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DESIGN AND CONSTRUCTION OF A FULL-WIDTH, FULL-DEPTH PRECAST CONCRETE DECK SLAB ON STEEL GIRDER BRIDGE

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Abstract

The West Sandusky Street over I-75 bridge replacement project consists of a single 170’ span hybrid steel plate girder bridge with concrete deck. To minimize closure times full-width, full-depth precast concrete deck panels are used for the construction of the bridge deck. The precast deck panels are post-tensioned both longitudinally and transversely to minimize cracking and improve durability. The deck panels are constructed with shear stud pockets to allow for the installation of shear studs after erection and post-tensioning. During detail design, a finite element analysis of the bridge deck was carried out to determine the required level of prestress in the deck. A time dependent analysis was subsequently carried out to determine the long term creep effects and post-tensioning losses, including the effects of restraint from the steel girders. A sensitivity analysis was carried out to determine the optimum curing time required prior to stressing the longitudinal post-tensioning tendons and grouting the shear pockets. The steel plate girders were designed for the long term creep effects due to the post-tensioning of the deck, which imposed additional axial loads and moments on the steel girders. Bridge construction was completed in the Fall of 2004.

1. Introduction

The West Sandusky Bridge replacement project, located in Findlay, Ohio, was scheduled for replacement in 2004. In order to minimize bridge closure times, this bridge was selected as a pilot project by the Ohio Department of Transportation for the construction of a full-width, full-depth precast concrete deck panel alternative, to determine the time savings associated with this type of construction, and to monitor the performance of the transverse joints.

2. Structural configuration

The West Sandusky Street Bridge over I-75, located in Findlay, Ohio, was a 3 span steel rolled girder bridge with concrete riding deck, scheduled for replacement in 2004. The replacement structure consisted of a single span steel plate girder bridge with a full-depth precast concrete deck riding surface. The span of the replacement bridge was 170'. Elevation of the new bridge is shown in Figure 1. and a cross-section is shown in Figure 2.

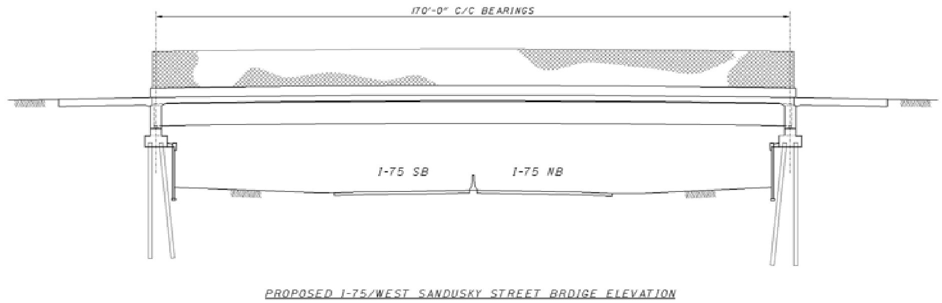

Figure 1

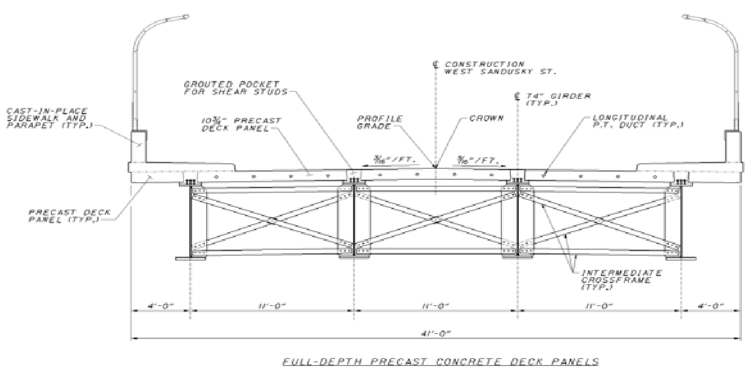

Figure 2

2.1 Deck panel details

A variety of deck panel configurations were evaluated at the preliminary design stage, including a traditional cast-in-place concrete deck, precast concrete deck form panels with a reinforced cast-in-place concrete deck, full-width precast deck panels with concrete topping, and full-width, full-depth precast concrete deck panels. The results of the evaluation indicated that up to five weeks of time savings could be achieved through the use of full-width, full-depth precast concrete deck panels, at an additional cost of approximately $200,000.

The deck panel segments were 10’ 3” wide, 10 ¾” thick, with shear keyways provided between the panels which are grouted after erection. The deck panels were milled ½” after installation to provide a nominal 10 ¼” deck thickness. Smaller sized units would have increased the number of joints in the deck, and increased the time required for lifting operations during erection, prolonging the construction schedule. Larger panel sizes would have resulted in handling and transportation difficulties due to the additional weight of the panels. The gap between the top flange of the girder and the deck underside was grouted after post-tensioning. The deck end blocks at the abutments were cast after post-tensioning. Pockets were blocked out of the deck panels to allow shear studs to be installed on the top flange of the girders, which were grouted after post-tensioning. The deck was milled ½” to achieve the desired roadway profile. Figure 3 shows a layout of the deck panels.

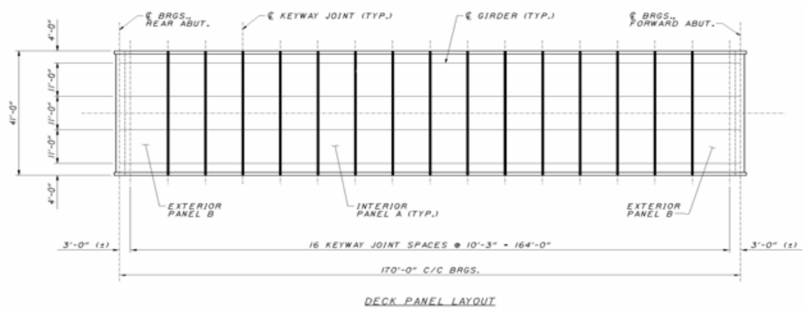

Figure 3

2.2 Post-tensioning arrangement

A variety of flat and round post-tensioning duct arrangements were considered during preliminary design. A combination of round and flat ducts was selected as the preferred alternative. Flat ducts were used in the transverse direction over the full width of the deck panels, as they have a shallower duct profile, which allows for a thinner deck thickness. However flat ducts are difficult to splice and feed strands through. Round ducts were therefore selected for the longitudinal direction, as they are simpler to splice and facilitate placement of the longitudinal prestressing strands through the ducts after installing the panels and grouting the shear keyways.

The round ducts were placed along the centerline of the slab in the longitudinal direction, and the flat ducts alternated above and below the longitudinal ducts in the transverse direction. This duct arrangement was selected to minimize the eccentricity of the centroid of the prestressing force and the centroid of the deck. The transverse tendons were stressed and grouted in the plant to prevent cracking of the panels during handling and transportation. The ducts were draped slightly at the ends to maintain minimum cover to the tendon anchor plates. Internal type anchorages were specified for both the transverse and longitudinal tendons and were galvanized for additional protection. A thixotropic non-shrink grout was specified for grouting of the tendons. The transverse tendons were stressed and grouted prior to shipping.

2.3 Shear stud pockets

The concrete deck was designed to act compositely with the steel plate girders to minimize the required steel plate girder size. Shear studs were required between the girders and the precast deck to develop the composite action of the system. Bevelled shear stud pockets, 12” wide x 10” long and spaced at 2’ centers were located in the deck panels over the girder top flanges to allow shear studs to be installed in the field. Each block-out contains two rows of three 7/8” diameter shear studs which were installed after erection of the deck panels. The shear stud pockets were grouted along with the girder haunches after all post-tensioning operations were completed to ensure post-tensioning forces were not transferred to the steel plate girders during stressing operations.

A variety of methods were considered to form the haunch between the deck panels and the top of the girder flange. A bedding strip of rigid insulation glued to the edge of the top flange on both sides, shaped to match the final road profile forms the haunch. Leveling bolts were used to make any adjustments to the final grade once the deck panels are in place.

2.4 Transverse shear keyway joint

A grouted shear keyway joint was used for the transverse joint between the deck panels. Match casting was not considered to be practical for the transverse joints, as grouted shear keyway joints have a greater allowance for out-of-tolerance in the fabrication of the deck panels. A ¾” rubber backing rod compressed to ½” was placed at the bottom of the keyway to prevent grout from leaking through the joint. The post-tensioning ducts between adjacent panels were connected with duct couplers.

3. Design considerations

The structural design of the deck panels and steel girders was carried out in two parts. During preliminary design, an initial structural analysis was carried out to determine the required deck configuration, prestressing layout and forces, and preliminary plate girder sizes. Subsequently, a refined analysis was carried out to determine the long term load redistribution effects due to creep. The refined analysis consisted of a finite element analysis of the deck and girder to determine the imposed stresses in the bridge deck due to dead and live loads, as well as temperature and shrinkage effects. The time-dependent analysis was carried out to determine the effects of creep and prestressing losses, and to determine the additional stresses in the plate girders due to load redistribution effects. The design of the bridge was carried out in accordance with AASHTO Standard Specifications, Sixteen Edition.

3.1 Finite element analysis

A finite element analysis was carried out using SAP 2000 software to determine the magnitude of tensile stresses and the required level of prestressing in the bridge deck. The finite element model consisted of plate elements to model the concrete deck, and beam elements to model the girders. The entire bridge was modeled using 80 plate elements for the deck, and 72 beam elements for the girders.

The loading conditions included all applicable dead loads, live loads, temperature effects, as well as shrinkage of the concrete deck. Both single and multi-lane loading conditions were considered for the live load case, with the live load truck being applied at locations producing maximum load effects. The temperature range considered was 60 degrees Fahrenheit. The results of the analysis indicated that a maximum tensile stress in the deck of 400 psi, with the governing load case being shrinkage of the concrete deck. The deck was primarily under compression for the majority of the remaining load cases, as the girders are simply supported inducing compression in the deck under positive bending moments.

Previous studies by Issa et al [2] indicated that the performance of the transverse joints in precast deck panels is significantly improved if the decks are post-tensioned and maintained in compression. The longitudinal prestressing was therefore designed for 400 psi compression in the bridge deck in both directions after all prestressing losses. This level of prestressing would ensure the joints remain in compression under all loading conditions.

3.2 Time dependant analysis

A time dependent analysis was performed to determine the load redistribution effects of creep and shrinkage, the magnitude of prestressing losses over time, and the additional stresses imposed on the plate girders due to the time related deformations of the deck. The time dependent analysis for dead load, prestressing, creep and shrinkage was performed using Bridge Designer II [BD2]. This analysis tool uses matrix analysis and time dependent material properties to determine the force effects in each structural component over time, considering loading conditions and structural configuration of the bridge at each stage of the analysis. The time dependent concrete material properties were determined in accordance with Comite Euro-International du Beton, *CEB-FIP Model Code*, 1990 [4]. AASHTO code requirements were not considered suitable for the time dependent analysis of the precast deck.

3.2.1 Model description

The girders and deck were modeled as two independent elements. The construction sequence, including casting dates of the concrete bridge deck was modeled in the analysis. Prior to grouting the shear stud pocket, the deck was assumed to be sitting on top of the plate girders, with no composite action. After grouting of the shear stud pockets, rigid links were added to the model between the centroid of the concrete deck and the centroid of the steel plate girders to model the composite action of the bridge deck. The rigid links were located at the shear stud pocket block-out locations, with joints located at these locations, and beam and slab elements spanning the joints. The bridge was assumed to be fully pinned at one end and on roller supports at the opposite end. The entire width of the bridge deck was modeled in BD2.

3.2.2 Construction sequence

The time dependent analysis was carried out to reflect the actual construction sequence of the bridge deck. Loads were applied to the model as they would be applied in the field. In order to minimize additional loads imposed on the deck girders due to creep effects, the effects of age of the precast deck panels at the time of stressing was considered. The first model assumed that the deck was cast 45 days prior to stressing the longitudinal tendons. The second model assumed that the longitudinal tendons were stressed after 7 days. By comparing the difference between the two models, the effects of age of the precast deck panels at the time of stressing the longitudinal post-tensioning tendons could be determined, and incorporated into the design of the deck panels and plate girders.

3.2.3 Time dependent properties

Creep and shrinkage time dependent properties for the concrete deck panels were calculated in accordance with the CEB FIP 90, and ACI 209R-82 was used for time related changes in Modulus of Elasticity. These parameters were used in the time dependent analysis to determine the long term stresses in the deck, including all losses due to creep, shrinkage, elastic shortening and relaxation of prestressing steel.

3.2.4 Time dependent analysis results

The unfactored stress history of the bottom and top flanges at the splice locations is given for the 45 day cycle in Figures 4 and 5. Note the vertical changes in shear force representing application, or removal, of static loads. The linear changes in stress are caused by time dependent effects.

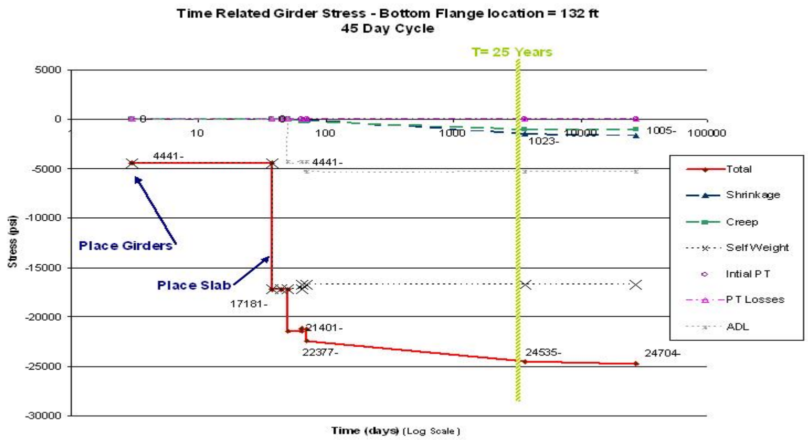

Figure 4: 45 day unfactored stress in bottom flange

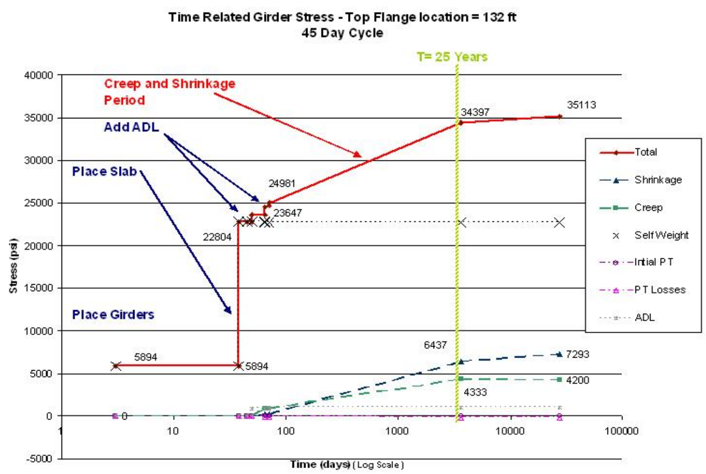

Figure 5: 45 day unfactored stress in top flange

The results of the time dependent analysis indicate an 11.5 ksi increase in compressive girder stress in the top flange due creep and shrinkage for the 45 day cycle. For the 7 day cycle, the increase in compressive stress is 12.8 ksi, or 11% greater than the 45 day cycle. The additional stress results from a combination of axial load due to creep and bending due to the eccentricity of the axial load.

The additional axial load acting on each individual girder is 380 kips at the splice location and 437.5 kips at midspan, and the corresponding bending moments acting on the steel section only are 1621.7 k-ft and 1875.5 k-ft respectively.

The linear changes in girder stress indicate the application of loads and changes in structural configuration. The gradual rise in stress after application of additional dead load (ADL) indicates the increase in stress from the time the deck is composite until 25 years and 75 years time. This increase in stress indicates the load redistribution from the deck to the plate girders due to creep.

The compressive stress in the deck decreased from 435 psi at the time of stressing to 405 psi for the 45 day cycle, and to 400 psi for the 7 day cycle. The shortening of the deck at the time of stressing was calculated to be 0.2”, with an additional 0.65” shortening occurring at 75 years. These values are based on the assumed material properties, and are derived from the time dependent analysis. These values will vary depending on the actual properties of the concrete deck. A future study is monitoring the actual performance of the deck for comparison with theoretical values.

3.2.5 Steel plate girder design

The design of the steel plate girders included the time dependent effects from the BD2 analysis. The additional loads calculated by BD2 were applied to the bare girder section, and were added to previously calculated dead and live load effects. The factored loads were compared to member resistances given in AASHTO, based on the Hybrid Steel Girder properties.

In addition to the increase in loads on the girders, the design of the splices, connections, and other components also needed to incorporate the results of the time dependent analysis. The additional stresses acting on the girders at the splice locations were converted to additional shears, axial loads, and bending moments acting at the splice points, and added to the dead and live load effects to design the splice connection. Other components such as shear studs and connections were also designed for the additional time dependent effects.

4. Construction details

The construction contract was awarded in the spring of 2004, and construction of the bridge began later that summer. Fabrication of the deck panels started before closure of the bridge to minimize the duration of the road closure.

Figure 6: deck panel casting bed

Figure 7: deck panel ready to be cast

4.1 Deck panel fabrication

The deck panels were fabricated by Carr Concrete at their Waverly plant in West Virginia. A modular steel casting bed was manufactured which simplified fabrication of the panels, see Figure 6. The panels were constructed on a two day cycle, with deck pours scheduled for the afternoon, and stripping the next morning. Forming of the panels typically required one and a half days. A deck panel ready to be cast is shown in Figure 7.

The specified concrete strength was 5,500 psi at transfer and 6,000 psi at 28 days. The specified air content was 6 ± 2%, and the maximum design permeability limit was 1500 coulombs. Concrete strengths of 6000 to 7000 psi were typically reached at 18 hours, with 28 day strengths in the order of 9,000 to 10,000 psi. The panels were wet cured for seven days.

4.2 Deck panel erection

The deck panels were erected during night time to minimize traffic disruptions on I-75. Fifteen minute lane closures were permitted while erecting deck panels over traveled lanes. All sixteen deck panels were erected in two nights, with the majority of lane closures lasting less than fifteen minutes.

Following casting of the shear keyways, the stressing and grouting of the longitudinal tendons was carried out. The stressing and grouting operations took approximately one week to complete, which was followed by the casting of the girder haunches, end diaphragms, sidewalks and railings. The deck was milled ½" prior to casting the sidewalks to achieve the desired roadway profile.

5. References

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