12

Task | Progress | Summary | External Tasks | Deadline
Split | Milestone | Project Summary | External Milestone

ID		Task Name	Duration	Start	Finish
105		Stay In Place Forms	2.5 days	Fri 11/10/17	Tue 11/14/17
106		Rebar	2 days	Tue 11/14/17	Thu 11/16/17
107		Setup Bidwell	1.5 days	Thu 11/16/17	Fri 11/17/17
108		Pour Deck and Closure Pour	1 day	Mon 11/20/17	Mon 11/20/17
109		Finalize CUY-480-1842R including Parapet - Second Half	8 days	Tue 11/21/17	Thu 11/30/17
110		End Deck Construction CUY-480-1842R	0 days	Thu 11/30/17	Thu 11/30/17
111		**CUY-480-1842 L Rehabilitation**	**260 days**	**Thu 11/30/17**	**Thu 11/29/18**
112		Begin Deck Construction CUY-480-1842L	0 days	Thu 11/30/17	Thu 11/30/17
113		Mobilization	20 days	Fri 12/1/17	Thu 12/28/17
114		Install Falsework/Protective Netting	40 days	Fri 12/29/17	Thu 2/22/18
115		**Maintenance of Traffic Setup**	**40 days**	**Fri 2/23/18**	**Thu 4/19/18**
116		Install Signing	40 days	Fri 2/23/18	Thu 4/19/18
117		Install PCB	40 days	Fri 2/23/18	Thu 4/19/18
118		**CUY-480-1842L Deck Construction - First Half**	**80 days**	**Fri 4/20/18**	**Thu 8/9/18**
119		**Pier 8 (300' +/-) - Half Width**	**9 days**	**Fri 4/20/18**	**Wed 5/2/18**
120		Demo Slab	2 days	Fri 4/20/18	Mon 4/23/18
121		Stay In Place Forms	2.5 days	Tue 4/24/18	Thu 4/26/18
122		Rebar	2 days	Thu 4/26/18	Mon 4/30/18
123		Setup Bidwell	1.5 days	Mon 4/30/18	Tue 5/1/18
124		Pour Deck	1 day	Wed 5/2/18	Wed 5/2/18
125		**Piers 7 & 9 (300' +/-) - Half Width**	**9 days**	**Thu 5/3/18**	**Tue 5/15/18**
126		Demo Slab	2 days	Thu 5/3/18	Fri 5/4/18
127		Stay In Place Forms	2.5 days	Mon 5/7/18	Wed 5/9/18
128		Rebar	2 days	Wed 5/9/18	Fri 5/11/18
129		Setup Bidwell	1.5 days	Fri 5/11/18	Mon 5/14/18
130		Pour Deck	1 day	Tue 5/15/18	Tue 5/15/18
131		**Piers 6 & 10 (300' +/-) - Half Width**	**9 days**	**Wed 5/16/18**	**Mon 5/28/18**
132		Demo Slab	2 days	Wed 5/16/18	Thu 5/17/18
133		Stay In Place Forms	2.5 days	Fri 5/18/18	Tue 5/22/18
134		Rebar	2 days	Tue 5/22/18	Thu 5/24/18
135		Setup Bidwell	1.5 days	Thu 5/24/18	Fri 5/25/18
136		Pour Deck	1 day	Mon 5/28/18	Mon 5/28/18
137		**Piers 5 & 11 (300' +/-) - Half Width**	**9 days**	**Tue 5/29/18**	**Fri 6/8/18**
138		Demo Slab	2 days	Tue 5/29/18	Wed 5/30/18
139		Stay In Place Forms	2.5 days	Thu 5/31/18	Mon 6/4/18
140		Rebar	2 days	Mon 6/4/18	Wed 6/6/18
141		Setup Bidwell	1.5 days	Wed 6/6/18	Thu 6/7/18
142		Pour Deck	1 day	Fri 6/8/18	Fri 6/8/18
143		**Piers 4 & 12 (300' +/-) - Half Width**	**9 days**	**Mon 6/11/18**	**Thu 6/21/18**
144		Demo Slab	2 days	Mon 6/11/18	Tue 6/12/18
145		Stay In Place Forms	2.5 days	Wed 6/13/18	Fri 6/15/18
146		Rebar	2 days	Fri 6/15/18	Tue 6/19/18
147		Setup Bidwell	1.5 days	Tue 6/19/18	Wed 6/20/18
148		Pour Deck	1 day	Thu 6/21/18	Thu 6/21/18
149		**Piers 3 & 13 (300' +/-) - Half Width**	**9 days**	**Fri 6/22/18**	**Wed 7/4/18**
150		Demo Slab	2 days	Fri 6/22/18	Mon 6/25/18
151		Stay In Place Forms	2.5 days	Tue 6/26/18	Thu 6/28/18
152		Rebar	2 days	Thu 6/28/18	Mon 7/2/18
153		Setup Bidwell	1.5 days	Mon 7/2/18	Tue 7/3/18
154		Pour Deck	1 day	Wed 7/4/18	Wed 7/4/18
155		**Piers 2 & 14 and Span 15 (300' +/-) - Half Width**	**9 days**	**Thu 7/5/18**	**Tue 7/17/18**
156		Demo Slab	2 days	Thu 7/5/18	Fri 7/6/18



Project: CUY-480-18.42 Construction :
Date: Mon 8/6/12

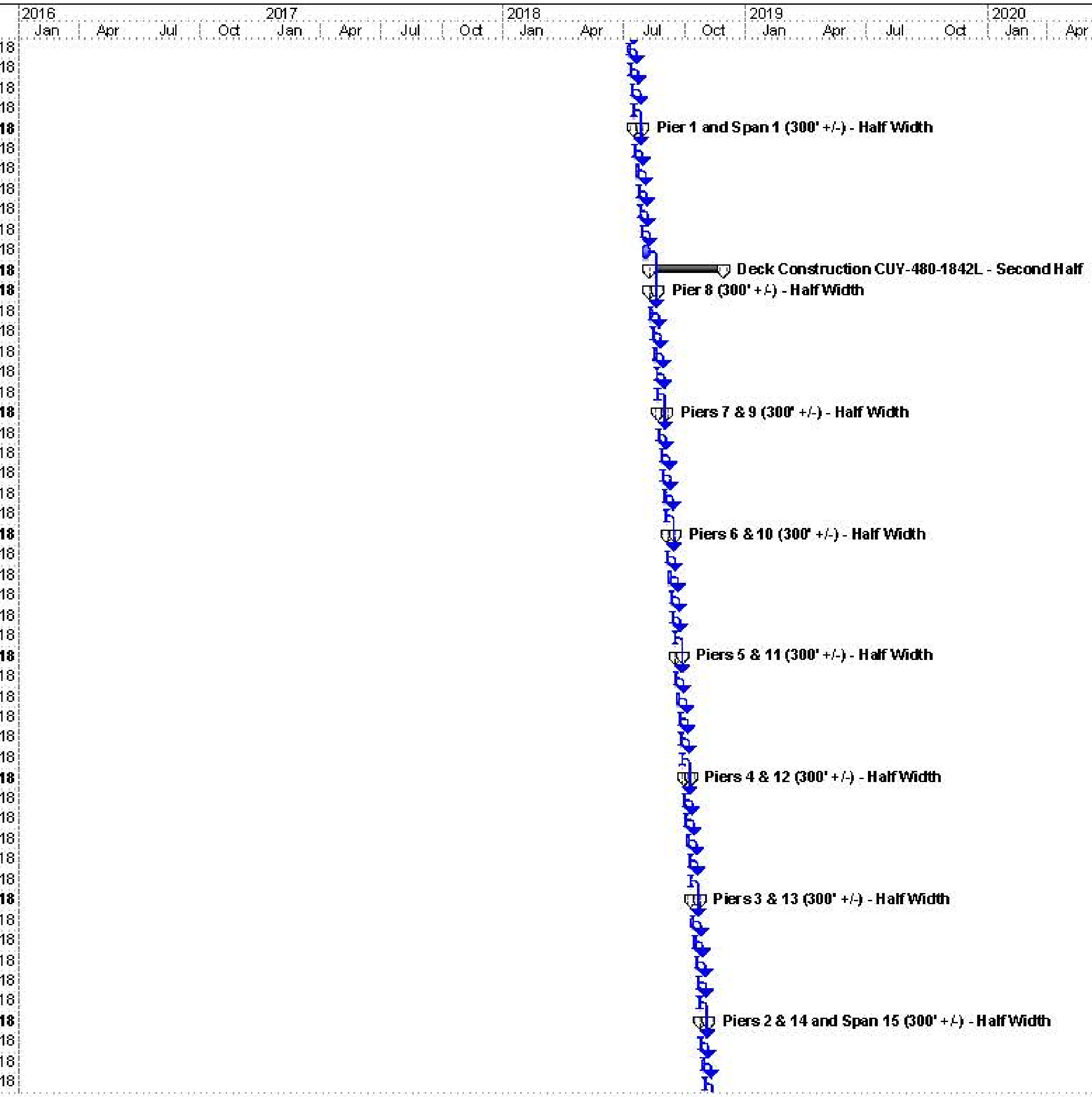
Task		Progress		Summary		External Tasks	
Split		Milestone		Project Summary		External Milestone	

Deadline

E.L. ROBINSON ENGINEERING

CUY-480-18.42
PID 90591

ID	Task Name	Duration	Start	Finish
157	Stay In Place Forms	2.5 days	Mon 7/9/18	Wed 7/11/18
158	Rebar	2 days	Wed 7/11/18	Fri 7/13/18
159	Setup Bidwell	1.5 days	Fri 7/13/18	Mon 7/16/18
160	Pour Deck	1 day	Tue 7/17/18	Tue 7/17/18
161	**Pier 1 and Span 1 (300' +/-) - Half Width**	**9 days**	**Wed 7/18/18**	**Mon 7/30/18**
162	Demo Slab	2 days	Wed 7/18/18	Thu 7/19/18
163	Stay In Place Forms	2.5 days	Fri 7/20/18	Tue 7/24/18
164	Rebar	2 days	Tue 7/24/18	Thu 7/26/18
165	Setup Bidwell	1.5 days	Thu 7/26/18	Fri 7/27/18
166	Pour Deck	1 day	Mon 7/30/18	Mon 7/30/18
167	Finalize CUY-480-1842L including Parapet - First Half	8 days	Tue 7/31/18	Thu 8/9/18
168	**Deck Construction CUY-480-1842L - Second Half**	**80 days**	**Fri 8/10/18**	**Thu 11/29/18**
169	**Pier 8 (300' +/-) - Half Width**	**9 days**	**Fri 8/10/18**	**Wed 8/22/18**
170	Demo Slab	2 days	Fri 8/10/18	Mon 8/13/18
171	Stay In Place Forms	2.5 days	Tue 8/14/18	Thu 8/16/18
172	Rebar	2 days	Thu 8/16/18	Mon 8/20/18
173	Setup Bidwell	1.5 days	Mon 8/20/18	Tue 8/21/18
174	Pour Deck	1 day	Wed 8/22/18	Wed 8/22/18
175	**Piers 7 & 9 (300' +/-) - Half Width**	**9 days**	**Thu 8/23/18**	**Tue 9/4/18**
176	Demo Slab	2 days	Thu 8/23/18	Fri 8/24/18
177	Stay In Place Forms	2.5 days	Mon 8/27/18	Wed 8/29/18
178	Rebar	2 days	Wed 8/29/18	Fri 8/31/18
179	Setup Bidwell	1.5 days	Fri 8/31/18	Mon 9/3/18
180	Pour Deck	1 day	Tue 9/4/18	Tue 9/4/18
181	**Piers 6 & 10 (300' +/-) - Half Width**	**9 days**	**Wed 9/5/18**	**Mon 9/17/18**
182	Demo Slab	2 days	Wed 9/5/18	Thu 9/6/18
183	Stay In Place Forms	2.5 days	Fri 9/7/18	Tue 9/11/18
184	Rebar	2 days	Tue 9/11/18	Thu 9/13/18
185	Setup Bidwell	1.5 days	Thu 9/13/18	Fri 9/14/18
186	Pour Deck	1 day	Mon 9/17/18	Mon 9/17/18
187	**Piers 5 & 11 (300' +/-) - Half Width**	**9 days**	**Tue 9/18/18**	**Fri 9/28/18**
188	Demo Slab	2 days	Tue 9/18/18	Wed 9/19/18
189	Stay In Place Forms	2.5 days	Thu 9/20/18	Mon 9/24/18
190	Rebar	2 days	Mon 9/24/18	Wed 9/26/18
191	Setup Bidwell	1.5 days	Wed 9/26/18	Thu 9/27/18
192	Pour Deck	1 day	Fri 9/28/18	Fri 9/28/18
193	**Piers 4 & 12 (300' +/-) - Half Width**	**9 days**	**Mon 10/1/18**	**Thu 10/11/18**
194	Demo Slab	2 days	Mon 10/1/18	Tue 10/2/18
195	Stay In Place Forms	2.5 days	Wed 10/3/18	Fri 10/5/18
196	Rebar	2 days	Fri 10/5/18	Tue 10/9/18
197	Setup Bidwell	1.5 days	Tue 10/9/18	Wed 10/10/18
198	Pour Deck	1 day	Thu 10/11/18	Thu 10/11/18
199	**Piers 3 & 13 (300' +/-) - Half Width**	**9 days**	**Fri 10/12/18**	**Wed 10/24/18**
200	Demo Slab	2 days	Fri 10/12/18	Mon 10/15/18
201	Stay In Place Forms	2.5 days	Tue 10/16/18	Thu 10/18/18
202	Rebar	2 days	Thu 10/18/18	Mon 10/22/18
203	Setup Bidwell	1.5 days	Mon 10/22/18	Tue 10/23/18
204	Pour Deck	1 day	Wed 10/24/18	Wed 10/24/18
205	**Piers 2 & 14 and Span 15 (300' +/-) - Half Width**	**9 days**	**Thu 10/25/18**	**Tue 11/6/18**
206	Demo Slab	2 days	Thu 10/25/18	Fri 10/26/18
207	Stay In Place Forms	2.5 days	Mon 10/29/18	Wed 10/31/18
208	Rebar	2 days	Wed 10/31/18	Fri 11/2/18



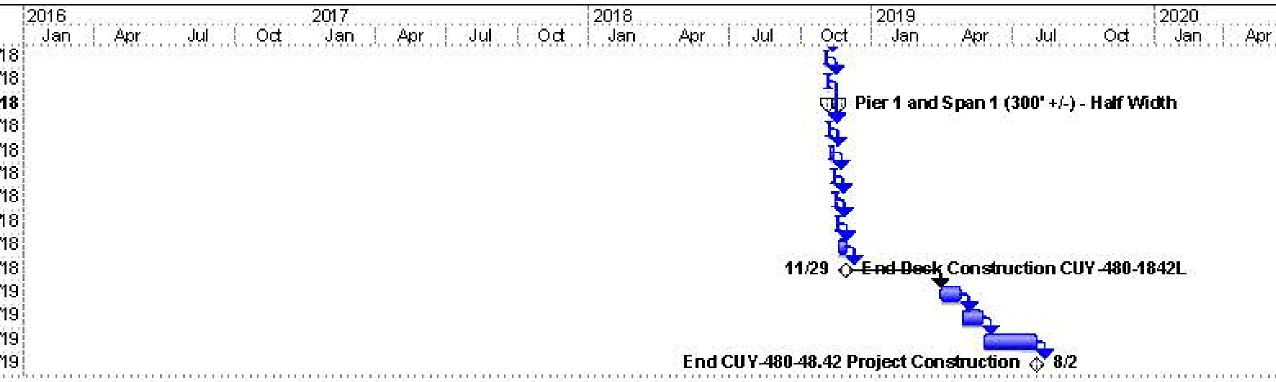
Project: CUY-480-18.42 Construction :
Date: Mon 8/6/12

Task | Split | Progress | Milestone | Summary | Project Summary | External Tasks | External Milestone | Deadline

E.L. ROBINSON ENGINEERING

CUY-480-18.42
PID 90591

ID	Task Name	Duration	Start	Finish
209	Setup Bidwell	1.5 days	Fri 11/2/18	Mon 11/5/18
210	Pour Deck	1 day	Tue 11/6/18	Tue 11/6/18
211	**Pier 1 and Span 1 (300' +/-) - Half Width**	**9 days**	**Wed 11/7/18**	**Mon 11/19/18**
212	Demo Slab	2 days	Wed 11/7/18	Thu 11/8/18
213	Stay In Place Forms	2.5 days	Fri 11/9/18	Tue 11/13/18
214	Rebar	2 days	Tue 11/13/18	Thu 11/15/18
215	Setup Bidwell	1.5 days	Thu 11/15/18	Fri 11/16/18
216	Pour Deck and Closure Pour	1 day	Mon 11/19/18	Mon 11/19/18
217	Finalize CUY-480-1842L including Parapet - Second Half	8 days	Tue 11/20/18	Thu 11/29/18
218	End Deck Construction CUY-480-1842L	0 days	Thu 11/29/18	Thu 11/29/18
219	Grind-in Grade	20 days	Mon 4/1/19	Fri 4/26/19
220	Seal Parapets	20 days	Mon 4/29/19	Fri 5/24/19
221	Erect Vandal Protection Fencing	50 days	Mon 5/27/19	Fri 8/2/19
222	End CUY-480-48.42 Project Construction	0 days	Fri 8/2/19	Fri 8/2/19



Project: CUY-480-18.42 Construction :
Date: Mon 8/6/12

Task — Progress — Summary — External Tasks — Deadline
Split — Milestone — Project Summary — External Milestone

E.L. ROBINSON ENGINEERING

CUY-480-18.42
PID 90591

APPENDIX B

Existing Plans Including Retrofit Plans

U-3

T-18

CONVENTIONAL SIGNS

PROPERTY LINE	P
EXISTING RIGHT OF WAY	
SUBDIVISION LINE	
SUBLOT LINE OR EXISTING EASEMENT	Z Z
ORIGINAL TOWNSHIP LOT LINE	
CORPORATION LINE	
LIMITED ACCESS LINE	LA
LIMITED ACCESS LINE AND RIGHT OF WAY LINE	LA & RW
RIGHT OF WAY LINE AND HIGHWAY EASEMENT LINE	R/W
AERIAL EASEMENT LINE	AERIAL
TEMPORARY RIGHT OF WAY	T
SEWER EASEMENT LINE	S S
SLOPE EASEMENT LINE	SL SL
CHANNEL EASEMENT	X X
PARTICIPATION LINE	P P P
CENTER LINE	
FENCE LINE	x x
GUARD RAIL (EXISTING)	
GUARD RAIL (PROPOSED)	
RAILROAD	
POWER POLES	
TELEPHONE POLES	
POWER AND TELEPHONE POLES	
LIGHT POLES	
TREES (EXISTING)	
ELECTRICAL TOWER	
WATER LINE	/ /
GAS LINE	G G
TELEPHONE CONDUIT	T T
EXISTING SEWERS (R/W PLANS)	S S
EXISTING STORM SEWER (DRAINAGE PLANS)	
EXISTING SANITARY SEWER (DRAINAGE PLANS)	
OIL LINE	o o
FIRE HYDRANT (EXISTING)	
FIRE HYDRANT (PROPOSED)	
MANHOLE (EXISTING)	
MANHOLE (PROPOSED STORM)	
MANHOLE (PROPOSED SANITARY)	
CATCH BASIN OR INLET (EXISTING)	
CATCH BASIN OR INLET (PROPOSED)	

MAR 27 ...

DEC 2 0 ...

STATE OF OHIO
DEPARTMENT OF HIGHWAYS
CUY-480-18.43
PART 2 - SUPERSTRUCTURE
CUYAHOGA COUNTY
CITY OF GARFIELD HEIGHTS
CITY OF INDEPENDENCE
VILLAGE OF VALLEY VIEW
GRADE SEPARATION WITH THE BALTIMORE & OHIO R.R.

FED. RD. DIVISION	STATE	PROJECT	
2	OHIO	I-480-4(41)172	

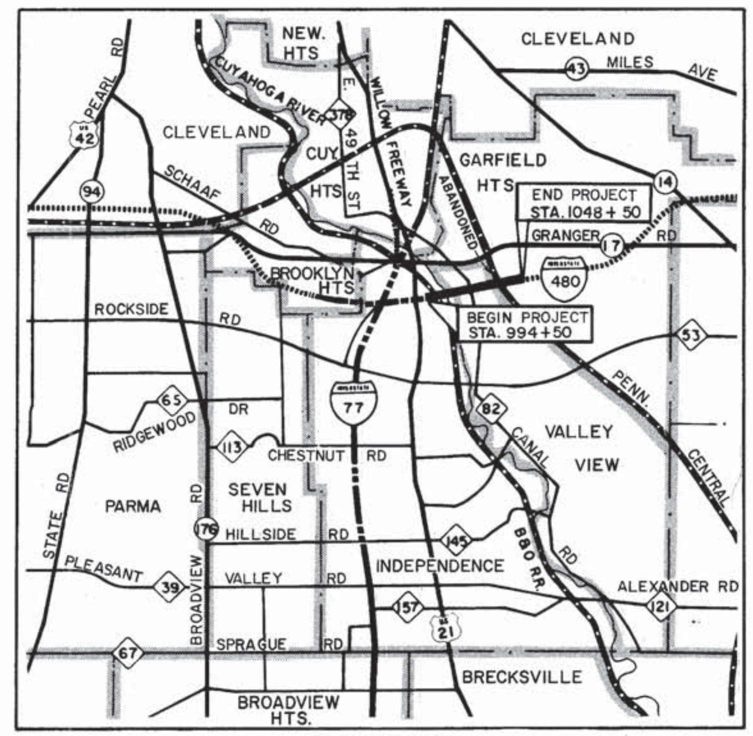
1
112

CUYAHOGA COUNTY
CUY-480-18.43 PART 2

PROJECT DESIGNATION CUY.-80-18.43 PART 2 APPEARING THROUGHOUT THIS PLAN SHALL BE CONSIDERED TO READ CUY.-480-18.43 PART 2.

LIMITED ACCESS

THIS IMPROVEMENT IS ESPECIALLY DESIGNED FOR THROUGH TRAFFIC AND HAS BEEN DECLARED A LIMITED ACCESS HIGHWAY OR FREEWAY BY ACTION OF THE DIRECTOR OF HIGHWAYS IN ACCORDANCE WITH THE PROVISIONS OF SECTION 5511.02, REVISED CODE OF OHIO.



LOCATION MAP

SCALE

0 1 2 3 MILES

N

PORTION TO BE IMPROVED	
STATE ROADS	
COUNTY ROADS	
OTHER ROADS	
FUTURE CONSTRUCTION	
UNDER CONSTRUCTION	

SCALE
PLAN 1" = 50' PROFILE HOR 1" = 100'
CROSS SECTIONS 1" = 10' PROFILE VERT 1" = 10'

I-480-4(41)172
1969 SPECIFICATIONS

THE STANDARD SPECIFICATIONS OF THE STATE OF OHIO, DEPARTMENT OF HIGHWAYS, INCLUDING CHANGES AND SUPPLEMENTAL SPECIFICATIONS LISTED IN THE PROPOSAL SHALL GOVERN THIS IMPROVEMENT.

THE RIGHT OF WAY FOR THIS IMPROVEMENT WILL BE PROVIDED BY THE STATE OF OHIO.

I HEREBY APPROVE THESE PLANS AND DECLARE THAT THE MAKING OF THIS IMPROVEMENT WILL NOT REQUIRE THE CLOSING OF THE HIGHWAY TO TRAFFIC AND THAT PROVISIONS FOR THE MAINTENANCE AND SAFETY OF TRAFFIC WILL BE AS SET FORTH ON THE PLANS AND ESTIMATES.

APPROVED DATE 9-1-70	Charles M. Yurick	DIVISION DEPUTY DIRECTOR
APPROVED DATE 9-2-70	C. H. Albrator	ENGINEER OF BRIDGES
APPROVED DATE 9-2-70	R. E. Catlin	ENGINEER OF LOCATION AND DESIGN
APPROVED DATE 9-3-70	George J. Thormyer	DEPUTY DIRECTOR OF DESIGN AND CONSTRUCTION
APPROVED DATE ___		DEPUTY DIRECTOR OF RIGHT OF WAY
APPROVED DATE 9-10-70	Thomas M. Major	DEPUTY DIRECTOR OF PLANNING AND PROGRAMMING
APPROVED DATE 9-10-70	[illegible]	FIRST ASSISTANT DIRECTOR
APPROVED DATE 9-10-70	[illegible]	DIRECTOR OF HIGHWAYS

INDEX OF SHEETS

Sheet Nos. 69, 79, 81, 82, 85, 98, & 100 revised 11-16-70 EBL
Sheet Nos. 2, 10, 11, 13, 14, 23, 24, 25, 26, 27, 28, 46, 73, 74, 75, 76, 86, 88, 89, 90, 91, 93, 96, 97, 98, & 99, revised; 86 A & 88A added, 8-15-73. NAA

SUPPLEMENTAL SPECIFICATIONS

NUMBER	DATE	NUMBER	DATE
801	1-1-69	1001	1-1-69
808	11-14-69	816	1-1-69
811	1-1-69		
836	6-17-69		
815	1-1-69		
938	8-12-69		

STANDARD DRAWINGS

NUMBER	DATE	NUMBER	DATE
BP-1	6-1-65	AS-1-67	1-11-68
BP-2	12-1-68	RB-1-55	2-2-59
BP-3	12-1-68	SD-1-69 Sh.1,2,3,4	6-12-69
BP-4	12-1-68	BR-1-67 sh. 1,2,3	2-1-68
BP-7	1-1-66		
F-1	3-10-69		
F-2	3-10-69		
GR-2B	2-15-68		
GR-5	1-15-68		
GR-6	7-15-68		
HL-1	11-1-65		
HL-2	11-1-65		
HL-3	11-1-65		
HL-4	1-1-66		
MC-3	6-20-69		
MC-4	6-13-69		
GR-1	1-1-67		

LINE DATA

BEGIN PROJECT STA. 994 + 50 ℄ I-480
END PROJECT STA. 1048 + 50 ℄ I-480
ADD FOR STATION EQUATION: STA. 1040 + 50.00 BACK EQUALS
STA. 1040 + 25.85 AHEAD = 24.15 LIN. FT.
NET LENGTH OF PROJECT: 5424.15 LIN. FT. = 1.027 MILES
ADD WORK STA. 992 + 48 TO STA. 994+50 = 202 LIN. FT.
STA. 1048 + 50 TO STA. 1049 + 30 = 80 LIN. FT.
NET LENGTH OF WORK: 5706.15 LIN. FT. = 1.081 MILES.

PREPARED AND RECOMMENDED BY

HOWARD NEEDLES TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

H.G. SOURS
ASSOCIATE
COLUMBUS

Browning Crow
BROWNING CROW

STATE OF OHIO
BROWNING CROW
18873
REGISTERED PROFESSIONAL ENGINEER

FILE NO.	CUYAHOGA COUNTY 00347 DATE OF LETTING CONTRACT NO.

DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
BUREAU OF PUBLIC ROADS

APPROVED

DIVISION ENGINEER DATE

SCHEMATIC PLAN

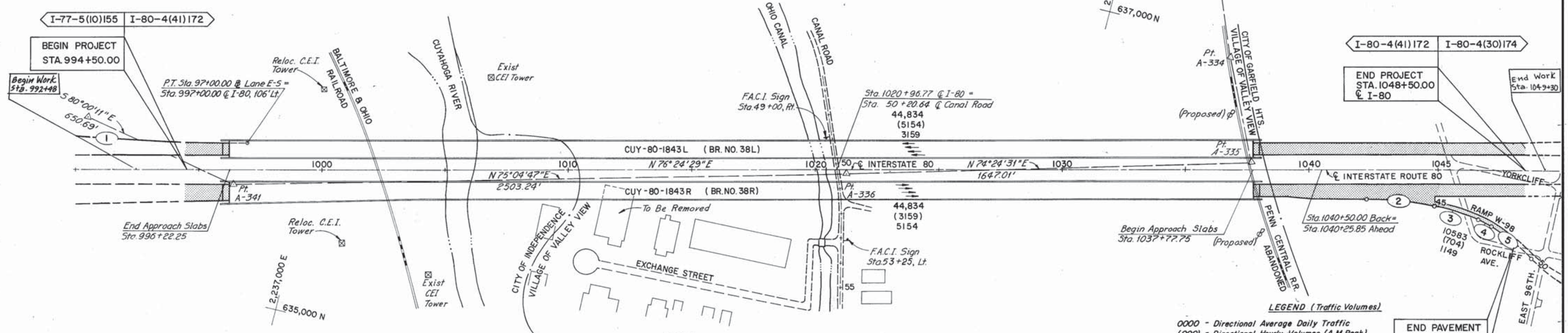
FED. RD. DIVISION	STATE	PROJECT	
2	OHIO		

CUYAHOGA COUNTY
CUY.-80-18.43 Part 2

PROPOSED STRUCTURE

TYPE: Continuous welded steel girder with floor system and reinforced concrete deck and substructure.
SPANS: Unit I - 220', 300' and 275'
Units 2,3 and 4 - 25' cantilever, 2 @ 300' and 275'
Unit 5 - 25' cantilever, 2 @ 225' and 180'
ROADWAY: Unit IL - Varies 71'-0" to 71'-4" face to face of parapets.
Unit IR - Varies 71'-0" to 86'-6" face to face of parapets.
Units 2,3,4 and 5 - 71'-0" face to face of parapets.
LOADING: HS 20-44 and Interstate Alternate Loading
SKEW: None
WEARING SURFACE: $2\frac{1}{2}$" Asphalt Concrete ~~1" Monolithic Concrete~~
APPROACH SLABS: AS-1-67 (25' long)
ALIGNMENT: Tangent
SUPERELEVATION: Normal Crown

URBAN RADIAL (WITHOUT LEE AND BEDFORD FREEWAYS)		
DESIGN DESIGNATIONS	EAST END OF PROJECT	WEST END OF PROJECT
Current A.D.T. 1970	69,199	70,460
Design Year A.D.T. 1990	79,085	89,668
D.D.H.V.	7,567A.M. 7,096P.M.	8,313
D (Directional Distribution)	62% - 38%	62% - 38%
T (Per Cent B.C. Trucks)	3	3
V (Design Speed)	60 MPH	60 MPH



PLAN

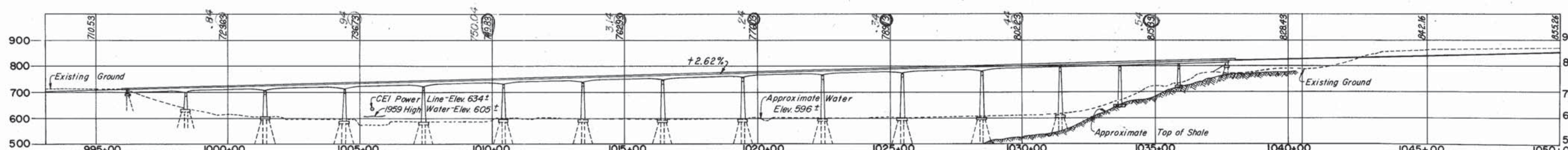
Note: For Curve Data See Sheet No. 3
Curves are identified like this: (2)

Profile Grades have been revised to accommodate the addition of the $2\frac{1}{2}$" Surface Course to the Bridge.

LEGEND (Traffic Volumes)

0000 - Directional Average Daily Traffic
(000) - Directional Hourly Volumes (A.M. Peak)
00 - Directional Hourly Volumes (P.M. Peak)

Note:
Directional Design Hourly Volumes for each movement were computed at 5.7474% of the A.D.T. (or 11.4949% of the directional A.D.T.) based on a 9% peak hour factor, a 62%-38% distribution by direction and 3% trucks.



PROFILE

Note: The Superstructure is not part of this contract.

STA. 1040+50.00 BACK = AHEAD STA. 1040+25.85
STA. EQUATION

SCALE 1" = 200 HOWARD, NEEDLES, TAMMEN & BERGENDOFF
MADE I.M. DATE 7-15-69 CONSULTING ENGINEERS
TRCD. I.N. DATE 10-16-69 KANSAS CITY CLEVELAND NEW YORK
CKD. DATE

100 0 100 200 300 400
SCALE IN FEET

REV. 6-15-73

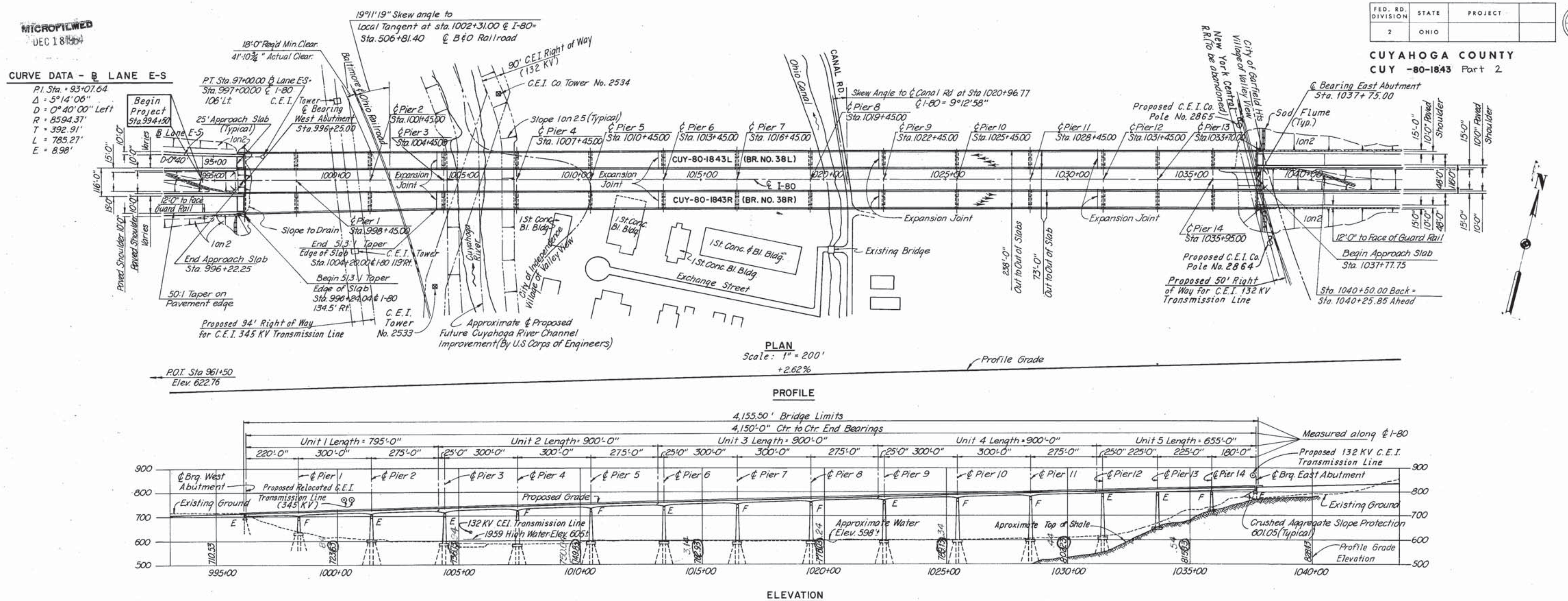
SCHEMATIC PLAN AND DESIGN DESIGNATION

U-3

FED. RD. DIVISION	STATE	PROJECT
2	OHIO	

73/112

CUYAHOGA COUNTY
CUY -80-1843 Part 2



PLAN
Scale: 1" = 200'

PROFILE

ELEVATION

FOUNDATION DATA

All piles are 12 BP 53 or 14" C.I.P. concrete with an allowable design load of 65 tons per pile at the piers and 40 tons at the abutments. The tabulation to the right gives the pile type, and estimated average pay length.

Substructure Unit	Pile Type	Estimated Pile Length
West Abutment	14"φ C.I.P.	64'
Pier 1L	14"φ C.I.P.	75'
Pier 1R	14"φ C.I.P.	75'
Pier 2L	14"φ C.I.P.	105'
Pier 2R	14"φ C.I.P.	110'
Pier 3	14"φ C.I.P.	103'
Pier 4	14"φ C.I.P.	92'
Pier 5	14"φ C.I.P.	98'
Pier 6	14"φ C.I.P.	99'
Pier 7	14"φ C.I.P.	104'
Pier 8	14"φ C.I.P.	94'
Pier 9	14"φ C.I.P.	99'
Pier 10	14"φ C.I.P.	97'
Pier 11	14"φ C.I.P.	86'
Pier 12	14"φ C.I.P.	54'
Pier 13L	—	Spread Footing Allowable Bearing Capacity - 6 Tons per sq. ft.
Pier 13R	—	
Pier 14L	—	
Pier 14R	—	
East Abutment	12 BP53	38'

PROPOSED STRUCTURE

TYPE: Continuous welded steel girder with floor system and reinforced concrete deck and substructure.
SPANS: Unit 1 - 220', 300' and 275'
Units 2, 3 and 4 - 25' cantilever, 2 @ 300' and 275'
Unit 5 - 25' cantilever, 2 @ 225' and 180'
ROADWAY: Unit 1L - Varies 71'-0" to 71'-4" face to face of parapets.
Unit 1R - Varies 71'-0" to 86'-6" face to face of parapets.
Units 2, 3, 4 and 5 - 71'-0" face to face of parapets.
LOADING: HS 20-44 and Interstate Alternate Loading
SKEW: None
WEARING SURFACE: $2\frac{1}{2}$" Asphalt Concrete / 1" Monolithic Concrete
APPROACH SLABS: AS-1-67 (25' long)
ALIGNMENT: Tangent
SUPERELEVATION: Normal Crown

PROPOSED 132 KV TRANSMISSION LINE DATA

	Minimum Clear Vertical	Lateral
Left Bridge	45' ft.±	20' ft.±
Right Bridge	44' ft.±	20' ft.±
Future Bridge	44 ft.±	—

All clearances are measured at 60°F. Lateral clearances are measured from power line with 45° sideswing to the nearest light pole.

Note: For light pole locations see Lighting Plans.

PROPOSED 345 KV TRANSMISSION LINE DATA

	Minimum Clear Vertical	Lateral
Left Bridge	41 ft.±	47 ft.±
Right Bridge	42 ft.±	45 ft.±
Future Bridge	41 ft.±	37 ft.±

All clearances are measured at 60°F. Lateral clearances are measured from power line with 45° sideswing to the nearest light pole.

EXISTING 132 KV TRANSMISSION LINE DATA

	Elevation Ground Wire	Power Line
Tower No. 2533	709.5	672.0
Tower No. 2534	694.5	657.0
Low Pt. at 60°F (350'± South of Tower No. 2534)	671.0	632.0

To be replaced by the proposed 345 KV transmission line prior to start of bridge construction.

TRAFFIC DATA: (1990)

I-80 - 44,834 A.D.T. (Each Way)
5,154 D.D.H.V.

H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

GENERAL PLAN AND ELEVATION

I-80 OVER CUYAHOGA RIVER VALLEY
BR. NO. CUY-80-1843 L & R STA. 996+22.25 TO STA. 1037+77.75
CUYAHOGA COUNTY OHIO

DRAWN HPH DATE 8-18-66 | TRACED DATE | CHECKED DATE 10-22-66 | REVIEWED WF DATE 8-22-71 | REVISED | SHEET 1/28

REV. 8-15-73

1. DESIGN SPECIFICATION
This structure conforms to Standard Specifications for Highway Bridges adopted by the American Association of State Highway Officials, 1965, including the 1966 - 1967 Interim Specifications and the Ohio "Supplement" to these specifications.

2. DESIGN DATA
Design Loading - HS-20-44 and the Interstate Alternate Loading.
Concrete Class C - unit stress 1,200 p.s.i. for superstructure.
unit stress 1,333 p.s.i. for abutments.
Structural Steel - ASTM A588 - unit stress 27,000 p.s.i.
ASTM A36 - unit stress 20,000 p.s.i.
ASTM A237, Class B - Minimum Yield Point 55,000 p.s.i.
ASTM A486, Class 90 - Minimum Yield Point 60,000 p.s.i.
ASTM A193, Grade B7 - Minimum Yield Point 105,000 p.s.i.
Reinforcing Steel - ASTM A615, A616, A617 - unit stress 20,000 p.s.i.

3. SUPPLEMENTAL SPECIFICATIONS
Reference shall be made to Supplemental Specifications No. 808, Chemical Admixtures for Concrete, dated 11-14-69, No. 811, Examination of Welds, dated 1-1-69, and No. 836, Concrete Curing and Protective Membrane, dated 6-17-69.

4. REFERENCE DRAWINGS
Reference shall be made to Standard Bridge Drawings RB-1-55, revised 2-2-59, SD-1-69, dated 6-12-69 (Sheets 1, 2 and 3 of 4) and to AS-1-67, revised 6-12-69.

5. DIMENSIONS
Dimensions given are measured horizontally and at 60° F. unless otherwise noted.

6. CONCRETE DECK
(a) The steel girders shall be fabricated with camber, as specified on the plans, to compensate for the deflections due to the weight of concrete and steel. The theoretical deflections are tabulated on the plans.
(b) The final surface of the roadway shall conform to the elevations shown on the plans. To compensate for deflections due to dead load of the concrete, the screeds used to strike off the surface of the concrete to the final desired grade line shall be adjusted by amounts equal to deflections shown for this dead load. The theoretical elevations required at the curbs before concrete is placed are tabulated on Sheet $\boxed{27/28}$. Screeds may require further adjustments due to irregularities in the fabricated steel.
(c) The depth of concrete over each stringer or girder (top of concrete to top of flange or top of web) at the supports is given on the plans. The concrete slab shall be of uniform thickness between stringers and girders with adjustments obtained by varying the thickness of the haunches over the stringers or girders.
(d) The aforementioned depth of concrete over each stringer or girder is the nominal dimension. The quantity of deck concrete to be paid for shall be based on this dimension even though deviation from it may be necessary because the top flanges may not have the exact camber or conformation required to place it parallel to the finished grade. Deduction shall be made for the volume of encased steel plates in accordance with Section 511.19 of the Construction and Material Specifications.

7. REINFORCING STEEL
(a) All bars are designated in the plans by bar numbers. The bar size is indicated by the first digit of three-digit numbers and by the first two digits of four-digit numbers. All bar dimensions are given out to out. All bars of a series shall vary in length by a constant increment.
(b) The clear distance between reinforcing steel and face of concrete shall be 2" unless otherwise shown on the plans.

8. STRUCTURAL STEEL
All girder webs, flanges, splice plates, web stiffeners and sign support parts on the exterior face of exterior girders, and certain parts of the expansion joints identified in the plans shall be ASTM A588 Structural Steel. The ASTM A588 structural steel shall meet Supplementary Requirement S1 of AASHO M222. All bolted connections of ASTM A588 Structural Steel shall be made with High Strength Steel Bolts having the corrosion characteristics of ASTM A588 steel and having all the mechanical properties of ASTM A325 High Strength Steel Bolts.
All other structural steel parts, except as noted in the plans, shall be ASTM A36 Structural Steel. All bolted connections of ASTM A36 Structural Steel shall be made with ASTM A325, 1" ∅ High Strength Steel Bolts, except as noted in the plans.
All bolting shall be in accordance with Item 513.10.

9. WELDING
(a) Electrodes and flux-electrode combinations for welding A588 steel shall be as listed in the following table:

Base Metal	Shielded Metal-Arc	Submerged Arc	Gas Metal-Arc	Flux Cored Arc
A588 used in a painted application	AWS A5.1 E7015, 16, 18 or 28	AWS A5.17 F71, F72, F73 or F74-Exxxx	AWS A5.18 E70S-1B, 2, 3, 6 or E70U-1	AWS A5.20 E70T-1, 5 or 6

(b) Welds on non-stress carrying members are shown thus:

N

10. COORDINATION OF WORK
(a) The work under this Contract shall be coordinated with the work of the Contractor for Part 1 - Substructure.
(b) All anchor bolts will be installed by the Contractor for Part 1 - Substructure, but the locations shall be checked by survey as a part of this Contract. The Contractor shall submit an as built anchor bolt layout plan to the Director in triplicate for approval prior to start of steel erection.
(c) Top of masonry elevations shall be checked as a part of this Contract. Variations from plan elevation will be corrected by the Contractor for Part 1 - Substructure in accordance with Section 513.24.
(d) The surveys of anchor bolt locations and top of masonry elevations shall be included for payment in the unit prices bid for other items.

11. ITEMS NOT INCLUDED IN BRIDGE PLANS
The following items are not included in the bridge plans.
See Roadway Plans for details.
(1) Approach roadway
(2) Approach slabs
(3) Lighting
(4) Signing
(5) Guard Rail

12. DECK POURING SEQUENCE AND METHOD:
See notes on sheet 25/28.

ESTIMATED QUANTITIES									
ITEM	TOTAL	UNIT	DESCRIPTION	ABUT-MENTS		SUPER-STRUCTURE	GENERAL		
509	5,029,864	Lbs.	Reinforcing Steel	1223		5028,641			
	5,031,502					5,030,279			
511	3534	Cu. Yd.	Class C Concrete, Abutments	3534					
511	18064 17,795	Cu. Yd.	Class C Concrete, Superstructure		18064	17,795			
513	27,669,500	Lb.	Structural Steel (ASTM A588)			27,669,500			
513	11,263,400	Lb.	*Structural Steel (ASTM A36)			11,263,400			
514	38,932,900	Lb.	Field Painting of Structural Steel			38,932,900			
518	972	Lin. Ft.	8" ∅ Standard Pipe Collector System Including Specials and Accessories			972			
518	112	Each	Scuppers Including Supports			112			
518	224	Cu. Yd.	Porous Backfill	224					
625			See Sheet 62 for Lighting Summary						
808	18064 17,795	Units	Chemical Admixture for Concrete Type A, B or D		18064	17,795			

For additional Bridge Quantities refer to Sheet 88/112

* Item 513, Structural Steel (ASTM A36), includes 606,800 pounds of ASTM A237 - Class B, Steel Forgings; 575,800 pounds of ASTM A486 - Class 90, Steel Castings and all other miscellaneous metals except ASTM A588.

JACKING HOLES, as specified on Standard Drawing AS-1-67 shall not be provided.

H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN, & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

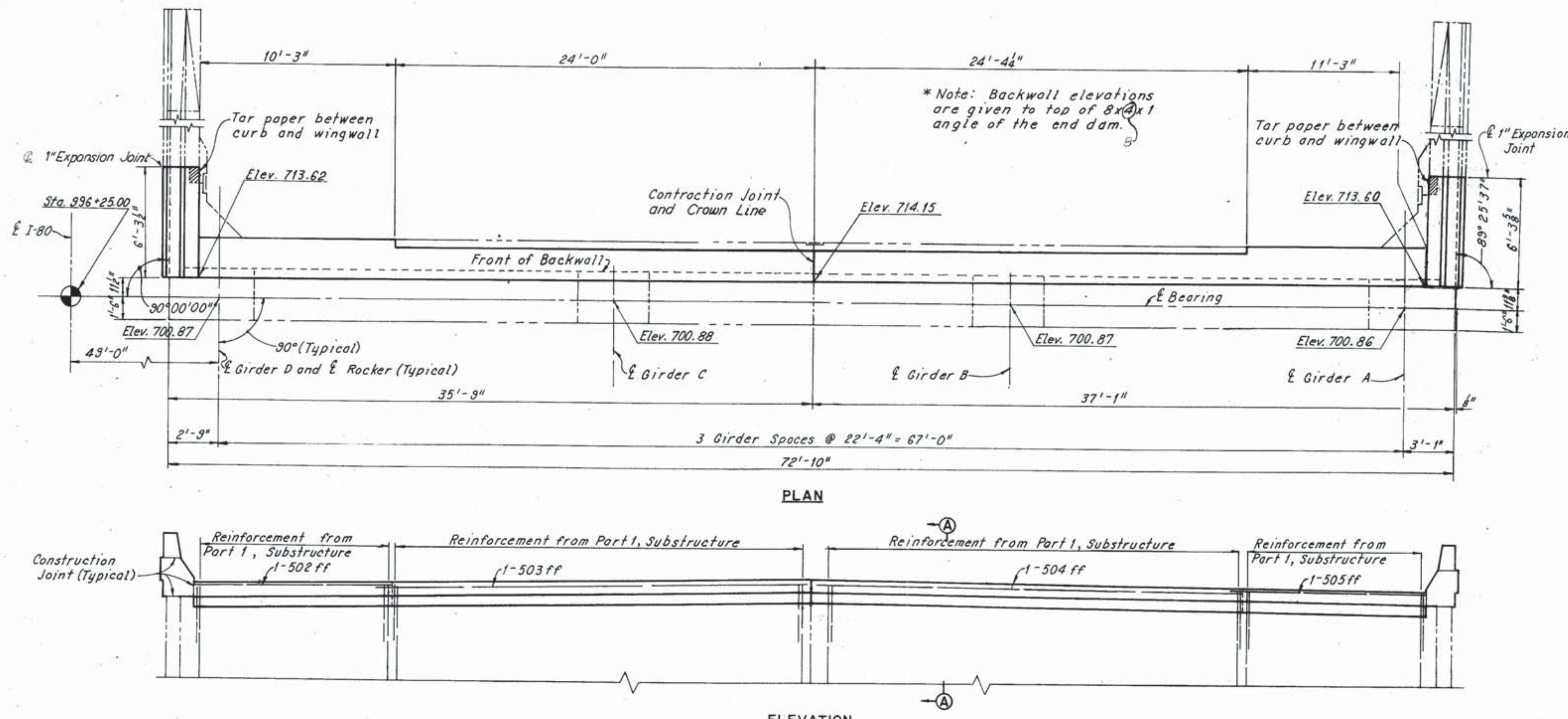
GENERAL NOTES AND ESTIMATED QUANTITIES

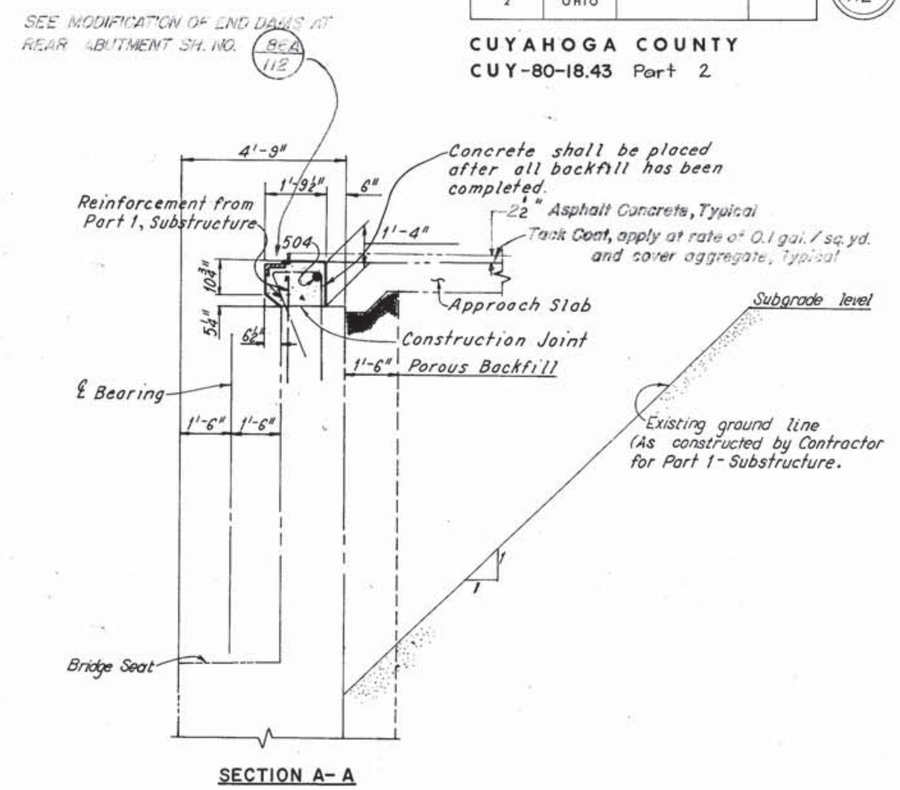
I-80 OVER CUYAHOGA RIVER VALLEY
BR. NO. CUY-80-1843 L & R STA. 996+22.25 TO STA. 1037+77.75
CUYAHOGA COUNTY OHIO

DRAWN CHB DATE 2-19-69	TRACED GEM DATE 2-26-69	CHECKED WSF DATE 3-22-69	REVIEWED WF DATE 8-22-70	REVISED 11-16-70 SHEET 2/28

REV. 8-15-73

CUYAHOGA COUNTY
CUY-80-18.43 Part 2

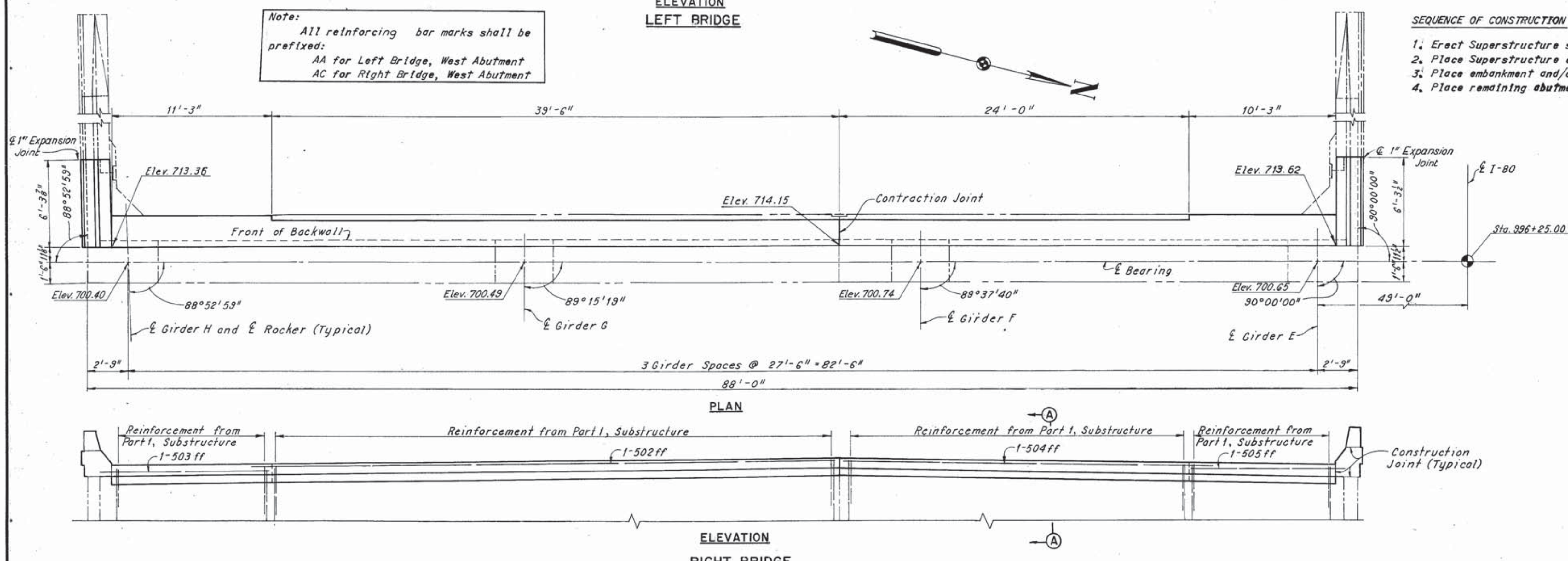




Note:
All reinforcing bar marks shall be prefixed:
AA for Left Bridge, West Abutment
AC for Right Bridge, West Abutment

SEQUENCE OF CONSTRUCTION

1. Erect Superstructure steelwork.
2. Place Superstructure concrete.
3. Place embankment and/or backfill.
4. Place remaining abutment concrete.



Notes:
For Curb and Parapet Details see Sheet 4/28.
Portions of abutments shown in phantom will be constructed with Part 1 - Substructure.
For end dam and curb plate details see Sheet 14/28.
The following abbreviation is used:
ff = far face

H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
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KANSAS CITY CLEVELAND NEW YORK

WEST ABUTMENTS

I-80 OVER CUYAHOGA RIVER VALLEY
BR. NO. CUY-80-1843 L&R STA. 996+22.25 TO STA. 1037+77.75

CUYAHOGA COUNTY OHIO

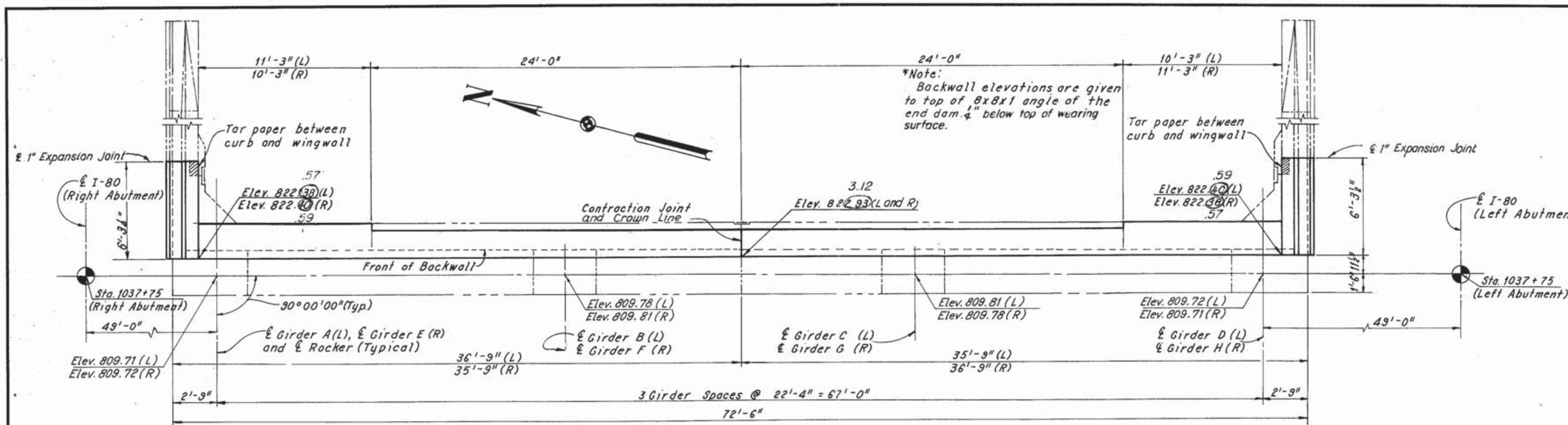
DRAWN MCB DATE 1-14-69	TRACED MCB DATE 2-13-69	CHECKED CHB DATE 3-10-69	REVIEWED WF DATE 8-21-70	REVISED	SHEET 3/28

REV. 8-15-73

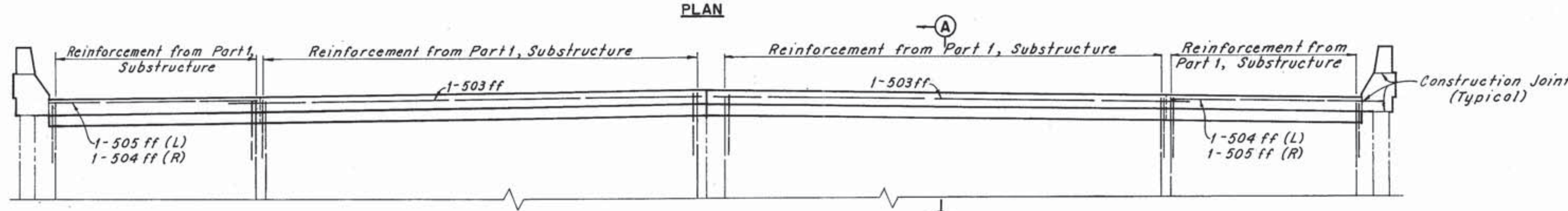
FED. RD. DIVISION	STATE	PROJECT	
2	OHIO		

76/112

CUYAHOGA COUNTY
CUY-80-18.43 Part 2

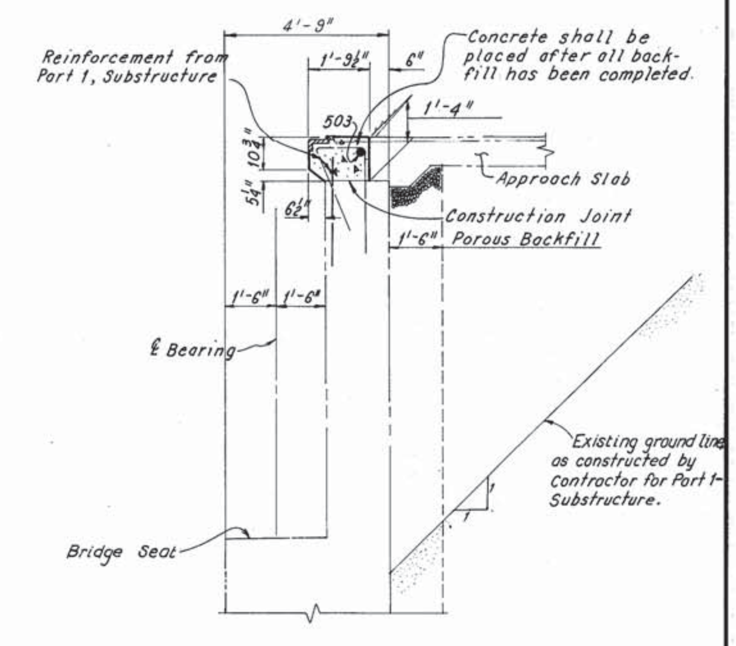


PLAN

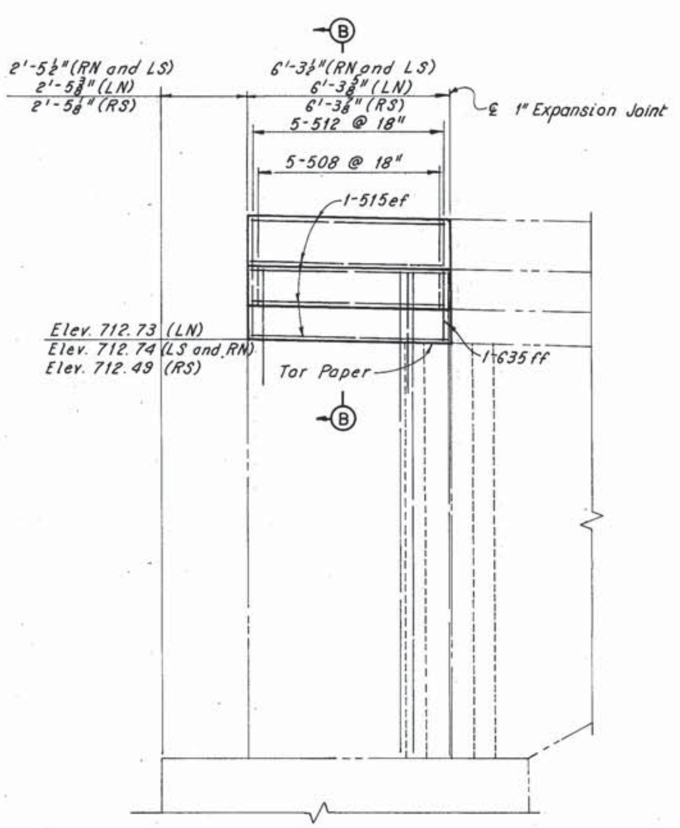


ELEVATION

Left Abutment shown, Right Abutment Similar.

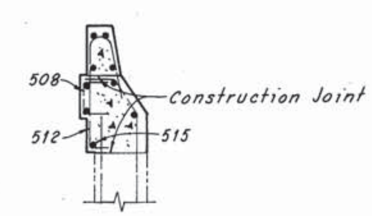


SECTION A-A



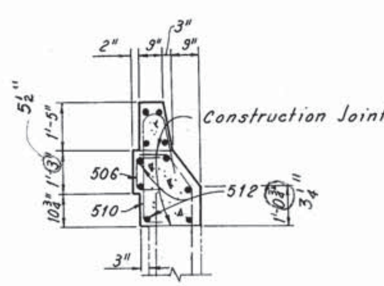
WEST ABUTMENTS - CURB AND PARAPET ELEVATION

(North Wingwalls Shown, South Wingwalls similar except as noted)

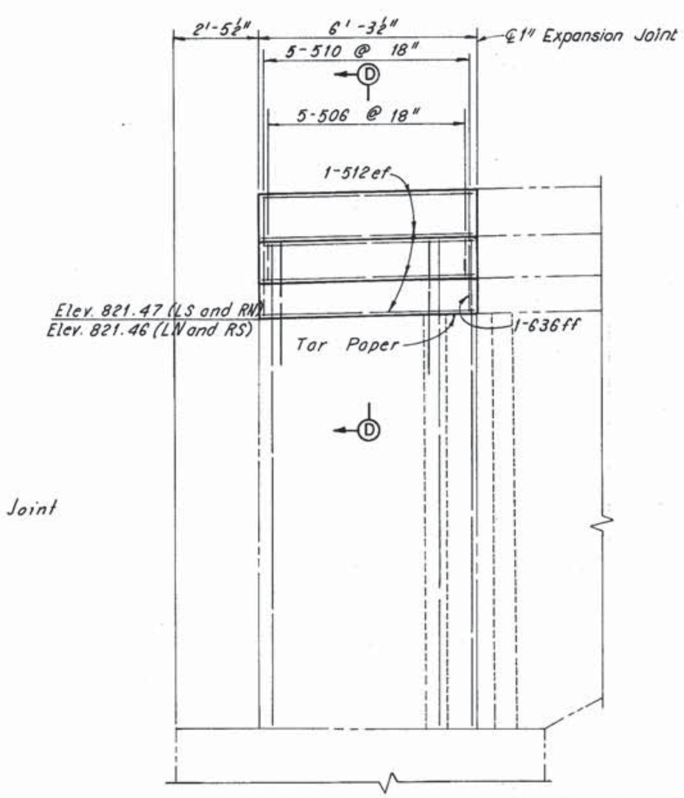


SECTION B-B

Curb and parapet dimensions same as Section D-D.



SECTION D - D



EAST ABUTMENTS - CURB AND PARAPET ELEVATION

(South Wingwalls Shown, North Wingwalls similar except as noted)

Notes:
For Sequence of Construction and additional notes see Sheet 3/28.
The following abbreviations are used:

- RN = Right Bridge, North Wingwall
- LS = Left Bridge, South Wingwall
- RS = Right Bridge, South Wingwall
- LN = Left Bridge, North Wingwall
- L = Left Bridge
- R = Right Bridge
- ff = far face
- ef = each face

All reinforcing bar marks shall be prefixed as follows:
- AA-West Abutment, Left Bridge
- AB-East Abutment, Right Bridge
- AC-West Abutment, Right Bridge
- AD-East Abutment, Left Bridge

Note:
Tar paper to be included with "Item 511" for payment.

H.N.T.B. BR. NO. 38L AND 38R

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EAST ABUTMENT AND CURB AND PARAPET DETAILS

I-80 OVER CUYAHOGA RIVER VALLEY
BR. NO. CUY-80-1843 L&R STA. 996+22.25 TO STA. 1037+77.75

CUYAHOGA COUNTY OHIO

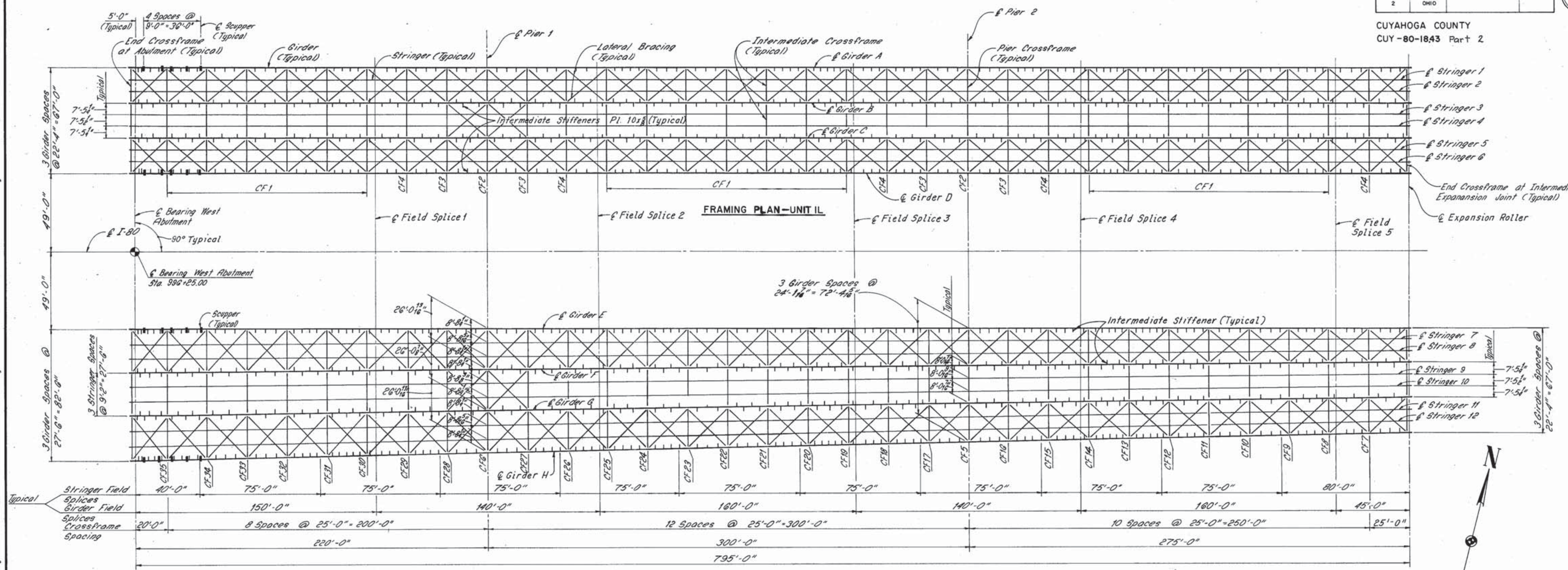
DRAWN MCB DATE 7-14-69	TRACED GEM DATE 7-17-69	CHECKED CHB DATE 3-10-69	REVIEWED WF DATE 8-21-70	REVISED	SHEET 4/28

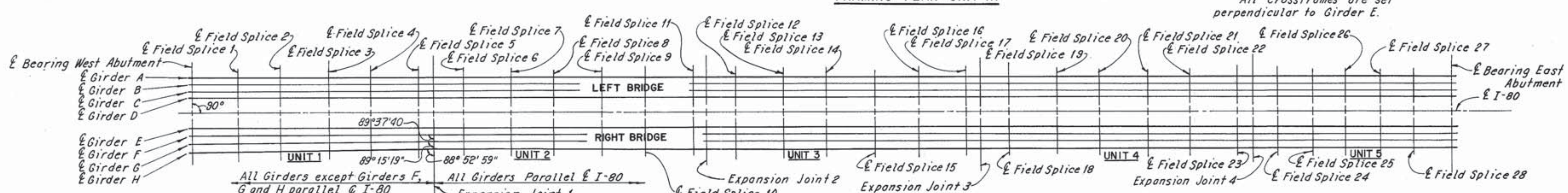
REV. 8-15-73

FED. RD. DIVISION	STATE	PROJECT	
2	OHIO		

77/112

CUYAHOGA COUNTY
CUY-80-1843 Part 2





SCHEMATIC FRAMING PLAN

Notes:
The Contractor shall submit to the Director for approval three prints showing his proposed erection procedure.
All stringers are 24 WF 68.
For stringer field splice details see Ohio Standard Drawing SD-1-69.
Crossframe types are designated with the prefix CF.
The bearings under the flared Girders E, F and G shall be set parallel with ℄ bearings of the West Abutment and Piers 1 and 2.

Girder Notes:
The girders shall be fabricated to compensate for the effects of dead load deflections and under full dead load shall parallel the profiles formed by the top of pavement elevations directly over the center line of the girders.
Top and bottom flange plates are to be the same.
Additional shop splices in the webs and flanges, as required by available plate lengths, will be permitted. The locations and details of all additional splices shall be submitted to the Director for approval prior to ordering of materials.
Intermediate stiffeners shall be placed as shown on the framing plan equally spaced between crossframes or between crossframes and girder field splices except the first two stiffener spaces from the abutments shall be one-half of this spacing. Additional intermediate stiffeners shall be placed in pairs at crossframes and 1'-2" from bearing stiffeners at piers.
Intermediate stiffeners, which are not placed in pairs, shall be welded to the bottom flange between the ℄ of pier and a point midway between the top and bottom longitudinal stiffener overlap, or between the ℄ of pier and an intermediate expansion joint, whichever comes first. All other intermediate stiffeners which are not placed in pairs shall be welded to the top flange. These stiffeners shall be welded to the flange on both sides of the stiffener with fillet welds of the same size as the web to flange weld at the same location. The end of the stiffener which is not welded to the flange shall have a clearance of 1¼" from the flange.
Intermediate stiffeners which are placed in pairs shall not be welded to a flange but shall have contact bearing with the flange to which adjacent stiffeners are welded. The end of the stiffener which is not in contact with the flange shall have a clearance of 1¼" from the flange.
Bearing stiffeners at piers and abutments shall be placed in pairs on all girders.
Bearing stiffeners at abutments shall be vertical but all other transverse stiffeners shall be perpendicular to the top flange.
Longitudinal stiffener plates shall be placed only on the interior side of the webs. They shall be placed in segments between transverse stiffeners and web splice plates.
All girder field splices shall be made with 1" diameter high strength steel bolts. The bolts shall be placed with their heads on the outside face of exterior girders and on the bottom of all flange plates.

H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

FRAMING PLAN UNIT I

I-80 OVER CUYAHOGA RIVER VALLEY
BR. NO. CUY-80-1843 STA. 996+22.25 TO STA. 1037+77.75

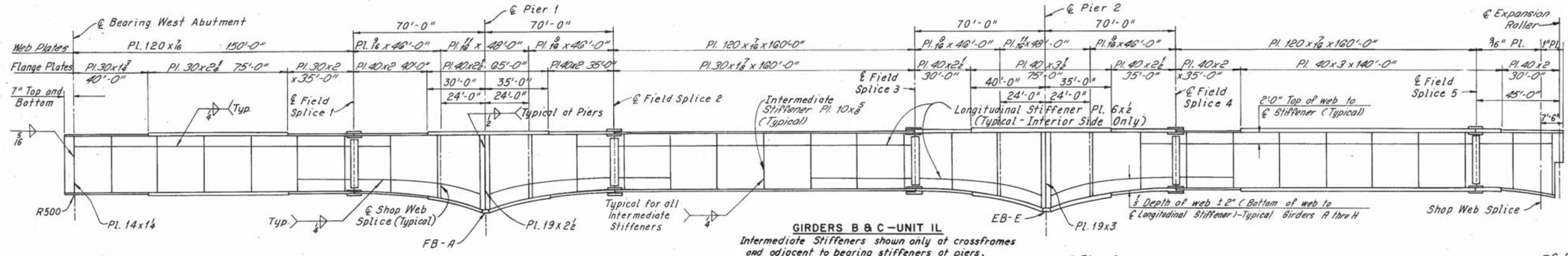
CUYAHOGA COUNTY OHIO

DRAWN JEH	TRACED AJJ	CHECKED TJS	REVIEWED WF	REVISED
DATE 6-10-68	DATE 6-20-68	DATE 11-21-68	DATE 8-22-70	SHEET 5/28

FED. RD. DIVISION	STATE	PROJECT	
2	OHIO		

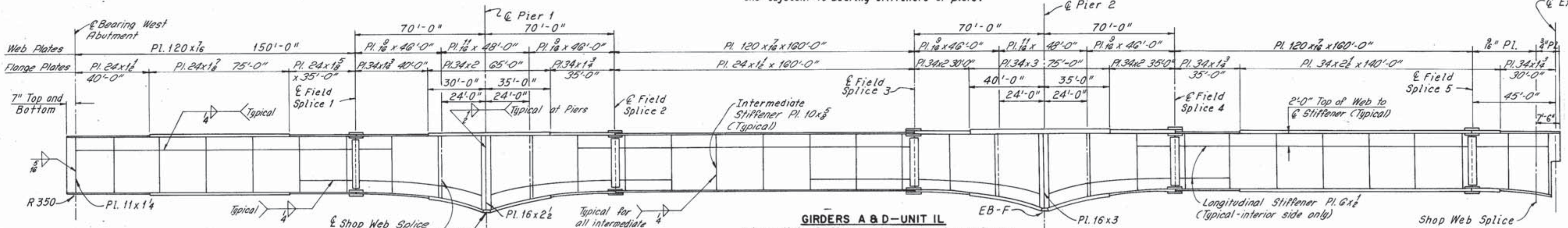
78 / 112

CUYAHOGA COUNTY
CUY -80-1843 Part 2



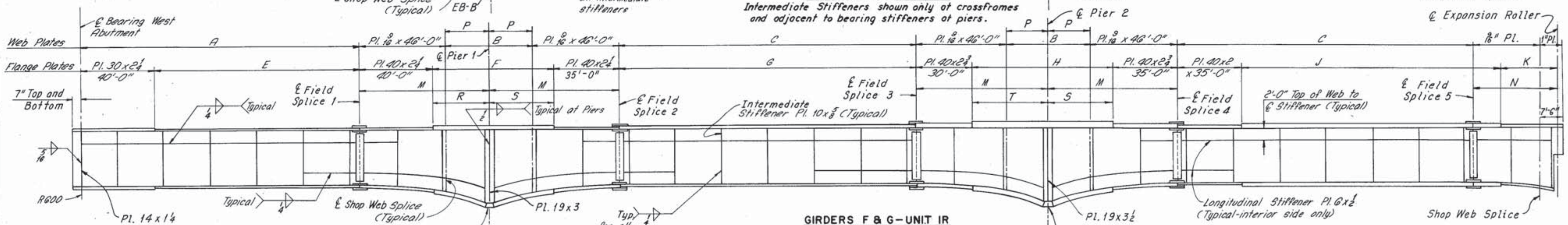
GIRDERS B & C – UNIT IL

Intermediate Stiffeners shown only at crossframes and adjacent to bearing stiffeners at piers.



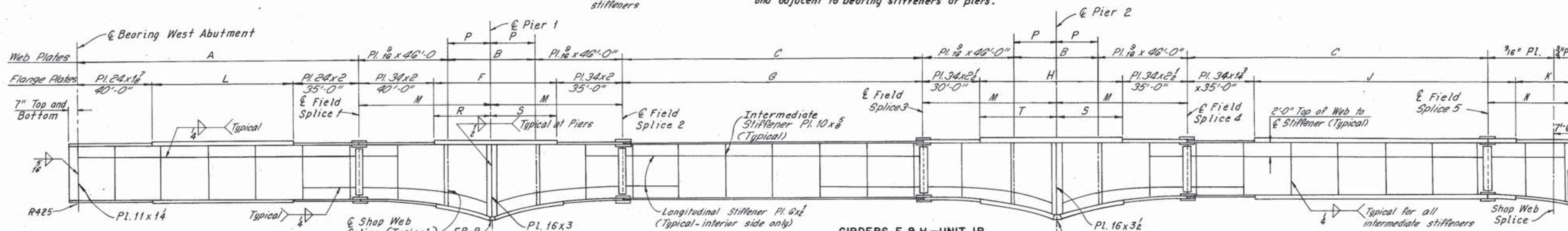
GIRDERS A & D – UNIT IL

Intermediate Stiffeners shown only at crossframes and adjacent to bearing stiffeners at piers.



GIRDERS F & G – UNIT IR

Intermediate Stiffeners shown only at crossframes and adjacent to bearing stiffeners at piers.



GIRDERS E & H – UNIT IR

Intermediate Stiffeners shown only at crossframes and adjacent to bearing stiffeners at piers.

WELD SIZE WEB TO FLANGE	
Flange Plate Thickness	Fillet Weld Size
1 1/2"	5/16"
1 3/4" thru 2 1/4"	3/8"
2 1/2" thru 4"	1/2"

Notes:
For girder notes see Sheet 5/28
For locations of intermediate stiffeners between crossframes see Sheet 5/28
For girder web and flange plate transitions see Sheet 13/28
For girder shop weld details see Sheet 10/28

DIMENSIONS						
	M	N	P	R	S	T
GIRDER E	70'-0"	45'-0"	24'-0"	30'-0"	35'-0"	40'-0"
GIRDER F	70'-0"	45'-0"	24'-0"	30'-0"	35'-0"	40'-0"
GIRDER G	70'-0 1/16"	45'-0"	24'-0 1/16"	30'-0 1/16"	35'-0 1/16"	40'-0 1/16"
GIRDER H	70'-0 3/16"	45'-0 1/8"	24'-0 3/16"	30'-0 3/16"	35'-0 3/16"	40'-0 3/16"

	WEB PLATES			FLANGE PLATES												
	A	B	C	E	F		G		H		J		K		L
SIZE	120x7/16	11/16	120x7/16	40x2	34x2 1/2	40x3	24x1 3/4	30x2 1/4	34x3 1/2	40x4	34x2 3/4	40x3	34x1 3/4	40x2	24x2 1/4
DIMENSION GIRDER E	150'-0"	48'-0"	160'-0"	—	65'-0"	—	160'-0"	—	75'-0"	—	140'-0"	—	30'-0"	—	75'-0"
DIMENSION GIRDER F	150'-0 1/16"	48'-0"	160'-0 1/16"	110'-0 1/8"	—	65'-0"	—	160'-0 1/16"	—	75'-0"	—	140'-0 1/16"	—	30'-0"	—
DIMENSION GIRDER G	150'-0 3/16"	48'-0 1/8"	160'-0 3/16"	110'-0 1/8"	—	65'-0 1/8"	—	160'-0 3/16"	—	75'-0 1/8"	—	140'-0 3/16"	—	30'-0"	—
DIMENSION GIRDER H	150'-0 5/16"	48'-0 3/8"	160'-0 5/16"	—	65'-0 3/8"	—	160'-0 5/16"	—	75'-0 3/8"	—	140'-0 5/16"	—	30'-0 1/8"	—	75'-0 5/16"

H.N.T.B. BR. NO. 38L AND 38R

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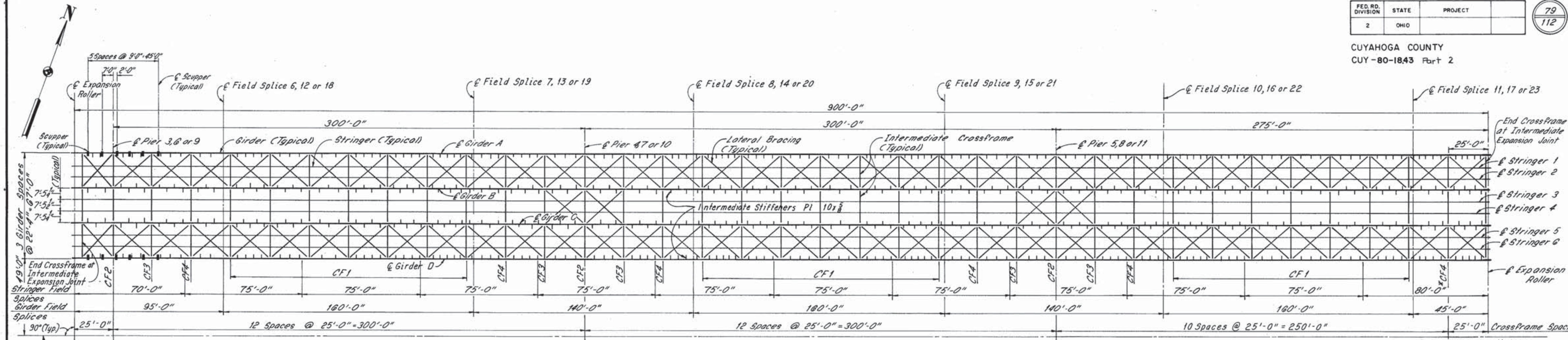
KANSAS CITY CLEVELAND NEW YORK

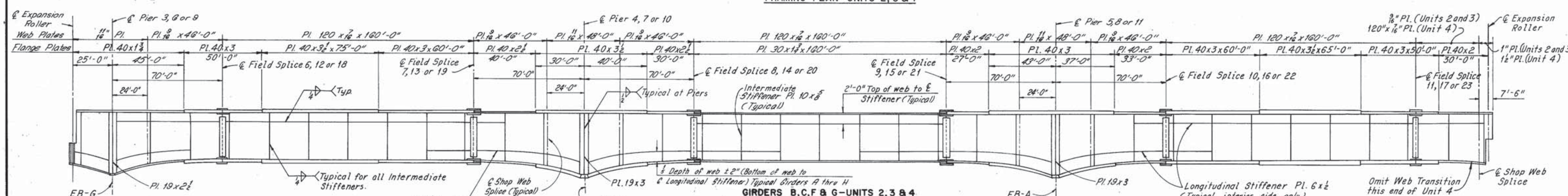
GIRDER DETAILS-UNIT I

I-80 OVER CUYAHOGA RIVER VALLEY
BR.NO.CUY-80-1843 STA.996+22.25 TO STA.1037+77.75
CUYAHOGA COUNTY OHIO

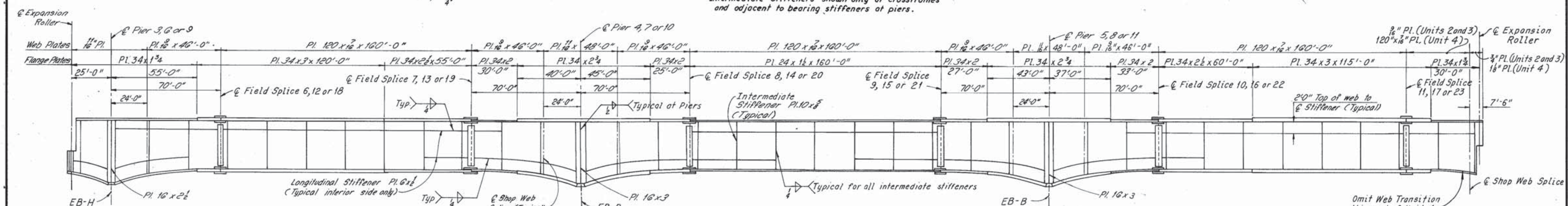
DRAWN KYH	TRACED MS	CHECKED TJS	REVIEWED WF	REVISED
DATE 6-11-68	DATE 6-20-68	DATE 11-21-68	DATE 8-21-70	SHEET 6/28



FRAMING PLAN-UNITS 2, 3 & 4



GIRDERS B, C, F & G-UNITS 2, 3 & 4
Intermediate Stiffeners shown only at crossframes and adjacent to bearing stiffeners at piers.



GIRDERS A, D, E & H-UNITS 2, 3 & 4
Intermediate Stiffeners shown only at crossframes and adjacent to bearing stiffeners at piers.

WELD SIZE WEB TO FLANGE	
Flange Plate Thickness	Fillet Weld Size
1 1/2"	5/16"
1 3/4" and 2"	3/8"
2 1/2" thru 3 1/2"	1/2"

Notes:
For girder notes see Sheet 5/28.
For girder web and flange plate transitions see Sheet 13/28.
For girder shop weld details see Sheet 10/28.
For details of flange width taper at Expansion Joint 4, see View K-K Sheet 17/28.
For girder and stringer designation for the right bridge see sheet 5/28.

H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

FRAMING PLAN AND GIRDER DETAILS UNITS 2, 3 & 4

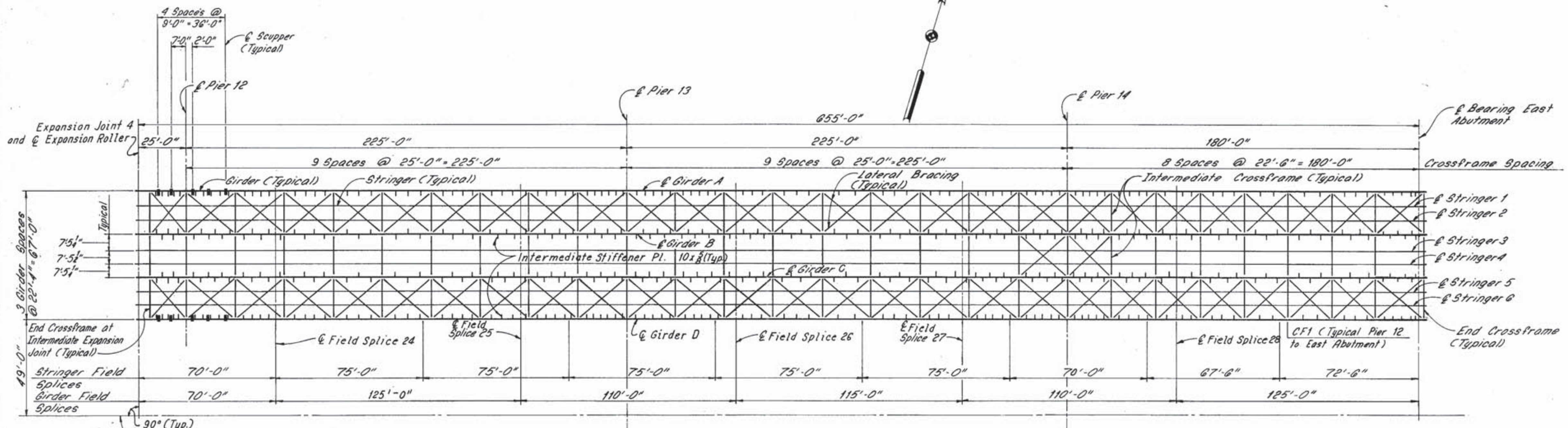
I-80 OVER CUYAHOGA RIVER VALLEY
BR. NO. CUY-80-1843 STA. 996+22.25 TO STA. 1037+77.75
CUYAHOGA COUNTY OHIO

DRAWN JEH DATE 6-12-68	TRACED AJJ DATE 6-20-68	CHECKED TJS DATE 11-21-68	REVIEWED WF DATE 8-21-70	REVISED

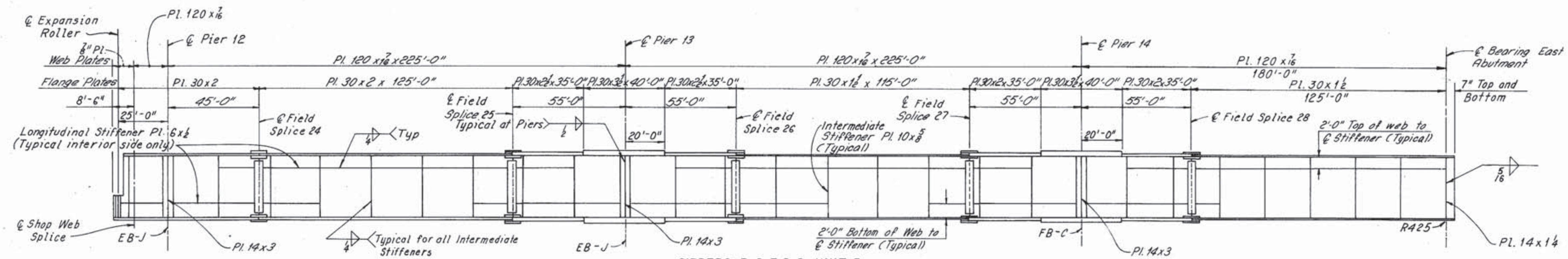
FED. RD. DIVISION	STATE	PROJECT	
2	OHIO		

80/112

CUYAHOGA COUNTY
CUY-80-18.43 Part 2

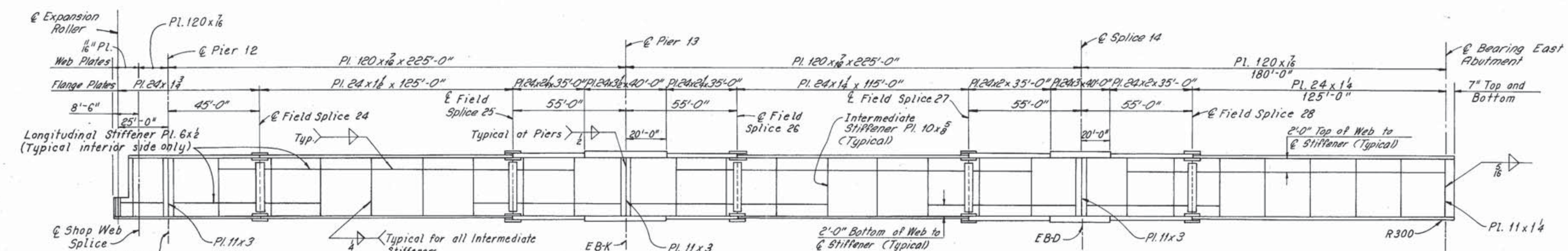


FRAMING PLAN - UNIT 5



GIRDERS B, C, F & G - UNIT 5

Intermediate Stiffeners shown only at crossframes and adjacent to bearing stiffeners at piers.



GIRDERS A, D, E & H - UNIT 5

Intermediate Stiffeners shown only at crossframes and adjacent to bearing stiffeners at piers.

WELD SIZE WEB TO FLANGE	
Flange Plate Thickness	Fillet Weld Size
1¼" and 1½"	5/16"
1¾" thru 2¼"	⅜"
2½" thru 3½"	½"

Notes:
For girder notes see Sheet 5/28
For girder web and flange plate transitions see Sheet 13/28
For girder shop weld details see Sheet 10/28
For girder and stringer designation for the right bridge see sheet 5/28

H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

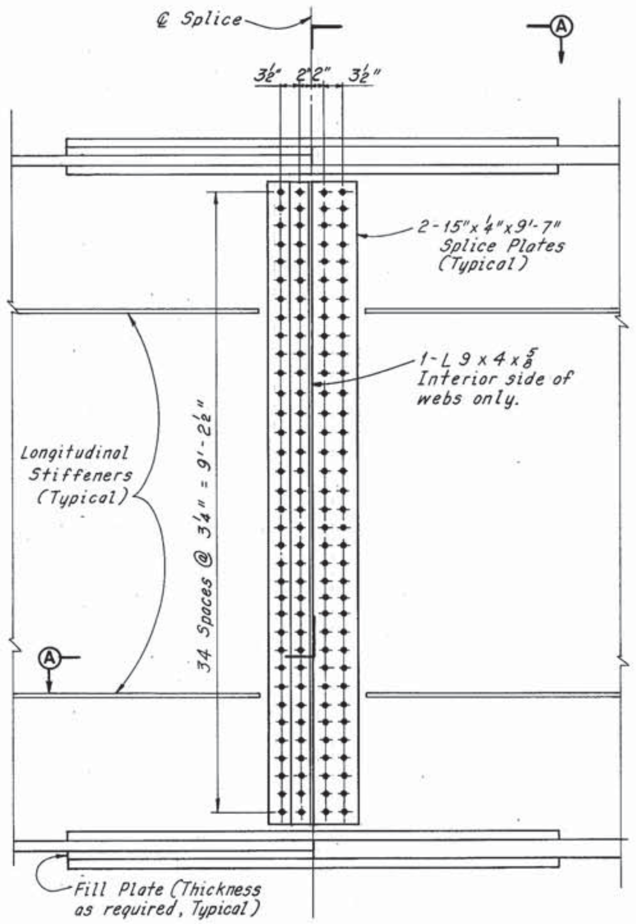
FRAMING PLAN AND GIRDER DETAILS UNIT 5

I-80 OVER CUYAHOGA RIVER VALLEY

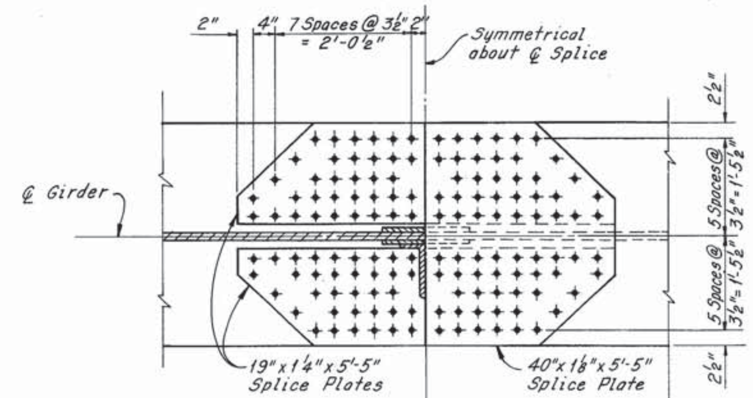
BR. NO. CUY-80-1843 STA. 996+22.25 TO STA. 1037+77.75

CUYAHOGA COUNTY OHIO

DRAWN JEH	TRACED AJJ	CHECKED TJS	REVIEWED WF	REVISED
DATE 6-13-68	DATE 6-21-68	DATE 11-20-68	DATE 8-21-70	SHEET 8/28

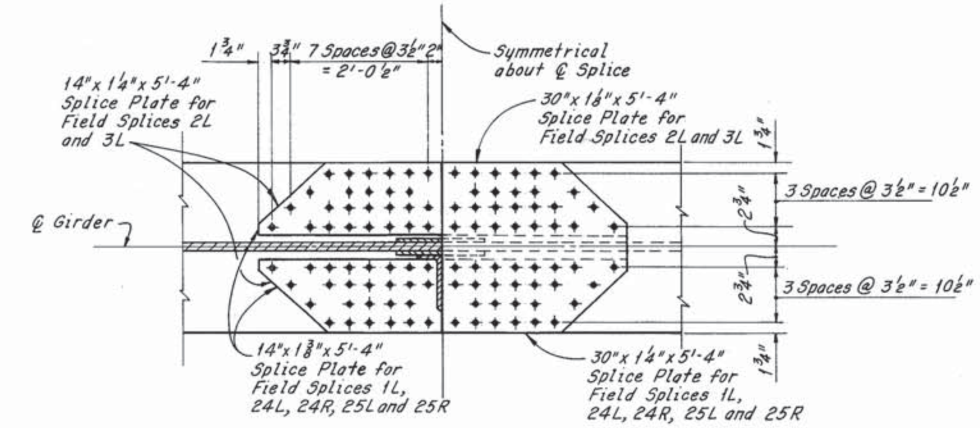


TYPICAL FIELD WEB SPLICE



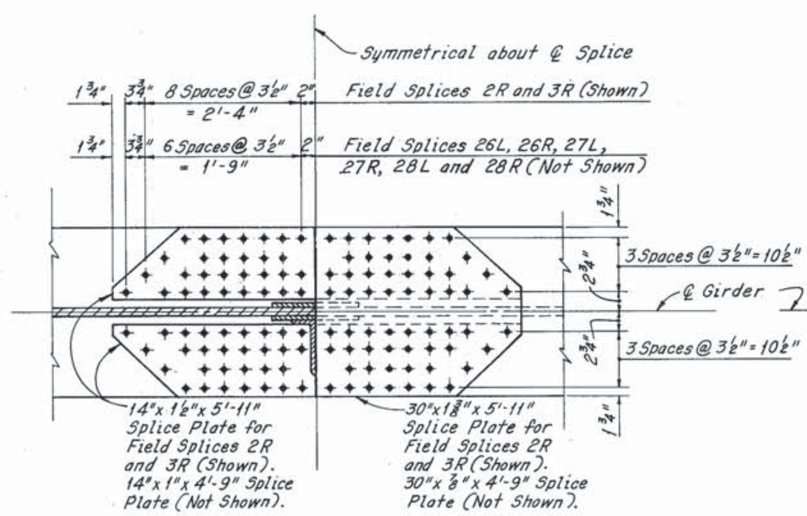
SECTION A-A

Shown for Splices
1R
4L and R
10L and R
16L and R
22L and R.



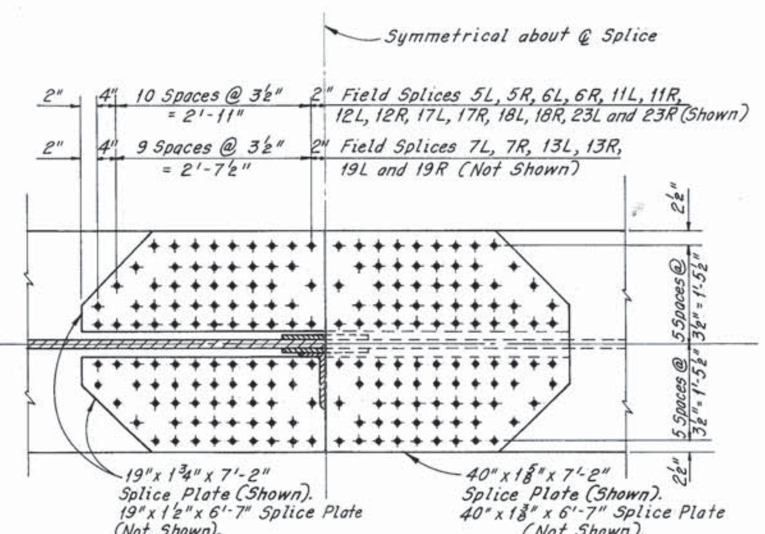
SECTION A-A

Shown for Splices
1L
2L
3L
24L and R
25L and R.



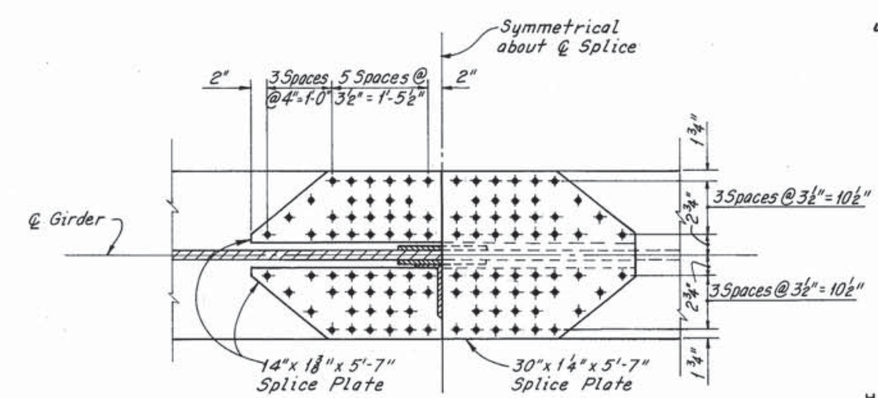
SECTION A-A

Shown for Splices 2R and 3R.
Not shown but similar, except as noted for Splices
26L and R
27L and R
28L and R.



SECTION A-A

Shown for Splices
5L and R 17L and R
6L and R 18L and R
11L and R 23L and R
12L and R.
Not shown but similar, except as noted for Splices
7L and R 19L and R
13L and R.



SECTION A-A

Shown for Splices
8L and R
9L and R
14L and R
15L and R
20L and R
21L and R.

The following abbreviations are used:
L = Left Bridge
R = Right Bridge

For Flange Transition Detail see sheet 13/28

H.N.T.B. BR. NO. 38L AND 38R

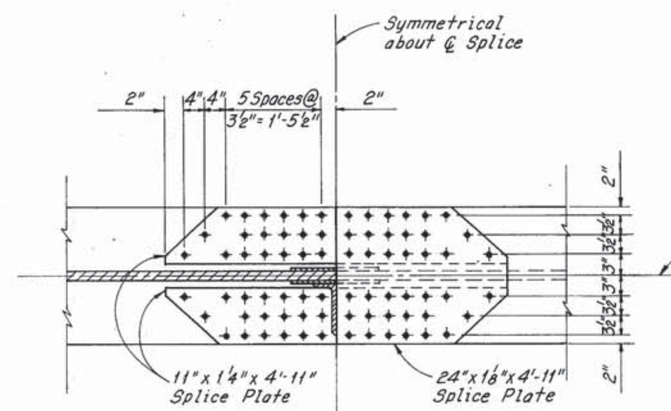
HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

FIELD SPLICE DETAILS
INTERIOR GIRDERS

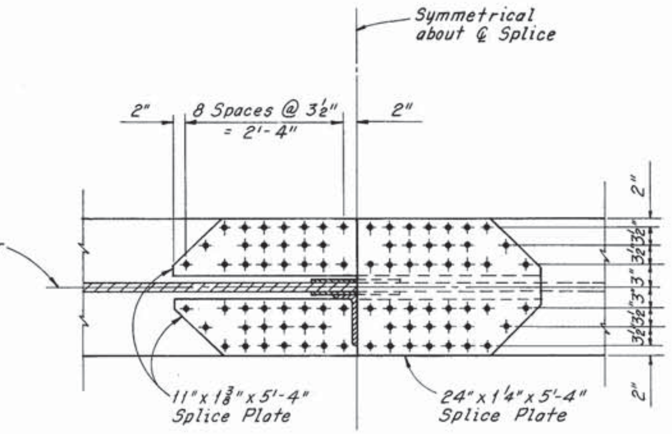
I-80 OVER CUYAHOGA RIVER VALLEY
BR. NO. CUY-80-1843
STA. 996+22.25 TO STA. 1037+77.75
CUYAHOGA COUNTY OHIO

DRAWN M.C.B. DATE 9-19-68	TRACED C.P. DATE 9-20-68	CHECKED T.S. DATE 11-14-68	REVIEWED W.F. DATE 9-21-70	REVISED 11-16-70 SHEET 9/28

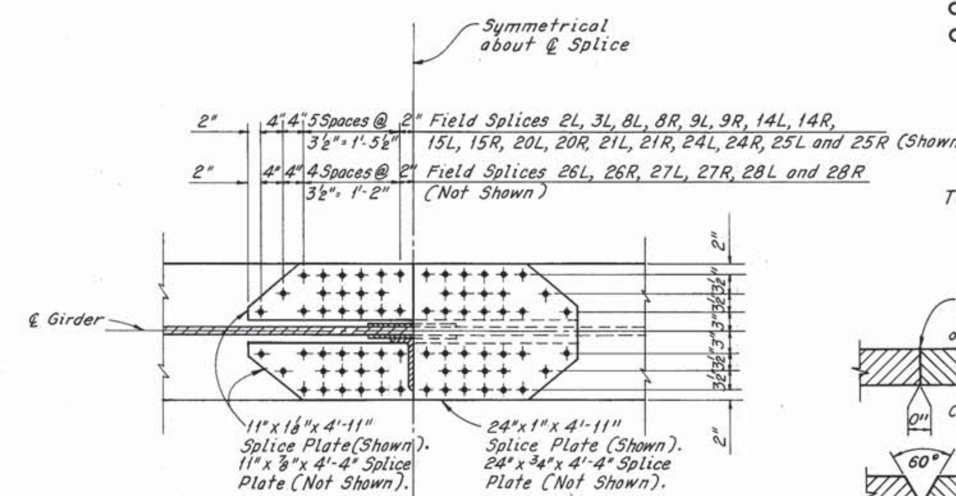
CUYAHOGA COUNTY
CUY-80-18.43 Part 2



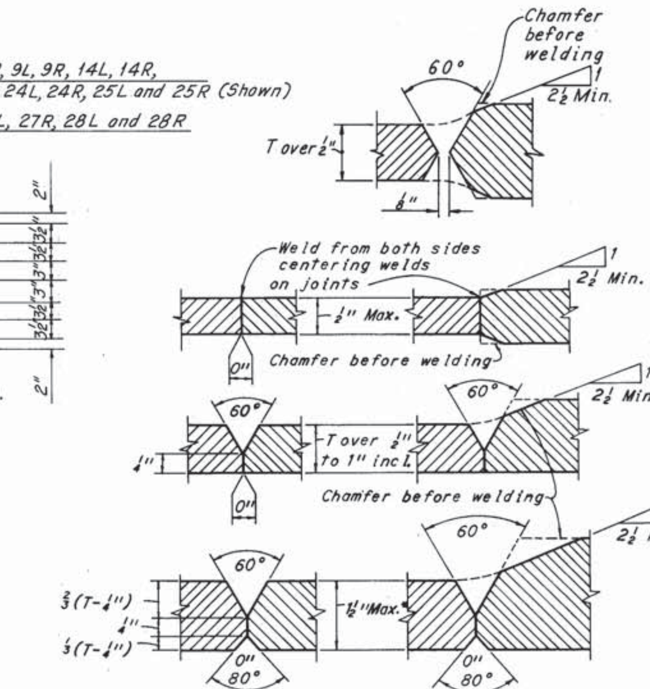
SECTION A-A
Shown for Splices
1L
2R
3R.



SECTION A-A
Shown for Splice 1R.

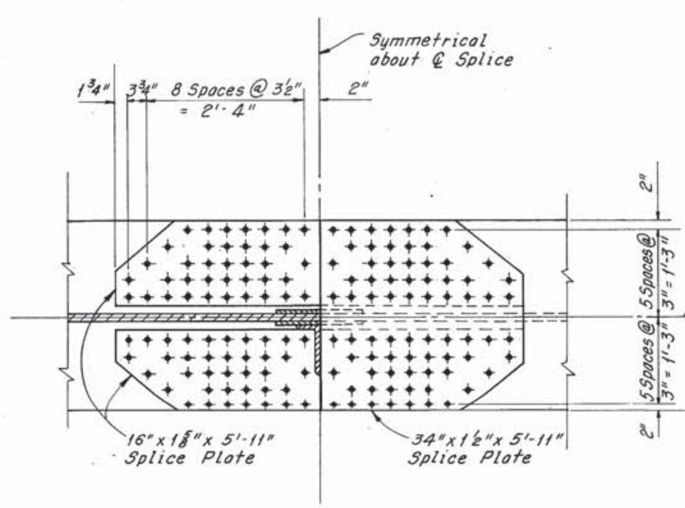


SECTION A-A
Shown for Splices
2L 15L and R
3L 20L and R
8L and R 21L and R
9L and R 24L and R
14L and R 25L and R.
Not shown but similar, except as noted for Splices
26L and R
27L and R
28L and R.

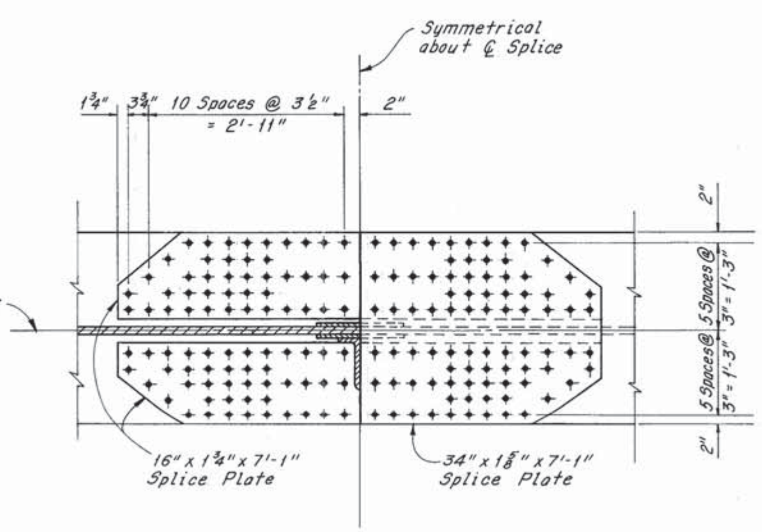


Note:
All of the above full penetration welds shall be back-gouged and welded after welding far side.
Butt welds on girder flange plates shall be ground flush, the finish grinding being parallel to the direction of stress.
*For plate thickness over 1½", use joint preparation conforming to B-U3a-S or B-U7-S, Figure 216, A.W.S. Specifications.

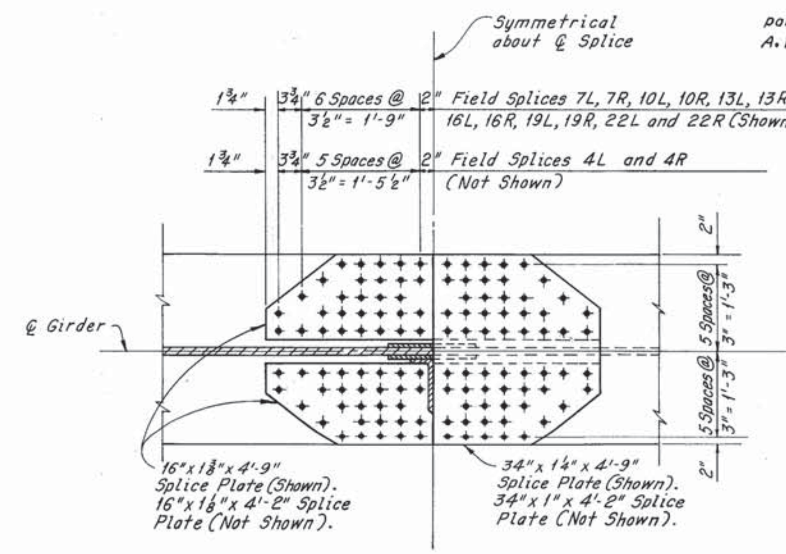
GIRDER SHOP WELDING DETAILS



SECTION A-A
Shown for Splices 5L and R.



SECTION A-A
Shown for Splices
6L and R 17L and R
11L and R 18L and R
12L and R 23L and R.



SECTION A-A
Shown for Splices 7L and R
10L and R
13L and R
16L and R
19L and R
22L and R.
Not shown but similar, except as noted for Splices 4L and R.

Notes:
For flange shop weld at Pier, see Sheet 13/28.
For typical field web splice and additional note see Sheet 9/28.

For Flange Transition Detail see sheet 13/28

FIELD SPLICE DETAILS - EXTERIOR GIRDERS

H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

GIRDER SPLICE DETAILS

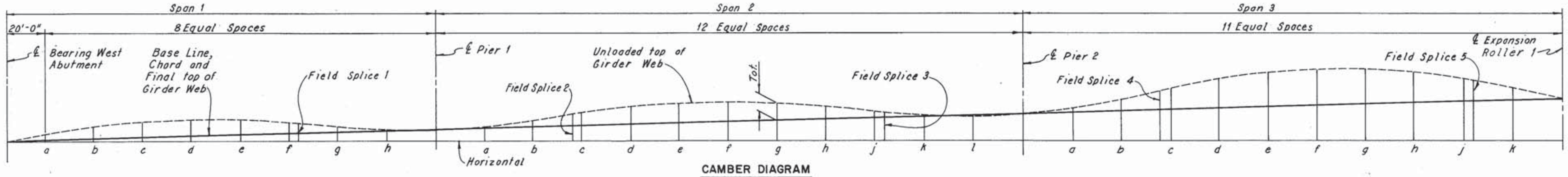
I-80 OVER CUYAHOGA RIVER VALLEY
BR. NO. CUY-80-1843 STA. 996+22.25 TO STA. 1037+77.75
CUYAHOGA COUNTY OHIO

DRAWN M.C.B. DATE 3-24-68	TRACED C.P. DATE 3-25-68	CHECKED T.J.S. DATE 11-14-68	REVIEWED W.F. DATE 8-21-70	REVISED 11-16-70 SHEET 10/28

FED. RD. DIVISION	STATE	PROJECT	
2	OHIO		

83
112

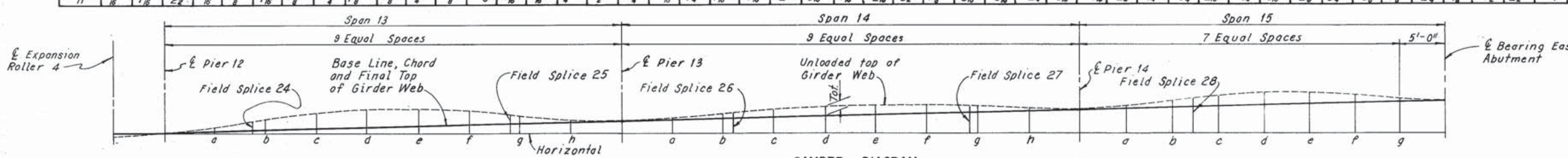
CUYAHOGA COUNTY
CUY -80-1843 Part 2



CAMBER DIAGRAM
Unit 1

DEAD LOAD DEFLECTION AND CAMBER - UNIT I																																																			
	Span 1																											Span 2																							
	a			b			c			d			e			f			Field Splice 1			g			h			a			b			Field Splice 2			c			d			e			f			g		
Girder	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.
A	7/16	7/8	1 5/16	13/16	1 7/8	2 11/16	1 1/16	2 3/8	3 7/16	1 1/8	2 3/8	3 1/2	15/16	2 1/16	3	5/8	1 7/16	2 1/16	9/16	1 5/16	1 7/8	5/16	3/4	1 1/16	1/16	1/4	5/16	1/8	3/16	5/16	3/8	11/16	1 1/16	5/8	1 5/16	1 15/16	11/16	1 7/16	2 1/8	1 1/16	2 1/8	3 3/16	1 1/4	2 9/16	3 13/16	1 1/4	2 5/8	3 7/8	1 1/16	2 1/4	3 5/16
B	7/16	7/8	1 5/16	7/8	1 11/16	2 9/16	1 1/16	2 1/4	3 5/16	1 1/8	2 5/16	3 7/16	1	2	3	11/16	1 3/8	2 1/16	5/8	1 1/4	1 7/8	5/16	11/16	1	1/8	3/16	5/16	1/8	3/16	5/16	3/8	11/16	1 1/16	5/8	1 1/4	1 7/8	3/4	1 3/8	2 1/8	1 1/16	2 1/16	3 1/8	1 1/4	2 1/2	3 3/4	1 3/16	2 7/16	3 3/4	1 1/8	2 1/16	3 3/16
C	7/16	7/8	1 5/16	7/8	1 11/16	2 9/16	1 1/16	2 1/4	3 5/16	1 1/8	2 5/16	3 7/16	1	2	3	11/16	1 3/8	2 1/16	5/8	1 1/4	1 7/8	5/16	11/16	1	1/8	3/16	5/16	1/8	3/16	5/16	3/8	11/16	1 1/16	5/8	1 1/4	1 7/8	3/4	1 3/8	2 1/8	1 1/16	2 1/16	3 1/8	1 1/4	2 1/2	3 3/4	1 3/16	2 7/16	3 3/4	1 1/8	2 1/16	3 3/16
D	7/16	7/8	1 5/16	13/16	1 7/8	2 11/16	1 1/16	2 3/8	3 7/16	1 1/8	2 3/8	3 1/2	15/16	2 1/16	3	5/8	1 7/16	2 1/16	9/16	1 5/16	1 7/8	5/16	3/4	1 1/16	1/16	1/4	5/16	1/8	3/16	5/16	3/8	11/16	1 1/16	5/8	1 5/16	1 15/16	11/16	1 7/16	2 1/8	1 1/16	2 1/8	3 3/16	1 1/4	2 9/16	3 13/16	1 1/4	2 5/8	3 7/8	1 1/16	2 1/4	3 5/16
E	7/16	15/16	1 3/8	7/8	1 15/16	2 13/16	1 1/8	2 1/2	3 5/8	1 1/8	2 5/8	3 3/4	1	2 1/4	3 1/4	11/16	1 9/16	2 1/4	5/8	1 7/16	2 1/16	3/8	13/16	1 3/16	1/8	1/4	3/8	1/8	3/16	5/16	3/8	3/4	1 1/8	5/8	1 3/8	2	3/4	1 1/2	2 1/4	1 1/16	2 1/4	3 5/16	1 3/16	2 11/16	4	1 3/8	2 13/16	4 1/8	1 1/8	2 7/16	3 9/16
F	7/16	1	1 7/16	7/8	1 15/16	2 13/16	1 1/8	2 9/16	3 11/16	1 3/16	2 5/8	3 13/16	1	2 5/16	3 5/16	11/16	1 5/8	2 5/16	5/8	1 7/16	2 1/16	3/8	13/16	1 3/16	1/8	1/4	3/8	1/16	3/16	1/4	5/16	11/16	1	5/8	1 1/4	1 7/8	11/16	1 3/8	2 1/16	1 1/16	2 1/16	3 1/8	1 1/4	2 1/2	3 3/4	1 5/16	2 9/16	3 7/8	1 1/8	2 1/4	3 3/8
G	7/16	1	1 7/16	7/8	1 15/16	2 13/16	1 1/8	2 9/16	3 11/16	1 3/16	2 5/8	3 13/16	1	2 5/16	3 5/16	11/16	1 5/8	2 5/16	5/8	1 7/16	2 1/16	3/8	13/16	1 3/16	1/8	1/4	3/8	1/16	3/16	1/4	5/16	11/16	1	5/8	1 1/4	1 7/8	11/16	1 3/8	2 1/16	1 1/16	2 1/16	3 1/8	1 1/4	2 1/2	3 3/4	1 5/16	2 9/16	3 7/8	1 1/8	2 1/4	3 3/8
H	7/16	15/16	1 3/8	7/8	1 15/16	2 13/16	1 1/8	2 1/2	3 5/8	1 1/8	2 5/8	3 3/4	1	2 1/4	3 1/4	11/16	1 9/16	2 1/4	5/8	1 7/16	2 1/16	3/8	13/16	1 3/16	1/8	1/4	3/8	1/8	3/16	5/16	3/8	3/4	1 1/8	5/8	1 3/8	2	3/4	1 1/2	2 1/4	1 1/16	2 1/4	3 5/16	1 3/16	2 11/16	4	1 5/16	2 13/16	4 1/8	1 1/8	2 7/16	3 9/16

DEAD LOAD DEFLECTION AND CAMBER - UNIT I																																																		
	Span 2															Span 3																																			
	h			j			Field Splice 3			k			l			a			b			Field Splice 4			c			d			e			f			g			h			j			Field Splice 5			k		
Girder	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.
A	3/4	1 1/2	2 1/4	3/8	11/16	1 1/16	5/16	9/16	7/8	1/16	1/8	3/16	-1/16	-1/8	-3/16	5/16	1/2	13/16	3/4	1 5/16	2 1/16	1 1/4	2 1/8	3 3/8	1 3/8	2 3/8	3 3/4	1 15/16	3 7/16	5 3/8	2 3/8	4 1/8	6 1/2	2 9/16	4 1/2	7 1/16	2 9/16	4 7/16	7	2 1/4	3 7/8	6 1/8	1 11/16	2 7/8	4 9/16	1 1/2	2 11/16	4 3/16	7/8	1 9/16	2 7/16
B	3/4	1 1/2	2 1/4	3/8	11/16	1 1/16	5/16	9/16	7/8	1/16	3/16	1/4	-1/16	-1/16	-1/8	5/16	7/16	3/4	3/4	1 1/4	2	1 1/4	2 1/16	3 5/16	1 3/8	2 1/4	3 5/8	1 15/16	3 1/4	5 3/16	2 3/8	3 15/16	6 5/16	2 9/16	4 5/16	6 7/8	2 9/16	4 3/16	6 3/4	2 1/4	3 11/16	5 15/16	1 11/16	2 3/4	4 7/16	1 1/2	2 1/2	4	7/8	1 1/2	2 3/8
C	3/4	1 1/2	2 1/4	3/8	11/16	1 1/16	5/16	9/16	7/8	1/16	3/16	1/4	-1/16	-1/16	-1/8	5/16	7/16	3/4	3/4	1 1/4	2	1 1/4	2 1/16	3 5/16	1 3/8	2 1/4	3 5/8	1 15/16	3 1/4	5 3/16	2 3/8	3 15/16	6 5/16	2 9/16	4 5/16	6 7/8	2 9/16	4 3/16	6 3/4	2 1/4	3 11/16	5 15/16	1 11/16	2 3/4	4 7/16	1 1/2	2 1/2	4	7/8	1 1/2	2 3/8
D	3/4	1 1/2	2 1/4	3/8	11/16	1 1/16	5/16	9/16	7/8	1/16	1/8	3/16	-1/16	-1/8	-3/16	5/16	1/2	13/16	3/4	1 5/16	2 1/16	1 1/4	2 1/8	3 3/8	1 3/8	2 3/8	3 3/4	1 15/16	3 7/16	5 3/8	2 3/8	4 1/8	6 1/2	2 9/16	4 1/2	7 1/16	2 9/16	4 7/16	7	2 1/4	3 7/8	6 1/8	1 11/16	2 7/8	4 9/16	1 1/2	2 11/16	4 3/16	7/8	1 9/16	2 7/16
E	13/16	1 11/16	2 1/2	7/16	7/8	1 5/16	3/8	3/4	1 1/8	1/8	1/4	3/8	0	-1/16	-1/16	1/4	1/2	3/4	11/16	1 1/4	1 15/16	1 3/16	2	3 3/16	1 5/16	2 3/16	3 1/2	1 7/8	3 3/16	5 1/16	2 1/4	3 15/16	6 3/16	2 1/2	4 1/4	6 3/4	2 7/16	4 1/4	6 11/16	2 1/8	3 3/4	5 7/8	1 5/8	2 3/4	4 3/8	1 1/2	2 1/2	4	7/8	1 1/2	2 3/8
F	13/16	1 9/16	2 3/8	7/16	13/16	1 1/4	3/8	11/16	1 1/16	1/8	1/4	3/8	0	-1/16	-1/16	1/4	1/2	3/4	3/4	1 1/4	2	1 3/16	2 1/16	3 1/4	1 5/16	2 5/16	3 5/8	1 15/16	3 5/16	5 1/4	2 3/8	4 1/16	6 7/16	2 5/8	4 7/16	7 1/16	2 9/16	4 3/8	6 15/16	2 1/4	3 7/8	6 1/8	1 11/16	2 7/8	4 9/16	1 9/16	2 5/8	4 3/16	15/16	1 9/16	2 1/2
G	13/16	1 9/16	2 3/8	7/16	13/16	1 1/4	3/8	11/16	1 1/16	1/8	1/4	3/8	0	-1/16	-1/16	1/4	1/2	3/4	3/4	1 1/4	2	1 3/16	2 1/16	3 1/4	1 5/16	2 5/16	3 5/8	1 15/16	3 5/16	5 1/4	2 3/8	4 1/16	6 7/16	2 5/8	4 7/16	7 1/16	2 9/16	4 3/8	6 15/16	2 1/4	3 7/8	6 1/8	1 11/16	2 7/8	4 9/16	1 9/16	2 5/8	4 3/16	15/16	1 9/16	2 1/2
H	13/16	1 11/16	2 1/2	7/16	7/8	1 5/16	3/8	3/4	1 1/8	1/8	1/4	3/8	0	-1/16	-1/16	1/4	1/2	3/4	11/16	1 1/4	1 15/16	1 3/16	2	3 3/16	1 5/16	2 3/16	3 1/2	1 7/8	3 3/16	5 1/16	2 1/4	3 15/16	6 3/16	2 1/2	4 1/4	6 3/4	2 7/16	4 1/4	6 11/16	2 1/8	3 3/4	5 7/8	1 5/8	2 3/4	4 3/8	1 1/2	2 1/2	4	7/8	1 1/2	2 3/8



CAMBER DIAGRAM

DEAD LOAD DEFLECTION AND CAMBER - UNIT 5																																													
	℄ Expansion Roller 4			Span 13																														Span 14											
				a			Field Splice 24			b			c			d			e			f			Field Splice 25			g			h			a			b			Field Splice 26			c		
Girder	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.
A	-1/16	-1/4	-5/16	1/4	11/16	15/16	9/16	1 5/16	1 7/8	5/8	1 7/16	2 1/16	7/8	2 3/16	3 1/16	1	2 1/2	3 1/2	1	2 3/8	3 3/8	3/4	1 7/8	2 5/8	9/16	1 1/4	1 13/16	1/2	1 1/8	1 5/8	3/16	7/16	5/8	0	1/16	1/16	3/16	7/16	5/8	3/16	9/16	3/4	3/8	7/8	1 1/4
B	-1/16	-1/8	-3/16	1/4	9/16	13/16	1/2	1 1/8	1 5/8	9/16	1 1/4	1 13/16	13/16	1 7/8	2 11/16	15/16	2 1/8	3 1/16	15/16	2	2 15/16	3/4	1 9/16	2 5/16	1/2	1 1/16	1 9/16	7/16	15/16	1 3/8	1/8	3/8	1/2	1/16	1/16	1/8	3/16	7/16	5/8	1/4	1/2	3/4	3/8	7/8	1 1/4
C	-1/16	-1/8	-3/16	1/4	9/16	13/16	1/2	1 1/8	1 5/8	9/16	1 1/4	1 13/16	13/16	1 7/8	2 11/16	15/16	2 1/8	3 1/16	15/16	2	2 15/16	3/4	1 9/16	2 5/16	1/2	1 1/16	1 9/16	7/16	15/16	1 3/8	1/8	3/8	1/2	1/16	1/16	1/8	3/16	7/16	5/8	1/4	1/2	3/4	3/8	7/8	1 1/4
D	-1/16	-1/4	-5/16	1/4	11/16	15/16	9/16	1 5/16	1 7/8	5/8	1 7/16	2 1/16	7/8	2 3/16	3 1/16	1	2 1/2	3 1/2	1	2 3/8	3 3/8	3/4	1 7/8	2 5/8	9/16	1 1/4	1 13/16	1/2	1 1/8	1 5/8	3/16	7/16	5/8	0	1/16	1/16	3/16	7/16	5/8	3/16	9/16	3/4	3/8	7/8	1 1/4
E	-1/16	-1/4	-5/16	1/4	11/16	15/16	9/16	1 5/16	1 7/8	5/8	1 7/16	2 1/16	7/8	2 3/16	3 1/16	1	2 1/2	3 1/2	1	2 3/8	3 3/8	3/4	1 7/8	2 5/8	9/16	1 1/4	1 13/16	1/2	1 1/8	1 5/8	3/16	7/16	5/8	0	1/16	1/16	3/16	7/16	5/8	3/16	9/16	3/4	3/8	7/8	1 1/4
F	-1/16	-1/8	-3/16	1/4	9/16	13/16	1/2	1 1/8	1 5/8	9/16	1 1/4	1 13/16	13/16	1 7/8	2 11/16	15/16	2 1/8	3 1/16	15/16	2	2 15/16	3/4	1 9/16	2 5/16	1/2	1 1/16	1 9/16	7/16	15/16	1 3/8	1/8	3/8	1/2	1/16	1/16	1/8	3/16	7/16	5/8	1/4	1/2	3/4	3/8	7/8	1 1/4
G	-1/16	-1/8	-3/16	1/4	9/16	13/16	1/2	1 1/8	1 5/8	9/16	1 1/4	1 13/16	13/16	1 7/8	2 11/16	15/16	2 1/8	3 1/16	15/16	2	2 15/16	3/4	1 9/16	2 5/16	1/2	1 1/16	1 9/16	7/16	15/16	1 3/8	1/8	3/8	1/2	1/16	1/16	1/8	3/16	7/16	5/8	1/4	1/2	3/4	3/8	7/8	1 1/4
H	-1/16	-1/4	-5/16	1/4	11/16	15/16	9/16	1 5/16	1 7/8	5/8	1 7/16	2 1/16	7/8	2 3/16	3 1/16	1	2 1/2	3 1/2	1	2 3/8	3 3/8	3/4	1 7/8	2 5/8	9/16	1 1/4	1 13/16	1/2	1 1/8	1 5/8	3/16	7/16	5/8	0	1/16	1/16	3/16	7/16	5/8	3/16	9/16	3/4	3/8	7/8	1 1/4

DEAD LOAD DEFLECTION AND CAMBER - UNIT 5																																										
	Span 14																		Span 15																							
	d			e			f			Field Splice 27			g			h			a			b			Field Splice 28			c			d			e			f			g		
Girder	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.
A	1/2	1 3/16	1 11/16	1/2	1 3/16	1 11/16	3/8	15/16	1 5/16	3/16	9/16	3/4	3/16	1/2	11/16	1/16	1/16	1/8	1/8	1/4	3/8	5/16	13/16	1 1/8	3/8	15/16	1 5/16	9/16	1 3/8	1 15/16	11/16	1 11/16	2 3/8	5/8	1 5/8	2 1/4	7/16	1 1/16	1 1/2	1/16	3/16	1/4
B	1/2	1 3/16	1 11/16	9/16	1 3/16	1 3/4	7/16	15/16	1 3/8	1/4	9/16	13/16	3/16	1/2	11/16	1/16	1/16	1/8	1/8	1/4	3/8	5/16	13/16	1 1/8	3/8	15/16	1 5/16	9/16	1 3/8	1 15/16	11/16	1 5/8	2 5/16	5/8	1 9/16	2 3/16	7/16	1	1 7/16	1/16	3/16	1/4
C	1/2	1 3/16	1 11/16	9/16	1 3/16	1 3/4	7/16	15/16	1 3/8	1/4	9/16	13/16	3/16	1/2	11/16	1/16	1/16	1/8	1/8	1/4	3/8	5/16	13/16	1 1/8	3/8	15/16	1 5/16	9/16	1 3/8	1 15/16	11/16	1 5/8	2 5/16	5/8	1 9/16	2 3/16	7/16	1	1 7/16	1/16	3/16	1/4
D	1/2	1 3/16	1 11/16	1/2	1 3/16	1 11/16	3/8	15/16	1 5/16	3/16	9/16	3/4	3/16	1/2	11/16	1/16	1/16	1/8	1/8	1/4	3/8	5/16	13/16	1 1/8	3/8	15/16	1 5/16	9/16	1 3/8	1 15/16	11/16	1 11/16	2 3/8	5/8	1 5/8	2 1/4	7/16	1 1/16	1 1/2	1/16	3/16	1/4
E	1/2	1 3/16	1 11/16	1/2	1 3/16	1 11/16	3/8	15/16	1 5/16	3/16	9/16	3/4	3/16	1/2	11/16	1/16	1/16	1/8	1/8	1/4	3/8	5/16	13/16	1 1/8	3/8	15/16	1 5/16	9/16	1 3/8	1 15/16	11/16	1 11/16	2 3/8	5/8	1 5/8	2 1/4	7/16	1 1/16	1 1/2	1/16	3/16	1/4
F	1/2	1 3/16	1 11/16	9/16	1 3/16	1 3/4	7/16	15/16	1 3/8	1/4	9/16	13/16	3/16	1/2	11/16	1/16	1/16	1/8	1/8	1/4	3/8	5/16	13/16	1 1/8	3/8	15/16	1 5/16	9/16	1 3/8	1 15/16	11/16	1 5/8	2 5/16	5/8	1 9/16	2 3/16	7/16	1	1 7/16	1/16	3/16	1/4
G	1/2	1 3/16	1 11/16	9/16	1 3/16	1 3/4	7/16	15/16	1 3/8	1/4	9/16	13/16	3/16	1/2	11/16	1/16	1/16	1/8	1/8	1/4	3/8	5/16	13/16	1 1/8	3/8	15/16	1 5/16	9/16	1 3/8	1 15/16	11/16	1 5/8	2 5/16	5/8	1 9/16	2 3/16	7/16	1	1 7/16	1/16	3/16	1/4
H	1/2	1 3/16	1 11/16	1/2	1 3/16	1 11/16	3/8	15/16	1 5/16	3/16	9/16	3/4	3/16	1/2	11/16	1/16	1/16	1/8	1/8	1/4	3/8	5/16	13/16	1 1/8	3/8	15/16	1 5/16	9/16	1 3/8	1 15/16	11/16	1 11/16	2 3/8	5/8	1 5/8	2 1/4	7/16	1 1/16	1 1/2	1/16	3/16	1/4

Notes:
Negative values of total required camber indicate values below the chord.
Negative values for deflection indicate deflection above the chord.
Letter designations given at deflections, are spaced at 25'-0" increments except as noted.
Deflections and convexities or concavities are given to the nearest 1/16 inch.
The following abbreviations are used:
Stl. = Dead load deflection due to the weight of steel.
Rem.D.L. = Remaining dead load deflection.
Tot. = Total required camber.

H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

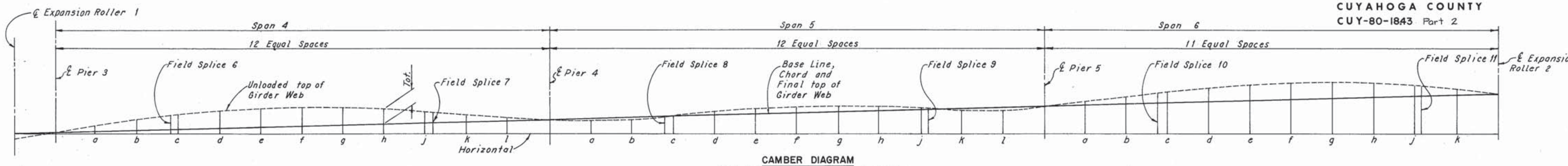
DEAD LOAD DEFLECTION AND CAMBER DIAGRAMS UNITS 1 AND 5
I-80 OVER CUYAHOGA RIVER VALLEY
BR. NO. CUY-80-1843 STA. 996+22.25 TO STA. 1037+77.75
CUYAHOGA COUNTY OHIO

DRAWN JEH	TRACED GEM	CHECKED TJS	REVIEWED WA	REVISED	
DATE 11-12-68	DATE 11-15-68	DATE 12-1968	DATE 8-21-70		SHEET 11 / 28

FED. RD. DIVISION	STATE	PROJECT	
2	OHIO		

84/112

CUYAHOGA COUNTY
CUY-80-1843 Part 2



CAMBER DIAGRAM
Unit 2 Shown, Units 3 and 4 Similar

DEAD LOAD DEFLECTION AND CAMBER - UNIT 2

Girder	℄ Expansion Roller 1			Span 4: a			b			Field Splice 6			c			d			e		
	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.
A	-13/16	1 5/16	-2 1/8	7/8	1 7/16	2 5/16	1 11/16	2 13/16	4 1/2	2 5/16	3 3/4	6 1/16	2 7/16	3 15/16	6 3/8	2 15/16	4 3/4	7 11/16	3 3/16	5 3/16	8 3/8
B	-3/4	1 1/4	-2	7/8	1 3/8	2 1/4	1 11/16	2 3/4	4 7/16	2 3/8	3 11/16	6	[illegible]	3 7/8	6 3/8	2 15/16	4 5/8	7 9/16	3 1/4	5 1/16	8 3/16
C	-3/4	1 1/4	-2	7/8	1 3/8	2 1/4	1 11/16	2 3/4	4 7/16	2 5/16	3 11/16	6	2 7/16	3 7/8	6 5/16	2 15/16	4 5/8	7 9/16	3 1/4	5 1/16	8 3/16
D	-13/16	1 5/16	-2 1/8	7/8	1 7/16	2 5/16	1 11/16	2 13/16	4 1/2	2 5/16	3 3/4	6 1/16	2 7/16	3 15/16	6 3/8	2 15/16	4 3/4	7 11/16	3 3/16	5 3/16	8 3/8
E	-3/4	1 1/4	-2	7/8	1 3/8	2 1/4	1 11/16	2 3/4	4 7/16	2 1/4	3 5/8	5 7/8	2 3/8	3 7/8	6 1/4	2 7/8	4 11/16	7 9/16	3 1/8	5 1/8	8 1/4
F	-3/4	1 3/16	-1 15/16	13/16	1 3/8	2 3/16	1 11/16	2 5/8	4 5/16	2 1/4	3 9/16	5 13/16	2 3/8	3 13/16	6 3/16	2 7/8	4 9/16	7 7/16	3 1/8	4 15/16	8 1/16
G	-3/4	1 3/16	-1 15/16	13/16	1 3/8	2 3/16	1 11/16	2 5/8	4 5/16	2 1/4	3 9/16	5 13/16	2 3/8	3 13/16	6 3/16	2 7/8	4 9/16	7 7/16	3 1/8	4 15/16	8 1/16
H	-3/4	1 1/4	-2	7/8	1 3/8	2 1/4	1 11/16	2 3/4	4 7/16	2 1/4	3 5/8	5 7/8	2 3/8	3 7/8	6 1/4	2 7/8	4 11/16	7 9/16	3 1/8	5 1/8	8 1/4

Girder	Span 4: f			g			h			j			Field Splice 7			k			l		
	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.
A	3 3/16	5 3/16	8 3/8	2 15/16	4 3/4	7 11/16	2 7/16	3 15/16	6 3/8	1 3/4	2 7/8	4 5/8	1 5/8	2 5/8	4 1/4	1 1/16	1 11/16	2 3/4	7/16	3/4	1 3/16
B	3 1/8	5	8 1/8	2 7/8	4 9/16	7 7/16	2 3/8	3 13/16	6 3/16	1 11/16	2 3/4	4 7/16	1 9/16	2 1/2	4 1/16	1	1 5/8	2 5/8	7/16	5/8	1 1/16
C	3 1/8	5	8 1/8	2 7/8	4 9/16	7 7/16	2 3/8	3 13/16	6 3/16	1 11/16	2 3/4	4 7/16	1 9/16	2 1/2	4 1/16	1	1 5/8	2 5/8	7/16	5/8	1 1/16
D	3 3/16	5 3/16	8 3/8	2 15/16	4 3/4	7 11/16	2 7/16	3 15/16	6 3/8	1 3/4	2 7/8	4 5/8	1 5/8	2 5/8	4 1/4	1 1/16	1 11/16	2 3/4	7/16	3/4	1 3/16
E	3 1/8	5 1/8	8 1/4	2 7/8	4 11/16	7 9/16	2 3/8	3 15/16	6 5/16	1 3/4	2 13/16	4 9/16	1 9/16	2 5/8	4 3/16	1	1 3/4	2 3/4	7/16	11/16	1 1/8
F	3 1/8	4 15/16	8 1/16	2 13/16	4 9/16	7 3/8	2 5/16	3 3/4	6 1/16	1 11/16	2 11/16	4 3/8	1 9/16	2 1/2	4 1/16	1	1 5/8	2 5/8	3/8	11/16	1 1/16
G	3 1/8	4 15/16	8 1/16	2 13/16	4 9/16	7 3/8	2 3/8	3 3/4	6 1/16	1 11/16	2 11/16	4 3/8	1 9/16	2 1/2	4 1/16	1	1 5/8	2 5/8	3/8	11/16	1 1/16
H	3 1/8	5 1/8	8 1/4	2 7/8	4 11/16	7 9/16	2 3/8	3 15/16	6 5/16	1 3/4	2 13/16	4 9/16	1 9/16	2 5/8	4 3/16	1	1 3/4	2 3/4	7/16	11/16	1 1/8

Girder	Span 5: a			b			Field Splice 8			c			d			e		
	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.
A	-3/16	-3/8	-9/16	-3/16	-7/16	-5/8	-1/8	-3/16	-5/16	-1/16	-1/8	-3/16	3/16	5/16	1/2	3/8	13/16	1 3/16
B	-3/16	-5/16	-1/2	-1/8	-5/16	-7/16	-1/16	-1/16	-1/8	0	0	0	1/4	1/2	3/4	1/2	15/16	1 7/16
C	-3/16	-5/16	-1/2	-1/8	-5/16	-7/16	-1/16	-1/16	-1/8	0	0	0	1/4	1/2	3/4	1/2	15/16	1 7/16
D	-3/16	-3/8	-9/16	-3/16	-7/16	-5/8	-1/8	-3/16	-5/16	-1/16	-1/8	-3/16	3/16	5/16	1/2	3/8	13/16	1 3/16
E	-3/16	-3/8	-9/16	-3/16	-7/16	-5/8	-1/8	-3/16	-5/16	-1/16	-1/8	-3/16	3/16	5/16	1/2	3/8	13/16	1 3/16
F	-3/16	-5/16	-1/2	-1/8	-5/16	-7/16	-1/16	-1/16	-1/8	0	0	0	1/4	1/2	3/4	1/2	15/16	1 7/16
G	-3/16	-5/16	-1/2	-1/8	-5/16	-7/16	-1/16	-1/16	-1/8	0	0	0	1/4	1/2	3/4	1/2	15/16	1 7/16
H	-3/16	-3/8	-9/16	-3/16	-7/16	-5/8	-1/8	-3/16	-5/16	-1/16	-1/8	-3/16	3/16	5/16	1/2	3/8	13/16	1 3/16

DEAD LOAD DEFLECTION AND CAMBER - UNIT 2

Girder	Span 5: f			g			h			j			Field Splice 9			k			l		
	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.
A	1/2	15/16	1 7/16	7/16	13/16	1 1/4	1/4	7/16	11/16	0	0	0	-1/16	-1/16	-1/8	-3/16	-5/16	-1/2	-3/16	-5/16	-1/2
B	5/8	1 1/8	1 3/4	1/2	1	1 1/2	5/16	5/8	15/16	1/16	1/8	3/16	0	0	0	-1/8	-1/4	-3/8	-1/8	-5/16	-7/16
C	5/8	1 1/8	1 3/4	1/2	1	1 1/2	5/16	5/8	15/16	1/16	1/8	3/16	0	0	0	-1/8	-1/4	-3/8	-1/8	-5/16	-7/16
D	1/2	15/16	1 7/16	7/16	13/16	1 1/4	1/4	7/16	11/16	0	0	0	-1/16	-1/16	-1/8	-3/16	-5/16	-1/2	-3/16	-5/16	-1/2
E	1/2	15/16	1 7/16	7/16	13/16	1 1/4	1/4	7/16	11/16	0	0	0	-1/16	-1/16	-1/8	-3/16	-5/16	-1/2	-3/16	-5/16	-1/2
F	5/8	1 1/8	1 3/4	1/2	1	1 1/2	5/16	5/8	15/16	1/16	1/8	3/16	0	0	0	-1/8	-1/4	-3/8	-1/8	-5/16	-7/16
G	5/8	1 1/8	1 3/4	1/2	1	1 1/2	5/16	5/8	15/16	1/16	1/8	3/16	0	0	0	-1/8	-1/4	-3/8	-1/8	-5/16	-7/16
H	1/2	15/16	1 7/16	7/16	13/16	1 1/4	1/4	7/16	11/16	0	0	0	-1/16	-1/16	-1/8	-3/16	-5/16	-1/2	-3/16	-5/16	-1/2

Girder	Span 6: a			b			Field Splice 10			c			d			e		
	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.
A	7/16	11/16	1 1/8	1	1 5/8	2 5/8	1 9/16	2 1/2	4 1/16	1 11/16	2 3/4	4 7/16	2 1/4	3 3/4	6	2 11/16	4 3/8	7 1/16
B	7/16	5/8	1 1/16	1	1 5/8	2 5/8	1 9/16	2 7/16	4	1 11/16	2 11/16	4 3/8	2 1/4	3 11/16	5 15/16	2 11/16	4 3/8	7 1/16
C	7/16	5/8	1 1/16	1	1 5/8	2 5/8	1 9/16	2 7/16	4	1 11/16	2 11/16	4 3/8	2 1/4	3 11/16	5 15/16	2 11/16	4 3/8	7 1/16
D	7/16	11/16	1 1/8	1	1 5/8	2 5/8	1 9/16	2 1/2	4 1/16	1 11/16	2 3/4	4 7/16	2 1/4	3 3/4	6	2 11/16	4 3/8	7 1/16
E	7/16	11/16	1 1/8	1	1 5/8	2 5/8	1 9/16	2 1/2	4 1/16	1 11/16	2 3/4	4 7/16	2 1/4	3 3/4	6	2 11/16	4 3/8	7 1/16
F	7/16	5/8	1 1/16	1	1 5/8	2 5/8	1 9/16	2 7/16	4	1 11/16	2 11/16	4 3/8	2 1/4	3 11/16	5 15/16	2 11/16	4 3/8	7 1/16
G	7/16	5/8	1 1/16	1	1 5/8	2 5/8	1 9/16	2 7/16	4	1 11/16	2 11/16	4 3/8	2 1/4	3 11/16	5 15/16	2 11/16	4 3/8	7 1/16
H	7/16	11/16	1 1/8	1	1 5/8	2 5/8	1 9/16	2 1/2	4 1/16	1 11/16	2 3/4	4 7/16	2 1/4	3 3/4	6	2 11/16	4 3/8	7 1/16

Girder	Span 6: f			g			h			j			Field Splice 11			k		
	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.
A	2 7/8	4 5/8	7 1/2	2 3/4	4 1/2	7 1/4	2 3/8	3 7/8	6 1/4	1 3/4	2 7/8	4 5/8	1 5/8	2 5/8	4 1/4	15/16	1 9/16	2 1/2
B	2 7/8	4 5/8	7 1/2	2 13/16	4 1/2	7 5/16	2 7/16	3 15/16	6 3/8	1 13/16	2 15/16	4 3/4	1 11/16	2 11/16	4 3/8	1	1 9/16	2 9/16
C	2 7/8	4 5/8	7 1/2	2 13/16	4 1/2	7 5/16	2 7/16	3 15/16	6 3/8	1 13/16	2 15/16	4 3/4	1 11/16	2 11/16	4 3/8	1	1 9/16	2 9/16
D	2 7/8	4 5/8	7 1/2	2 3/4	4 1/2	7 1/4	2 3/8	3 7/8	6 1/4	1 3/4	2 7/8	4 5/8	1 5/8	2 5/8	4 1/4	15/16	1 9/16	2 1/2
E	2 7/8	4 5/8	7 1/2	2 3/4	4 1/2	7 1/4	2 3/8	3 7/8	6 1/4	1 3/4	2 7/8	4 5/8	1 5/8	2 5/8	4 1/4	15/16	1 9/16	2 1/2
F	2 7/8	4 5/8	7 1/2	2 13/16	4 1/2	7 5/16	2 7/16	3 15/16	6 3/8	1 13/16	2 15/16	4 3/4	1 11/16	2 11/16	4 3/8	1	1 9/16	2 9/16
G	2 7/8	4 5/8	7 1/2	2 13/16	4 1/2	7 5/16	2 7/16	3 15/16	6 3/8	1 13/16	2 15/16	4 3/4	1 11/16	2 11/16	4 3/8	1	1 9/16	2 9/16
H	2 7/8	4 5/8	7 1/2	2 3/4	4 1/2	7 1/4	2 3/8	3 7/8	6 1/4	1 3/4	2 7/8	4 5/8	1 5/8	2 5/8	4 1/4	15/16	1 9/16	2 1/2

DEAD LOAD DEFLECTION AND CAMBER - UNIT 3

Girder	℄ Expansion Roller 2			Span 7: a			b			Field Splice 12			c			d			e		
	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.
A	-3/4	-1 1/4	-2	7/8	1 3/8	2 1/4	1 11/16	2 3/4	4 7/16	2 1/4	3 5/8	5 7/8	2 3/8	3 7/8	6 1/4	2 7/8	4 11/16	7 9/16	3 1/8	5 1/8	8 1/4
B	-3/4	-1 3/16	-1 15/16	13/16	1 3/8	2 3/16	1 11/16	2 5/8	4 5/16	2 1/4	3 9/16	5 13/16	2 3/8	3 13/16	6 3/16	2 7/8	4 9/16	7 7/16	3 1/8	4 15/16	8 1/16
C	-3/4	-1 3/16	-1 15/16	13/16	1 3/8	2 3/16	1 11/16	2 5/8	4 5/16	2 1/4	3 9/16	5 13/16	2 3/8	3 13/16	6 3/16	2 7/8	4 9/16	7 7/16	3 1/8	4 15/16	8 1/16
D	-3/4	-1 1/4	-2	7/8	1 3/8	2 1/4	1 11/16	2 3/4	4 7/16	2 1/4	3 5/8	5 7/8	2 3/8	3 7/8	6 1/4	2 7/8	4 11/16	7 9/16	3 1/8	5 1/8	8 1/4
E	-3/4	-1 1/4	-2	7/8	1 3/8	2 1/4	1 11/16	2 3/4	4 7/16	2 1/4	3 5/8	5 7/8	2 3/8	3 7/8	6 1/4	2 7/8	4 11/16	7 9/16	3 1/8	5 1/8	8 1/4
F	-3/4	-1 3/16	-1 15/16	13/16	1 3/8	2 3/16	1 11/16	2 5/8	4 5/16	2 1/4	3 9/16	5 13/16	2 3/8	3 13/16	6 3/16	2 7/8	4 9/16	7 7/16	3 1/8	4 15/16	8 1/16
G	-3/4	-1 3/16	-1 15/16	13/16	1 3/8	2 3/16	1 11/16	2 5/8	4 5/16	2 1/4	3 9/16	5 13/16	2 3/8	3 13/16	6 3/16	2 7/8	4 9/16	7 7/16	3 1/8	4 15/16	8 1/16
H	-3/4	-1 1/4	-2	7/8	1 3/8	2 1/4	1 11/16	2 3/4	4 7/16	2 1/4	3 5/8	5 7/8	2 3/8	3 7/8	6 1/4	2 7/8	4 11/16	7 9/16	3 1/8	5 1/8	8 1/4

Girder	Span 7: f			g			h			j			Field Splice 13			k			l		
	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.
A	3 1/8	5 1/8	8 1/4	2 7/8	4 11/16	7 9/16	2 3/8	3 15/16	6 5/16	1 3/4	2 13/16	4 9/16	1 9/16	2 5/8	4 3/16	1	1 3/4	2 3/4	7/16	11/16	1 1/8
B	3 1/8	4 15/16	8 1/16	2 13/16	4 9/16	7 3/8	2 5/16	3 3/4	6 1/16	1 11/16	2 11/16	4 3/8	1 9/16	2 1/2	4 1/16	1	1 5/8	2 5/8	3/8	11/16	1 1/16
C	3 1/8	4 15/16	8 1/16	2 13/16	4 9/16	7 3/8	2 5/16	3 3/4	6 1/16	1 11/16	2 11/16	4 3/8	1 9/16	2 1/2	4 1/16	1	1 5/8	2 5/8	3/8	11/16	1 1/16
D	3 1/8	5 1/8	8 1/4	2 7/8	4 11/16	7 9/16	2 3/8	3 15/16	6 5/16	1 3/4	2 13/16	4 9/16	1 9/16	2 5/8	4 3/16	1	1 3/4	2 3/4	7/16	11/16	1 1/8
E	3 1/8	5 1/8	8 1/4	2 7/8	4 11/16	7 9/16	2 3/8	3 15/16	6 5/16	1 3/4	2 13/16	4 9/16	1 9/16	2 5/8	4 3/16	1	1 3/4	2 3/4	7/16	11/16	1 1/8
F	3 1/8	4 15/16	8 1/16	2 13/16	4 9/16	7 3/8	2 5/16	3 3/4	6 1/16	1 11/16	2 11/16	4 3/8	1 9/16	2 1/2	4 1/16	1	1 5/8	2 5/8	3/8	11/16	1 1/16
G	3 1/8	4 15/16	8 1/16	2 13/16	4 9/16	7 3/8	2 5/16	3 3/4	6 1/16	1 11/16	2 11/16	4 3/8	1 9/16	2 1/2	4 1/16	1	1 5/8	2 5/8	3/8	11/16	1 1/16
H	3 1/8	5 1/8	8 1/4	2 7/8	4 11/16	7 9/16	2 3/8	3 15/16	6 5/16	1 3/4	2 13/16	4 9/16	1 9/16	2 5/8	4 3/16	1	1 3/4	2 3/4	7/16	11/16	1 1/8

Span 8	Span 9
Note: Span 8 deflection and camber same as Unit 2 Span 5.	Note: Span 9 deflection and camber same as Unit 2 Span 6.

Note:
For notes see Sheet 11/28.

DEAD LOAD DEFLECTION AND CAMBER - UNIT 4

Span 10	Span 11
Note: Span 10 deflection and camber same as Unit 3 Span 7.	Note: Span 11 deflection and camber same as Unit 2 Span 5.

Girder	Span 12: a			b			Field Splice 22			c			d			e		
	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.
A	7/16	11/16	1 1/8	1	1 5/8	2 5/8	1 9/16	2 1/2	4 1/16	1 11/16	2 11/16	4 3/8	2 1/4	3 3/4	6	2 11/16	4 3/8	7 1/16
B	7/16	5/8	1 1/16	1	1 5/8	2 5/8	1 9/16	2 1/2	4 1/16	1 11/16	2 11/16	4 3/8	2 5/16	3 5/8	5 15/16	2 11/16	4 3/8	7 1/16
C	7/16	5/8	1 1/16	1	1 5/8	2 5/8	1 9/16	2 1/2	4 1/16	1 11/16	2 11/16	4 3/8	2 5/16	3 5/8	5 15/16	2 11/16	4 3/8	7 1/16
D	7/16	11/16	1 1/8	1	1 5/8	2 5/8	1 9/16	2 1/2	4 1/16	1 11/16	2 11/16	4 3/8	2 1/4	3 3/4	6	2 11/16	4 3/8	7 1/16
E	7/16	11/16	1 1/8	1	1 5/8	2 5/8	1 9/16	2 1/2	4 1/16	1 11/16	2 11/16	4 3/8	2 1/4	3 3/4	6	2 11/16	4 3/8	7 1/16
F	7/16	5/8	1 1/16	1	1 5/8	2 5/8	1 9/16	2 1/2	4 1/16	1 11/16	2 11/16	4 3/8	2 5/16	3 5/8	5 15/16	2 11/16	4 3/8	7 1/16
G	7/16	5/8	1 1/16	1	1 5/8	2 5/8	1 9/16	2 1/2	4 1/16	1 11/16	2 11/16	4 3/8	2 5/16	3 5/8	5 15/16	2 11/16	4 3/8	7 1/16
H	7/16	11/16	1 1/8	1	1 5/8	2 5/8	1 9/16	2 1/2	4 1/16	1 11/16	2 11/16	4 3/8	2 1/4	3 3/4	6	2 11/16	4 3/8	7 1/16

Girder	Span 12: f			g			h			j			Field Splice 23			k		
	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.	Stl.	Rem. D.L.	Tot.
A	2 7/8	4 5/8	7 1/2	2 3/4	4 1/2	7 1/4	2 3/8	3 15/16	6 5/16	1 3/4	2 15/16	4 11/16	1 5/8	2 5/8	4 1/4	15/16	1 9/16	2 1/2
B	2 7/8	4 5/8	7 1/2	2 13/16	4 1/2	7 5/16	2 7/16	3 15/16	6 3/8	1 13/16	3	4 13/16	1 11/16	2 11/16	4 3/8	1	1 5/8	2 5/8
C	2 7/8	4 5/8	7 1/2	2 13/16	4 1/2	7 5/16	2 7/16	3 15/16	6 3/8	1 13/16	3	4 13/16	1 11/16	2 11/16	4 3/8	1	1 5/8	2 5/8
D	2 7/8	4 5/8	7 1/2	2 3/4	4 1/2	7 1/4	2 3/8	3 15/16	6 5/16	1 3/4	2 15/16	4 11/16	1 5/8	2 5/8	4 1/4	15/16	1 9/16	2 1/2
E	2 7/8	4 5/8	7 1/2	2 3/4	4 1/2	7 1/4	2 3/8	3 15/16	6 5/16	1 3/4	2 15/16	4 11/16	1 5/8	2 5/8	4 1/4	15/16	1 9/16	2 1/2
F	2 7/8	4 5/8	7 1/2	2 13/16	4 1/2	7 5/16	2 7/16	3 15/16	6 3/8	1 13/16	3	4 13/16	1 11/16	2 11/16	4 3/8	1	1 5/8	2 5/8
G	2 7/8	4 5/8	7 1/2	2 13/16	4 1/2	7 5/16	2 7/16	3 15/16	6 3/8	1 13/16	3	4 13/16	1 11/16	2 11/16	4 3/8	1	1 5/8	2 5/8
H	2 7/8	4 5/8	7 1/2	2 3/4	4 1/2	7 1/4	2 3/8	3 15/16	6 5/16	1 3/4	2 15/16	4 11/16	1 5/8	2 5/8	4 1/4	15/16	1 9/16	2 1/2

H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

DEAD LOAD DEFLECTION AND CAMBER DIAGRAM
UNITS 2, 3 & 4

I-80 OVER CUYAHOGA RIVER VALLEY

BR. NO. CUY-80-1843 STA. 996+22.25 TO STA. 1037+77.75

CUYAHOGA COUNTY OHIO

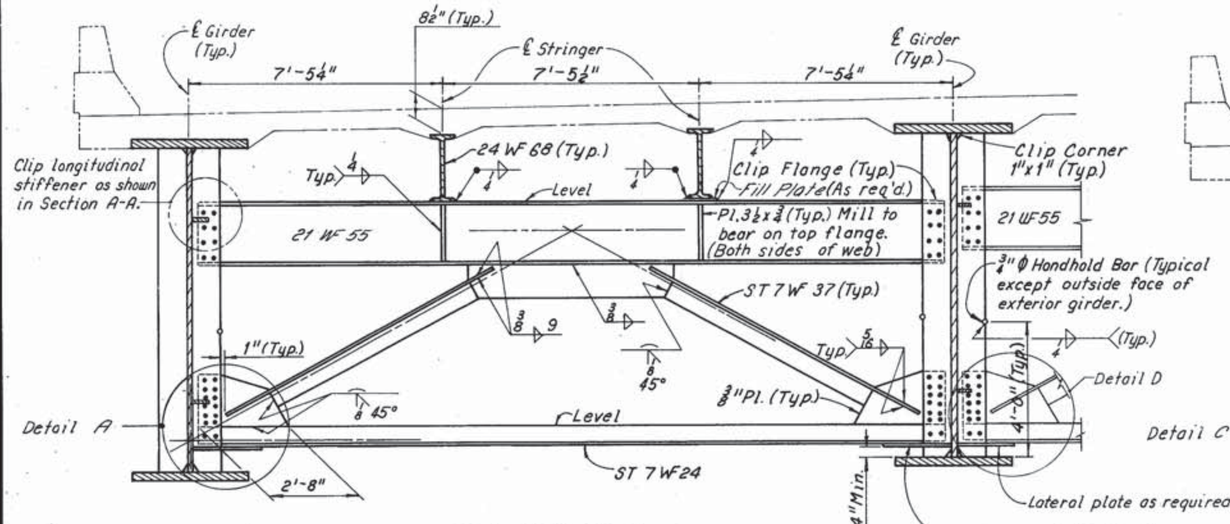
DRAWN JEH	TRACED GEM	CHECKED TJS	REVIEWED WF	REVISED
DATE 11-5-68	DATE 11-11-68	DATE 12-16-68	DATE 8-21-70	SHEET 12/28

FED. RD. DIVISION	STATE	PROJECT		
2	OHIO			

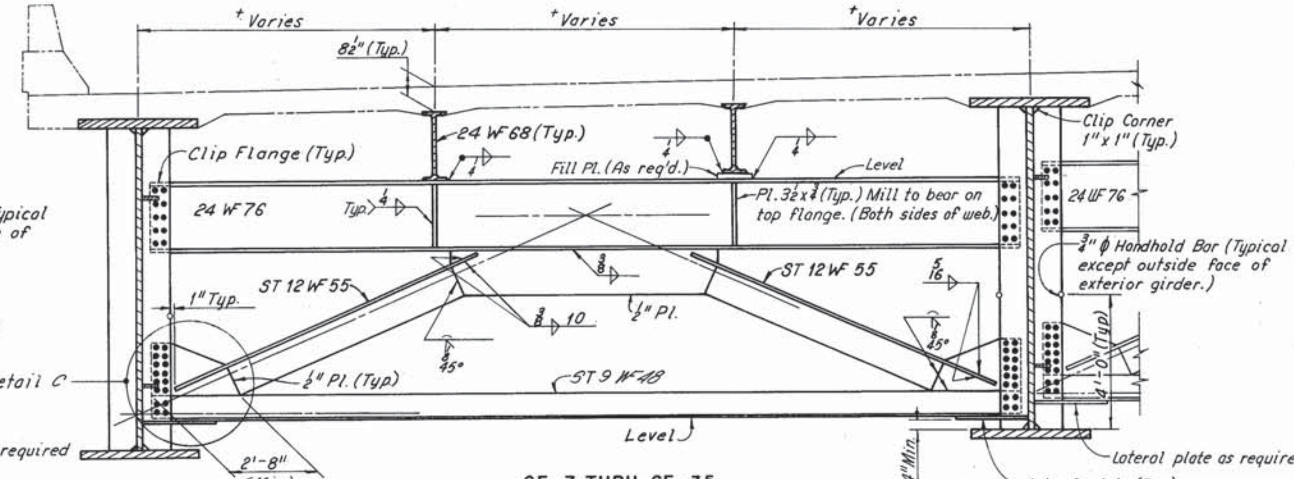
85/112

CUYAHOGA COUNTY
CUY-80-18.43 Part 2

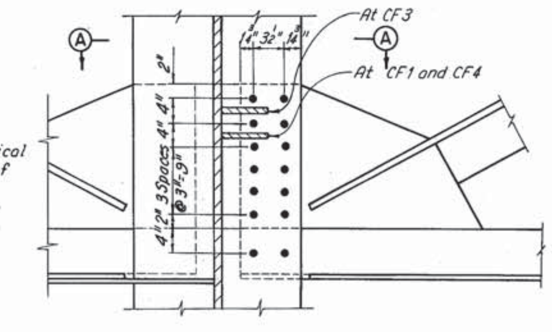
†Note: See Framing Plan - Unit 1R Sheet 5/28 for Stringer Spacing



CF-1, CF-3, CF-4

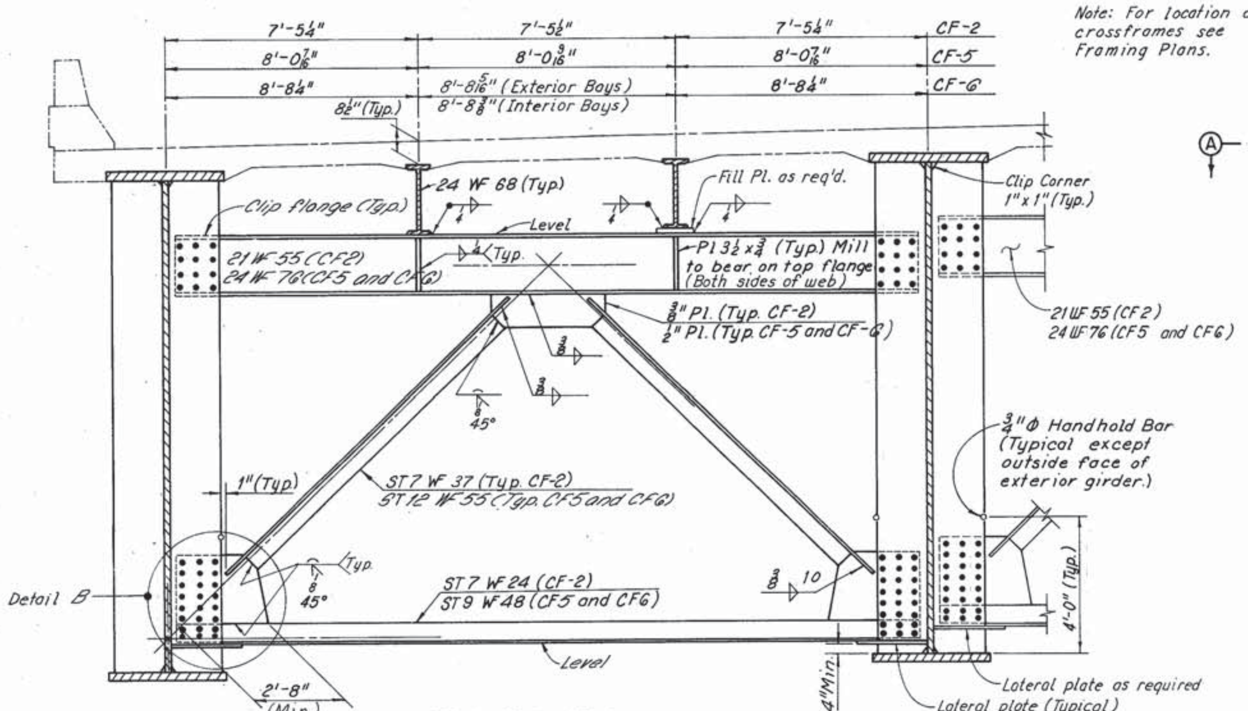


CF-7 THRU CF-35



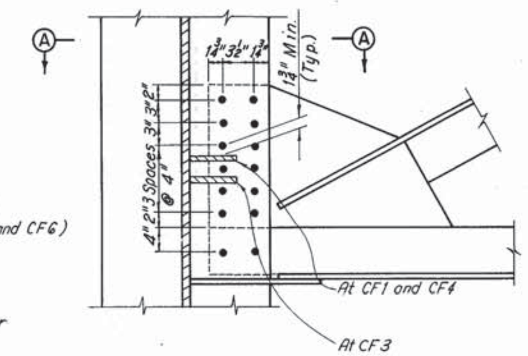
DETAIL D

(Typical except where lateral plate is required)

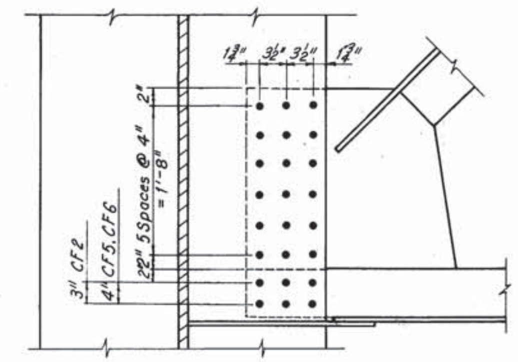


CF-2, CF-5, CF-6

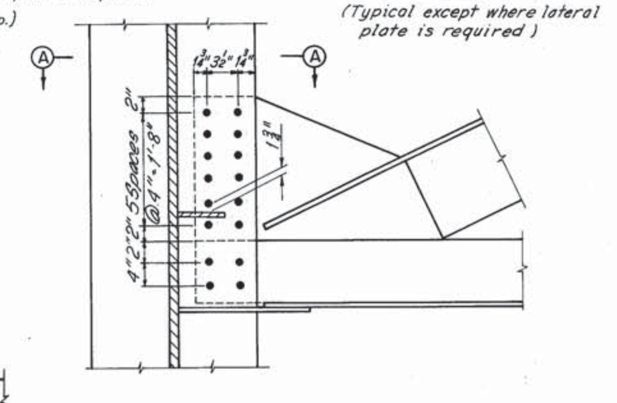
Note: For location of crossframes see Framing Plans.



DETAIL A

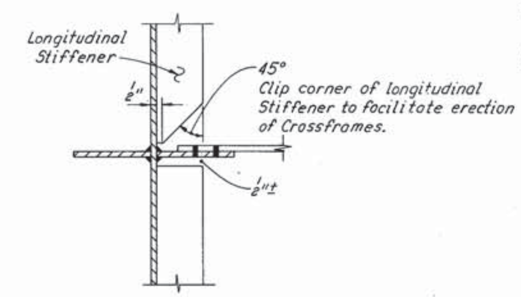


DETAIL B

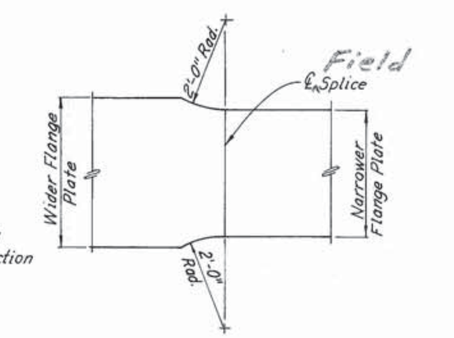


DETAIL C

Note:
Lateral plates are required in the interior bays at Piers 1, 4, 5, 7, 8, 10, 11 and 14 and at the first crossframe on both sides of these piers for connecting lateral bracing.

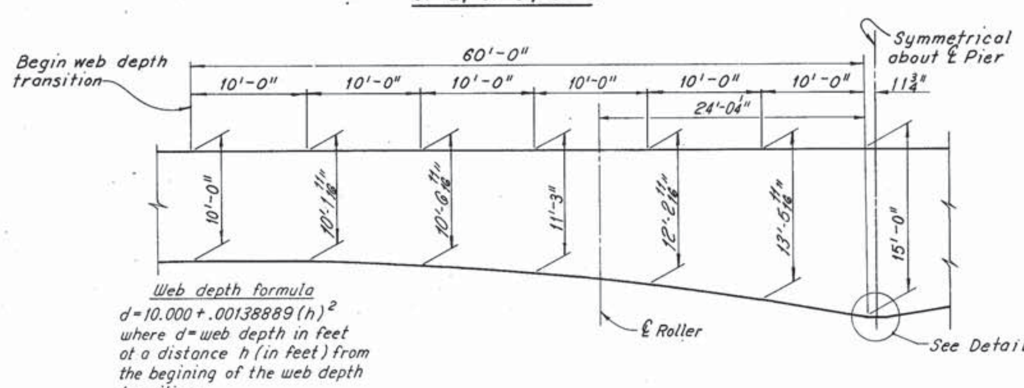


SECTION A-A



FLANGE TRANSITION DETAIL

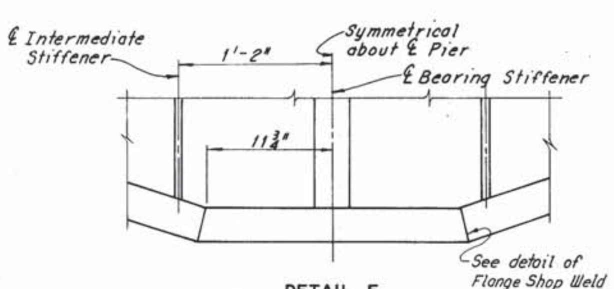
Note:
All bolts are 1" ϕ High Strength.



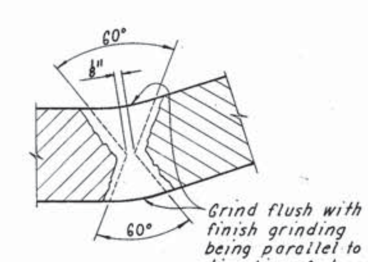
Web depth formula
$d = 10.000 + .00138889(h)^2$
where d = web depth in feet at a distance h (in feet) from the begining of the web depth transition.

GIRDER WEB TRANSITION DETAIL

Note: Dimensions are measured parallel or perpendicular to grade.



DETAIL E



FLANGE SHOP WELD AT PIER

Note: The above full penetration weld shall be back gouged and welded after welding far side.

H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

INTERMEDIATE CROSSFRAME AND GIRDER DETAILS

I-80 OVER CUYAHOGA RIVER VALLEY
BR. NO. CUY-80-1843 STA. 996+22.25 TO STA. 1037+77.75

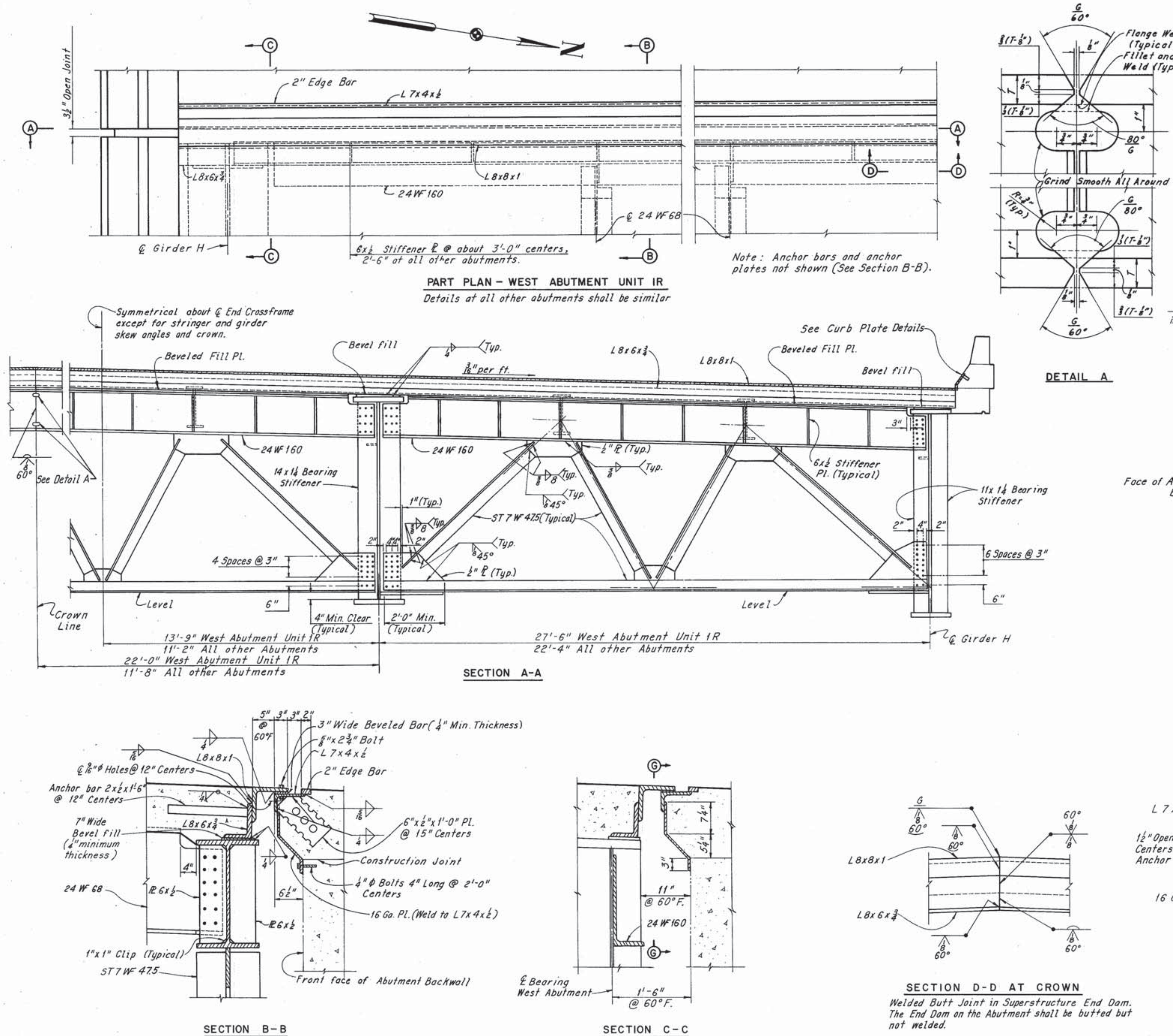
CUYAHOGA COUNTY OHIO

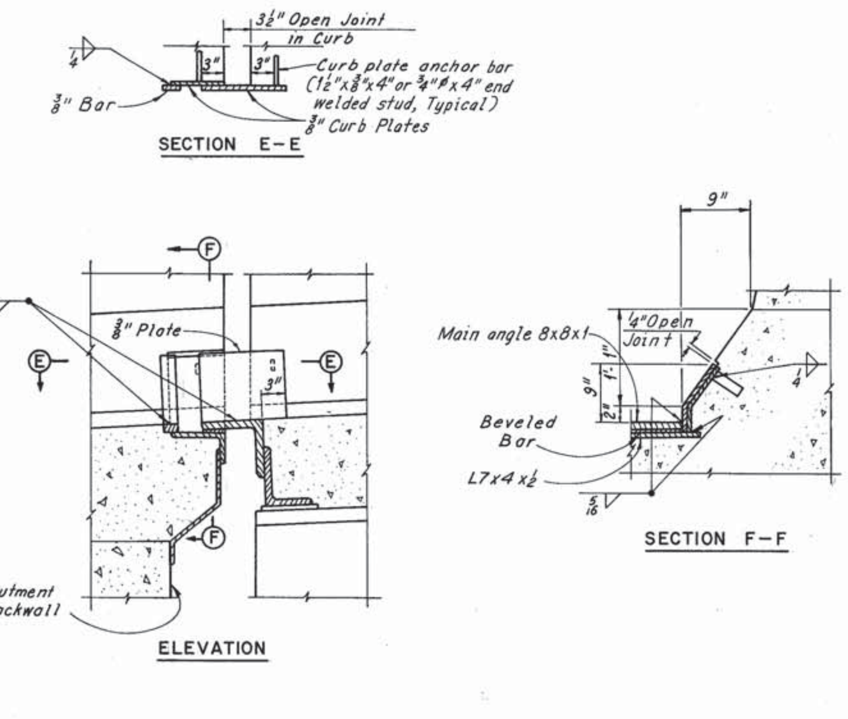
DRAWN TJS	TRACED CEM	CHECKED JEH	REVIEWED WR	REVISED 11-16-70
DATE 11-15-68	DATE 1-21-69	DATE 2-19-69	DATE 8-21-70	SHEET 13/28

FED. RD. DIVISION	STATE	PROJECT		
2	OHIO			

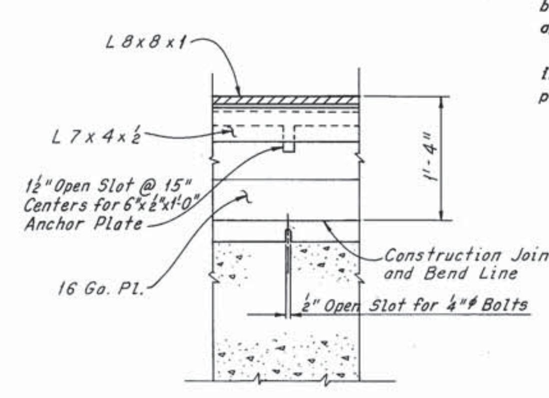
86/112

CUYAHOGA COUNTY
CUY -80-1843 Part 2





CURB PLATE DETAILS



SECTION G-G

Notes:

For additional notes concerning joint lubrication, ¾" φ bolts, anchor bars, anchor plates, painting and joints in end dam components see Ohio Standard Drawing SD-1-69, Sheet 1 of 3.

Portions of end dams which will be in contact with steel or with concrete shall not be painted. All other portions shall be cleaned and painted in accordance with Item 514.

All parts of the end dam and end crossframe, including the bearing stiffeners and lateral plates shall be ASTM A588 steel.

FOR END DAM MODIFICATIONS SEE SHEET 86A/112

H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

END CROSSFRAME DETAILS

I-80 OVER CUYAHOGA RIVER VALLEY
BR. NO. CUY-80-1843 STA. 996+22.25 TO STA. 1037+77.75

CUYAHOGA COUNTY OHIO

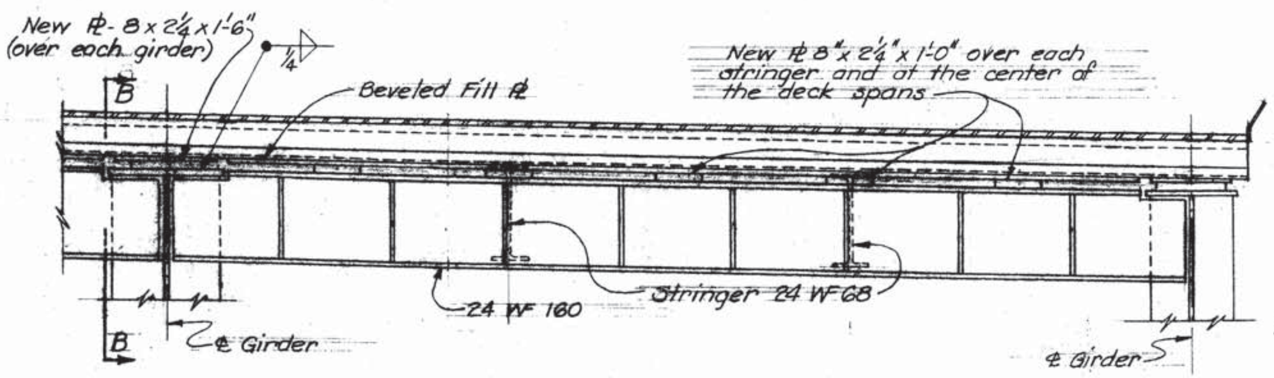
DRAWN JEH	TRACED GEM	CHECKED CHB	REVIEWED WF	REVISED
DATE 8-1-68	DATE 3-17-69	DATE 2-18-69	DATE 8-22-70	SHEET 14/28

REV. 8-15-73

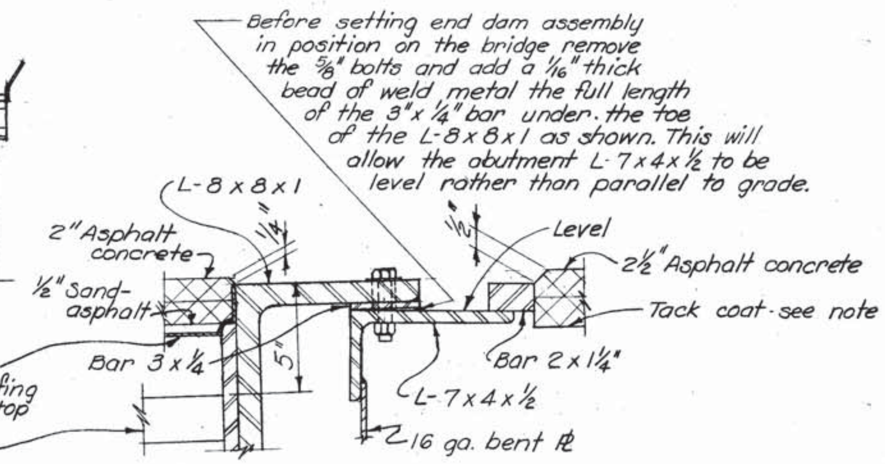
FED. RD. DIVISION	STATE	PROJECT		
2	OHIO			

BGA 112

CUY-480-18.43 Part 2

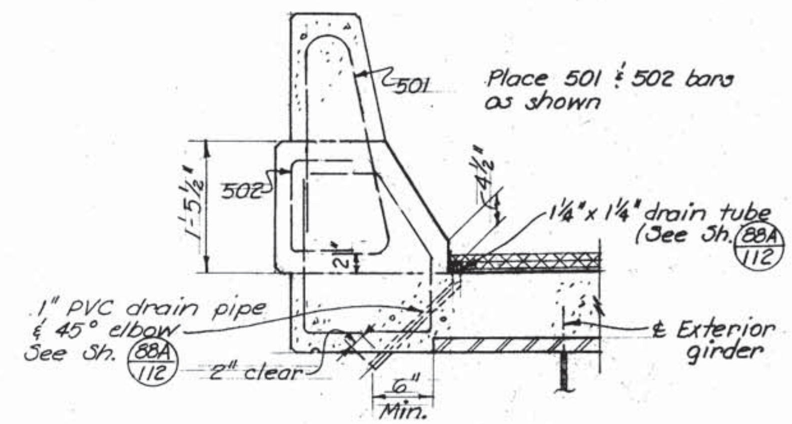


SECTION A-A

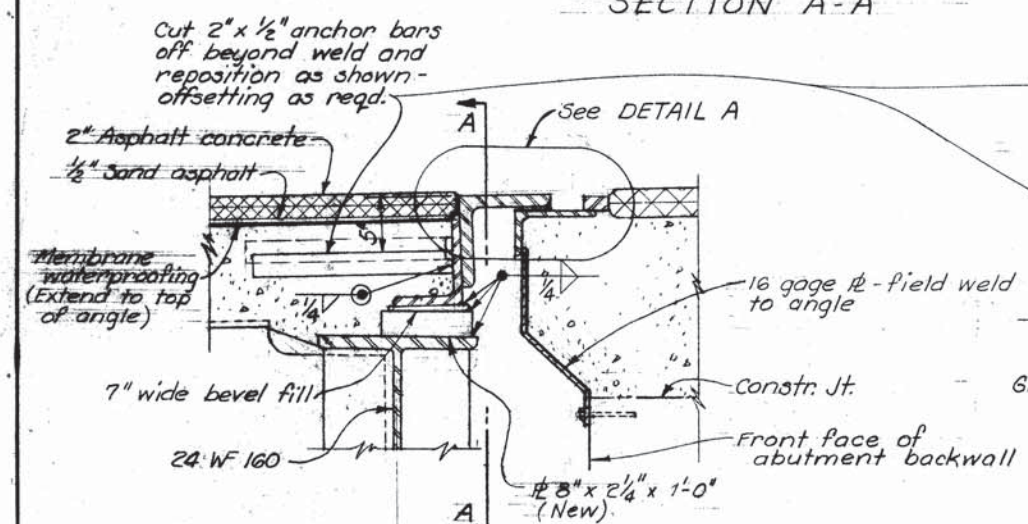


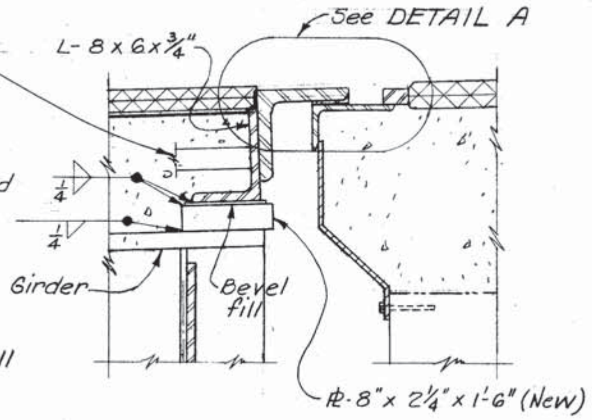
DETAIL A

TACK COAT: Apply a tack coat at the rate of 0.1 gal. per sq. yd. and cover aggregate to the surface of abutment backwalls & approach slabs.



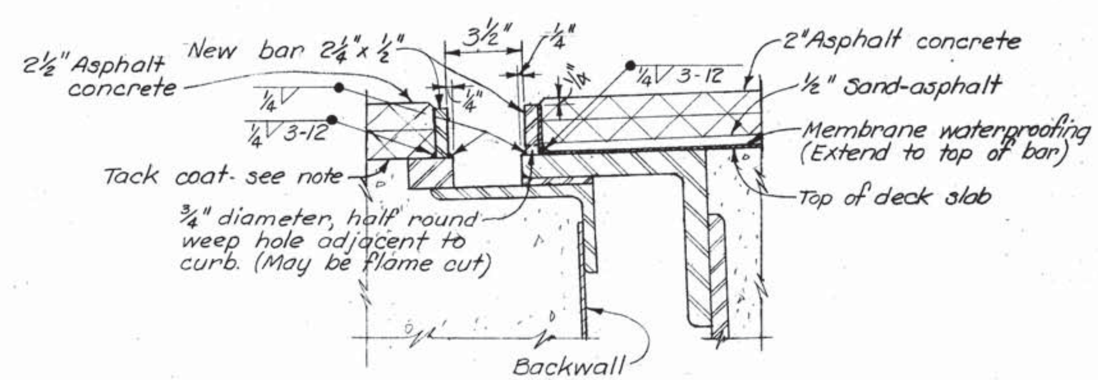
REVISED PART DECK SECTION
(See original plans for parapet dimensions not shown)





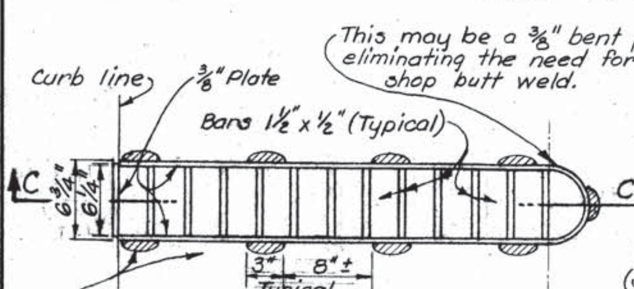
SECTION B-B

SECTION THRU ROADWAY END DAM & 24 WF 160 END BEAM



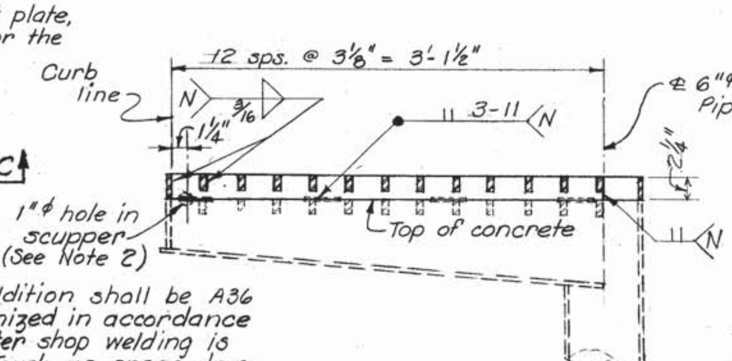
SECTION THRU ROADWAY END DAM

END DAM MODIFICATION AT FORWARD ABUTMENTS



Remove enough concrete in shaded areas to permit welding. Patch with mortar. Include with scupper modification for payment.

PLAN



Scupper addition shall be A36 steel, galvanized in accordance with 711 after shop welding is completed. Touch up areas damaged by field welding with zinc-rich paint.

SECTION C-C

TYPE 1 SCUPPER MODIFICATION

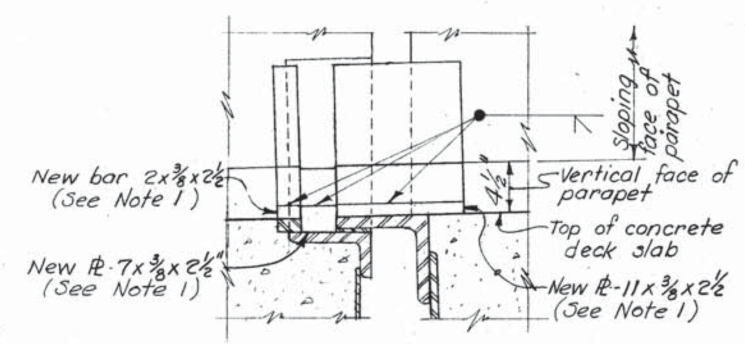
10 SCUPPERS AT THE REAR ABUTMENT END OF THE LEFT BRIDGE

ALL SCUPPERS:
Note 2: Cut a 1"φ hole in the uphill side of scupper as shown in SECTION C-C. Include with applicable scupper modification item for payment.

TYPE 2 SCUPPER MODIFICATION

10 SCUPPERS AT THE REAR ABUTMENT END OF THE RIGHT BRIDGE

Raise scuppers 2¼" providing longer support bolts if necessary. Weld a 10" length of 6"φ standard pipe (galvanized) to the bottom of each scupper pipe outlet. (This ends the pipe approx. 8" below the girder bottom flange). Touch up areas damaged by welding with zinc-rich paint.



ELEVATION OF CURB PLATES
(Asphalt concr., etc., & retainer bars not shown)

MODIFICATION OF END DAMS & CURB PLATES AT REAR ABUTMENTS

Note 1: New curb plates may be provided in lieu of welding extensions on the original plates.

TYPE 3 SCUPPER MODIFICATION

92 SCUPPERS AT PIERS 3, 6, 9 & 12

Raise scuppers 2¼" providing longer support bolts if necessary. Weld galv. 6"φ std. pipe to the bottom of each scupper pipe outlet as reqd. to project into the horizontal conductor assembly. (2¼" max.) The 6"φ pipe downspouts (8 ea.) from the transverse trough to the hor. collector system shall be lengthened as described above and included with Type 3 Scupper Modification for payment.

STATE OF OHIO
DEPARTMENT OF HIGHWAYS
DIVISION OF DESIGN AND CONSTRUCTION
BUREAU OF BRIDGES

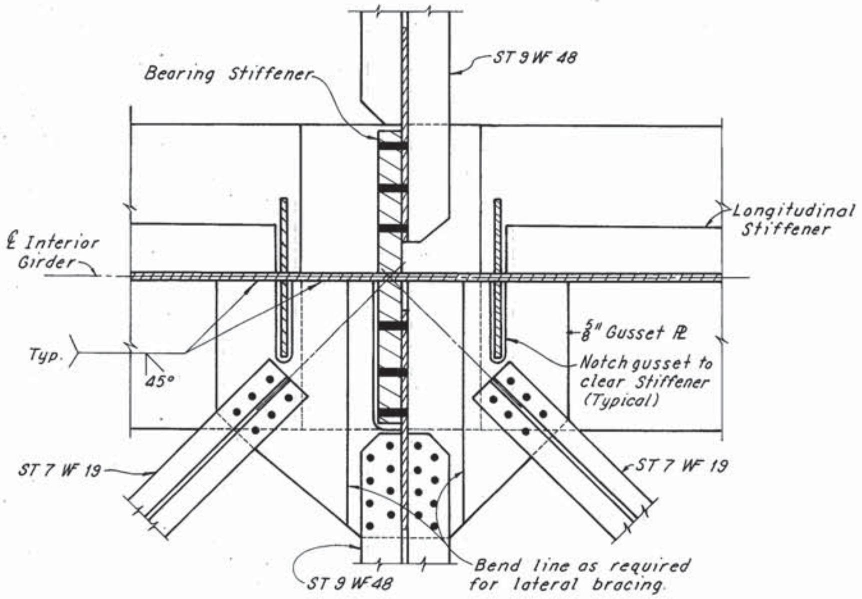
MODIFICATION OF END DAMS AND SCUPPERS, PARAPET REVISION
I-480 OVER CUYAHOGA RIVER VALLEY
CUYAHOGA COUNTY

DESIGNED	DRAWN	TRACED	CHECKED	REVIEWED	DATE	REVISED
NAA	NAA		CPD	WJJ	8-15-73	

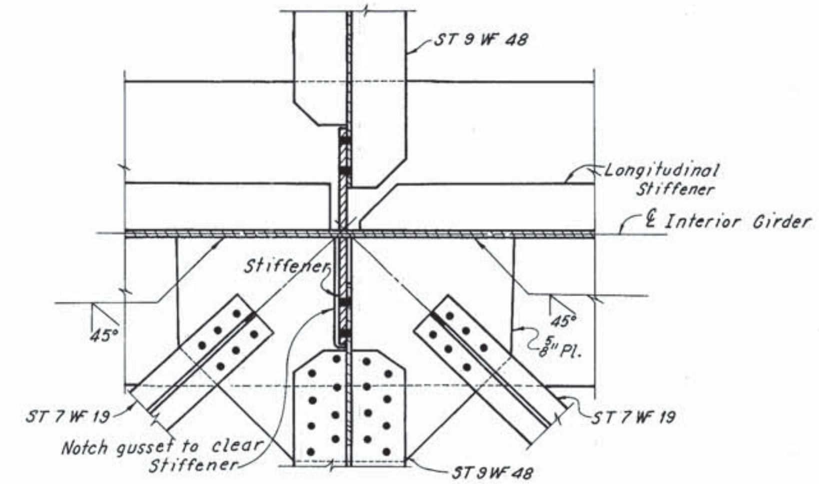
This sheet added by NAA, 8-15-73

FED RD. DIVISION	STATE	PROJECT	
2	OHIO		

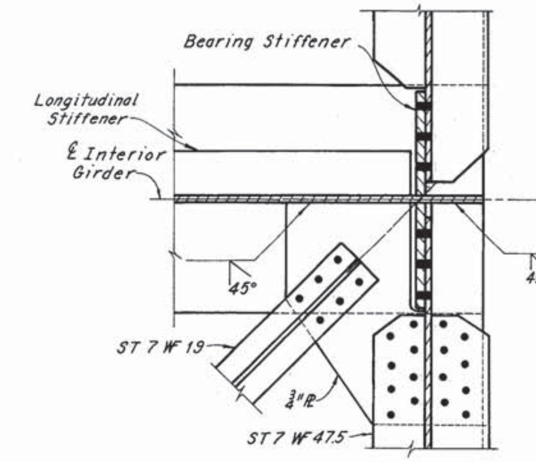
CUYAHOGA COUNTY
CUY-80-1843 Part 2



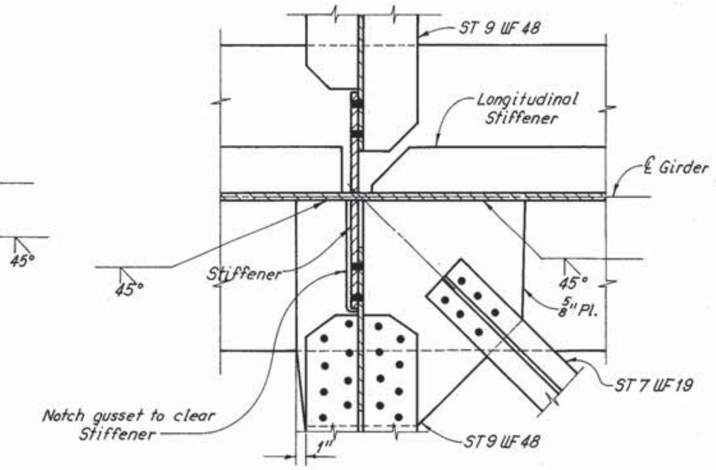
AT PIERS UNIT 1R



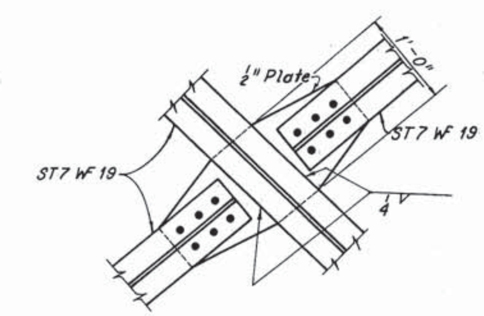
AT CROSSFRAMES CF-7 THROUGH CF-35
(UNIT 1R)



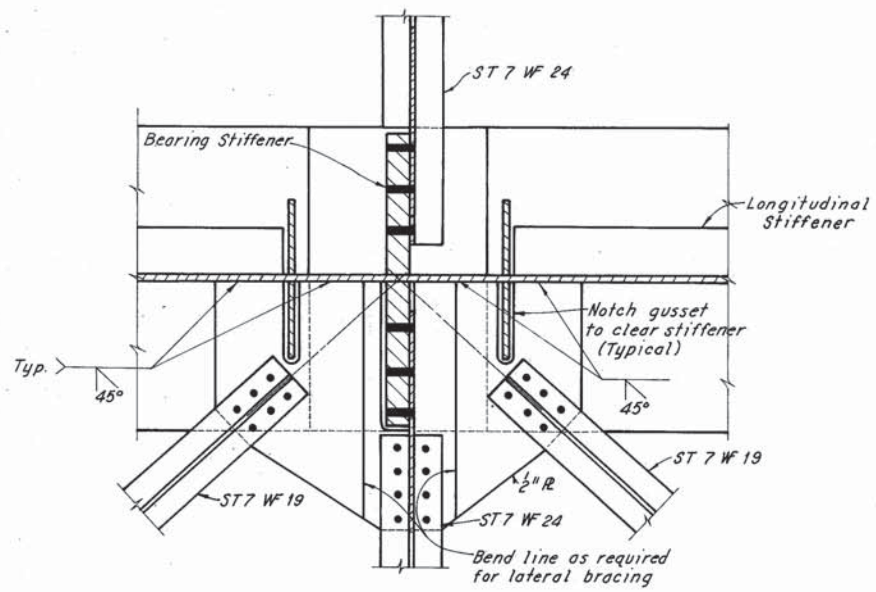
AT ABUTMENT END CROSSFRAME



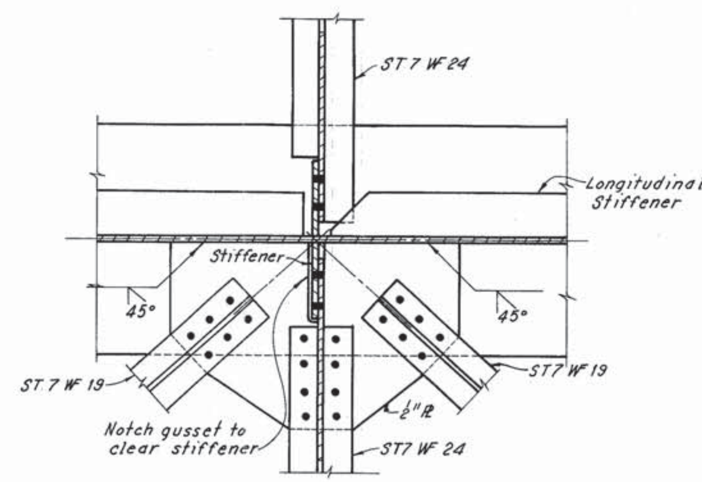
AT CROSSFRAMES WITH LATERAL BRACING EXTENDING IN ONE DIRECTION ONLY
(UNIT 1R)



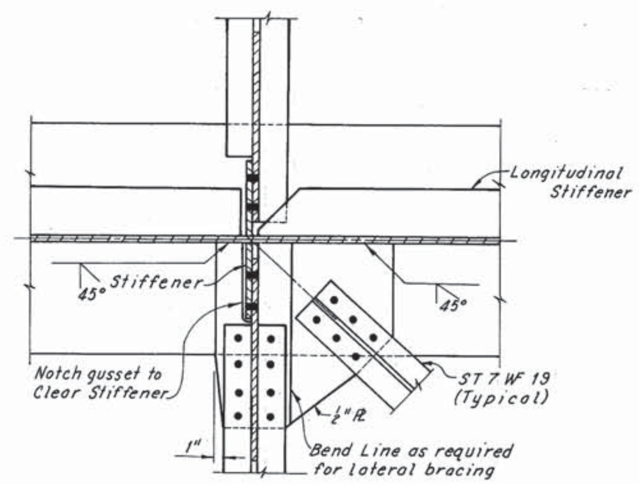
LATERAL BRACING CONNECTION AT MID-PANEL



AT PIERS (EXCEPT UNIT 1R)



AT INTERMEDIATE CROSSFRAME
(EXCEPT UNIT 1R)



AT CROSSFRAMES WITH LATERAL BRACING EXTENDING IN ONE DIRECTION ONLY
(EXCEPT UNIT 1R)

Note:
Lateral Bracing Connections in interior bays to be the same as for exterior bay lateral connections.

TYPICAL CONNECTIONS

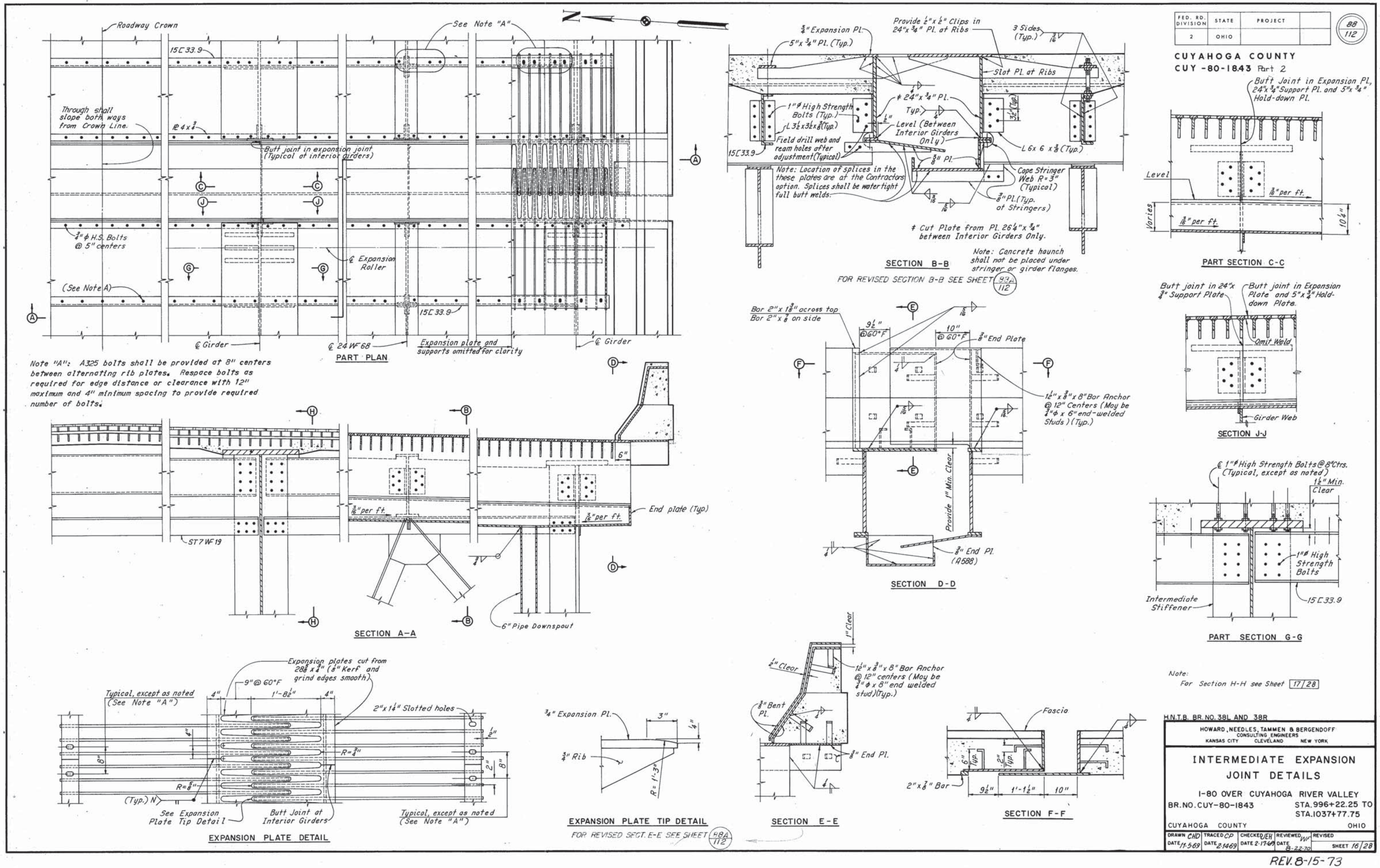
H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

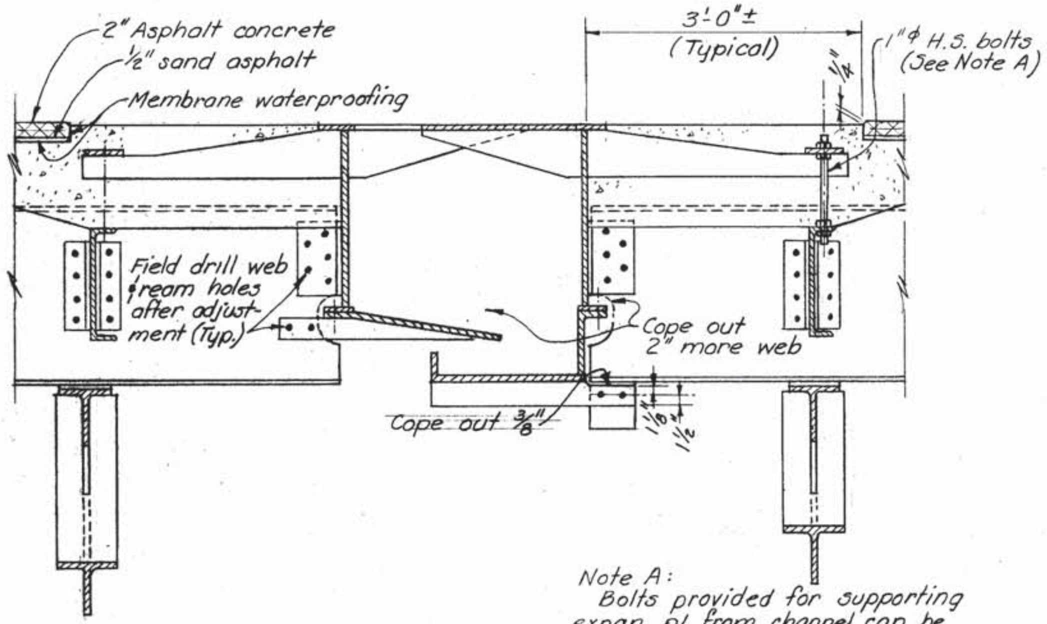
LATERAL BRACING DETAILS

I-80 OVER CUYAHOGA RIVER VALLEY
BR. NO. CUY-80-1843 L & R STA. 996+22.25 TO STA. 1037+77.75
CUYAHOGA COUNTY OHIO

DRAWN JEH DATE 1-29-69	TRACED GEM DATE 1-30-69	CHECKED CHB DATE 2-6-69	REVIEWED WF DATE 3-21-70	REVISED	SHEET 15/28

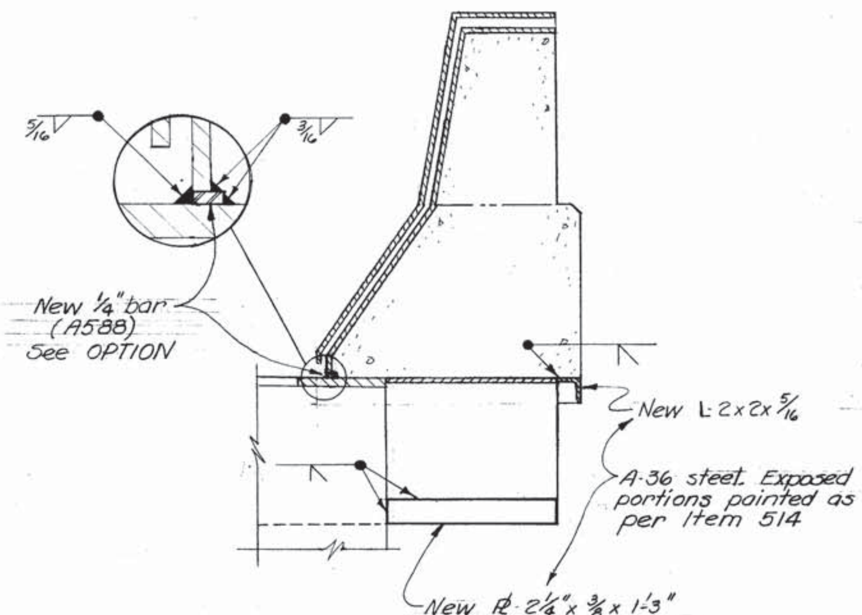


FED. RD. DIVISION 2 | STATE OHIO | PROJECT
88/112
CUYAHOGA COUNTY
CUY-80-1843 Part 2
PART PLAN
Note "A": A325 bolts shall be provided at 8" centers between alternating rib plates. Respace bolts as required for edge distance or clearance with 12" maximum and 4" minimum spacing to provide required number of bolts.
Roadway Crown
See Note "A"
15[33.9
Through shall slope both ways from Crown Line.
Butt joint in expansion joint (Typical at interior girders)
3/4" ϕ H.S. Bolts @ 5" centers
(See Note A)
℄ Expansion Roller
℄ Girder
℄ 24WF68
Expansion plate and supports omitted for clarity
SECTION A-A
ST7WF19
End plate (Typ)
6" Pipe Downspout
SECTION B-B
FOR REVISED SECTION B-B SEE SHEET 88A/112
3/4" Expansion Pl.
5"x3/4" Pl. (Typ.)
Provide 1/2"x1/2" Clips in 24"x3/4" Pl. at Ribs
3 Sides (Typ.)
Slot Pl. at Ribs
1"ϕ High Strength Bolts (Typ.)
L 3 1/2x3 1/2x3/8 (Typ.)
Field drill web and ream holes after adjustment (Typical)
24"x3/4" Pl.
Level (Between Interior Girders Only)
L 6x6x3/8 (Typ.)
Note: Location of splices in the these plates are at the Contractors option. Splices shall be water tight full butt welds.
Cope Stringer Web R=3" (Typical)
3/8" Pl. (Typ. at Stringers)
Cut Plate from Pl. 26 1/4"x3/4" between Interior Girders Only.
Note: Concrete haunch shall not be placed under stringer or girder flanges.
PART SECTION C-C
Butt Joint in Expansion Pl., 24"x3/4" Support Pl. and 5"x3/4" Hold-down Pl.
Level
3/16" per ft.
Varies
10 1/4"
SECTION D-D
Bar 2"x1 3/8" across top
Bar 2"x7/8" on side
9 1/2" @ 60°F
10" @ 60°F
3/8" End Plate
1 1/2"x3/8"x8" Bar Anchor @ 12" Centers (May be 3/4"ϕ x 6" end-welded Studs) (Typ.)
Provide 1" Min. Clear
3/8" End Pl. (A588)
SECTION J-J
Butt joint in 24"x3/4" Support Plate
Butt joint in Expansion Plate and 5"x3/4" Hold-down Plate.
Omit Weld
Girder Web
PART SECTION G-G
℄ 1"ϕ High Strength Bolts @ 8" Ctrs. (Typical, except as noted)
1 1/2" Min. Clear
1"ϕ High Strength Bolts
Intermediate Stiffener
15[33.9
Note: For Section H-H see Sheet 17/28
EXPANSION PLATE DETAIL
Expansion plates cut from 28 3/8 x 3/4" (1/8" Kerf and grind edges smooth)
Typical, except as noted (See Note "A")
9" @ 60°F
1'-8 1/2"
2"x1 1/4" Slotted holes
R=3/4"
R=5/8"
See Expansion Plate Tip Detail
Butt Joint at Interior Girders
EXPANSION PLATE TIP DETAIL
FOR REVISED SECT. E-E SEE SHEET 88A/112
3/4" Expansion Pl.
3/4" Rib
R=1'-3"
SECTION E-E
1" Clear
3/8" Bent Pl.
1 1/2"x3/8"x8" Bar Anchor @ 12" centers (May be 3/4"ϕ x 8" end welded stud) (Typ.)
3/8" End Pl.
SECTION F-F
Fascia
2"x7/8" Bar
9 1/2"
1'-1 1/2"
10"
H.N.T.B. BR. NO. 38L AND 38R
HOWARD, NEEDLES, TAMMEN & BERGENDOFF CONSULTING ENGINEERS KANSAS CITY CLEVELAND NEW YORK
INTERMEDIATE EXPANSION JOINT DETAILS
I-80 OVER CUYAHOGA RIVER VALLEY
BR. NO. CUY-80-1843 STA. 996+22.25 TO STA. 1037+77.75
CUYAHOGA COUNTY OHIO
DRAWN CHD DATE 11-5-69 | TRACED CP DATE 2-14-69 | CHECKED VEH DATE 2-17-69 | REVIEWED WF DATE 8-22-70 | REVISED | SHEET 16/28
REV. 8-15-73



Note A:
Bolts provided for supporting expan. pl. from channel can be used from stringer or girder flange. Provide new longer bolts for support from channel as needed.

REVISED SECTION B-B, SH. 88/112

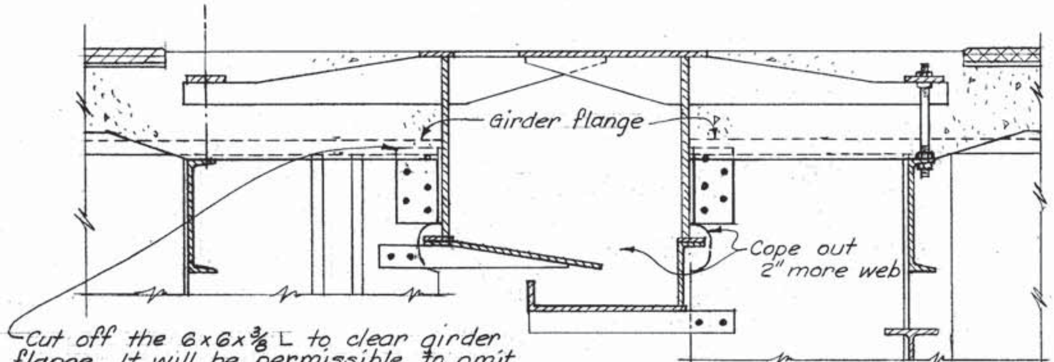


REVISED SECTION E-E, SH. 88/112

SECTION D-D, SH. 88/112

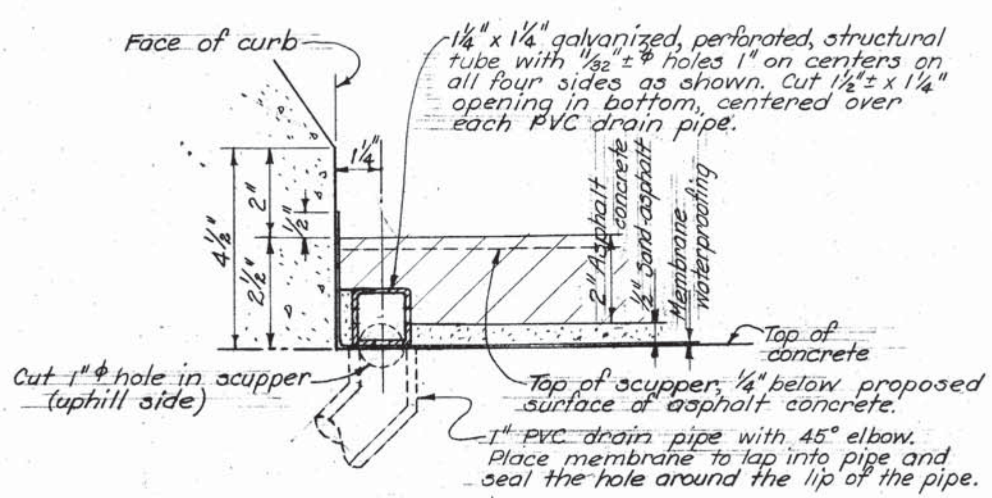
Field weld a ¼" filler bar to the bottom of the ⅜" end plates at the joint ends of the parapet. (A-36). See OPTION.

OPTION: At the option of the Contractor the ¼" filler bars may be omitted and a smooth transition in the parapet provided.



Cut off the 6x6x⅜ L to clear girder flange. It will be permissible to omit one bolt if necessary. - Typical for girders only.

REVISED PART OF SECTION H-H, SH. 89/112



SUBDRAINAGE & SURFACE COURSE DETAILS

The PVC drain pipes shall be spaced at approx. 6' intervals with a pipe placed as near as practicable to the downhill end of the drain tube adjacent to each expansion joint. The pipes shall be relocated or extended so that any discharge clears bridge seats or structural members, such as sign supports, by at least 6."

The structural tube may be placed in any convenient lengths using butt joints. The price per lin. ft. for this drainage system shall include all PVC pipe and fittings (schedule 40), structural tubes and all labor necessary to complete this item. The quantity will be the actual lengths of structural tube required.

The steel for the structural tube shall conform to the following:
PREGALVANIZED, ASTM A446, Grade A Steel, Galvanizing as per ASTM A525.
POSTGALVANIZED, ASTM A569 or A366, Galvanizing as per 711.02.
Any damaged galvanizing shall be repaired as per AASHO M36.
The minimum steel thickness shall be 0.105".

The drain pipe and elbow shall comply with the dimensions and markings of ASTM D1785 and ASTM D2466, Type I and II respectively. The solvent cement for the pipe and fittings shall conform to ASTM D2564.

ESTIMATED QUANTITIES

Item	Total	Unit	Description	As Built
511	*	Cu. yd.	Class C concrete, superstructure	
518	16,399	Lin. ft.	Subdrainage for wearing course, as per plan	
Spec.	2	Each	Rear end dam modification	
Spec.	2	Each	Forward end dam modification	
Spec.	8	Each	Expansion joint modification	
Spec.	10	Each	Type I scupper modification	
Spec.	10	Each	Type 2 scupper modification	
Spec.	92	Each	Type 3 scupper modification	
404	3570△	Cu. yd.	Asphalt concrete	
Spec.	893	Cu. yd.	Sand asphalt, see proposal note	
Spec.	64,705	Sq. yd.	Membrane waterproofing, see proposal note	
808	*	Units	Chemical admixture for concrete, Type A, B, or D	
407	‡	Tons	Cover aggregate	
407	‡	Gals.	Tack coat: 702.04, MS-2 or RS-1; or 702.02, RC-70 or RC 250	

*For quantity see Sh. 74/112

‡For quantity see Sh. 10/112

△Quantity shown is Bridge quantity. For quantity on approach slabs see Sh. 10/112.

PAYMENT for the modification items shall include all the new steel as well as the labor required to complete the modification. Unless otherwise noted all new steel shall be A588.

STATE OF OHIO
DEPARTMENT OF HIGHWAYS
DIVISION OF DESIGN AND CONSTRUCTION
BUREAU OF BRIDGES

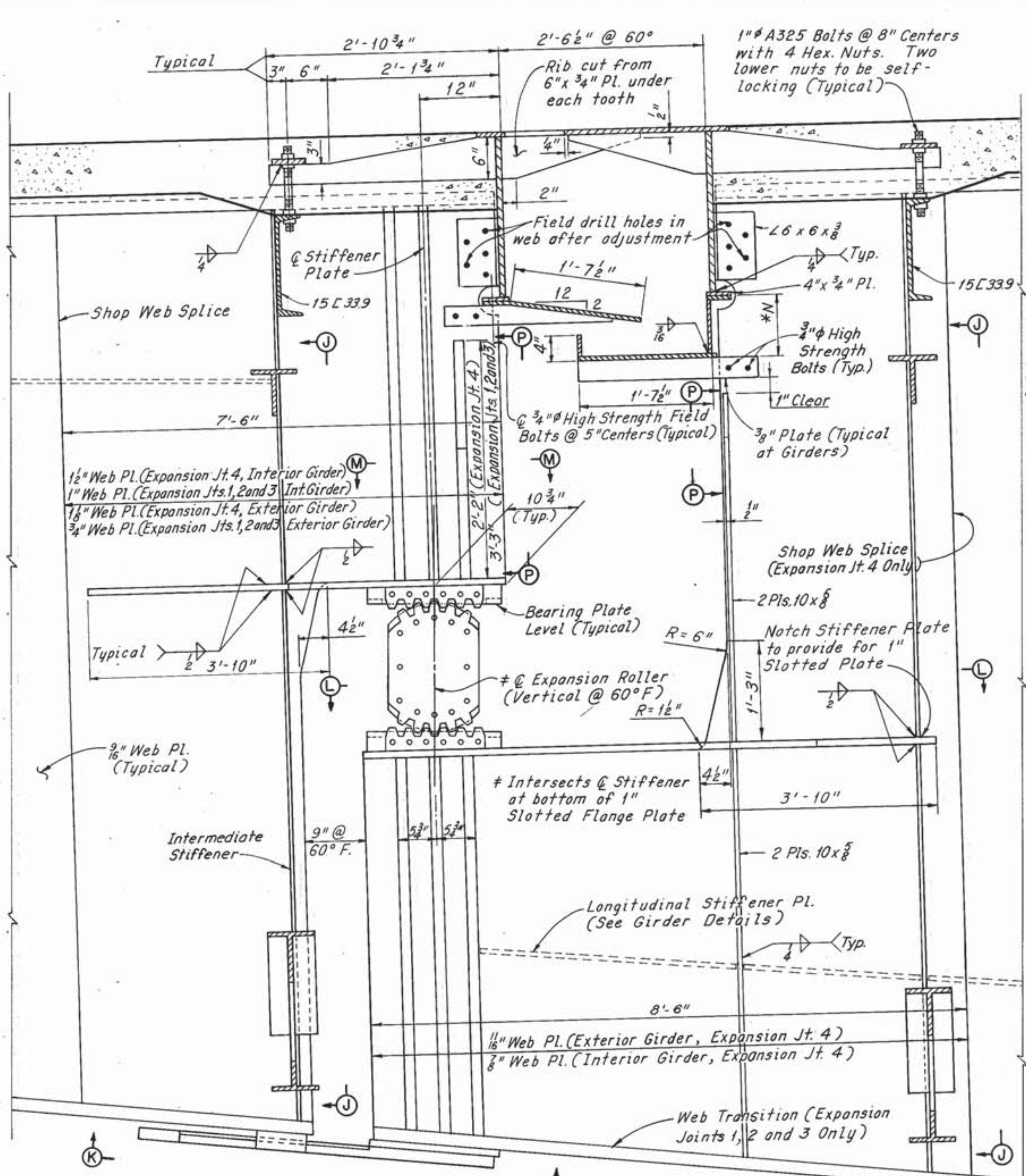
EXPANSION JOINT MODIFICATION
DECK DRAINAGE DETAILS
ESTIMATED QUANTITIES
I-480 OVER CUYAHOGA RIVER VALLEY
CUYAHOGA COUNTY

DESIGNED	DRAWN	TRACED	CHECKED	REVIEWED	DATE	REVISED
NAA	NAA		CPD	WJJ	8-15-73	

This sheet added by NAA, 8-15-73

FED. RD. DIVISION	STATE	PROJECT
2	OHIO	

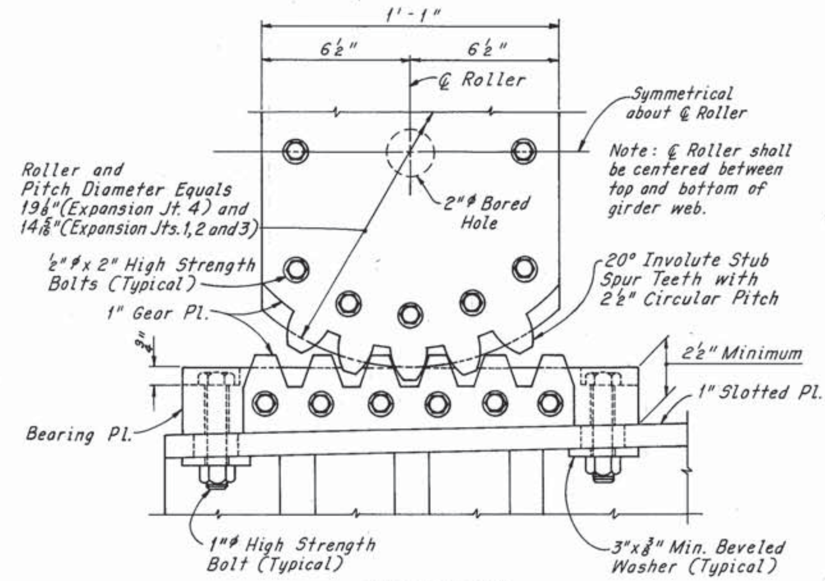
CUYAHOGA COUNTY
CUY - 80 - 1843 Part 2



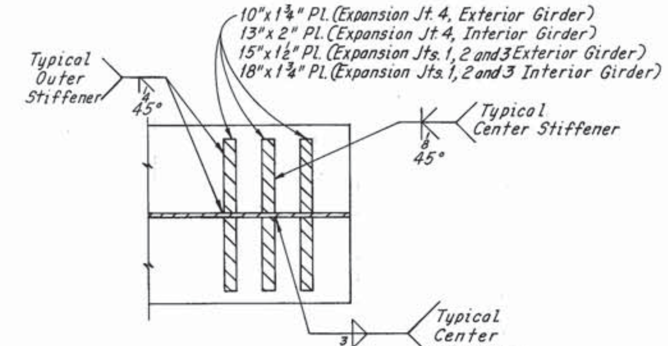
SECTION H - H
Dimensions shown are parallel or perpendicular to grade

FOR REVISED SECTION H-H SEE SHEET 88A/112

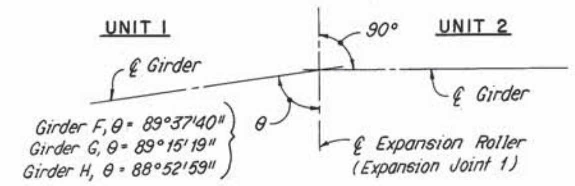
Note:
Details shown are for haunched girders. Details at expansion joint 4 are similar but girders are not haunched.



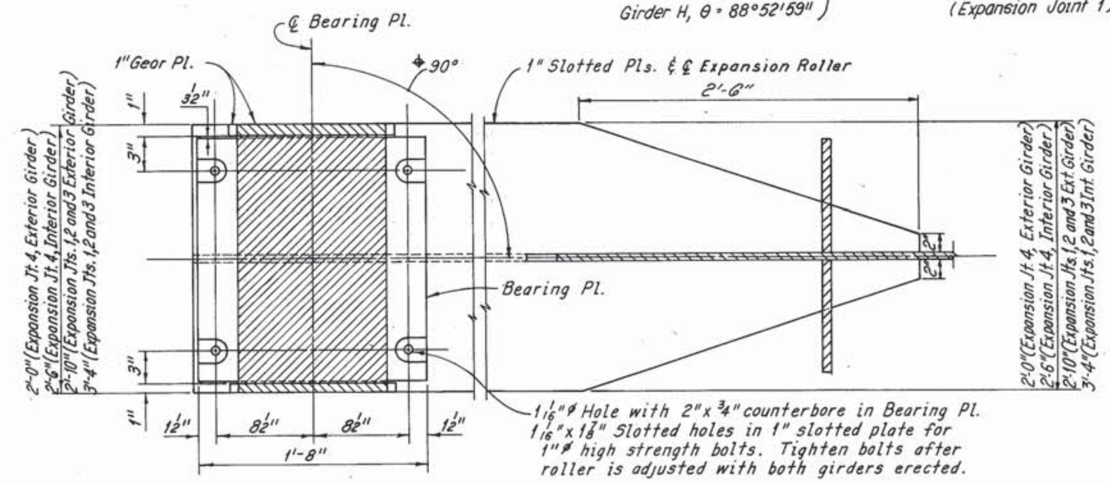
ROLLER DETAIL



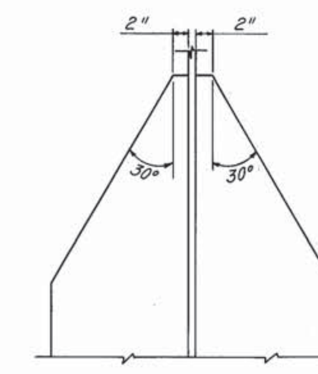
SECTION M-M



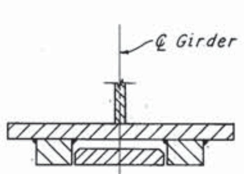
♦Note:
The upper bearing plates at expansion joint 1, shall be rotated with respect to Unit 1 Girders F, G and H.



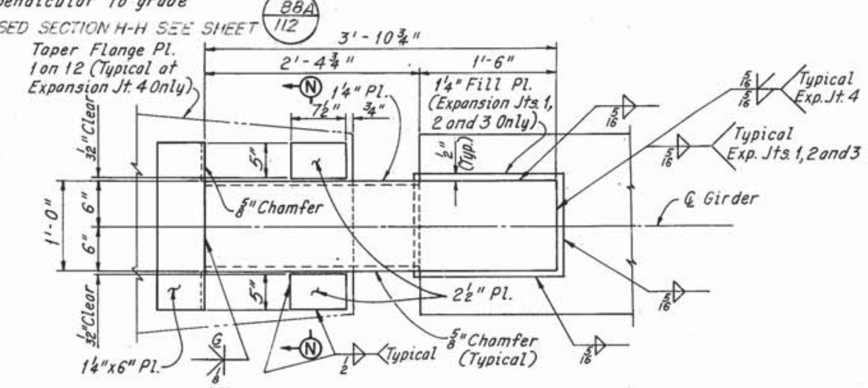
PART SECTION L-L



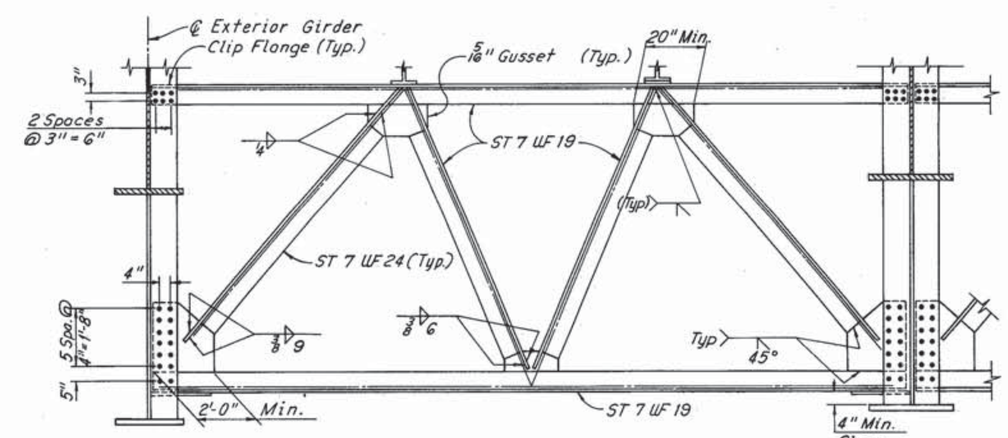
PARTIAL SECTION P-P



SECTION N-N



VIEW K-K



SECTION J - J
(Section J-J is shown for the supported span, Section J-J for the supporting span is similar)

Notes:
Rollers, Gear Plates and Bearing Plates shall be alloy steel forgings conforming to ASTM A237-63T, Class B with a minimum yield strength of 55,000 p.s.i. and shall be paid for at the unit price bid for "Item 513, Structural Steel (A36)".
For girder web shop splices see Sheet 10/28.
For material identification in expansion joint see Sheet 18/28.

H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

INTERMEDIATE EXPANSION JOINT AND CROSSFRAME DETAILS

I-80 OVER CUYAHOGA RIVER VALLEY
BR. NO. CUY-80-1843
STA. 996+22.25 TO STA. 1037+77.75

CUYAHOGA COUNTY OHIO

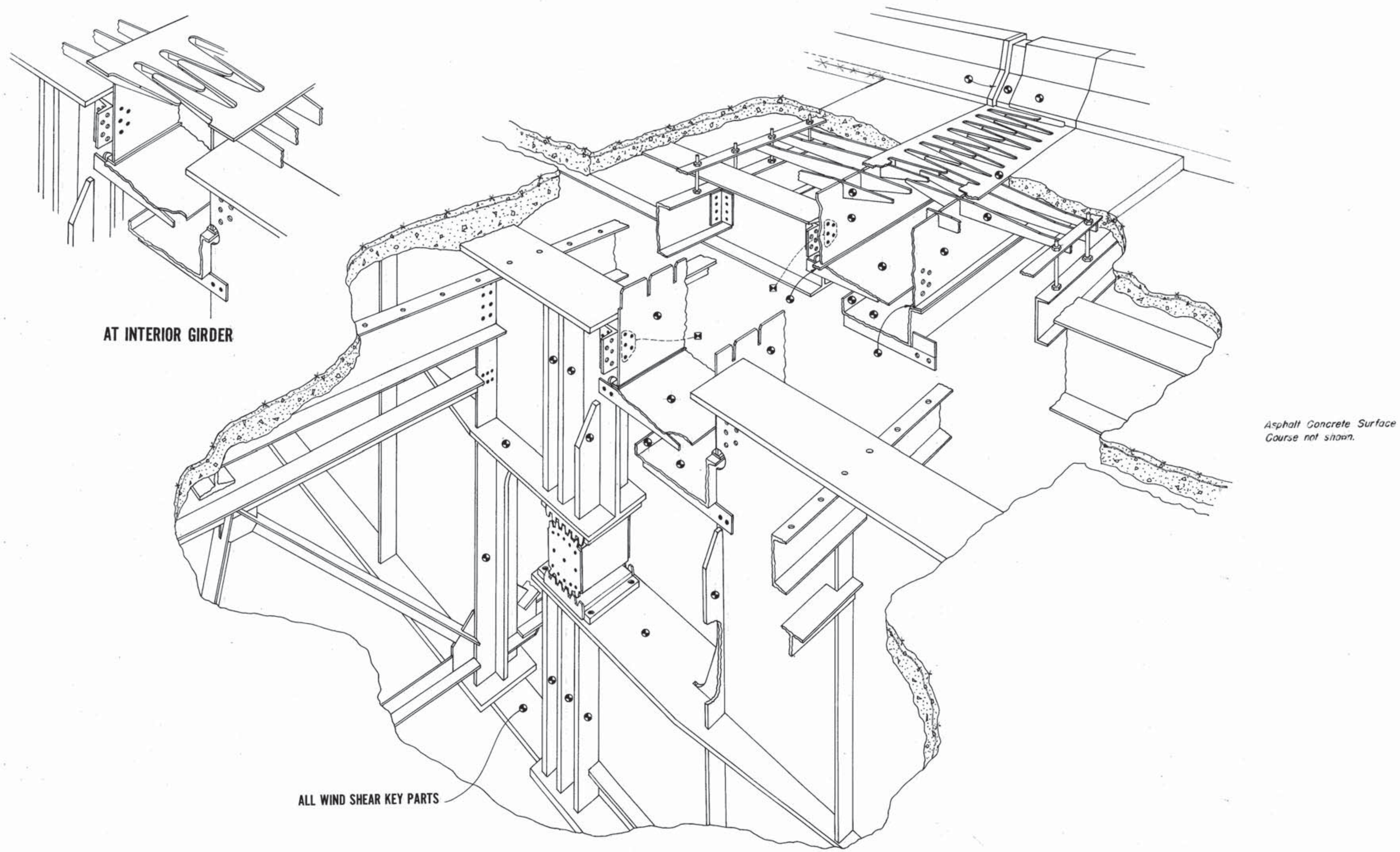
DRAWN CHO DATE 1-15-68	TRACED CP DATE 2-15-69	CHECKED JEH DATE 2-17-69	REVIEWED WF DATE 5-22-70	REVISED	SHEET 17/28

REV. 8-15-73

FED.RD. DIVISION	STATE	PROJECT	
2	OHIO		

90/112

CUYAHOGA COUNTY
CUY–80–18.43 Part 2



MATERIAL DESIGNATIONS (ALL LIKE PARTS)

- A.S.T.M. A-588 STRUCTURAL STEEL
- BOLTS WITH A.S.T.M. A-588 CORROSION CHARACTERISTICS AND A.S.T.M. A-325 MECHANICAL PROPERTIES

SEE PLANS FOR MATERIAL TYPES OF REMAINING PARTS

H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

PERSPECTIVE
INTERMEDIATE EXPANSION
JOINT

I-80 OVER CUYAHOGA RIVER VALLEY

BR. NO. CUY–80–1843 L & R STA. 996+22.25 TO STA. 1037+77.75

CUYAHOGA COUNTY OHIO

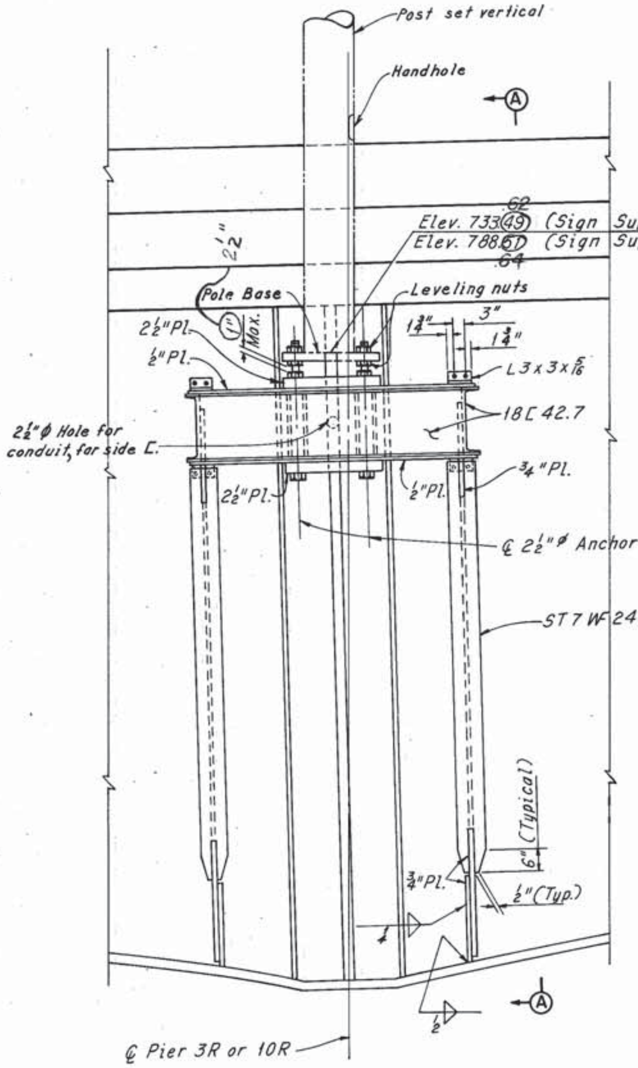
DRAWN H.R.H.	TRACED	CHECKED	REVIEWED WF	REVISED
DATE 3-26-69	DATE	DATE 4-4-69	DATE 5-22-70	SHEET 18/28

REV. 8-15-73

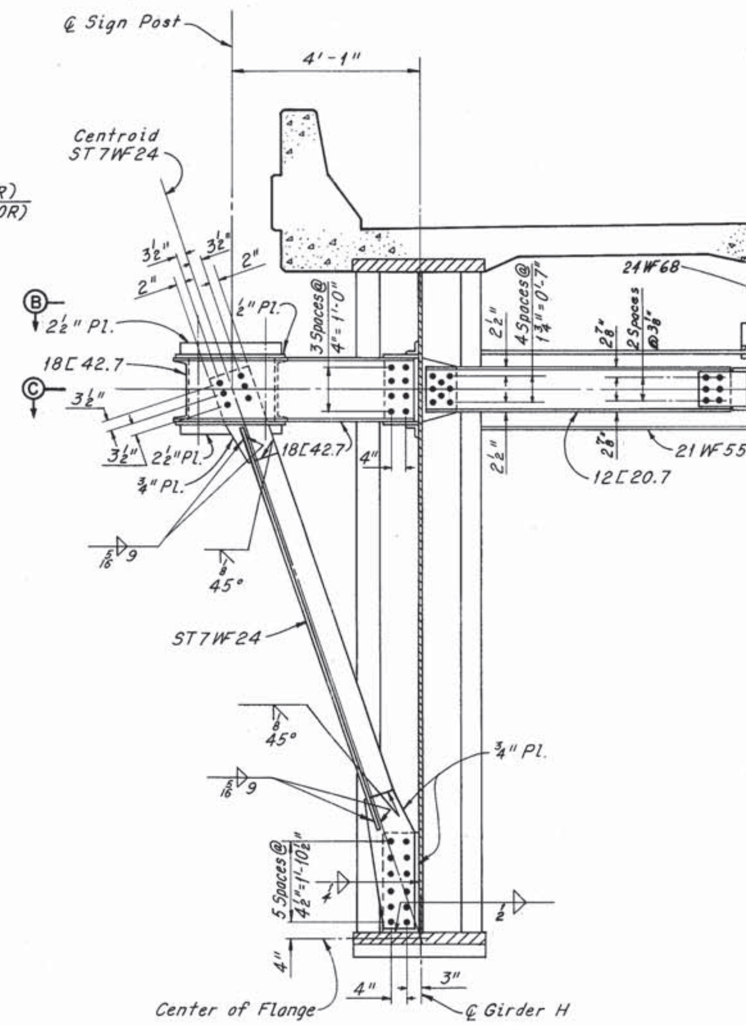
FED. RD. DIVISION	STATE	PROJECT		
2	OHIO			

91/112

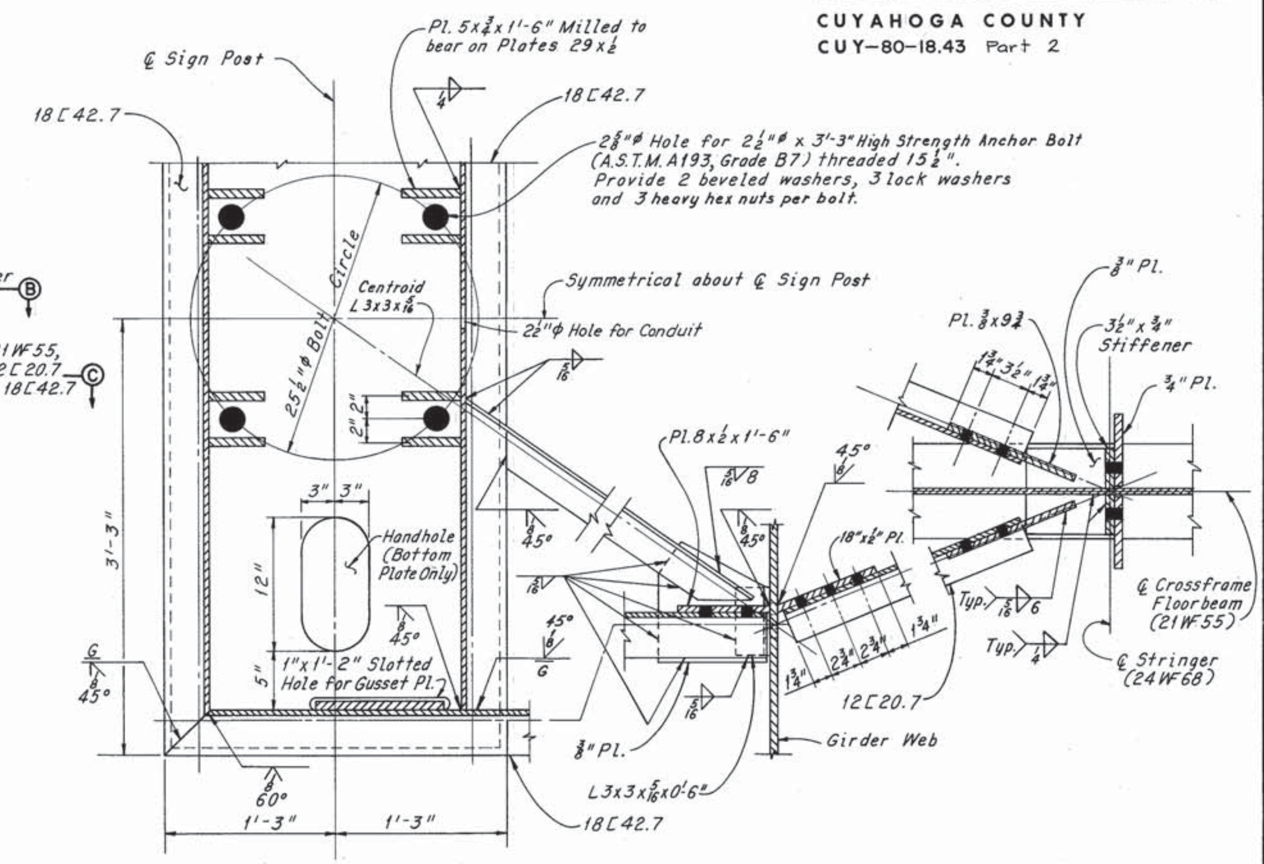
CUYAHOGA COUNTY
CUY-80-18.43 Part 2



ELEVATION

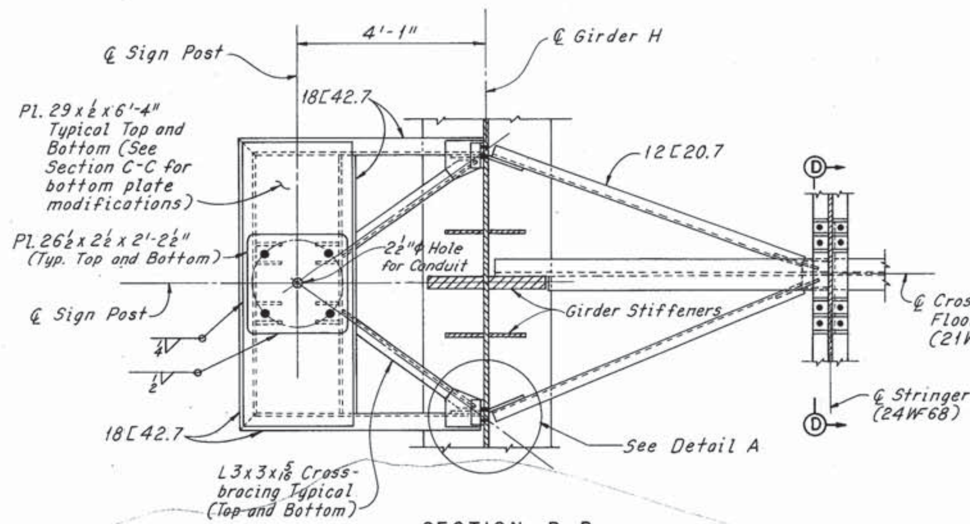


SECTION A-A

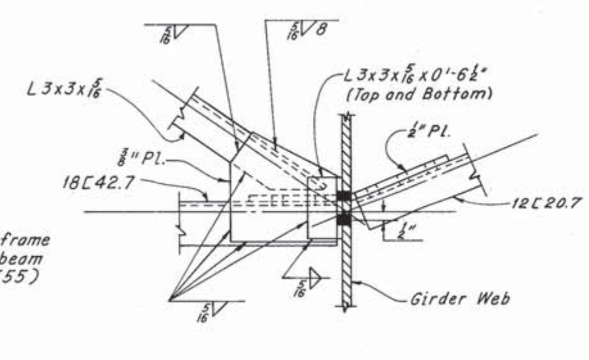


SECTION C-C

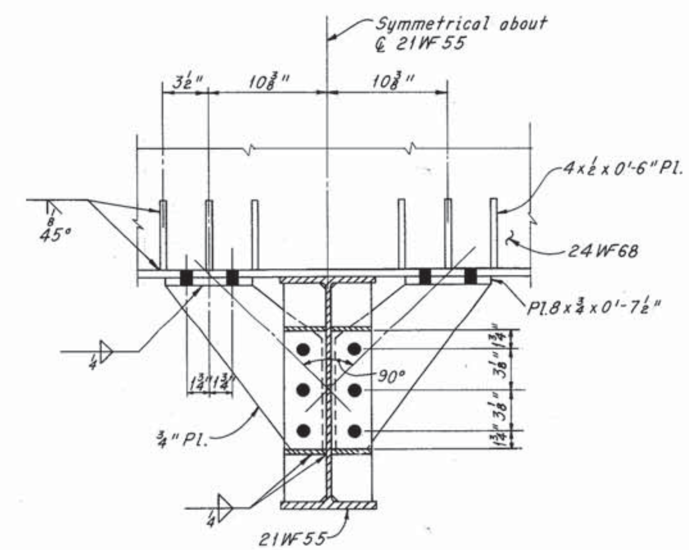
Note:
Only 21WF55 of crossframe has been shown. For details of crossframe and lateral bracing see Sheets 13/28 and 15/28.



SECTION B-B



DETAIL A



SECTION D-D
(Pl. $\frac{3}{8}$x$9\frac{3}{4}$ not shown)

Notes:
For signing and sign post details see Traffic Control Plans.
All connections shall be made with 1" Ø high strength steel bolts.
All sign support parts, except as otherwise noted, on the outside of the exterior girders shall be A.S.T.M. A-588 structural steel. All parts inside of the exterior girder shall be A.S.T.M. A-36 structural steel

H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

SIGN SUPPORT NO. 100 AND NO. 103

I-80 OVER CUYAHOGA RIVER VALLEY
BR. NO. CUY-80-1843 L&R STA. 996+22.25 TO STA. 1037+77.75
CUYAHOGA COUNTY OHIO

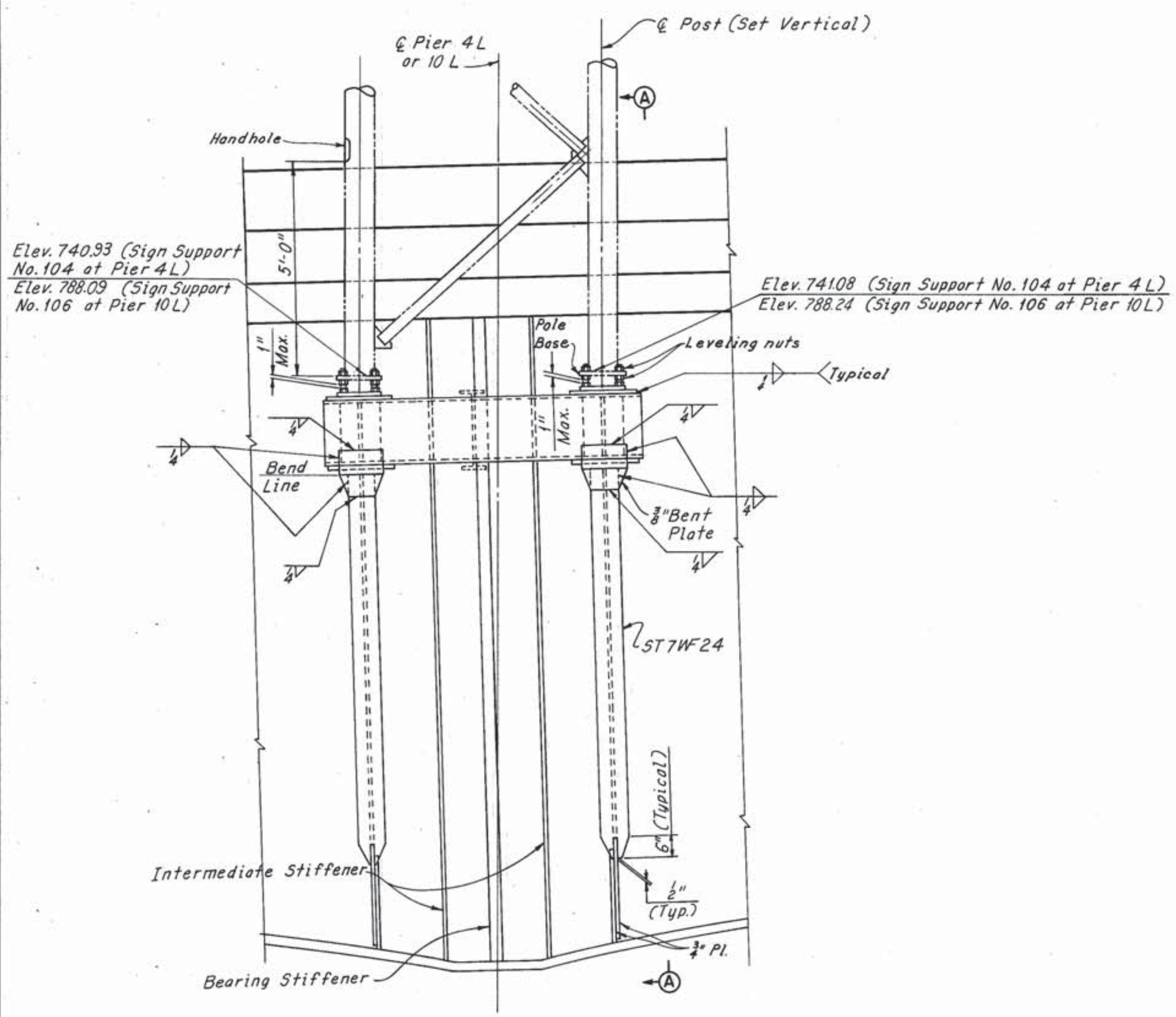
DRAWN C.H.B. DATE 3-7-69	TRACED C.P. DATE 3-13-69	CHECKED J.M.S. DATE 7-16-70	REVIEWED WF DATE 8-21-70	REVISED	SHEET 19/28

REV. 8-15-73

FED. RD. DIVISION	STATE	PROJECT		
2	OHIO			

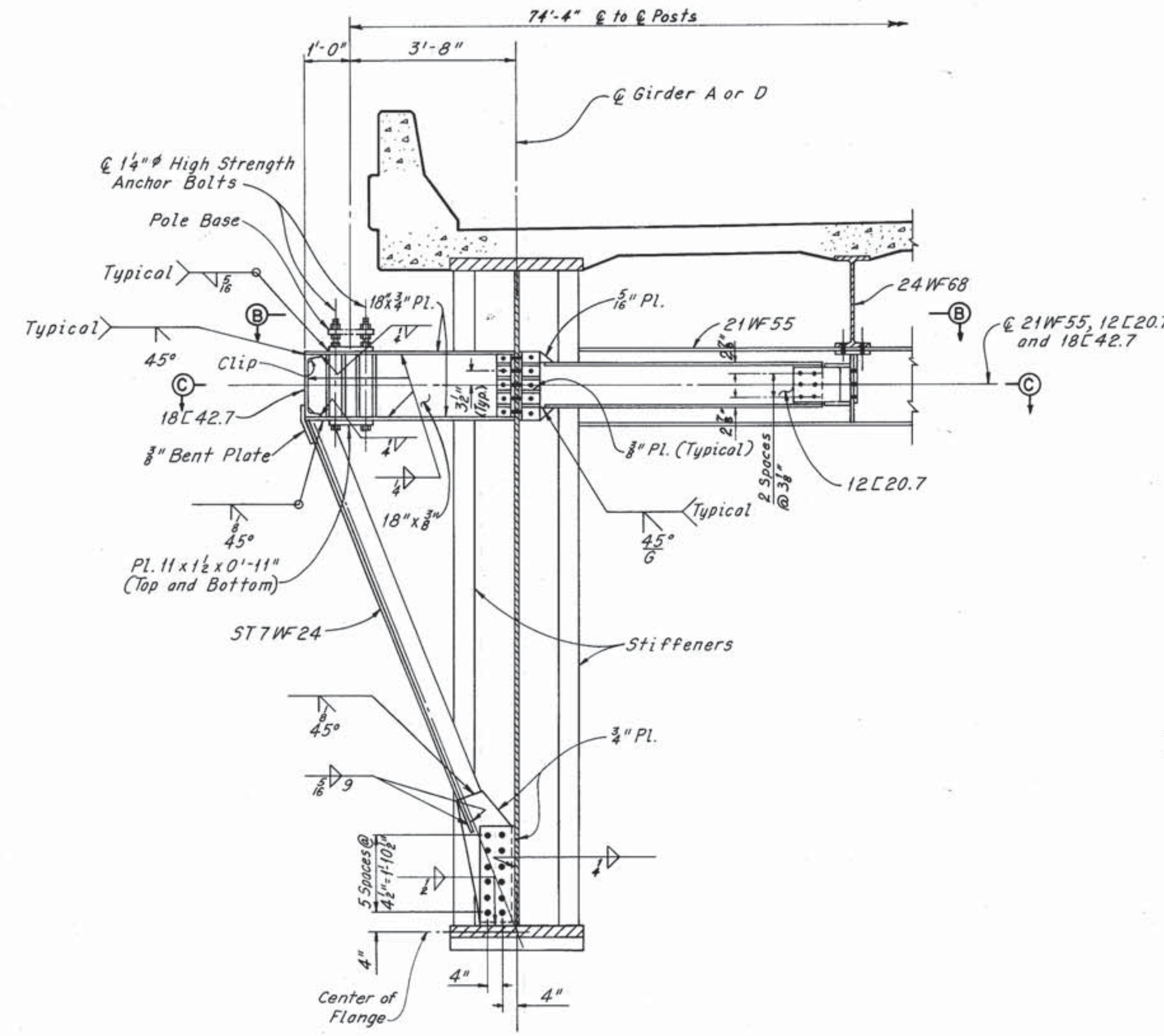
92/112

CUYAHOGA COUNTY
CUY-80-18.43 Part 2



SOUTH ELEVATION

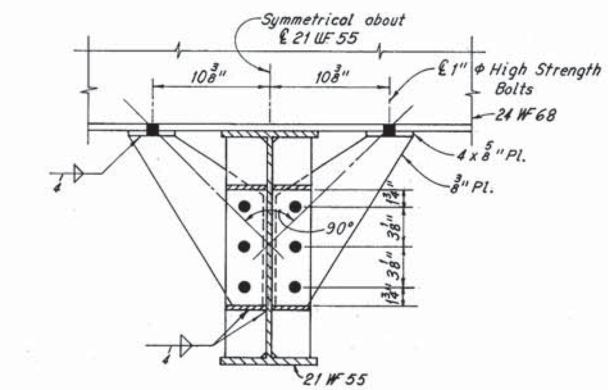
Girder D shown, Details at Girder A are Similar.



SECTION A-A

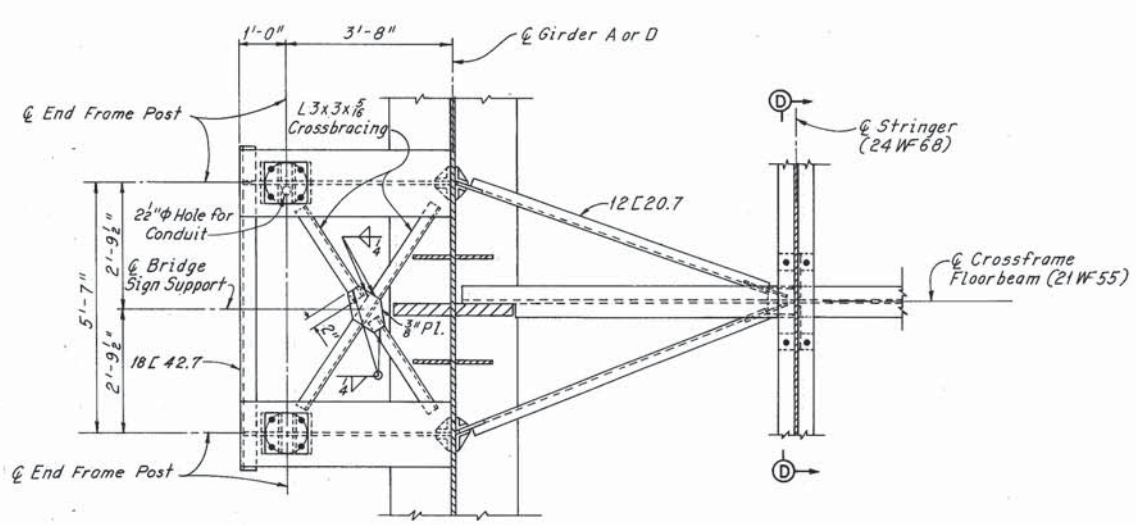
(Cross Bracing and Sign Post not shown)

Note:
Only 21WF55 of crossframe has been shown. For details of crossframe and lateral bracing see Sheets 13/28 and 15/28



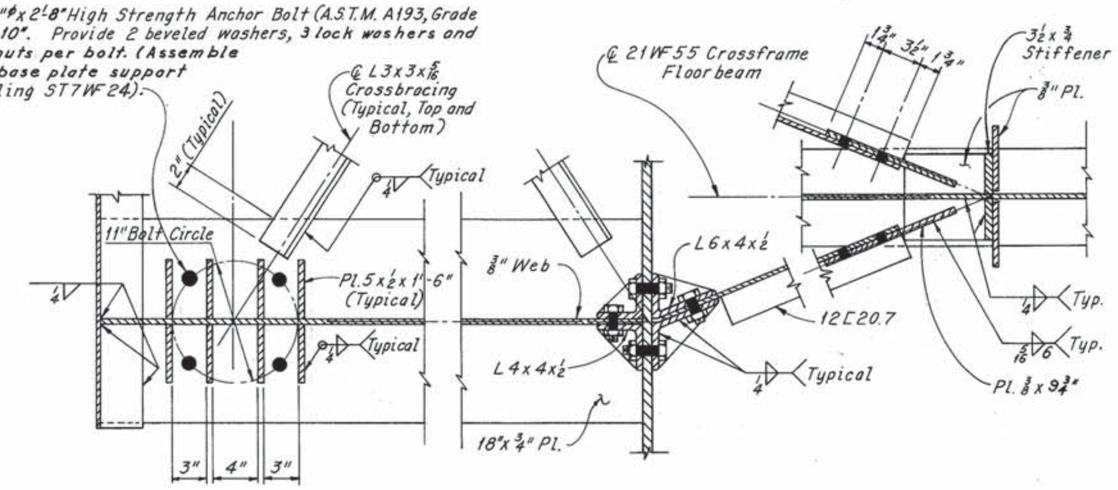
SECTION D-D

(Pl. ⅜ x 9¾" not shown)



SECTION B-B

1⅜"ϕHole for 1¼"ϕx2'-8" High Strength Anchor Bolt (A.S.T.M. A193, Grade B7) threaded 10". Provide 2 beveled washers, 3 lock washers and 3 heavy hex nuts per bolt. (Assemble and tighten base plate support before installing ST7WF24).



SECTION C-C

For notes see Sheet 19/28

H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

SIGN SUPPORT NO. 104 AND NO. 106

I-80 OVER CUYAHOGA RIVER VALLEY

BR. NO. CUY-80-1843 L&R — STA. 996+22.25 TO STA. 1037+77.75

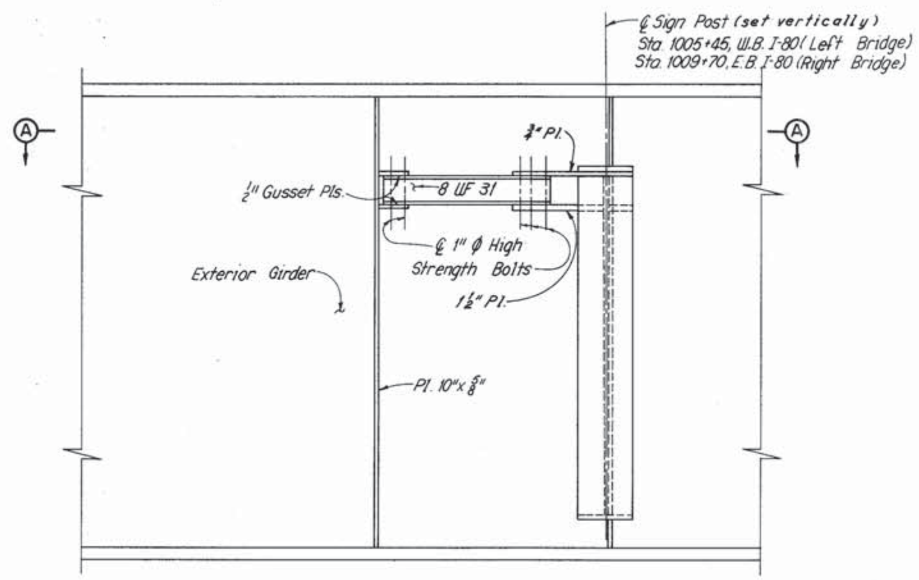
CUYAHOGA COUNTY — OHIO

DRAWN C.H.B. DATE 3-14-69	TRACED C.P. DATE 3-17-69	CHECKED J.M.S. DATE 7-16-70	REVIEWED W.F. DATE 9-22-70	REVISED	SHEET 20/28

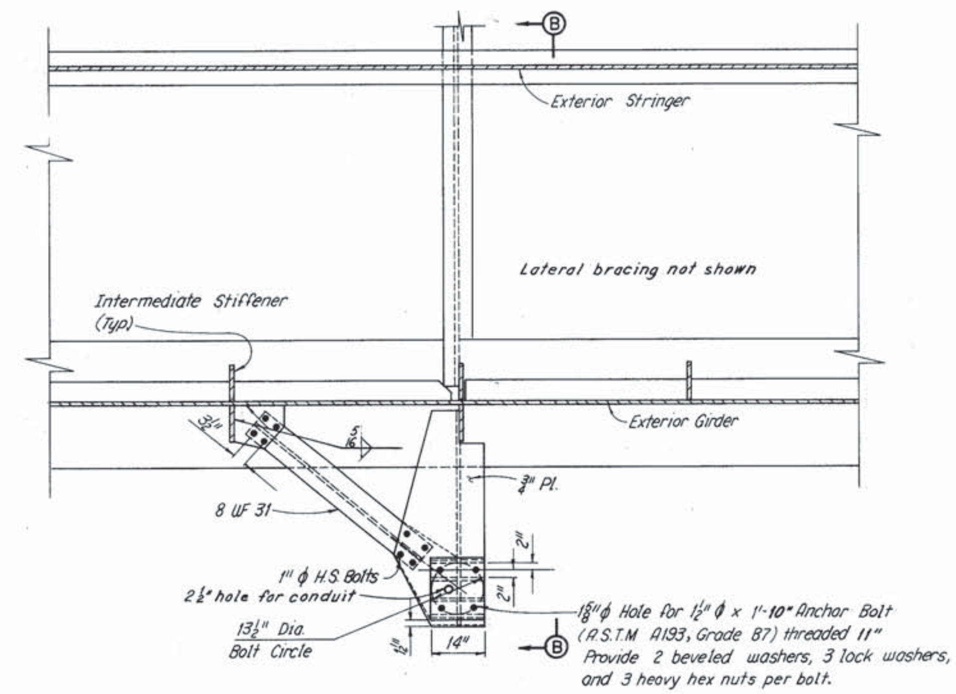
FED. RD. DIVISION	STATE	PROJECT	
2	OHIO		

93/112

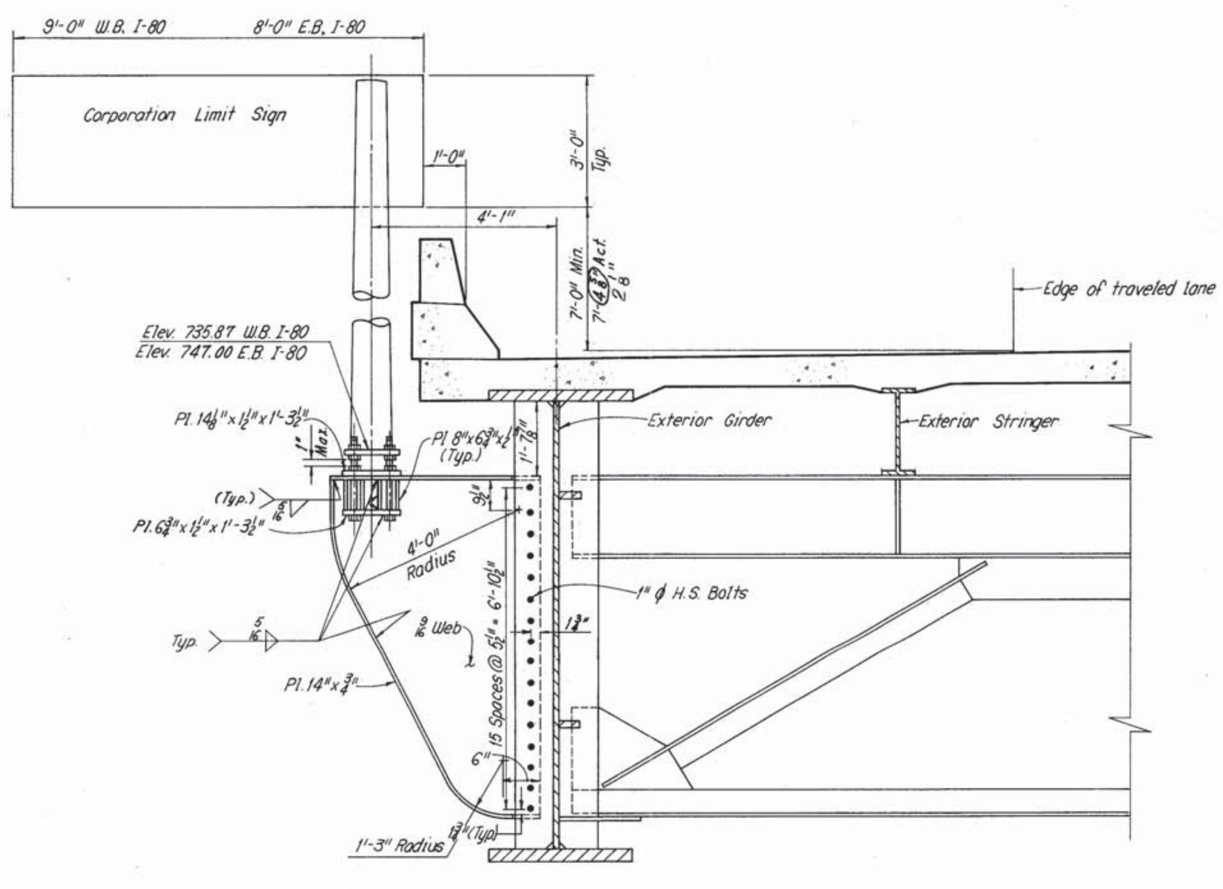
CUYAHOGA COUNTY
CUY-80-18.43 Part 2



ELEVATION



SECTION A—A



SECTION B—B

Note:
For notes see Sheet 19/28.

H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

CORPORATION LIMIT SIGN SUPPORTS

I-80 OVER CUYAHOGA RIVER VALLEY

BR. NO. CUY-80-1843 L & R STA. 996+22.25 TO STA. 1037+77.75

OHIO

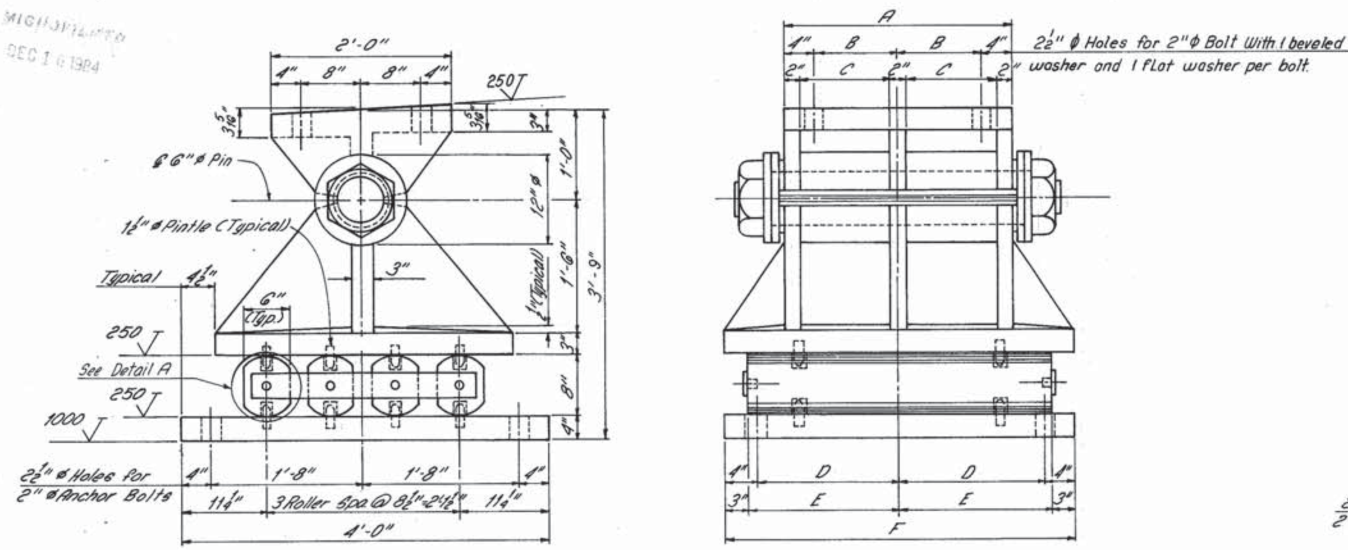
DRAWN MCB	TRACED AJT	CHECKED JMS	REVIEWED WF	REVISED
DATE 6-17-70	DATE 6-19-70	DATE 7-16-70	DATE 8-21-70	SHEET 21/28

REV. 8-15-73

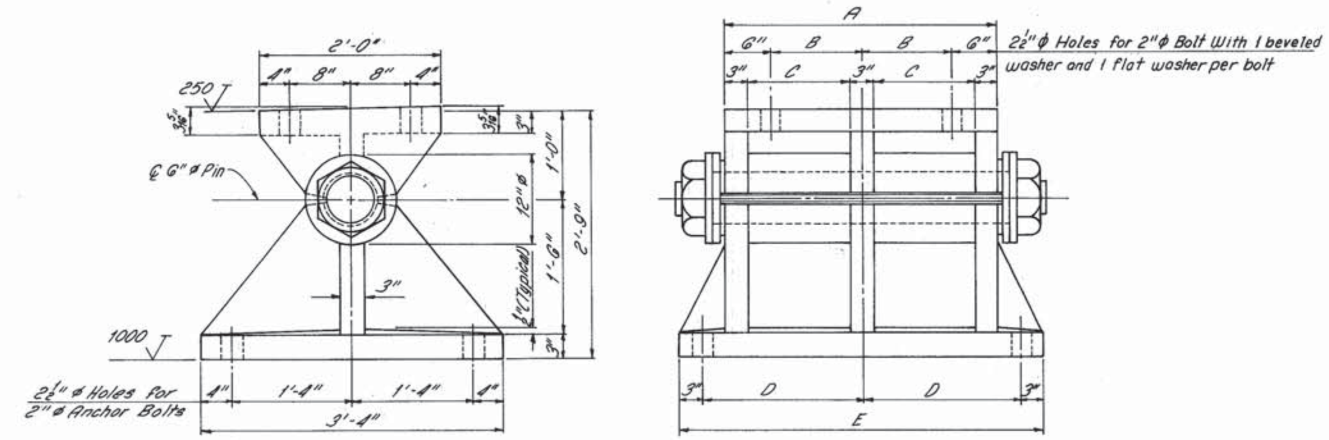
FED. RD. DIVISION	STATE	PROJECT	
2	OHIO		

94
112

CUYAHOGA COUNTY
CUY 80-18.43 . Part 2



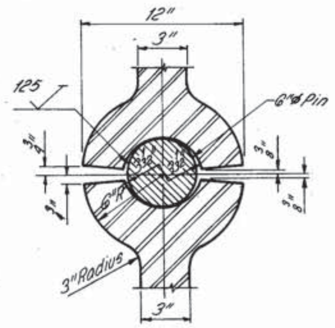
EXPANSION BEARING



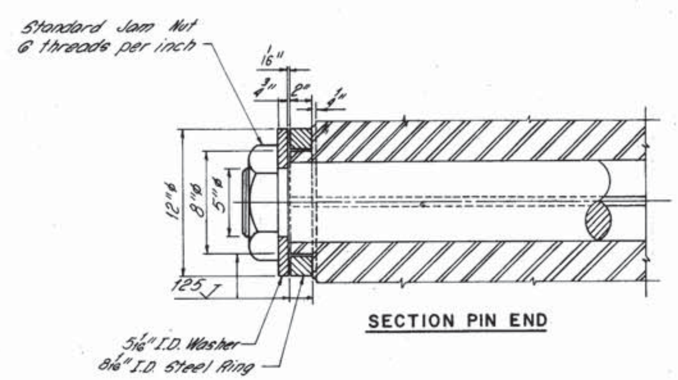
FIXED BEARING

BEARING DIMENSIONS											
BEARING	LOCATION	NO.REQ'D	GIRDER	TYPE	A	B	C	D	E	F	WEIGHT
FB-A	PIERS 1,4, 5,7,8,10 and 11	28	Interior	Fixed	3'-0"	1'-0"	1'-1½"	1'-9"	4'-0"	——	159,012
EB-B		28	Exterior	Exp.	2'-6"	11"	1'-0"	1'-7"	1'-8"	3'-10"	259,924
FB-C	PIER 14	4	Interior	Fixed	2'-0"	6"	7½"	1'-3"	3'-0"	——	17,844
EB-D		4	Exterior	Exp.	1'-8"	6"	7"	1'-1"	1'-2"	2'-10"	27,800

Note: Weights given are total for all bearings, including high strength bolts, nuts and washers and ⅛" sheet lead.

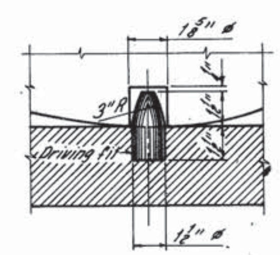


SECTION THRU PIN

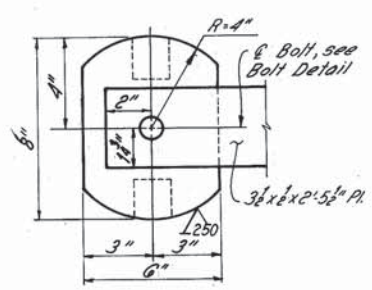


SECTION PIN END

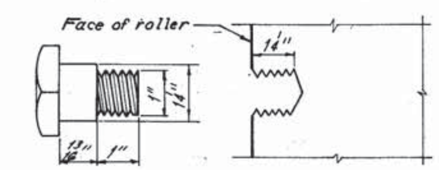
PIN DETAILS



PINTLE DETAIL



DETAIL A



BOLT DETAIL

BEARINGS

Notes:

Rockers for Abutments shall conform to the requirements set forth by Ohio Standard Drawing RB-1-55.

All castings shall be ASTM A 486, Class 90 Cast Steel, with a 60,000 p.s.i. minimum yield point. All fillets shall be ¾" radius, except as shown.

All rollers, pins, base plates, gears and gear racks shall be ASTM A 237, Class B Steel Forgings, with a 55,000 p.s.i. minimum yield point.

All nuts, bolts, washers and rings shall be Structural Carbon Steel (A 36).

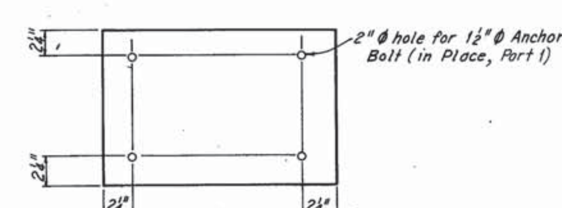
Provide ⅛" sheet lead between masonry and bearing and fill space around anchor bolts with molten lead before placing nuts.

All base plates and castings shall be scribed with longitudinal and transverse center lines.

ABUTMENT ROCKER DIMENSIONS (Inches)															
ROCKER NO.	NO. REQ'D	A	B	C	D	F	G	H	K	L	M	R	T	Y	WEIGHT
Mod. R-300	4	3½	22	3½	3¼	¾	12	19⅛	14	28	25	12½	3	1 11/16	4,392
R-350	2	3½	22	4	3¼	⅞	12	20⅞	15	30	25	13¼	3½	1 11/16	2,586
R-425	6	3½	24	4	3½	1	12	22⅛	16	32	28	14½	3½	1 11/16	9,414
R-500	2	4	26	4	3¾	1	13	24⅛	17	34	31	16	4	1 15/16	3,848
R-600	2	4	27	4	4	1	14	25⅜	22	37	34	17	4¼	1 15/16	4,938

Note: For dimension locations and details see Ohio Standard Drawing RB-1-55, revised 2-2-59. Rocker details, except for anchor bolt holes in base plate, shall be the same as given on Ohio Standard Drawing RB-1-55, but with dimensions as shown here. Bevel top sole plates to match grade.

ROCKERS—ABUTMENTS



ROCKER BASE PLATE

H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

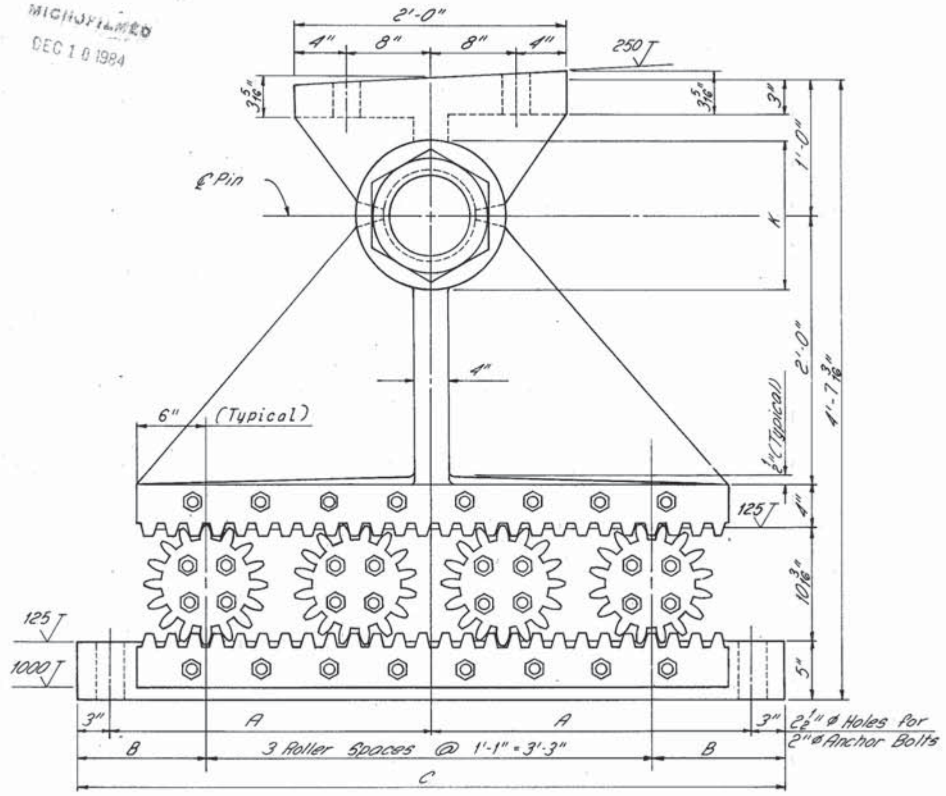
BEARINGS – PIERS 1, 4, 5, 7, 8, 10, 11 AND 14
ROCKERS – ABUTMENTS
I-80 OVER CUYAHOGA RIVER VALLEY
BR. NO. CUY-80-1843 STA. 996+22.25 TO STA. 1037+77.75
CUYAHOGA COUNTY OHIO

DRAWN	TRACED	CHECKED	REVIEWED	REVISED	
DRAWN TJS DATE 8-68	TRACED AJJ DATE 8-68	CHECKED CHG DATE 1-16-69	REVIEWED WF DATE 8-21-70		SHEET 22/28

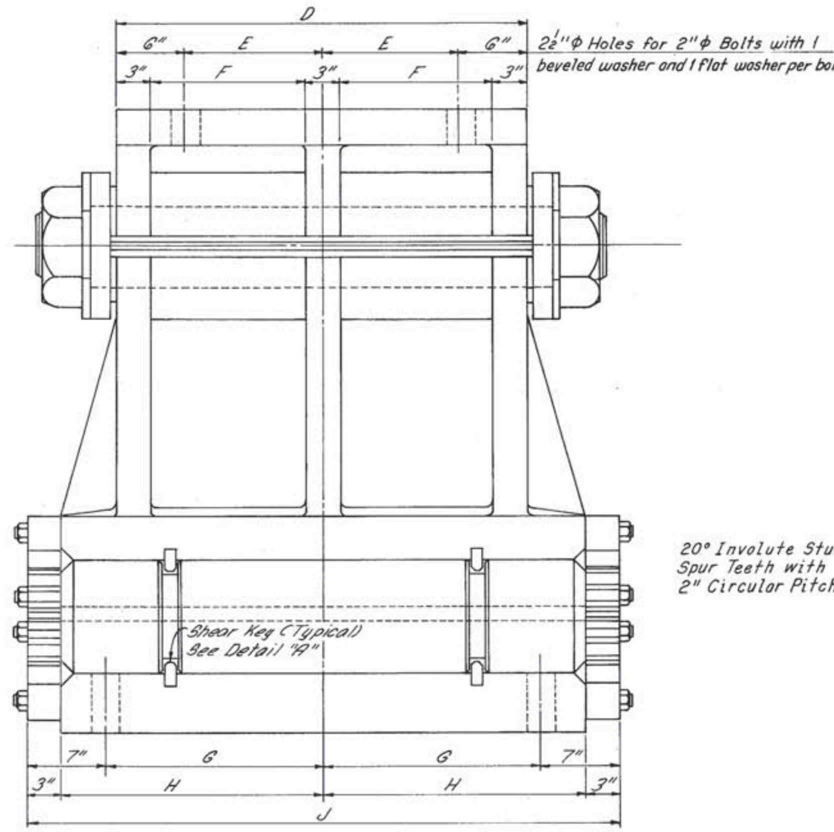
FED. RD. DIVISION	STATE	PROJECT		
2	OHIO			

95/112

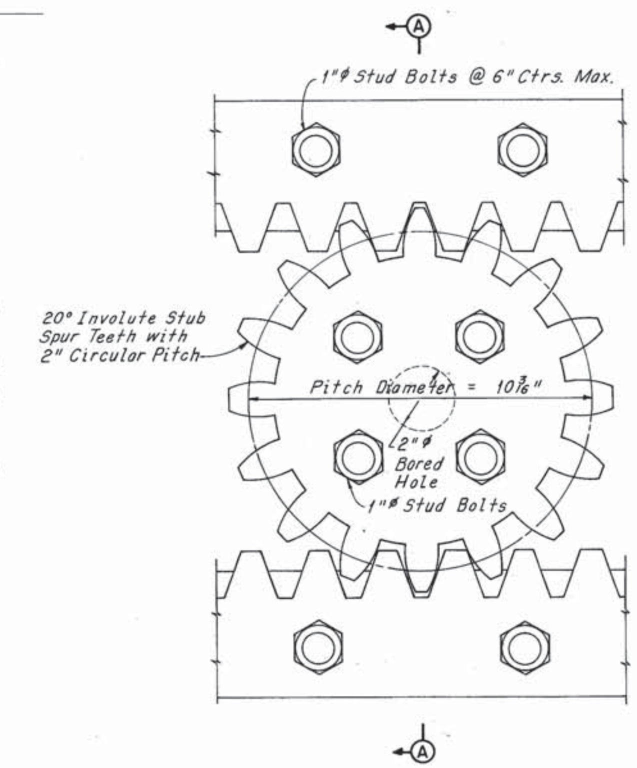
CUYAHOGA COUNTY
CUY -80-1843 Part 2



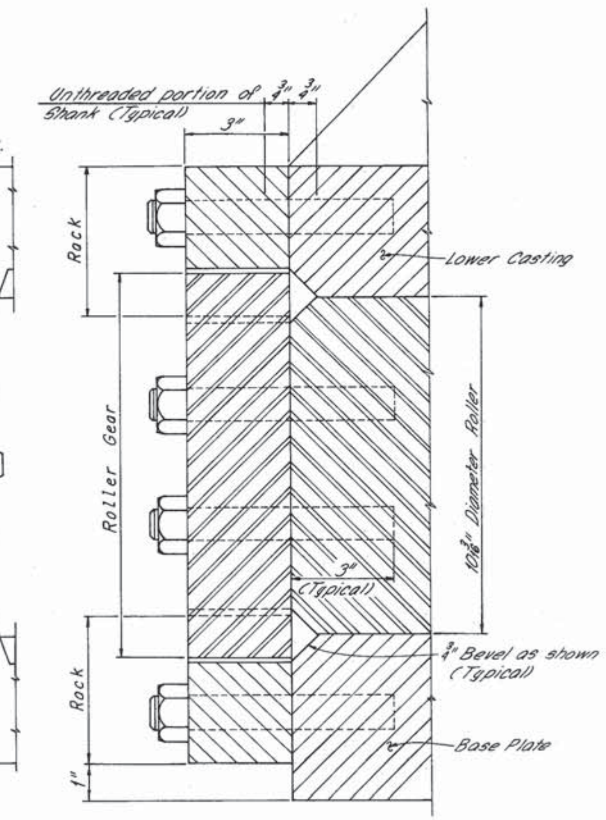
SIDE ELEVATION



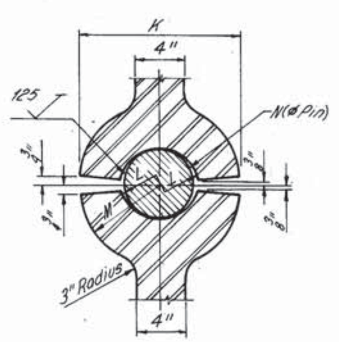
ELEVATION



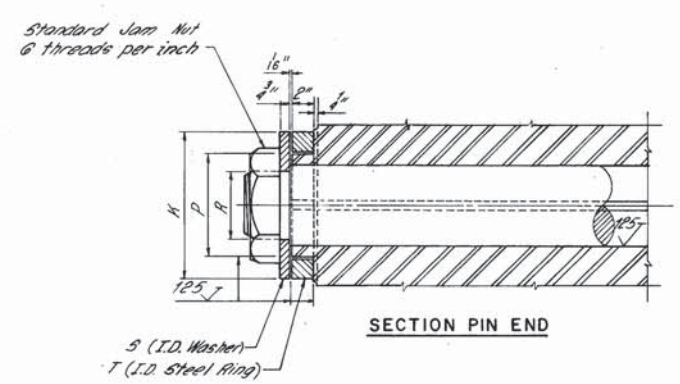
ELEVATION OF GEARING



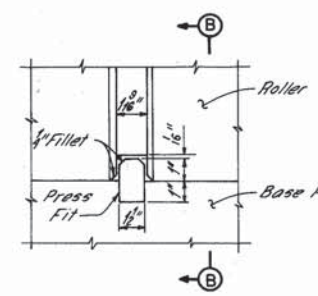
SECTION A-A



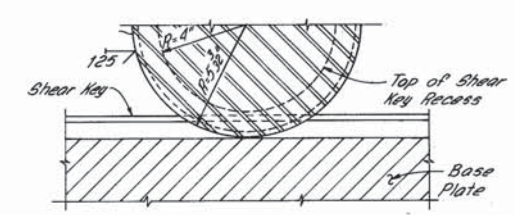
SECTION THRU PIN



SECTION PIN END



DETAIL A



SECTION B-B

For notes see Sheet 22/28.

BEARING DIMENSIONS																				
BEARING	LOCATION	NO.REQ'D	GIRDER	A	B	C	D	E	F	G	H	J	K	L	M	N	P	R	S	T	LBS.
EB-E	PIER 2	4	Interior	2'-3"	11"	5'-0"	3'-0"	1'-0"	1'-1 1/2"	1'-7"	1'-11"	4'-4"	13"	3 17/32"	6 1/8"	7"	9"	6"	6 1/16"	9 1/16"	68,808
EB-F		4	Exterior	2'-3"	11"	5'-0"	2'-6"	9"	10 1/2"	1'-2"	1'-6"	3'-6"	12"	3 1/32"	6"	6"	8"	5"	5 1/16"	8 1/16"	56,576
EB-G	PIERS 3,	12	Interior	2'-8 1/2"	1'-4"	5'-11"	3'-0"	1'-0"	1'-1 1/2"	1'-2"	1'-6"	3'-6"	11"	2 17/32"	5 1/8"	5"	7"	4"	4 1/16"	7 1/16"	190,248
EB-H	6 AND 9	12	Exterior	2'-8 1/2"	1'-4"	5'-11"	2'-6"	9"	10 1/2"	11"	1'-3"	3'-0"	11"	2 17/32"	5 1/8"	5"	7"	4"	4 1/16"	7 1/16"	166,392
EB-J	PIERS 12	8	Interior	2'-4"	1'-0"	5'-2"	2'-0"	6"	7 1/2"	1'-0 1/2"	1'-4 1/2"	3'-3"	13"	3 17/32"	6 1/8"	7"	9"	6"	6 1/16"	9 1/16"	107,728
EB-K	AND 13	8	Exterior	2'-4"	1'-0"	5'-2"	1'-8"	4"	5 1/2"	8 1/2"	1'-0 1/2"	2'-7"	12"	3 1/32"	6"	6"	8"	5"	5 1/16"	8 1/16"	88,288

Note: Weights given are total for all bearings, including high strength bolts, nuts and washers and 1/8" sheet lead bearing pad.

H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

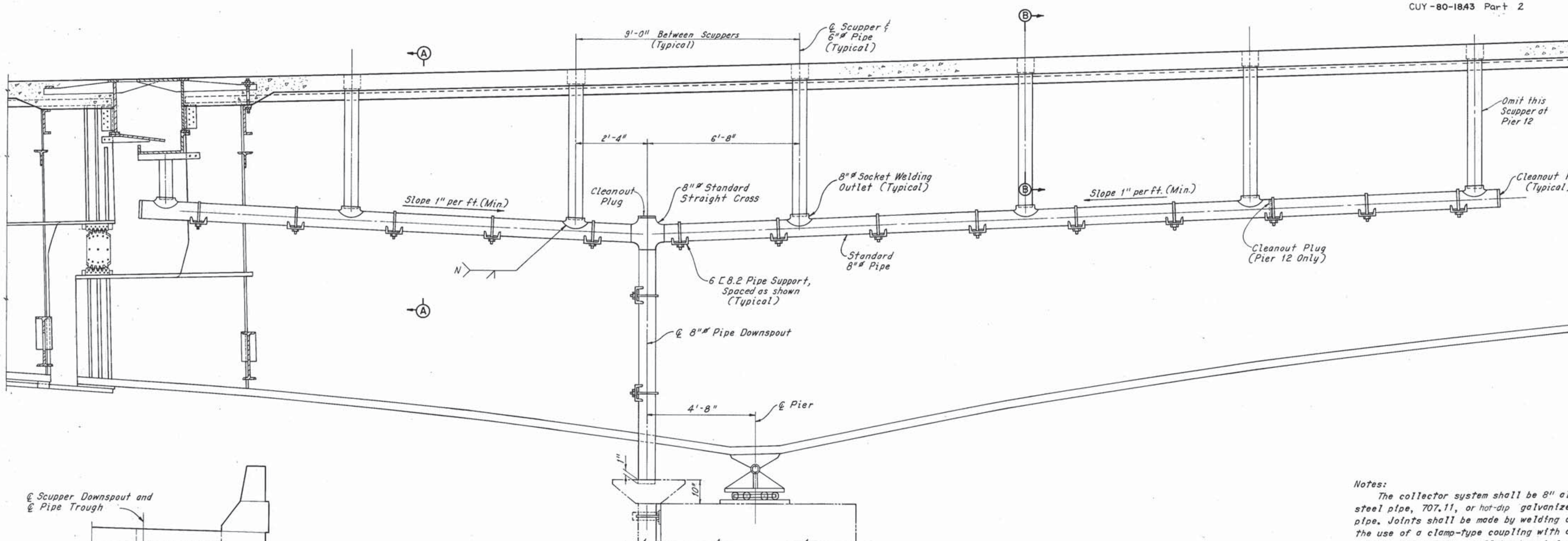
BEARINGS - PIERS 2, 3, 6, 9, 12 AND 13

I-80 OVER CUYAHOGA RIVER VALLEY
BR. NO. CUY-80-1843 STA. 996+22.25 TO STA. 1037+77.75

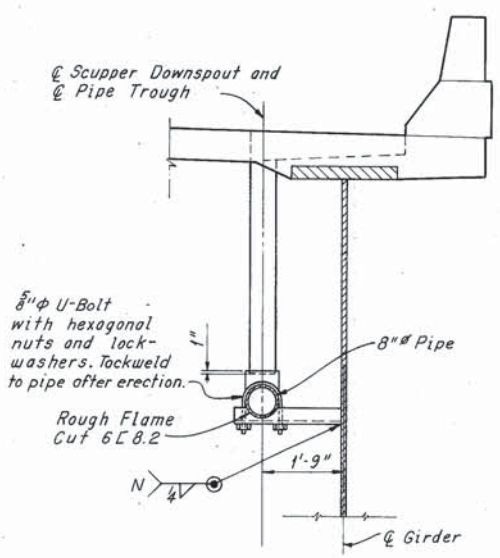
CUYAHOGA COUNTY OHIO

DRAWN TJS	TRACED RJJ	CHECKED	REVIEWED WF	REVISED	
DATE 8-68	DATE 8-68	DATE 1-14-69	DATE 8-22-70		SHEET 23/28

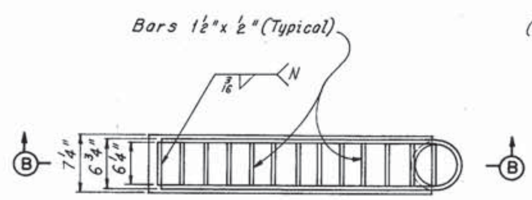
CUYAHOGA COUNTY
CUY-80-1843 Part 2



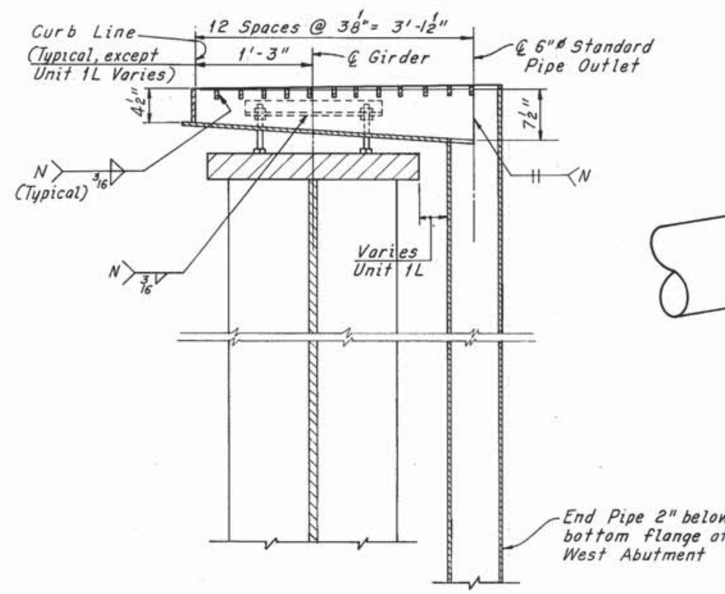
COLLECTOR SYSTEM AT PIERS 3, 6 AND 9
(Similar at Pier 12, except as noted. No Collector System at West Abutments)



SECTION A-A

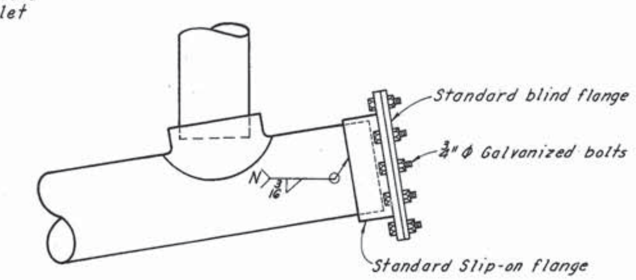


TYPICAL SCUPPER PLAN



SECTION B-B

FOR SCUPPER MODIFICATION SEE SHEET 86A/112



TYPICAL CLEANOUT DETAIL

PIPE SUPPORTS AT WEST ABUTMENTS

Each 6" ø scupper pipe at the West Abutments shall be supported by two 6[8.2 channel supports with ⅝" ø U-bolts, one 2'-0" above the bottom flange and the other 7'-0" above the bottom flange.

Notes:

The collector system shall be 8" alloy steel pipe, 707.11, or hot-dip galvanized steel pipe. Joints shall be made by welding or by the use of a clamp-type coupling with a ring gasket. All welding shall be done before galvanizing. Pipe supports shall be A588 steel or hot-dip galvanized steel. On bolts, galvanizing as called for in ASTM A153 is acceptabale.

The 8" ø pipe collector system including fittings, supports and accessories shall be paid for at the unit price bid for "Item 518, 8" ø Pipe Collector System."

Scuppers and supports shall be structural carbon steel ASTM A36. Support Ls 2½x2½x¼, pipe and supports at west abutment shall be included with "Item 518, Scuppers" for payment. For additional scupper details see Ohio Standard Drawing SD-1-69 Sheet 3 of 4.

H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

DRAINAGE DETAILS

I-80 OVER CUYAHOGA RIVER VALLEY
BR. NO. CUY-80-1843 STA. 996+22.25 TO STA. 1037+77.75
CUYAHOGA COUNTY OHIO

DRAWN T.J.S. DATE 9-11-68	TRACED C.P. DATE 9-30-68	CHECKED JEH DATE 2-17-69	REVIEWED WF DATE 8-21-70	REVISED	SHEET 24/28

REV. 8-15-73

Note: Longitudinal curb reinforcement per unit consists of 4 Lines of 32-S601 except as follows:
UNIT 1L - 4 Lines of 28-S601 and 1-S602.
UNIT 5L and 5R - 4 Lines of 23-S601 and 1-S602.

Note: A haunch width of 9" shall be used for computing quantity of concrete. However, the haunch width may vary between 6" and 12" provided that the slope shall be not more than 1:4 for a haunch less than 9" in width.
All Girder haunch depths are measured from top of web to top of pavement @ ℄ Bearing.

FED. RD. DIVISION	STATE	PROJECT		
2	OHIO			

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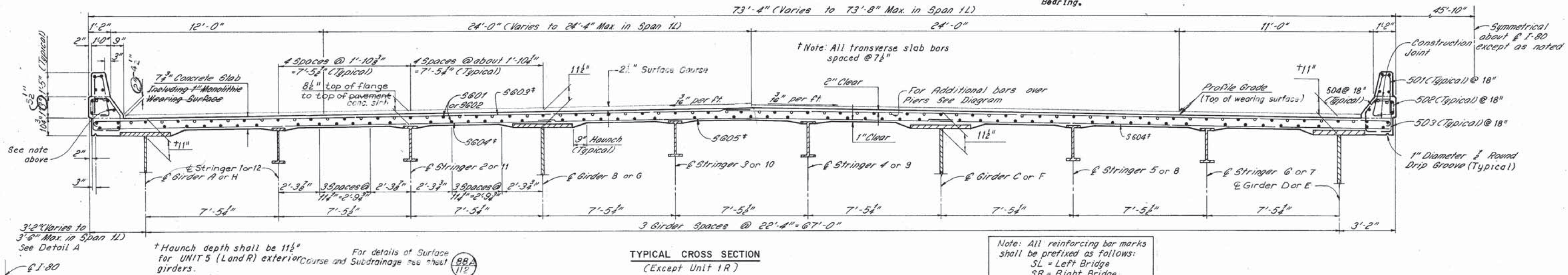
CUYAHOGA COUNTY
CUY-80-1843 Part 2

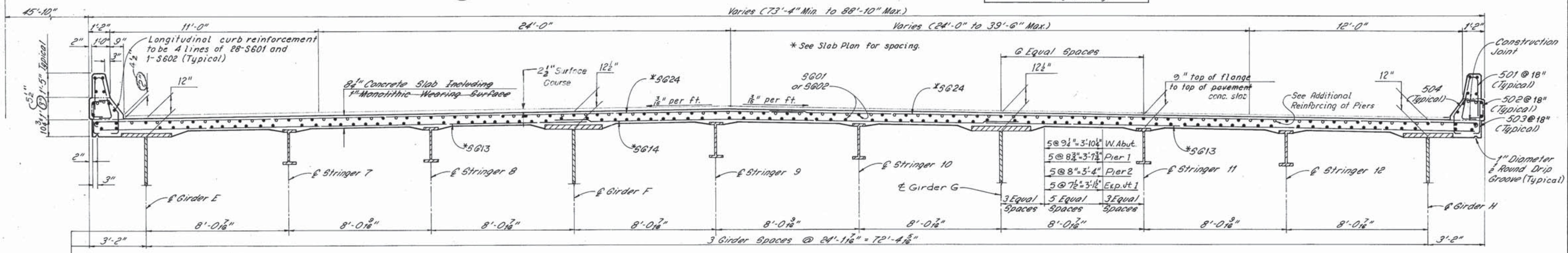


† Haunch depth shall be 11½" for UNIT 5 (L and R) exterior girders.

For details of Surface Course and Subdrainage see sheet 88A/112

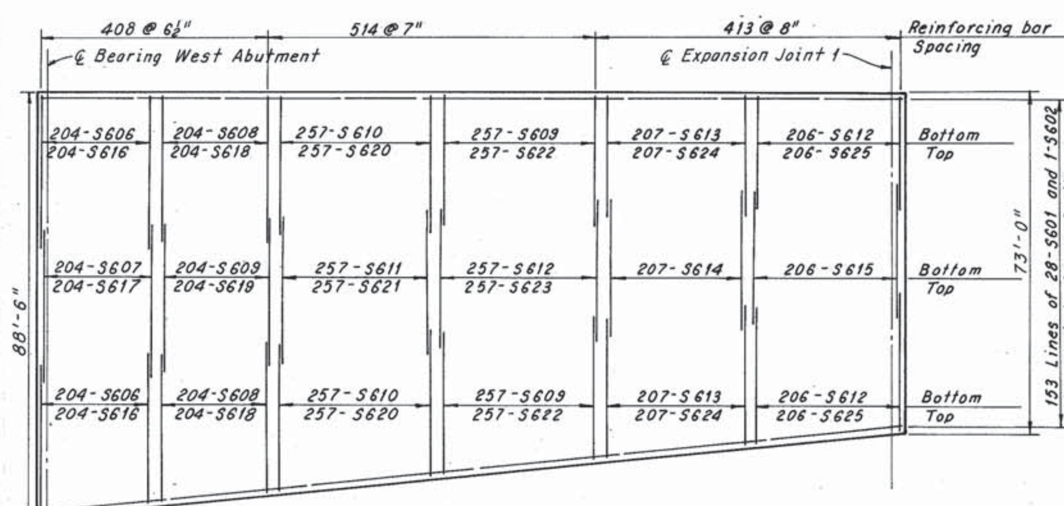
TYPICAL CROSS SECTION
(Except Unit 1R)

Note: All reinforcing bar marks shall be prefixed as follows:
SL = Left Bridge
SR = Right Bridge

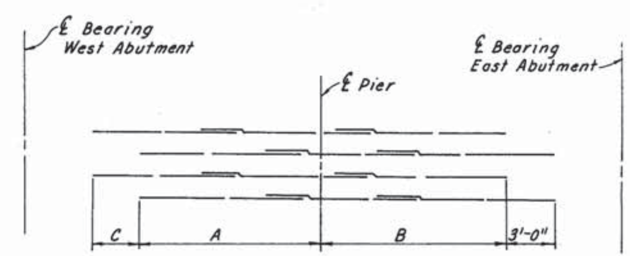


Dimensions are given for Pier 2R only
See Framing Plan, Sheet 5/28

TYPICAL CROSS SECTION - UNIT 1R



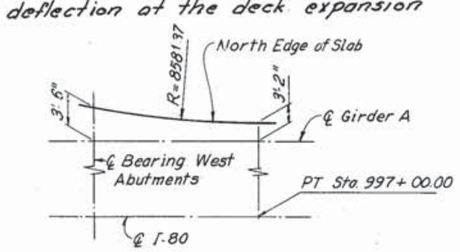
SLAB PLAN - UNIT 1R



ADDITIONAL REINFORCING BARS AT PIERS

Pier	A	B	C	No. of Bars
1L	57'-0"	57'-0"	3'-0"	42 Lines of 3-S626
1R	57'-0"	57'-0"	3'-0"	60 Lines of 3-S626
2L, 5L, 5R, 8L, 8R, 11L & 11R	63'-0"	63'-0"	3'-0"	42 Lines of 4-S627
2R	63'-0"	63'-0"	3'-0"	60 Lines of 4-S627
3L, 3R, 6L, 6R, 9L, 9R, 12L & 12R	21'-6"	30'-0"	0	42 Lines of 2-S628
4L, 4R, 7L, 7R, 10L & 10R	66'-0"	66'-0"	3'-0"	42 Lines of 4-S629
13L & 13R	52'-0"	52'-0"	3'-0"	42 Lines of 3-S630
14L & 14R	44'-0"	44'-0"	3'-0"	42 Lines of 3-S631

Note:
The Contractor shall submit a deck pouring sequence and method to the Director for approval.
Concrete shall be placed symmetrically about longitudinal ℄ of deck. No longitudinal construction joints will be permitted.
The deck shall be poured west to east, beginning at the West Abutment, due to design consideration of detrimental deflection at the deck expansion joints.



DETAIL A

SLAB REINFORCEMENT QUANTITIES				
Unit	Longitudinal reinforcement	S603	S604	S605
1L	99 Lines of 28-S601 and 1-S602	2548	2548	1274
1R	See Unit 1R Slab Plan	—	—	—
2, 3 and 4 (L and R)	99 Lines of 32-S601	2874	2874	1437
5L and 5R	99 Lines of 23-S601 and 1-S602	2090	2090	1045

Note:
For parapet joints and longitudinal reinforcement in the parapet see Sheet 26/28.

H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

TYPICAL CROSS SECTION AND DECK REINFORCEMENT

BR. NO. CUY-80-1843
STA. 996+22.25 TO STA. 1037+77.75
CUYAHOGA COUNTY OHIO

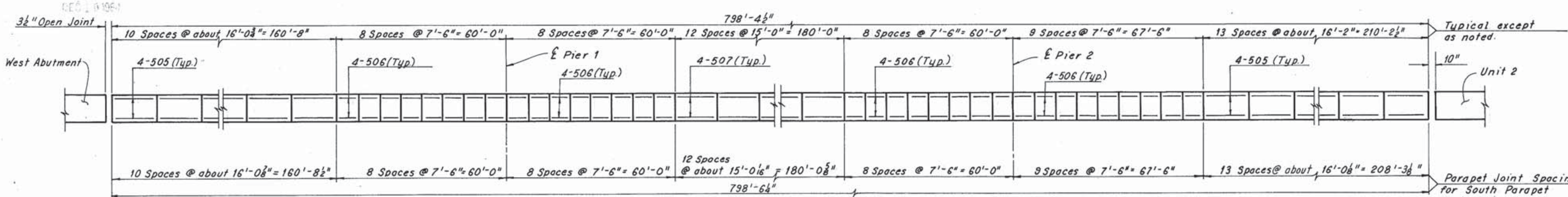
DRAWN JEH DATE 7-14-68	TRACED AJJ DATE 7-16-68	CHECKED JS DATE 9-9-68	REVIEWED WF DATE 8-22-70	REVISED	SHEET 25/28

REV. 8-15-73

FED. RD. DIVISION	STATE	PROJECT	
2	OHIO		

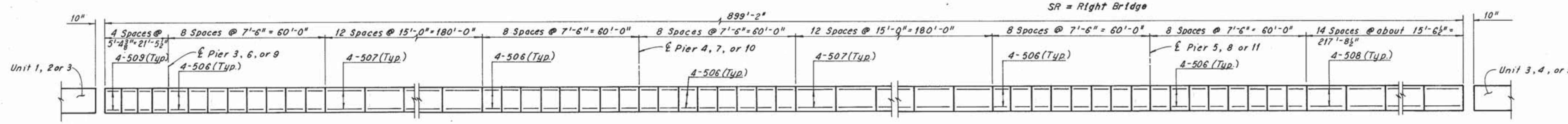
98/112

CUYAHOGA COUNTY
CUY-80-1843 Part 2

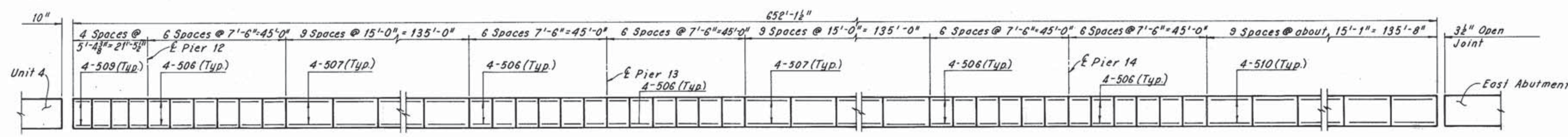


UNIT 1L AND UNIT 1R

Note:
All reinforcing bar marks for parapet shall be prefixed as follows:
SL = Left Bridge
SR = Right Bridge

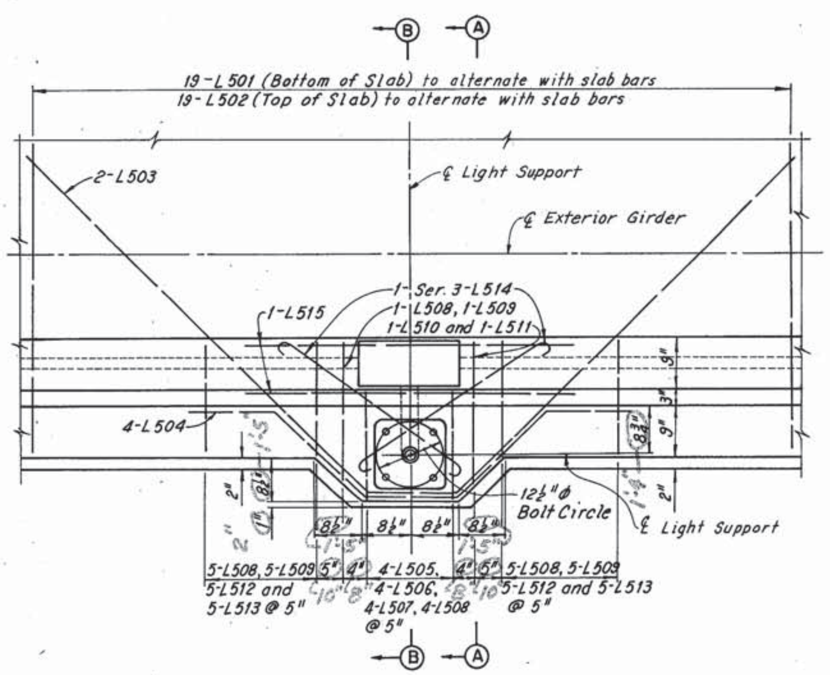


UNIT 2 (L AND R), UNIT 3 (L AND R) AND UNIT 4 (L AND R)



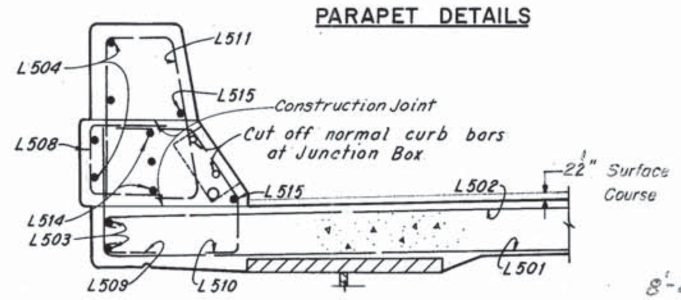
UNIT 5L AND UNIT 5R

PARAPET DETAILS

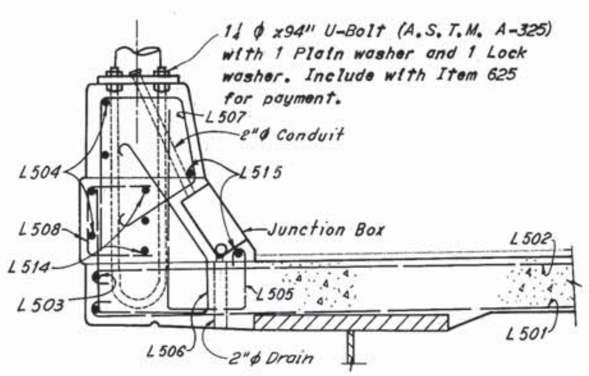


PLAN

Note: See Lighting Plans for additional details.



SECTION A-A

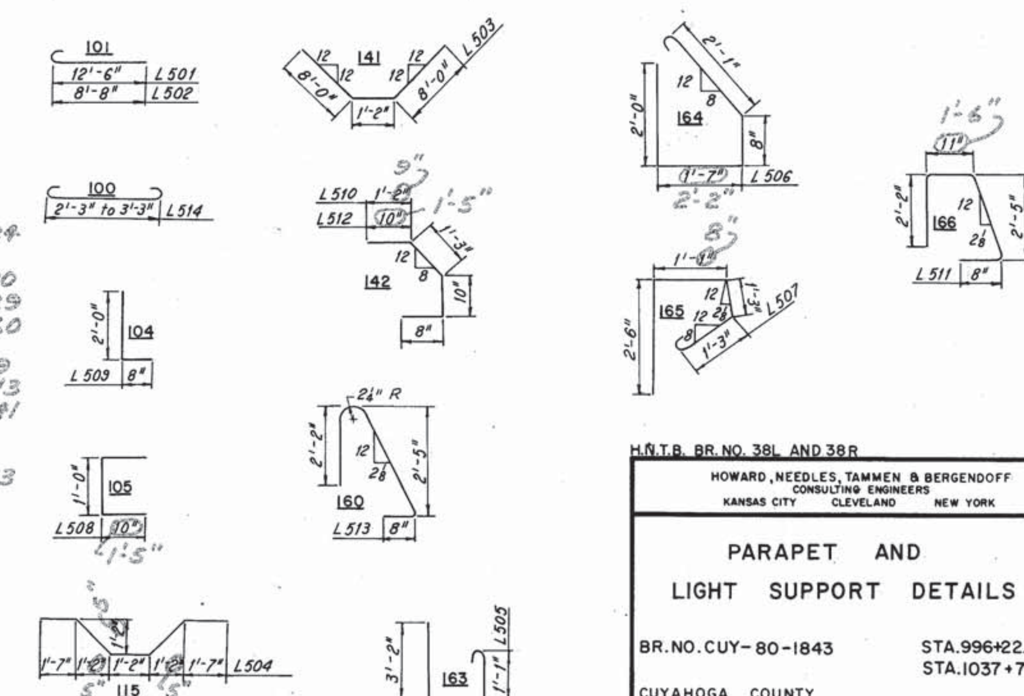


SECTION B-B

LIGHT SUPPORT DETAILS

REINFORCING SCHEDULE LIGHT STANDARD SUPPORT					
MARK	NO.	LENGTH	SHAPE	SER. INCR.	WEIGHT
L501	19	13'-1"	101		259
L502	19	9'-3"	101		183
L503	2	17'-0"	141		35
L504	4	7'-4" 8'-2"	115		31 34
L505	4	5'-8"	163		24
L506	4	6'-7" 7'-2"	164		27 30
L507	4	6'-4" 6'-11"	165		26 29
L508	16	2'-5" 3'-7"	105		40 60
L509	12	2'-6"	104		31
L510	2	3'-8" 4'-3"	142		8 9
L511	2	5'-9" 6'-4"	166		12 13
L512	10	3'-4" 3'-11"	142		35 41
L513	10	5'-4"	160		56
L514	2-S-3	3'-5" / 4'-5"	100	6"	25
L515	2	4'-9" 6'-0"	Str.		10 13
For 1 (one) Light Standard Support on Bridge					
		TOTAL WEIGHT =			802 842

BAR BENDING DIAGRAMS



H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

PARAPET AND LIGHT SUPPORT DETAILS

BR. NO. CUY-80-1843
STA. 996+22.25 TO STA. 1037+77.75

CUYAHOGA COUNTY OHIO

DRAWN J.E.H. DATE 9-5-68	TRACED G.E.M. DATE 7-26-68	CHECKED T.J.S. DATE 9-9-68	REVIEWED W.F. DATE 8-22-70	REVISED 11-16-70 SHEET 26/28

REV. 8-15-73

CUYAHOGA COUNTY
CUY-80-1843 Part 2

UNIT NO. 1

Station	Elev. A	Girder A	Girder B	Girder C	Girder D	Elev. B	Girder E	Girder F	Girder G	Girder H	Elev. C
996+25	713.63	713.65	714.00	714.01	713.66	713.64	713.66	714.09	713.84	713.41	713.39
996+45	714.22	714.17	714.52	714.54	714.19	714.24	714.19	714.62	714.36	713.94	714.00
996+70	714.97	714.83	715.18	715.19	714.84	714.98	714.84	715.27	715.02	714.60	714.74
996+95	715.66	715.48	715.83	715.85	715.50	715.68	715.50	715.92	715.68	715.26	715.45
997+20	716.32	716.14	716.49	716.50	716.15	716.34	716.15	716.57	716.34	715.92	716.12
997+45	716.94	716.79	717.14	717.16	716.81	716.97	716.81	717.22	717.00	716.59	716.76
997+70	717.55	717.45	717.80	717.81	717.46	717.57	717.46	717.88	717.66	717.25	717.36
991+95	718.14	718.10	718.45	718.47	718.12	718.17	718.12	718.53	718.32	717.91	717.96
998+20	718.76	718.76	719.11	719.12	718.77	718.77	718.77	719.18	718.98	718.58	718.58
998+45	719.39	719.41	719.76	719.78	719.43	719.41	719.43	719.83	719.64	719.24	719.22
998+70	720.07	720.07	720.42	720.43	720.08	720.08	720.08	720.48	720.31	719.90	719.90
998+95	720.76	720.72	721.07	721.09	720.74	720.78	720.74	721.14	720.97	720.57	720.61
999+20	721.48	721.38	721.73	721.74	721.39	721.49	721.39	721.79	721.63	721.23	721.34
999+45	722.19	722.03	722.38	722.40	722.05	722.21	722.05	722.44	722.29	721.89	722.06
999+70	722.88	722.69	723.04	723.05	722.70	722.90	722.70	723.09	722.95	722.56	722.76
999+95	723.54	723.34	723.69	723.71	723.36	723.57	723.36	723.74	723.61	723.22	723.43
1000+20	724.17	724.00	724.35	724.36	724.01	724.19	724.01	724.40	724.27	723.87	724.05
1000+45	724.76	724.65	725.00	725.02	724.67	724.78	724.67	725.05	724.93	724.53	724.65
1000+70	725.35	725.31	725.66	725.67	725.32	725.37	725.32	725.70	725.59	725.20	725.25
1000+95	725.95	725.96	726.31	726.33	725.98	725.98	725.98	726.35	726.25	725.86	725.86
1001+20	726.59	726.62	726.97	726.98	726.63	726.60	726.63	727.00	726.91	726.52	726.49
1001+45	727.26	727.28	727.62	727.64	727.29	727.27	727.29	727.66	727.57	727.19	727.17
1001+70	727.94	727.93	728.28	728.29	727.94	727.95	727.94	728.31	728.23	727.85	727.86
1001+95	728.65	728.58	728.93	728.95	728.60	728.66	728.60	728.96	728.89	728.51	728.57
1002+20	729.39	729.24	729.59	729.60	729.25	729.40	729.25	729.61	729.55	729.17	729.31
1002+45	730.12	729.89	730.24	730.26	729.91	730.13	729.91	730.26	730.21	729.84	730.05
1002+70	730.82	730.55	730.90	730.91	730.56	730.83	730.56	730.42	730.87	730.50	730.76
1002+95	731.50	731.20	731.55	731.57	731.22	731.51	731.22	731.57	731.53	731.16	731.43
1003+20	732.14	731.86	732.21	732.22	731.87	732.15	731.87	732.22	732.19	731.83	732.09
1003+45	732.73	732.51	732.86	732.88	732.53	732.75	732.53	732.87	732.85	732.49	732.70
1003+70	733.30	733.17	733.52	733.53	733.18	733.31	733.18	733.52	733.51	733.15	733.27
1003+95	733.83	733.82	734.17	734.19	733.84	733.84	732.84	734.18	734.19	733.82	733.83
1004+20	734.35	734.48	734.83	734.84	734.49	734.36	734.49	734.84	734.83	734.48	734.35

UNIT NO. 2

Station	Elev A and C	Girder A or H	Girder B or G	Girder C or F	Girder D or E	Elev. B
1004+45	735.11	735.13	735.48	735.50	735.15	735.13
1004+70	735.89	735.79	736.14	736.15	735.80	735.90
1004+95	736.65	736.44	736.79	736.81	736.46	736.67
1005+20	737.41	737.10	737.45	737.46	737.11	737.42
1005+45	738.13	737.75	738.10	738.12	737.77	738.15
1005+70	738.82	738.41	738.76	738.77	738.42	738.83
1005+95	739.47	739.06	739.41	739.43	739.08	738.49
1006+20	740.10	739.72	740.07	740.08	739.73	740.11
1006+45	740.68	740.37	740.72	740.74	740.39	740.70
1006+70	741.25	741.03	741.38	741.39	741.04	741.26
1006+95	741.80	741.68	742.03	742.05	741.70	741.82
1007+20	742.38	742.34	742.69	742.70	742.35	742.39
1007+45	742.97	742.99	743.34	743.36	743.01	742.99
1007+70	743.60	743.65	744.00	744.01	743.66	743.61
1007+95	744.24	744.30	744.65	744.67	744.32	744.26
1008+20	744.93	744.96	745.31	745.32	744.97	744.94
1008+45	745.62	745.61	745.96	745.98	745.63	745.64
1008+70	746.32	746.27	746.62	746.63	746.28	746.33
1008+95	746.98	746.92	747.27	747.29	746.94	747.00
1009+20	747.63	747.58	747.93	747.94	747.59	747.64
1009+45	748.25	748.23	748.58	748.60	748.25	748.27
1009+70	748.87	748.89	749.24	749.25	748.90	748.88
1009+95	749.49	749.54	749.89	749.91	749.56	749.51
1010+20	750.15	750.20	750.55	750.56	750.21	750.16
1010+45	750.83	750.85	751.20	751.22	750.87	750.85
1010+70	751.54	751.51	751.86	751.87	751.52	751.55
1010+95	752.26	752.16	753.51	752.53	752.18	752.28
1011+20	753.00	752.82	753.17	753.18	752.83	753.01
1011+45	753.72	753.47	753.82	753.84	753.49	753.74
1011+70	754.42	754.13	754.48	754.49	754.14	754.43
1011+95	755.10	754.79	755.13	755.15	754.80	755.11
1012+20	755.72	755.44	755.79	755.80	755.45	755.73
1012+45	756.31	756.09	756.44	756.46	756.11	756.33
1012+70	756.88	756.75	757.10	757.11	756.76	756.89
1012+95	757.41	757.40	757.75	757.77	757.42	757.43
1013+20	757.93	758.06	758.41	758.42	758.07	757.94

UNIT NO. 3

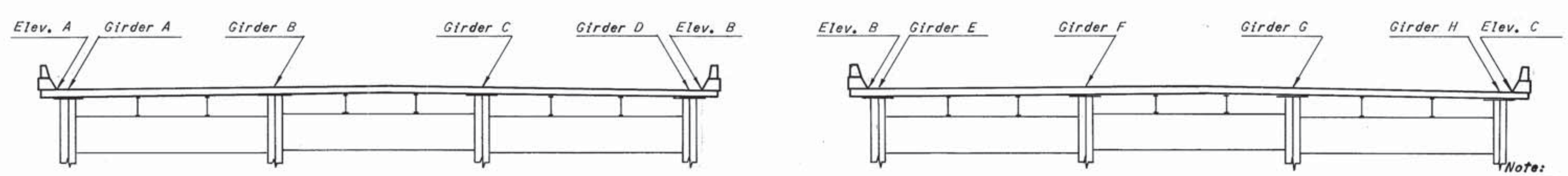
Station	Elev A or C	Girder A or H	Girder B or G	Girder C or F	Girder D or E	Elev. B
1013+45	758.69	758.71	759.06	759.08	758.73	758.71
1013+70	759.46	759.37	759.72	759.73	759.38	759.47
1013+95	760.23	760.02	760.37	760.39	760.04	760.25
1014+20	760.98	760.68	761.03	761.04	760.69	760.99
1014+45	761.70	761.33	761.68	761.70	761.35	761.72
1014+70	762.40	761.99	762.34	762.35	762.00	762.41
1014+95	763.05	762.64	762.99	763.01	762.66	763.07
1015+20	763.67	763.30	763.65	763.66	763.31	763.68
1015+45	764.26	763.95	764.30	764.32	763.97	764.27
1015+70	764.82	764.61	764.96	764.97	764.62	764.83
1015+95	765.39	765.26	765.61	765.63	765.28	765.41
1016+20	765.96	765.92	766.27	766.28	765.93	765.97
1016+45	766.55	766.57	766.92	766.94	766.59	766.57
1016+70	767.18	767.23	767.58	767.59	767.24	765.19
1016+95	767.82	767.88	768.23	768.25	767.90	767.84
1017+20	768.51	768.54	768.89	768.90	768.55	768.52
1017+45	769.20	769.19	769.54	769.56	769.21	769.22
1017+70	769.90	769.85	770.20	770.21	769.86	769.91
1017+95	770.56	770.50	770.85	770.87	770.52	770.58
1018+20	771.21	771.16	771.51	771.52	771.17	771.22
1018+45	771.83	771.81	772.16	772.18	771.83	771.85
1018+70	772.45	772.47	772.81	772.83	772.48	772.46
1018+95	773.07	773.12	773.47	773.49	773.14	773.09
1019+20	773.73	773.78	774.13	774.14	773.79	773.74
1019+45	774.41	774.43	774.78	774.80	774.45	774.43
1019+70	775.12	775.09	775.44	775.45	775.10	775.13
1019+95	775.84	775.74	776.09	776.11	775.76	775.86
1020+20	776.58	776.40	776.75	776.76	776.41	776.59
1020+45	777.30	777.05	777.40	777.42	777.07	777.32
1020+70	778.00	777.71	778.06	778.07	777.72	778.01
1020+95	778.67	778.36	778.71	778.73	778.38	778.69
1021+20	779.30	779.02	779.37	779.38	779.03	779.31
1021+45	779.89	779.67	780.02	780.04	779.69	779.91
1021+70	780.46	780.33	780.68	780.69	780.34	780.47
1021+95	780.99	780.98	781.33	781.35	781.00	781.02
1022+20	781.51	781.64	781.99	782.00	781.65	781.52

UNIT NO. 4

Station	Elev A or C	Girder A or H	Girder B or G	Girder C or F	Girder D or E	Elev. B
1022+45	782.27	782.29	782.64	782.66	782.31	782.29
1022+70	783.04	782.95	783.30	783.31	782.96	783.05
1022+95	783.81	783.60	783.95	783.97	783.62	783.83
1023+20	784.56	784.26	784.61	784.62	784.27	784.57
1023+45	785.28	784.91	785.26	785.28	784.93	785.30
1023+70	785.98	785.57	785.92	785.93	785.58	785.99
1023+95	786.63	786.22	786.57	786.59	786.24	786.65
1024+20	787.25	786.88	787.23	787.24	786.89	787.26
1024+45	787.84	787.53	787.88	787.90	787.55	787.86
1024+70	788.40	788.19	788.54	788.55	788.20	788.41
1024+95	788.97	788.84	789.19	789.21	788.86	788.99
1025+20	789.54	789.50	789.85	789.86	789.51	789.55
1025+45	790.13	790.15	790.50	790.52	790.17	790.15
1025+70	790.76	790.81	791.16	791.17	790.82	790.77
1025+95	791.40	791.46	791.81	791.83	791.48	791.42
1026+20	792.09	792.12	792.47	792.48	792.13	792.10
1026+45	792.78	792.77	793.12	793.14	792.79	792.80
1026+70	793.48	793.43	793.78	793.79	793.44	793.49
1026+95	794.14	794.08	794.43	794.45	794.10	795.16
1027+20	794.79	794.74	795.09	795.10	794.75	794.80
1027+45	795.41	795.39	795.74	795.76	795.41	795.43
1027+70	796.03	796.05	796.40	796.41	796.06	796.04
1027+95	796.65	796.70	797.05	797.07	796.72	796.67
1028+20	797.31	797.36	797.71	797.72	797.37	797.32
1028+45	797.99	798.01	798.36	798.38	798.03	798.01
1028+70	798.70	798.67	799.02	799.03	798.68	798.71
1028+95	799.42	799.32	799.67	799.69	799.34	799.44
1029+20	800.16	799.98	800.33	800.34	799.99	800.17
1029+45	800.88	800.63	800.98	801.00	800.65	800.90
1029+70	801.58	801.29	801.64	801.65	801.30	801.59
1029+95	802.25	801.94	802.29	802.31	801.96	802.27
1030+20	802.88	802.60	802.95	802.96	802.61	802.89
1030+45	803.47	803.25	803.60	803.62	803.27	803.49
1030+70	804.04	803.91	804.26	804.27	803.92	804.05
1030+95	804.57	804.56	804.91	804.93	804.58	804.59
1031+20	805.19	805.22	805.57	805.58	805.23	805.20

UNIT NO. 5

Station	Elev A or C	Girder A or H	Girder B or G	Girder C or F	Girder D or E	Elev. B
1031+45	805.85	805.87	806.22	806.24	805.89	805.87
1031+70	806.57	806.53	806.88	806.89	806.54	806.58
1031+95	807.28	807.18	807.53	807.55	807.20	807.30
1032+20	808.00	807.84	808.19	808.20	807.85	808.01
1032+45	809.68	808.49	808.85	808.86	808.51	808.70
1032+70	809.33	809.15	809.50	809.51	809.16	809.34
1032+95	809.94	809.80	810.15	810.17	809.82	809.96
1033+20	810.53	810.46	810.81	810.82	810.47	810.54
1033+45	811.18	811.16	811.46	811.48	811.18	811.20
1033+70	811.75	811.77	812.12	812.13	811.78	811.76
1033+95	812.41	812.42	812.77	812.79	812.44	812.43
1034+20	812.10	812.08	813.43	813.44	813.09	813.11
1034+45	813.78	813.73	814.08	814.10	813.75	813.80
1034+70	814.47	814.39	814.74	814.75	814.40	814.48
1034+95	815.12	815.04	815.39	815.41	815.06	815.14
1035+20	815.76	815.70	816.05	816.06	815.71	815.77
1035+45	816.37	816.35	816.70	816.72	816.37	816.39
1035+70	817.00	817.01	817.36	817.37	817.02	817.01
1035+95	817.64	817.66	818.01	818.03	817.68	817.66
1036+20	818.32	818.32	818.67	818.68	818.33	818.33
1036+45	819.02	818.97	819.32	819.34	818.99	819.04
1036+70	819.72	819.63	819.98	819.99	819.64	819.73
1036+95	820.40	820.28	820.63	820.65	820.30	820.42
1037+20	821.06	820.94	821.29	821.30	820.95	821.07
1037+45	821.66	821.59	821.94	821.96	821.61	821.68
1037+70	822.25	822.25	822.60	822.61	822.26	822.26
1037+75	822.36	822.38	822.73	822.74	822.39	822.37



TYPICAL CROSS-SECTION

$2\frac{1}{2}$" Surface Course not shown

TOP OF PIER ELEVATIONS

Location	Girder A	Girder B	Girder C	Girder D	Girder E	Girder F	Girder G	Girder H	Location	Girder A	Girder B	Girder C	Girder D	Girder E	Girder F	Girder G	Girder H
Pier 1	699.57	700.83	700.85	699.59	699.46	700.78	700.59	699.27	Pier 8	754.52	755.81	755.83	754.54	754.54	755.83	755.81	754.52
Pier 2	706.49	706.76	706.78	706.51	706.39	706.68	706.59	706.29	Pier 9	761.62	761.93	761.95	761.64	761.64	761.95	761.93	761.62
Pier 3	714.46	714.77	714.79	714.48	714.48	714.79	714.77	714.46	Pier 10	770.24	771.49	771.51	770.26	770.26	771.51	771.49	770.24
Pier 4	723.08	724.33	724.35	723.10	723.10	724.35	724.33	723.08	Pier 11	778.10	779.39	779.41	778.12	778.12	779.41	779.39	778.10
Pier 5	730.94	732.23	732.25	730.96	730.96	732.25	732.23	730.94	Pier 12	790.16	790.51	790.53	790.18	790.18	790.53	790.51	790.16
Pier 6	738.04	738.35	738.37	738.06	738.06	738.37	738.35	738.04	Pier 13	795.92	796.27	796.28	795.93	795.93	796.28	796.27	795.92
Pier 7	746.66	747.91	747.93	746.68	746.68	747.93	747.91	746.66	Pier 14	802.69	804.00	804.02	802.71	802.71	804.02	804.00	802.69

Note:

Elevations A, B and C shown at curbs are those which are required before concrete is placed. Proper allowance has been made for the dead load deflections caused by the weight of the reinforced concrete.

Elevations shown over the girders are final top of ~~pavement~~ reinforced concrete slab elevations.

H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

TOP OF PAVEMENT ELEVATIONS AND TOP OF PIER ELEVATIONS

I-80 OVER CUYAHOGA RIVER VALLEY

BR. NO. CUY-80-1843 L & R — STA. 996+22.25 TO STA. 1037+77.75

CUYAHOGA COUNTY — OHIO

DRAWN DLW	TRACED GEM	CHECKED TJS	REVIEWED WF	REVISED
DATE 10-18-68	DATE 10-22-68	DATE 10-28-68	DATE 8-22-70	SHEET 27/28

REV. 8-15-73

MICROFILMED
DEC 1 0 1984

FED. RD. DIVISION	STATE	PROJECT	
2	OHIO		

100
112

CUYAHOGA COUNTY
CUY-80-1843 Part 2

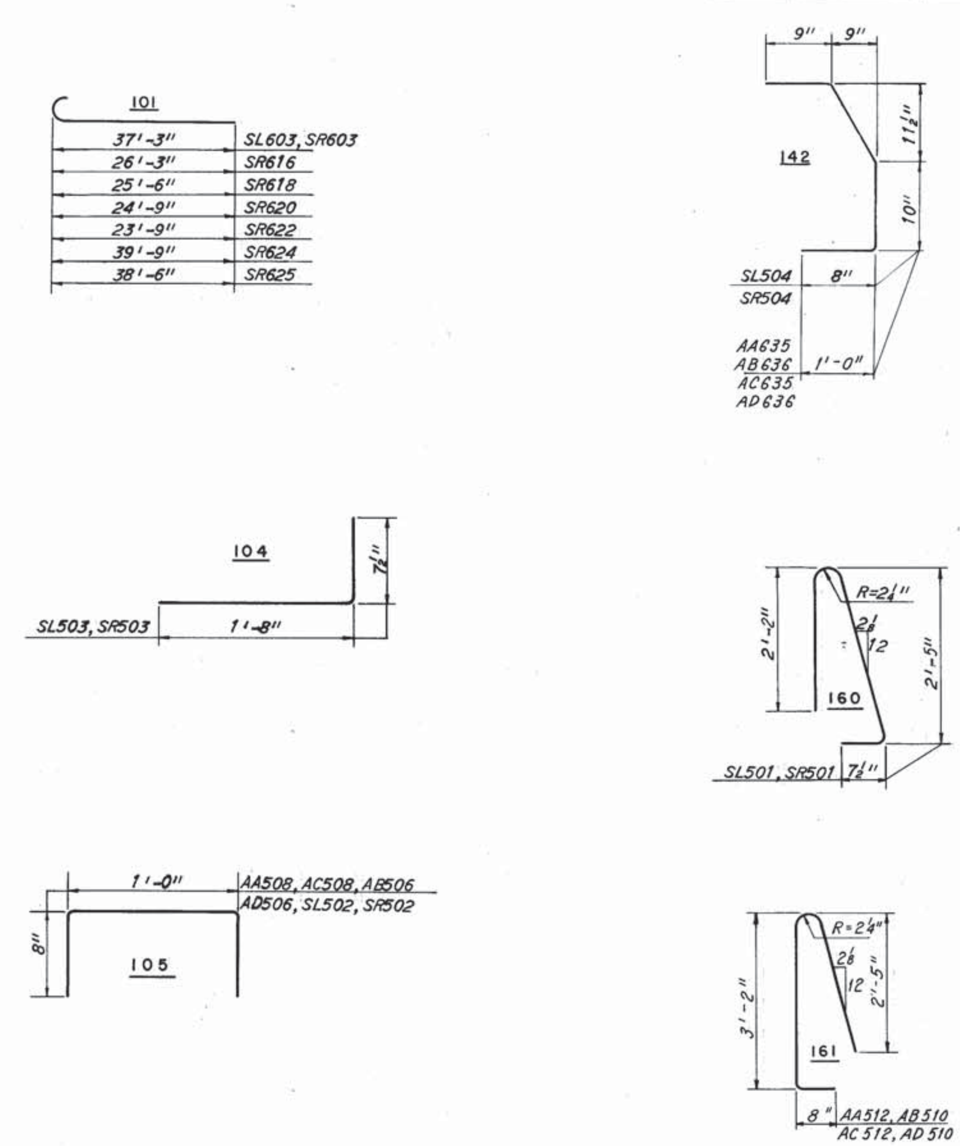
MARK	NO.	LENGTH	TYPE	SER. INCR.	WEIGHT (LBS.)
WEST ABUTMENT - LEFT BRIDGE					
AA502	1	11'-3"	Str.		12
AA503	1	25'-3"	Str.		26
AA504	1	26'-0"	Str.		27
AA505	1	12'-3"	Str.		13
AA508	10	2'-0"	105		21
AA512	10	6'-1"	161		64
AA515	20	6'-0"	Str		125
AA635	2	4'-3"	142		13
		TOTAL WEIGHT	=		301
EAST ABUTMENT - RIGHT BRIDGE					
AB503	2	26'-3"	Str.		55
AB504	1	11'-6"	Str.		12
AB505	1	12'-6"	Str.		13
AB506	10	2'-0"	105		21
AB510	10	6'-1"	161		63
AB512	20	6'-0"	Str.		125
AB636	2	4'-3"	142		13
		TOTAL WEIGHT	=		302
WEST ABUTMENT - RIGHT BRIDGE					
AC502	1	40'-9"	Str.		43
AC503	1	12'-3"	Str.		13
AC504	1	25'-6"	Str.		27
AC505	1	11'-3"	Str.		12
AC508	10	2'-0"	105		21
AC512	10	6'-1"	161		64
AC515	20	6'-0"	Str.		125
AC635	2	4'-3"	142		13
		TOTAL WEIGHT	=		318
EAST ABUTMENT-LEFT BRIDGE					
AD503	2	26'-3"	Str.		55
AD504	1	11'-6"	Str.		12
AD505	1	12'-6"	Str.		13
AD506	10	2'-0"	105		21
AD510	10	6'-1"	161		63
AD512	20	6'-0"	Str.		125
AD636	2	4'-3"	142		13
		TOTAL WEIGHT	=		302
TOTAL ABUTMENT WEIGHT			=		1,223
SLAB UNIT - 1R					
SR501	1064	5'-4"	161		5,915
SR502	1064	2'-0"	105		2,220
SR503	1064	2'-2"	104		2,405
SR504	1064	3'-2"	142		3,515
SR505	184	15'-6"	Str.		2,975
SR506	264	7'-0"	Str.		1,927
SR507	96	14'-6"	Str.		1,452
SR601	4508	30'-0"	Str.		203,130
SR602	161	10'-0"	Str.		2,418
SR606	408	31'-3"	Str.		19,151
SR607	204	29'-6"	Str.		9,039
SR608	408	30'-6"	Str.		18,691
SR609	718	28'-9"	Str.		31,005
SR610	514	29'-9"	Str.		22,968
SR611	257	28'-0"	Str.		10,808
SR612	669	27'-0"	Str.		27,131
SR613	414	27'-9"	Str.		17,256
SR614	207	26'-0"	Str.		8,084
SR615	206	25'-0"	Str.		7,735
SR616	408	26'-11"	101		16,495
SR617	204	39'-6"	Str.		12,103
SR618	408	26'-2"	101		16,036
SR619	204	38'-9"	Str.		11,873
SR620	514	25'-5"	101		19,623
SR621	257	38'-0"	Str.		14,669
SR622	514	24'-5"	101		18,851
SR623	257	37'-0"	Str.		14,283
SR624	414	40'-5"	101		25,132

MARK	NO.	LENGTH	TYPE	SER. INCR.	WEIGHT (LBS.)
SR625	412	39'-2"	101		24,237
SR626	180	40'-3"	Str.		10,882
SR627	240	33'-9"	Str.		12,166
		TOTAL WEIGHT	=		594,175
SLAB-UNIT 1L					
SL501	1064	5'-4"	160		5,915
SL502	1064	2'-0"	105		2,220
SL503	1064	2'-2"	104		2,405
SL504	1064	3'-2"	142		3,515
SL505	184	15'-6"	Str.		2,975
SL506	264	7'-0"	Str.		1,927
SL507	96	14'-6"	Str.		1,452
SL601	2996	30'-0"	Str.		135,000
SL602	107	10'-0"	Str.		1,607
SL603	2548	37'-11"	101		145,112
SL604	2548	26'-3"	Str.		100,461
SL605	1274	24'-3"	Str.		46,404
SL626	126	40'-3"	Str.		7,617
SL627	168	33'-9"	Str.		8,516
		TOTAL WEIGHT	=		465,126
SLAB-UNIT 2R					
SR501	1200	5'-4"	160		6,671
SR502	1200	2'-0"	105		2,503
SR503	1200	2'-2"	104		2,712
SR504	1200	3'-2"	142		3,964
SR506	320	7'-0"	Str.		2,336
SR507	192	14'-6"	Str.		2,904
SR508	112	15'-0"	Str.		1,752
SR509	32	5'-0"	Str.		167
SR601	3424	30'-0"	Str.		154,285
SR603	2874	37'-11"	101		163,678
SR604	2874	26'-3"	Str.		113,315
SR605	1437	24'-3"	Str.		52,341
SR627	168	33'-9"	Str.		8,516
SR628	84	28'-3"	Str.		3,564
SR629	168	35'-3"	Str.		8,895
		TOTAL WEIGHT	=		527,613
SLAB- UNIT 2L					
SL501	1200	5'-4"	160		6,671
SL502	1200	2'-0"	105		2,503
SL503	1200	2'-2"	104		2,712
SL504	1200	3'-2"	142		3,964
SL506	320	7'-0"	Str.		2,336
SL507	192	14'-6"	Str.		2,904
SL508	112	15'-0"	Str.		1,752
SL509	32	5'-0"	Str.		167
SL601	3424	30'-0"	Str.		154,285
SL603	2874	37'-11"	101		163,678
SL604	2874	26'-3"	Str.		113,315
SL605	1437	24'-3"	Str.		52,341
SL627	168	33'-9"	Str.		8,516
SL628	84	28'-3"	Str.		3,564
SL629	168	35'-3"	Str.		8,895
		TOTAL WEIGHT	=		527,613
SLAB-UNIT 3R					
SR501	1200	5'-4"	160		6,671
SR502	1200	2'-0"	105		2,503
SR503	1200	2'-2"	104		2,712
SR504	1200	3'-2"	142		3,964
SR506	320	7'-0"	Str.		2,336
SR507	192	14'-6"	Str.		2,904
SR508	112	15'-0"	Str.		1,752

MARK	NO.	LENGTH	TYPE	SER. INCR.	WEIGHT (LBS.)
SR509	32	5'-0"	Str.		167
SR601	3424	30'-0"	Str.		154,285
SR603	2874	37'-11"	101		163,678
SR604	2874	26'-3"	Str.		113,315
SR605	1437	24'-3"	Str		52,341
SR627	168	33'-9"	Str		8,516
SR628	84	28'-3"	Str.		3,564
SR629	168	35'-3"	Str.		8,895
		TOTAL WEIGHT	=		527,613
SLAB- UNIT 3L					
SL501	1200	5'-4"	160		6,671
SL502	1200	2'-0"	105		2,503
SL503	1200	2'-2"	104		2,712
SL504	1200	3'-2"	142		3,964
SL506	320	7'-0"	Str.		2,336
SL507	192	14'-6"	Str.		2,904
SL508	112	15'-0"	Str.		1,752
SL509	32	5'-0"	Str.		167
SL601	3424	30'-0"	Str		154,285
SL603	2874	37'-11"	101		163,678
SL604	2874	26'-3"	Str.		113,315
SL605	1437	24'-3"	Str.		52,341
SL627	168	33'-9"	Str.		8,516
SL628	84	28'-3"	Str.		3,564
SL629	168	35'-3"	Str.		8,895
		TOTAL WEIGHT	=		527,613
SLAB-UNIT 4 R					
SR501	1200	5'-4"	160		6,671
SR502	1200	2'-0"	105		2,503
SR503	1200	2'-2"	104		2,712
SR504	1200	3'-2"	142		3,964
SR506	320	7'-0"	Str.		2,336
SR507	192	14'-6"	Str.		2,904
SR508	112	15'-0"	Str.		1,752
SR509	32	5'-0"	Str.		167
SR601	3424	30'-0"	Str.		154,285
SR603	2874	37'-11"	101		163,678
SR604	2874	26'-3"	Str.		113,315
SR605	1437	24'-3"	Str.		52,341
SR627	168	33'-9"	Str.		8,516
SR628	84	28'-3"	Str.		3,564
SR629	168	35'-3"	Str.		8,895
		TOTAL WEIGHT	=		527,613
SLAB-UNIT 4L					
SL501	1200	5'-4"	160		6,671
SL502	1200	2'-0"	105		2,503
SL503	1200	2'-2"	104		2,712
SL504	1200	3'-2"	142		3,964
SL506	320	7'-0"	Str.		2,336
SL507	192	14'-6"	Str.		2,904
SL508	112	15'-0"	Str.		1,752
SL509	32	5'-0"	Str.		167
SL601	3424	30'-0"	Str.		154,285
SL603	2874	37'-11"	101		163,678
SL604	2874	26'-3"	Str		113,315
SL605	1437	24'-3"	Str		52,341
SL627	168	33'-9"	Str		8,516
SL628	84	28'-3"	Str		3,564
SL629	168	35'-3"	Str		8,895
		TOTAL WEIGHT	=		527,613

MARK	NO.	LENGTH	TYPE	SER. INCR.	WEIGHT (LBS.)
SLAB-UNIT 5R					
SR501	874	5'-4"	160		4,859
SR502	874	2'-0"	105		1,823
SR503	874	2'-2"	104		1,975
SR504	874	3'-2"	142		2,887
SR506	240	7'-0"	Str.		1,752
SR507	144	14'-6"	Str.		2,178
SR509	32	5'-0"	Str.		167
SR510	72	14'-9"	Str.		1,108
SR601	2461	30'-0"	Str.		110,893
SR602	107	10'-0"	Str.		1,607
SR603	2090	37'-11"	101		119,028
SR604	2090	26'-3"	Str.		82,403
SR605	1045	24'-3"	Str.		38,063
SR628	84	28'-3"	Str.		3,564
SR630	126	37'-0"	Str.		7,002
SR631	126	31'-9"	Str.		6,009
		TOTAL WEIGHT		=	385,328
SLAB-UNIT 5 L					
SL501	874	5'-4"	160		4,859
SL502	874	2'-0"	105		1,823
SL503	874	2'-2"	104		1,975
SL504	874	3'-2"	142		2,887
SL506	240	7'-0"	Str.		1,752
SL507	144	14'-6"	Str.		2,178
SL509	32	5'-0"	Str.		167
SL510	72	14'-9"	Str.		1,108
SL601	2461	30'-0"	Str.		110,893
SL602	107	10'-0"	Str.		1,607
SL603	2090	37'-11"	101		119,028
SL604	2090	26'-3"	Str.		82,403
SL605	1045	24'-3"	Str.		38,063
SL628	84	28'-3"	Str.		3,564
SL630	126	37'-0"	Str.		7,002
SL631	126	31'-9"	Str.		6,009
		TOTAL WEIGHT		=	385,328
	TOTAL SUPERSTRUCTURE			=	4,994,915
TOTAL WEIGHT 42 LIGHT STANDARD SUPPORTS				=	33,726 35,364
SUPERSTRUCTURE GRAND TOTAL				=	5,028,641 5,030,279
	GRAND TOTAL			=	5,029,864 5,031,502

BENDING DIAGRAMS



* REPLACEMENT REINFORCEMENT SCHEDULE

Size	No.	Length	Type
5	11	6'-7"	Str.
6	240	6'-11"	Str.

H.N.T.B. BR. NO. 38L AND 38R

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
KANSAS CITY CLEVELAND NEW YORK

REINFORCEMENT SCHEDULE

I-80 OVER CUYAHOGA RIVER VALLEY
BR. NO. CUY-80-1843 L & R STA. 996+22.25 TO STA. 1037+77.75
CUYAHOGA COUNTY OHIO

DRAWN JJS DATE 1-10-69	TRACED GEM DATE 1-13-69	CHECKED CHB DATE 1-28-69	REVIEWED WF DATE 8-21-70	REVISED 11-16-70	SHEET 28/28

APPENDIX C
Project Background Documents

C. RECOMMENDATIONS

To be further determined and finalized based on the meeting with ODOT on March 26, 2012. A draft matrix is provided below that will be further enhanced. All the options seem to have merit and there are reasonable increased benefits as cost increase. The "other" considerations for the owner in the decision making would seem to be the deciding factor. At this time, ELR did not make a recommendation prior to the March 26 meeting but if desired by ODOT we could.

CUY-480-18.42- COST MATRIX

Options	Cost (millions)	Time of Disruption (years)	Most Benefit	Least Benefit
1A-New structure	$255	3	All new	Most cost
1B-New median structure	$215	2	Least disruption to traffic	Span type/configuration dictated by existing
2-New superstructure	$160	4	All new superstructure	No additional roadway capacity
3-New deck and intermediate girders	$100	4 ½	Elimination of fatigue concerns	Most disruption to traffic
4-New deck	$65	3	Least cost	Uncertainty

Meeting Minutes

Date of Meeting: **August 19, 2011**
1:00 p.m.

To: Poonsak Sritalapat
ODOT – District 12

From: Dave Traini
E.L. Robinson Engineering of Ohio Co.

Subject: CUY-480-1842 L&R
Deck Replacement Study Scope of Services

Attached for your reference and use are the meeting minutes from the Deck Replacement Study Scope of Services held August 19, 2011 at The Ohio Department of Transportation – District 12 office at 5500 Transportation Blvd, Garfield Heights, Ohio.

Attendees:		
	Poonsak Sritalapat	ODOT District 12
	Mike Kubek	"
	Dick Walters	"
	Jim Calanni	"
	Mike Herceg	"
	Chris Ondash	"
	Lou Hazapis	"
	Dennis O'Neil	"
	Tim Keller	ODOT Central Office
	Ananda Dharma	ODOT Central Office
	Dave Traini	E. L. Robinson Engineering
	Rick Rockich	"
	Jonathan Hren	"
	George Maki	"

Issues Discussed

Mr. Sritalapat opened the meeting.

A draft scope was distributed to all attendees. A listing of the various maintenance of traffic scenarios and cross-sections of the MOT phases were also provided.

It was emphasized that the objective of the study is to determine the optimum approach for the I-480 deck replacements that will provide the least inconvenience and be the safest to the motoring public.

A status of the I-77 bridge project over the canal and Granger Road to the west was given. Some construction is expected to begin in 2013 but will really begin in 2014 and last through 2016 with some clean-up expected in 2017.

It is expected to take two construction seasons to do the I-480 project. (One for each bridge) It would be very unwise to have restricting traffic control in place in the winter on the I-480 bridges due to plowing and maintenance reasons.

The District ideally would like work on I-480 to be performed during 2012 and 2013 or after I-77 Bridge over the canal and Granger Rd, in the year 2017 and 2018. However, the maintenance of traffic on the I-77 project and the I-480 project may be looked at together to develop the best maintenance of traffic plan.

If the findings of this study dictate the deck replacement will need to occur in the future, a parapet repair and fence replacement contract could be let to address the immediate safety concerns of the existing parapets. It is estimated about $2 to $3 million for the cost of parapet repairs; this work would eventually be removed when future deck replacement occurs.

Mr. Walters outlined the goal of the deck replacement study following the Draft Scope dated 8-15-2011.

Task 1
This task will be skipped based on the BARS analysis provided by Central Office. Note the controlling rating for the two bridges was HS19.4 and 19.8 rating.

Task 2.0 and 2.2
The strength of the cross-frames and out-of-plane bending is a perceived problem with part width deck removal. How the various stages of construction outlined in Task 2.0 affect the stresses in the crossframes and connection details shall be investigated. Out-of-plane bending stresses must be checked. The effects of fatigue shall be studied, attempting to predict future impacts due to the stage construction loadings.

MOT-Scenario #1 is the suggested MOT plan that detrimental effects such as differential deflection, out-of-plane bending and excessive crossframe/diaphragm stresses would be minimized by placing balanced/symmetrical loads on the superstructure. 3-D analysis response of the superstructure is required to verify whether cross members can be retained (no work, preferred), need reinforcement or have to be removed during the phase construction. Special deck placement sequences may be utilized to benefit the superstructure responses.

Task 3.0
Provide overview of the various possible rehabilitation and replacement solutions. Cost, assessment of future life, disruption to public and construction duration should all be considered.

This task can be performed concurrently with Tasks 2.0 and 2.2. The draft scope provides some options.

ELR should not limit their options to the items discussed or items in the scope but may suggest any innovative solution to replace the decks safely and at a minimum of inconvenience to the public. New ideas can be discussed with ODOT and studied further if deemed appropriate.

The following are misc. topics that were discussed.

- Stay-in-place forms would only be considered in a new bridge as they would add too much weight to the existing bridges. Lightweight concrete could be an option.
- For the replacement option for Task 3.0, a new bridge constructed in between the existing bridges is the best solution for Maintenance of Traffic.
- Six lanes of traffic on one bridge with 10'-6" lanes is feasible but most likely unreasonable. Minimum width of traffic lanes for MOT is 12 feet.
- The District determined that based on the findings of the recent inspection reports that the present condition of existing substructures is adequate. The substructures have been in service for 35 years and show no signs of distress. Some options may increase or rearrange the points of loading and if necessary will be investigated later. Jim Calanni will however look into past inspection reports and/or discuss with Youssef Seif.
- Mr. Maki mentioned that the original design had asphalt drains that dripped drainage onto the outside of the exterior girder bottom flanges and then ran down on top of the pier caps. A contract was let to plug the asphalt drains and redirect that drainage into the cross drains at the expansion joints. This may have caused some deicing salts to be present on the existing pier caps.
- Five lanes of traffic on one bridge and part width on the other will be necessary to adequately maintain the high traffic volume.
- Fatigue Life Analysis, to determine remaining life, is not required.

Request for Additional Information

- Deck cores were taken. The District will provide information from these deck cores including deck thickness and any other useful data such as compressive strength, if available.
- District will provide full BARS output.

Study time frame:

The time frame for completing this study is ASAP.

Tim Keller needs a proposal from ELR. After he receives the proposal he would have a contract ready in probably 2-3 days.

The meeting was adjourned at 2:30 p.m.

Meeting minutes as taken by Dave Traini, P.E. of EL Robinson Engineering. These minutes are presented to the best of my knowledge as recorded August 19, 2011. Any comments or revisions should be submitted within three days of receipt.

*1801 Watermark Drive, Suite 310 * Columbus, OH 43215 * 614-586-0642 * 614-586-0648 fax*