CONSTRUCTION AND COMPOSITE BEHAVIOR OF PRECAST BRIDGE DECK PANEL SYSTEMS

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ABSTRACT

Experimental and analytical studies were conducted at Virginia Tech to investigate the behavior of precast bridge deck panel systems. A full scale, single span bridge was constructed at Virginia Tech. Fatigue testing and ultimate load tests were done with different shear connector types and pocket spacings. The construction process was well documented and different construction details were compared for strength, durability, and ease of construction.

Finite element studies were performed using the finite element software DIANA. Once the modeling methodology was verified with experimental data, the models were extrapolated to study the behavior of the system by varying the number of shear connectors in the shear pockets and varying the distribution of the connectors among the pockets.

Results of the research program were used to recommend the best construction details to use and propose design practices that account for the variables that make precast bridge deck panel systems unique.

Keywords: Bridge, Concrete, Deck, Panels, Precast, Prestressed

INTRODUCTION

Precast bridge deck panels can be used in place of a cast-in-place concrete deck in order to reduce bridge closure times for deck replacements. The panels are prefabricated at a precasting plant providing optimal casting and curing conditions. The panels are then transported to the bridge site for immediate erection.

Composite action between the deck and girders is provided by shear connectors that extend out of the girder and into the shear pockets of the panels. The shear connectors are clustered together at the shear pockets instead of having a more uniform shear connector spacing found with cast-in-place concrete decks. The connectors typically consist of either hooked reinforcing bars or shear studs.

The discrete locations of the shear connectors raises questions about the proper way to design for horizontal shear transfer. The pocket spacing is typically 2 ft. Larger pocket spacing is desirable because it results in less grout that has to be poured during the bridge closure, and fewer blockout forms that have to be placed during fabrication. This also allows for shorter construction delays. Current design provisions do not address the design of shear connectors for precast bridge deck panel systems.

OBJECTIVES

The research program was developed to address the challenges and problems discussed in the previous section. By doing so, current design provisions and practices can be improved and modifications to code provisions can be made, if necessary.

The first objective was to examine the constructability of the system. A bridge consisting of precast deck panels and precast, prestressed concrete girders was built in the Virginia Tech Structures and Materials Laboratory. This bridge was referred to as the lab mockup. The construction process was well documented. Particular attention was given to the transverse joint details, the types of shear connectors, and the construction process. Some details may result in more relaxed casting tolerances and/or reduce construction time.

The second objective was to study the composite action of the system. The hooked reinforcing bars and the new detail with shear studs were both considered in the testing program. The new shear stud detail will be discussed in the next section. Both cyclic and overload tests were performed. The strains in the shear connectors and the vertical deflections of the system were used as the primary indicators for the level of composite action. The shear pocket spacing was also examined to see if 4 ft pocket spacing performed adequately compared to 2 ft pocket spacing. Finite element studies were also conducted to aid in making more general conclusions about the composite action of the system. Modifications to current code provisions were suggested as necessary.

RESEARCH PROGRAM

An experimental research program and analytical research program were developed in order to accomplish the objectives. The experimental research program consisted of static and cyclic tests on a simply supported, full scale bridge built at the Virginia Tech Structures Laboratory. The analytical program consisted of finite element analyses using the commercial software DIANA.

EXPERIEMENTAL RESEARCH PROGAM

Design of the Lab Mockup

The design was based upon a 40 ft long simply supported bridge with 5 girder lines, spaced at 8 ft center to center. This span length was selected because of the limited available floor space in the Virginia Tech Structures Laboratory. The lab mockup consisted of 2 AASHTO Type II girders, 40 ft long, spaced at 8 ft center to center. The AASHTO Type II girder was the most efficient girder to use for the 40 ft simple span. The deck was 8 in. thick, with a 2 ft overhang. The haunch between the panels and girders was 2 in. The lab mockup is shown in Figure 1.

Twelve - ½ in. diameter strands were provided in the longitudinal post-tensioning ducts to provide a compressive stress across the transverse joints. The calculated initial level of post-tensioning after all initial losses was 268 psi. The calculated effective level of post-tensioning after all long term losses was 200 psi.

Because the shear connectors were clustered together in shear pockets instead being dispersed in a more uniform manner along the length of the bridge, the number of required connectors for each pocket was selected instead of a required connector spacing at a given location. The following design procedure was followed for each pocket:

1. The vertical shear force at the location under consideration was calculated.
2. The horizontal shear force per inch was calculated using the equation:

$$V_h = \frac{V_u}{d_v} \qquad \text{Eqn. 1}$$

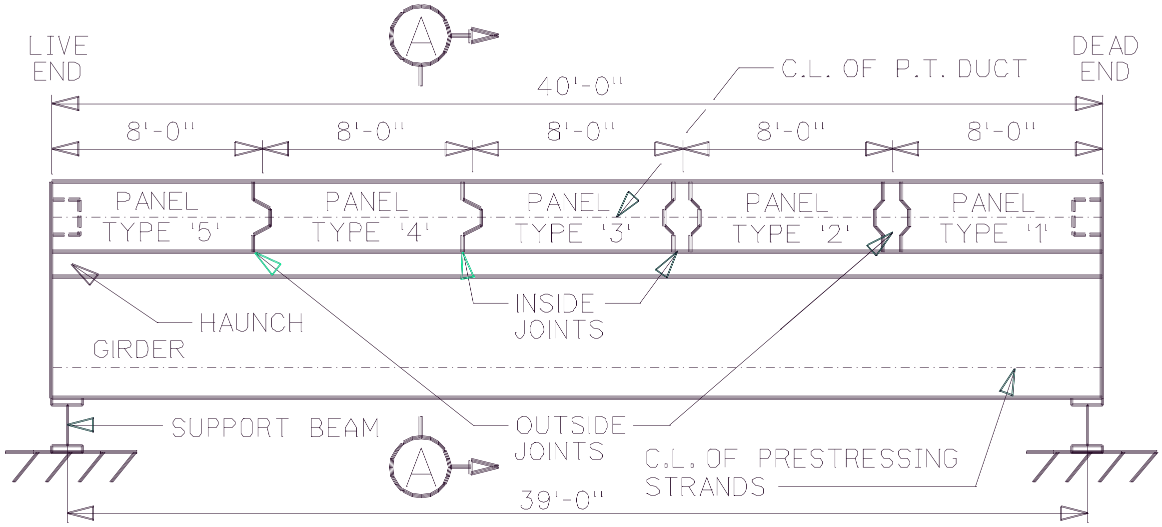
   where,
   V_u = vertical shear force
   d_v = distance between the centroid of the steel in the tension side of the girder to the resultant center of the compressive force in the deck.
3. The tributary pocket spacing was calculated. The tributary pocket spacing was half the pocket spacing on each side of the pocket under consideration.
4. The horizontal design shear force was calculated by multiplying the shear force per inch by the tributary pocket length.
5. The following equation was used to select the number of required shear connectors:

$$A_{s\_pocket} = \frac{\dfrac{\dfrac{V_u l_v}{d_v \phi} - b_v l_v c}{\mu} - P_c}{f_y} \qquad \text{Eqn. 2}$$

where,

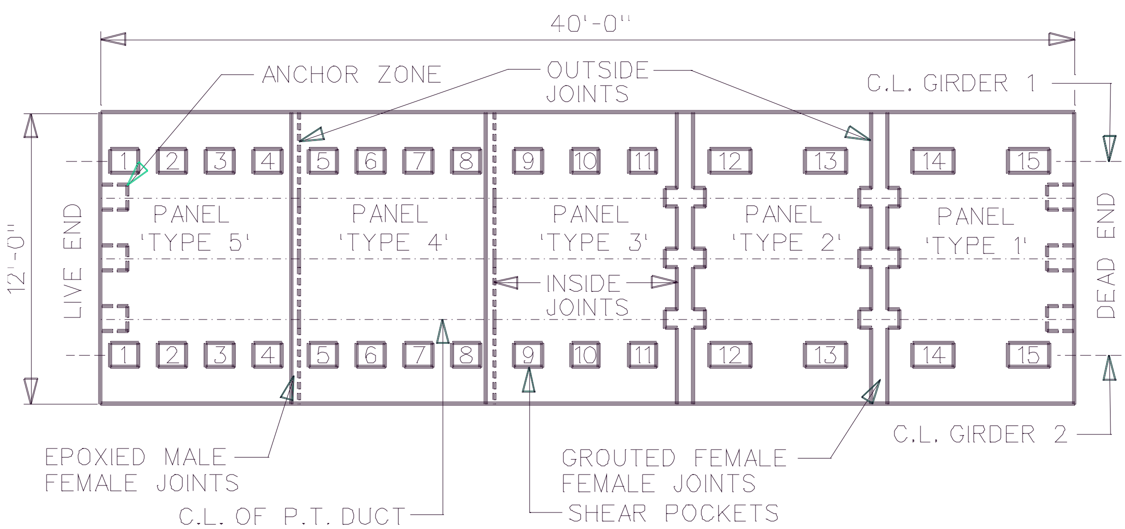
V_u = factored vertical shear force
ϕ = strength reduction factor = 0.9
d_v = distance between the centroid of the steel in the tension side of the girder to the resultant center of the compressive force in the deck
b_v = width of the surface area engaged in shear transfer
l_v = tributary pocket spacing
c = cohesion factor
= 75 psi for not intentionally roughened surface between two concrete surfaces cast at different times (used for girder 1)
= 25 psi for a surface formed by steel and concrete (used for girder 2)
μ = friction factor
= 0.6λ for not intentionally roughened surface between two concrete surfaces cast at different times (used for girder 1)
= 0.7λ for a surface formed by steel and concrete (used for girder 2)
λ = 1.0 for normal density concrete
f_y = yield strength of the shear reinforcement
P_c = permanent net compressive force normal to the interface

In order to provide a more uniform shear connector design, the same number of shear connectors was provided in several pockets. This caused many of the pockets in regions with small shear forces to be over designed. However, the system was intentionally overdesigned for flexure and vertical shear so that the behavior of the different types of shear connectors and pocket spacing could be studied.



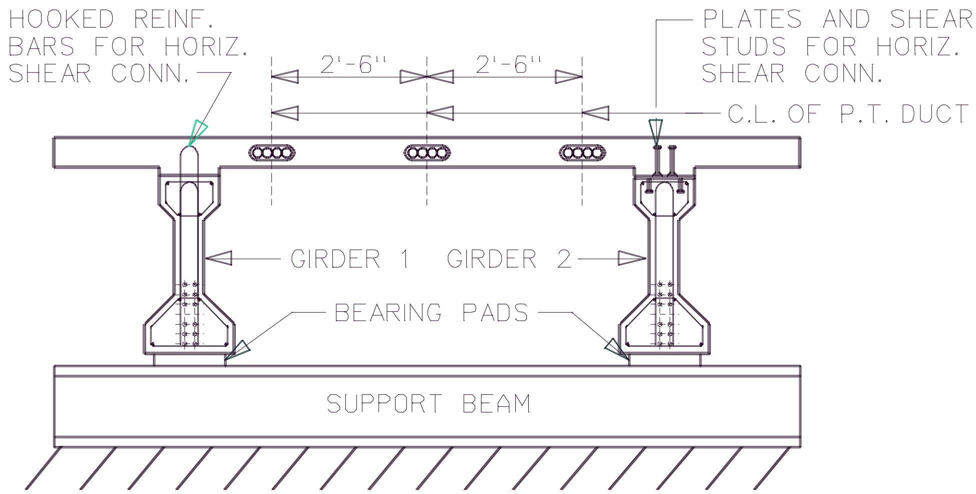
LIVE END
DEAD END
C.L. OF P.T. DUCT
40'-0"
8'-0"
8'-0"
8'-0"
8'-0"
8'-0"
PANEL TYPE '5'
PANEL TYPE '4'
PANEL TYPE '3'
PANEL TYPE '2'
PANEL TYPE '1'
HAUNCH
INSIDE JOINTS
GIRDER
SUPPORT BEAM
OUTSIDE JOINTS
C.L. OF PRESTRESSING STRANDS
39'-0"
A
A

(a)



40'-0"
ANCHOR ZONE
OUTSIDE JOINTS
C.L. GIRDER 1
12'-0"
LIVE END
PANEL 'TYPE 5'
PANEL 'TYPE 4'
PANEL 'TYPE 3'
PANEL 'TYPE 2'
PANEL 'TYPE 1'
DEAD END
INSIDE JOINTS
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
EPOXIED MALE FEMALE JOINTS
GROUTED FEMALE FEMALE JOINTS
C.L. GIRDER 2
C.L. OF P.T. DUCT
SHEAR POCKETS

(b)



(c)

Figure 1 Lab Mockup Details (a) Elevation View (b) Plan View (c) Section View

The new shear connector detail with the shear studs was fabricated by casting a steel plate in the top flange of a prestressed girder. Five, ¼ in. thick plates were placed in the top flange of girder 2 immediately after the concrete was placed in the formwork, as shown in Figure 2. Five smaller plates were used as opposed to one large plate in order to make placing the plates easier. The shear studs on the bottom of the plate were shot into place prior to casting the girders. Additional shear studs were then shot directly to the top of the steel plate after the girder was erected and the panels were placed. Figure 3 shows the required dimensions to satisfy AASHTO[1,2] cover and spacing requirements for the new connector detail. In Figure 3, 'd' is the stud diameter.

Figure 2 Placement of the Plates in Girder 2.

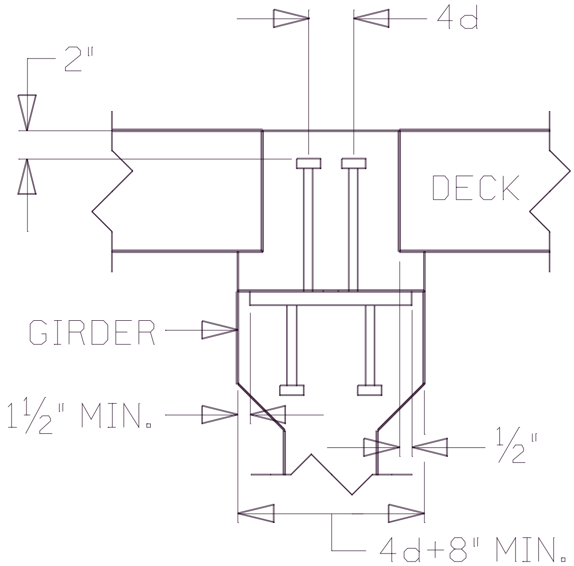


Figure 3 Requirements for New Shear Stud Detail

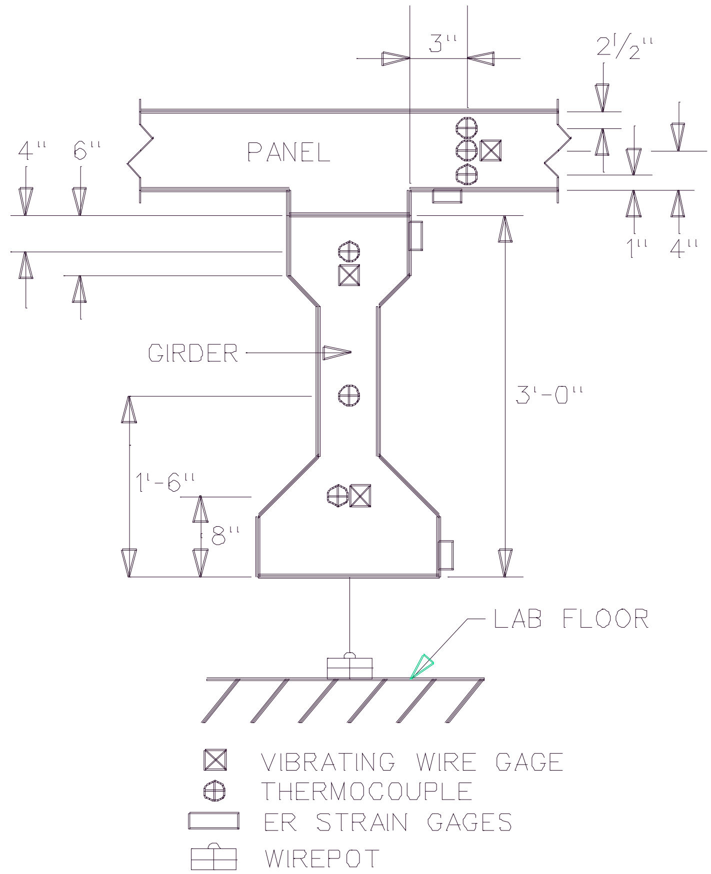


Figure 4 Location of Instrumentation for Panels and Girders

Instrumentation

During the casting operation the panels and girders were instrumented with thermocouples and VWGs (vibrating wire gages). The thermocouples and VWGs were placed such that they would be located at the 1/3 points of the span of the bridge. Figure 4 shows the location of the VWGs and thermocouples through the depth of the cross section. The VWGs and thermocouples were used in another phase of this research program to examine creep and shrinkage behavior.

The panels and girders were instrumented with ER (electrical resistance) strain gages and wirepots. Figure 4 shows the location of the ER strain gages, which were located at the 1/3 points of the span of the bridge. ER strain gages were also placed on selected shear connectors to measure the strain in the horizontal shear connectors during cyclic testing and static testing.

Wirepots were used to measure the vertical displacement of the bridge. The wirepots were placed under the load points. Figure 4 shows the location of the wirepots under the girders.

Constructability Study

The different stages of fabrication and construction of the lab mockup were examined. Recommendations were made for the transverse joint and shear connector type that worked best from a constructability point of view. The method for forming the transverse joints, method for creating an efficient strand pattern, and method for creating the shear pockets were examined during the fabrication of the panels. Other important observations were also recorded as seen fit.

Live Load Testing

The live load testing program consisted of cyclic testing up to 2 million cycles at a frequency of 2 Hz and a series of static tests. Figure 5 shows the test setup on the dead end and live end. The shear pockets are left out of the figure for clarity. The loading for each of the two test setups (dead end and live end) consisted of four load patches placed symmetrically about the longitudinal centerline of the bridge. Each frame applies 2 wheel loads. The symmetric loading was done to attempt to create the same loading on each girder. By doing so, the performance of different shear connectors could be compared. The two test setups are symmetric about the transverse centerline (midspan) of the bridge. This allowed the performance of the system with the different pocket spacings to be compared.

For the first 500,000 cycles, the load ranged from 2 k/frame to 29.4 k/frame. This created the AASHTO LRFD design fatigue moment of 2250 k-in for the girder, which was calculated during the design phase of the study. During the cyclic tests, a compressive force of at least 1 kip was always present to prevent rotational movement of the spreader beam and to prevent damage to the bridge. A wheel load for the next 1,500,000 cycles ranged from 2 k/frame to 44.7 k/frame. The range of the loading, 42.7 k/frame, corresponds to two typical AASHTO design wheel loads of 16 kips, multiplied by an impact factor of 1.33. This was greater than the AASHTO LRFD impact factor of 1.15 for fatigue.

Every 100,000 cycles to 300,000 cycles, the cyclic testing was stopped to conduct a static test on the system. The load was gradually increased up to 44.7 k/frame. These intermediate static tests were done to see if there was any loss in stiffness in the lab mockup due to loss of composite action, cracking, sliding at the joints, etc. throughout the cyclic test program.

After the cyclic testing was completed on the dead end and live ends of the bridge, a static test was performed on the dead end and then the live end. The purpose of this test was to see if the required flexural strength of 15,500 k-in and required vertical shear strength of 152 k could be reached before a failure was observed. The load was gradually increased until failure. An elastic analysis determined an applied load of 187 k/frame would produce a moment equal to the required flexural strength of 15,500 k-in and an applied load of 212 k/frame would produce a shear equal to the required vertical shear strength of 152 k.

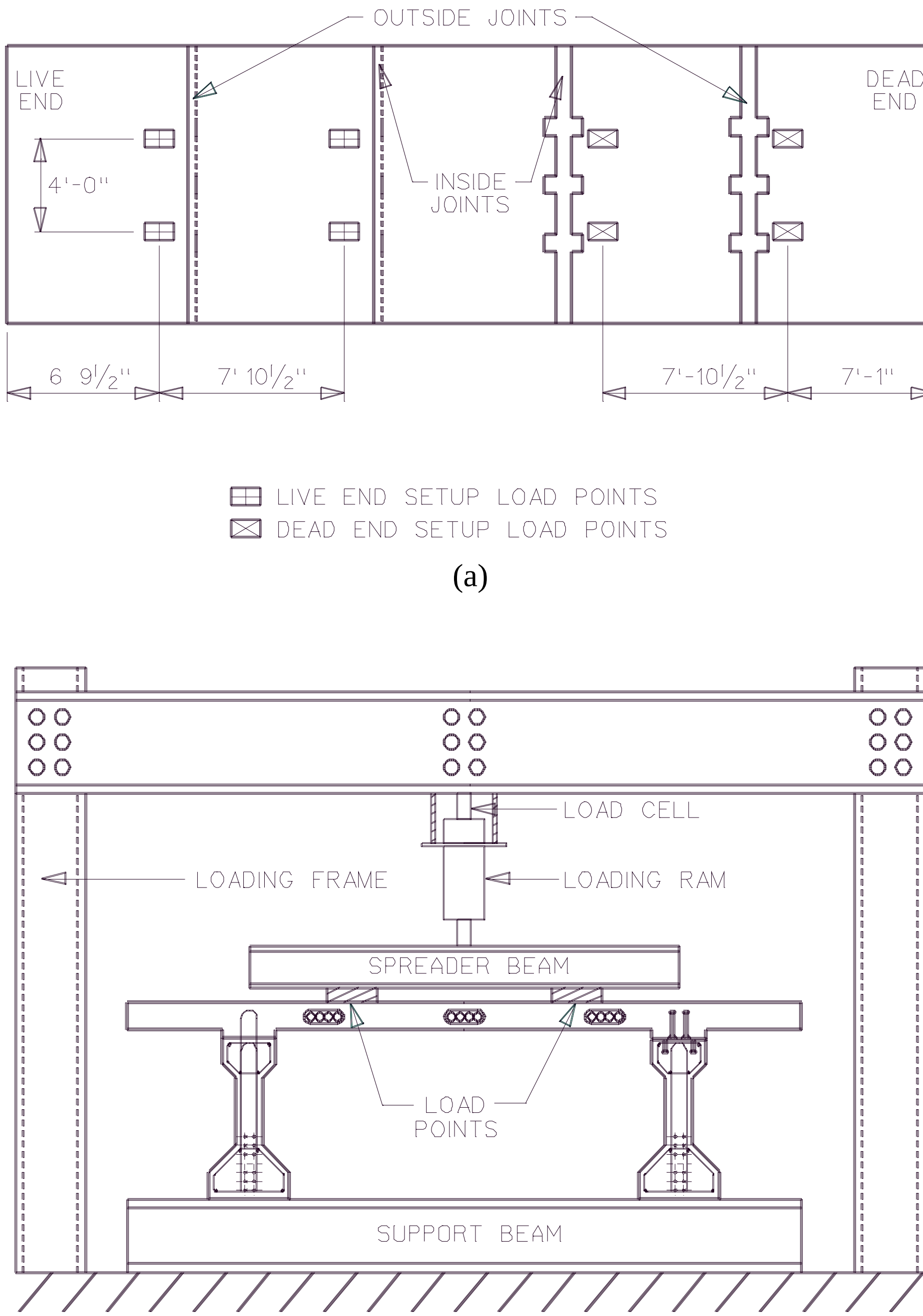


Figure 5 Live Load Test Setups (a) Plan View (b) Elevation View

FINITE ELEMENT RESEARCH PROGRAM

A series of plane stress finite element analyses were carried out to examine the flexural and shear capacities of the deck panel system compared to predicted values from the design calculations. One girder of the lab mockup was modeled at a time with a 6 ft tributary deck width.

Once the results of the finite element models were compared to the experimental results, the parametric study was conducted. The parametric study consisted of varying the amount of shear connectors in each pocket and the distribution of the shear connectors among the pockets. This allowed for additional insight in to the influence of the pocket spacing and connector type on the behavior of the deck panel system.

A total of 11 different models were examined for the parametric study. Six of the models were run with No. 5 hooked reinforcing bars as shear connectors and five of the models were run with ¾ in. diameter shear studs as shear connectors. Two different equations were used to determine the required number of shear connectors. One of the equations used was Equation 2. The other equation used was

$$Q_r = 0.5 A_{sc} \sqrt{f_c' E_c} \le A_{sc} F_u \qquad \text{Eqn. 3}$$

where,

Q = shear strength of an individual shear connector
A_{sc} = cross sectional area of a shear stud
f_c' = 28 day compressive strength of the concrete in the deck
E_c = modulus of elasticity of the concrete at 28 days
F_u = minimum tensile strength of the shear connectors

Model MOCKUP had the exact number of shear connectors used for the lab mockup. Model 2_100 had close to the exact number of shear connectors required per pocket using Equation 2. Model 2_75 had approximately 75% of shear connectors required per pocket using Equation 2. Model 2_50 had approximately 50% of shear connectors required per pocket using Equation 2. Model 3_R had close to the number of shear connectors required using Equation 3. The shear connectors were distributed in an even manner among the pockets. The connectors were also distributed among the shear pockets so the dead end and live end of the bridge had approximately the same amount of shear connectors. Model 3_L had close to the number of shear connectors required using Equation 3. Unlike model 3_R, the shear connectors were distributed among the pockets such that more shear connectors were placed in locations with high shear stresses.

Figure 6 shows the mesh for the finite element model. The mesh was refined in the vicinity of the interface between the haunch and girder. Eight node quadrilateral elements were used to model the panels, haunch, girder, and bearing pads. Three node beam elements were used to model the shear connectors. Only the beam elements provide rotational stiffness at the nodes that the beam elements share with the plane stress elements. Three node interface elements were used to model the interface between the haunch and girder. Embedded reinforcing bars were used to model the vertical shear stirrups, the mild longitudinal reinforcing steel, the strands for post-tensioning in the panels, and the strands for prestressing in the girder.

A smeared cracking approach was used with a linear tension softening model and a linear tension cut-off model to account for the influence of a multi-axial stress state. Crushing was defined by using the Von Mises failure criterion along with a uniaxial, multi-linear stress vs. strain curve to capture the plastic behavior of concrete and grout in compression. The constant shear retention model was used instead of the full shear retention model. For the models in this study, it was assumed that 50% of the shear stiffness was lost when the crack was formed (β=0.5).

Modeling the behavior of the shear connectors in a bridge is a complex problem. When a large shear is transferred from the deck to the girder, the interface cracks causing the two surfaces to separate and slip relative to one another. In turn, a tensile force is developed in the shear connector causing a compressive force and corresponding frictional force to develop at the interface. This frictional force increases the horizontal shear capacity of the system. Accurately modeling this "clamping effect" was difficult to accomplish in DIANA.

A nonlinear, elastic material was selected for the interfaces because of its stable and predictable behavior. The user specifies a normal stress vs. relative opening diagram and a tangential stress vs. slip diagram to define the behavior of the material. The tangential stress vs. slip diagram was defined such that there was still a small amount of shear resistance after the interface "cracks," as shown in Figure 7. The normal stress vs. relative opening diagram was defined such that the stiffness in the normal direction was very large.

Neglecting the "clamping effect" was conservative and acceptable for this type of analysis and it resulted in larger slip values and higher strains in the shear connectors. Because the strain levels and slip values were larger than expected, the acceptable number of shear connectors from the parametric study was conservative. More details on the modeling methodology and the material models can be found in Sullivan[3].

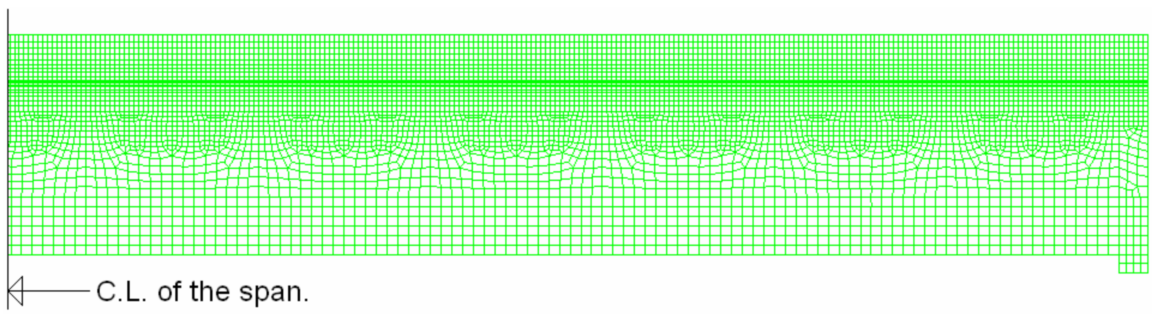


Figure 6 Mesh for Live Load Tests on Lab Mockup.

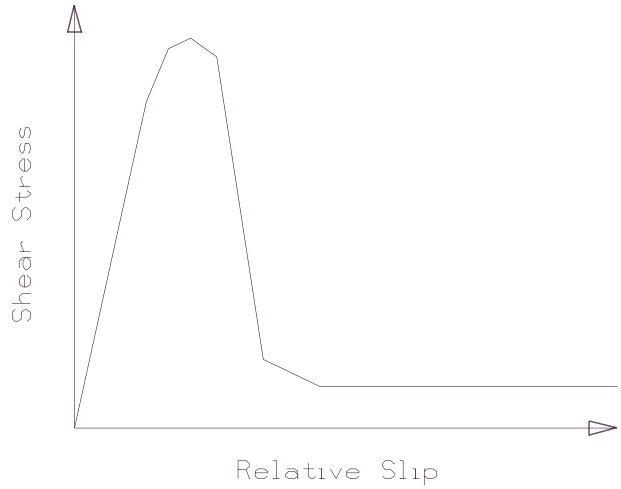


Figure 7 Tangential Stress vs. Slip Diagram for Interface Material Without Shear Connectors

EXPERIMENTAL RESULTS

CONSTRUCTABILITY RESULTS

Strand Pattern

When laying out the strand pattern for the precast panels, the panel production and stressing bed layout should be considered. Minimizing the number of strand patterns allows for more panels to be cast in a stressing bed at one time with fewer strands having to be debonded in panels that do not require a specific strand.

Transverse Joints

From purely a construction standpoint, the grouted female-female joint is better than the epoxied male-female joint. Neither of the joints takes significantly more time than the other to fabricate. The female-female joints allow the panels to be placed on the girders without having to slide the panels together while avoiding conflicts with the horizontal shear connector layout. The female-female joint configuration is also more forgiving if the edges of the panels are bowed. The variation in the gap between the adjacent panels along the length of the joint caused by bowing can be compensated for by allowing at least a ½ in. gap between the panels at the joint. A gap of at least 1 ½ in. is recommended if the grout is going to be vibrated in to place. The one advantage that the male-female joint has is that it is more aesthetically pleasing if an overlay is not going to be provided on the bridge deck. In the majority of cases, an overlay is provided.

Shear Connectors

The steel plate with post-installed shear studs is a better detail for the system to resist the horizontal shear forces compared to the hooked reinforcing bars, from a construction standpoint. It is quicker to place the steel plate with the shear studs in the wet concrete than tying the hooked reinforcing bars in with the reinforcing steel cage. However, this may not be the case if the hooked reinforcing bars are detailed such that some of the stirrups along the length of the beam are extended up in to the shear pockets.

The steel plate detail allows the leveling bolts to bear directly on the girders since the majority of the top surface of the girder is steel. Girders with hooked reinforcing bars need to have a steel plate placed on top of the girder so none of the post-tensioning force is transferred to the girder via a frictional force that develops between the leveling bolt and girder.

The steel plate detail with the post-installed shear studs also allows the panels to be moved during erection without having to worry about conflicts with the shear connector layout. Erecting panels with shear connectors already in place may damage the shear connectors if the panels collide with them. The steel plate detail is also better from a safety standpoint since the tripping hazard is eliminated.

Post-Tensioning Operation

During the post-tensioning operation, the force may be partially transferred from the deck to the girders via the leveling bolts. This happens if the interface between the leveling bolt and girder is rough enough to develop a significant frictional force. This frictional force can be reduced by using lubricated steel plates for the leveling bolts to bear on. This is shown in Figure 8. Dial gages, or similar instrumentation to measure the deflection of the system, can be used to monitor any change in deflection that may take place during the post-tensioning operation. In place of instrumentation to measure any change in deflection, strain gages are just as effective in detecting any force that may be transferred to the girder during the operation. Surveying equipment may be used to detect any force transfer to the girder when the use of strain gages or dial gages is not feasible.

Figure 8 Leveling Bolt Bearing on a Steel Plate

LIVE LOAD TESTING RESULTS

The cyclic testing had minimal effects, if any, on the degree of composite action in the lab mockup. There was no cracking at the transverse joints and no relative vertical movement between adjacent panels was measured by the wirepots. No cracking was observed in the girder or deck. There was a negligible difference in the vertical deflections measured during the intermediate static tests for the cyclic testing program. The strains in the shear connectors were less than 1% of the nominal yield strain. The shear connectors were not engaged in resisting the horizontal shear stresses developed during the cyclic testing for 2 ft pocket spacing and 4 ft pocket spacing. This indicates there was a negligible loss in the cohesive bond between the grout and concrete.

Figure 9 shows the deflections of the dead end (4 ft pocket spacing) of the lab mockup during the static test at the outside loading point and the inside loading point. The inside and outside loading points are shown in Figure 5. In Figure 9, G1_OUTSIDE stands for the vertical deflection as the outside loading point measured by a wirepot under girder 1 (with the hooked reinforcing bars as shear connectors), G2_INSIDE stands for the vertical deflection as the inside loading point measured by a wirepot under girder 2 (with the new shear stud detail as the shear connector system), etc. The vertical stiffness values were compared for the different static tests. The calculation of the vertical stiffness at a load point was calculated as $K_{vert}=P/\delta$, where P is the applied load at the load point, and δ is the vertical deflection at the load point,

For applied loads greater than 270 k/frame, the vertical stiffness at the inside load point and outside load point were 10.0 k/in. and 18.3 k/in., respectively. This corresponds to 1.3% and 1.4% of the initial vertical stiffness at the inside load point and outside load point, respectively.

At an applied load of 256 k/frame, there was a significant decrease in stiffness. This load does not include the dead load of the system. One reason for the decrease in stiffness may be from the prestressing strands in the girders exceeding the nominal yield strain of the prestressing strands. The load at which the prestressing strands exceed the nominal yield strain was calculated to be 269 k/frame. This was confirmed with the finite element models. The finite element results indicated the top layer of prestressing strands reach a strain of 0.012 in./in. at an applied load of 268 k/frame for the dead end load setup.

When 272 k/frame was reached, the load was intentionally reduced to 237 k/frame. The load was then increased up to 287 k/frame. Figure 9 shows how the vertical stiffness at the load points was greater upon reloading, compared to the vertical stiffness along the original load path at 272 k/frame. Once the applied load reached 272 k/frame again, the vertical stiffness had decreased back to 10.0 k/in. at the inside load point.

Figure 10 shows the strains in the shear connectors for the static test at the dead end. At an applied load of 256 k/frame, the rate at which the strains increased with respect to the load increased. This indicated that the shear connectors were engaged in resisting the horizontal shear stresses as the cracking at the interface between the haunch and girder continued to increase. The strains in the shear connectors in Figure 10 were less than 50% of the nominal yield strain for the entire range of applied loads. The strain behavior in the majority of the instrumented shear connectors was the same with the exception of shear connector G2_R4. Problems were encountered with this gage on preliminary live load tests. This may be a contributing factor to the difference in the strain for this connector when compared to other connectors and may not represent true physical behavior. However, the strain in shear connector G2_R4 was still small and does show the increase in the strain rate at an applied load of 256 k/frame like the rest of the shear connectors.

Figure 11 shows the deflections of the live end (2 ft pocket spacing) of the lab mockup during the static test at the outside loading point and the inside loading point. The initial vertical stiffness of the lab mockup at the loading points was less than the initial vertical stiffness at the loading points during the intermediate static tests for the cyclic testing program.

Cracking occurred at the interface between the haunch and girder at applied loads of 260 k/frame and 196 k/frame for girder 1 and girder 2, respectively. The rate the strains in the shear connectors increased with respect to the applied load was greater than the strain rate increase prior to cracking in the haunch. The increase in the strain rate with respect to the load indicates the shear connectors were engaged in resisting the horizontal shear stresses after cracking at the interface between the haunch and girder. However, the strains in the shear connectors are less than 50% of the nominal yield strain for the entire range of applied loads.

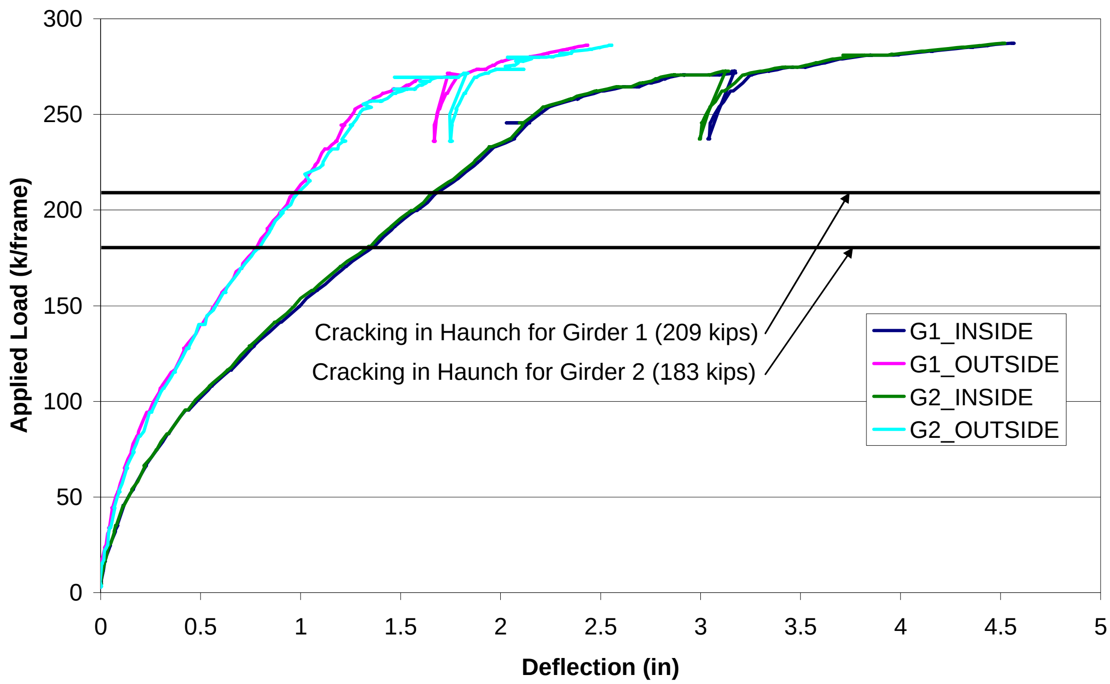


Figure 9 Deflections During the Static Test at the Dead End

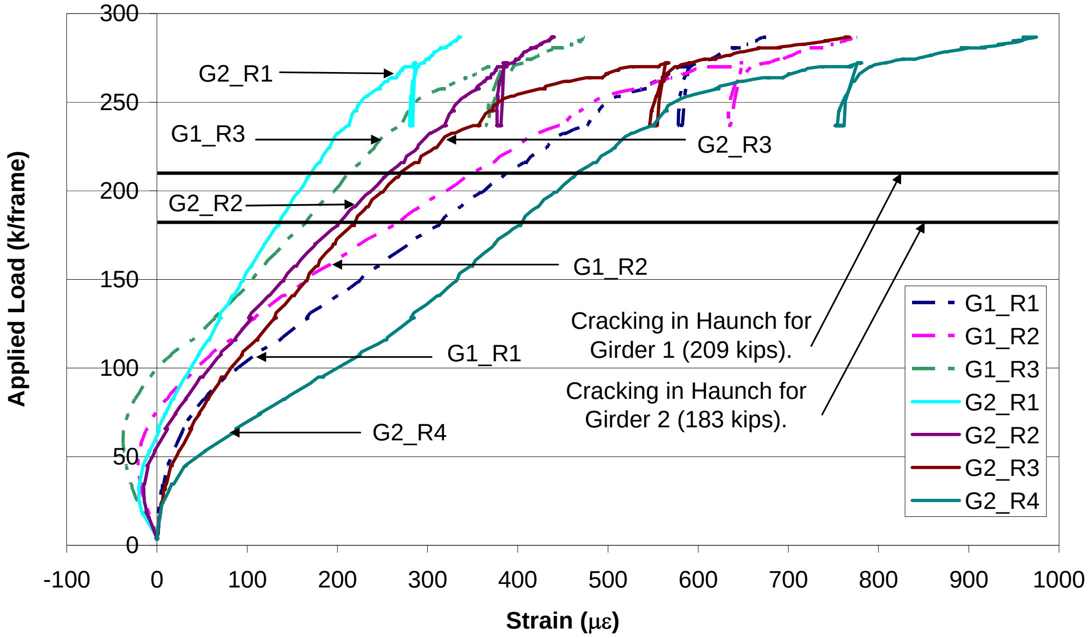


Figure 10 Connector Strains During the Static Test at the Dead End

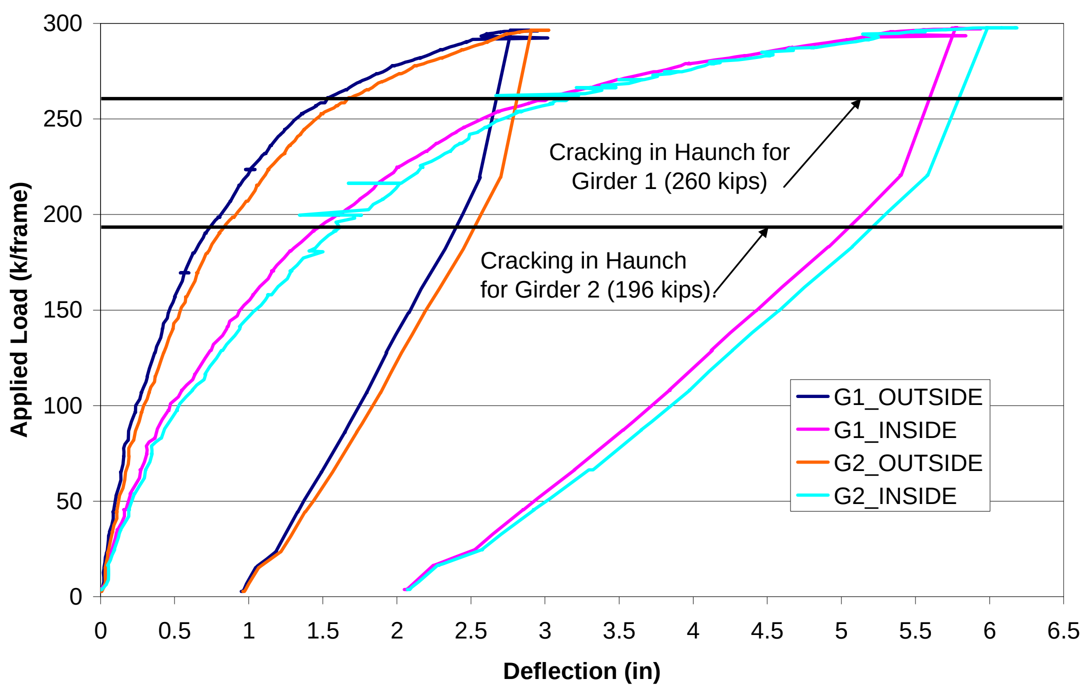


Figure 11 Deflections During the Static Test at the Live End

Both the live end and dead end of the lab mockup failed in flexure by crushing of the concrete on the top surface of the bridge deck. The maximum moment reached during the static tests on the dead and live ends of the lab mockup were 23,700 k-in. and 24,500 k-in., respectively. The difference in the maximum moments for the two tests is 3%. The AASHTO LRFD required flexural capacity of the lab mockup was 15,500 k-in. The required capacity is defined here as the factored design load divided by the strength reduction factor. The pocket spacing had very little influence upon the flexural capacity of the lab mockup. The maximum shear reached during the static tests on the dead and live ends of the lab mockup are 206 k and 213 k, respectively. The required vertical shear capacity of the lab mockup is 152 k. The lab mockup with either the 2 ft pocket spacing or 4 ft pocket spacing was capable of exceeding the required vertical shear strength. The flexural design and vertical shear design were modified such that the lab mockup would not fail before the horizontal shear forces exceeded the horizontal shear design capacities in the regions with high shear forces.

Table 1 shows the ratio of the resulting horizontal shear force at each shear pocket from the static tests to the nominal horizontal shear capacity at each shear pocket from the AASHTO LRFD shear friction equation. Pocket 1 was closest to the live end of the bridge with the 2 ft pocket spacing and pocket 15 was closest to the dead end of the bridge with the 4 ft pocket spacing. The regions from the support to the outside loading point had the highest shear, based on the shear diagram for the loading conditions. The region with the highest shear incorporates all the pockets within the exterior panel. The horizontal shear force developed at each pocket during the static tests is 19% higher and 24% higher than the nominal horizontal shear capacity at the dead end and live end, respectively. This indicates the number of shear connectors can be reduced and the lab mockup can still reach the required flexural strength, the required vertical shear strength, and the required horizontal shear strength. The finite element study investigated the issue of reducing the number of shear connectors in the shear pockets while still providing the required strength for the system.

Both the 2 ft pocket spacing and 4 ft pocket spacing are capable of providing the required strength. Additionally, the strain levels of less than 50% of the nominal yield strain at maximum loads in all of the connectors indicate both the hooked reinforcing bars and the shear studs perform well as shear connectors.

Table 1 Ratio of Applied Horizontal Shear to Horizontal Shear Capacity for the Final Static Tests

pocket #	$V_{applied}/V_{prov}$ Dead End	$V_{applied}/V_{prov}$ Live End
1	0.47	1.24
2	0.47	1.24
3	0.47	1.24
4	0.47	1.24
5	0.47	0.37
6	0.47	0.37
7	0.47	0.37
8	0.52	0.41
9	0.56	0.58
10	0.56	0.58
11	0.64	0.67
12	0.49	0.67
13	0.52	0.70
14	1.19	0.49
15	1.19	0.49

FINITE ELEMENT RESULTS

Figure 12 shows the differences in the response of the system for different hooked reinforcing bar quantities as shear connectors with 4 ft pocket spacing. At an applied load of 187 k/frame, the MOCKUP model had the lowest vertical deflection of 0.38 in. and the 3_R model had the highest vertical deflection of 0.67 in. The 2_100 model had a vertical deflection of 0.43 in., which was slightly greater than the vertical deflection of 0.38 in. for the model MOCKUP.

Figure 13 shows the differences in the response of the system for different shear stud quantities as shear connectors with 4 ft pocket spacing. Model 3_R provided the largest loss in composite action. The peak load for this model was 157 k/frame, which was 16% less than the load that produces the required nominal flexural strength. The peak loads reached by the rest of the models with shear studs as shear connectors with 4 ft pocket spacing were similar, ranging from 196 k/frame to 204 k/frame. Model 3_L had a vertical deflection more than 17% larger than the models that reach the load that produces the required nominal flexural strength.

When slip occurs at the interface between the haunch and girder, the shear stresses in the connectors can become large compared to the axial stresses. Because of the multi-axial stress state in the connectors, it was desirable to examine the Von Mises stresses. The Von Mises stresses can be directly compared to the yield stress for a connector in a uniaxial test.

Table 2 presents the maximum Von Mises stresses in the shear connectors at a load level that developed the required flexural strength (187 k/frame). The model 2_50 was only considered with hooked reinforcing bars as shear connectors. The model 3_R with shear studs as shear connectors was not able to reach the load that creates the required flexural strength. The maximum stress always occurred in one of the pockets near the supports. The models MOCKUP, 2_100, and 2_75 produced acceptable stress levels when hooked reinforcing bars were used as shear connectors. The models 2_50, 3_R, and 3_L exceed the nominal yield stress. However, the model 3_L exceeded the nominal yield stress of 60 ksi for the hooked reinforcing bars with 4 ft. pocket spacing by only 3.8%. This design was very similar to model 2_75.

All of the models with shear studs as the shear connectors exceeded the nominal yield stress of 50 ksi when the required flexural strength was reached. The models with shear studs overestimated the loss of composite action more than the models with hooked reinforcing bars. The experimental data from push-off tests[4] shows that the shear stud quantities used for the finite element model MOCKUP was satisfactory. Therefore, the Von Mises stresses for the models with shear studs were compared to the Von Mises stresses for the model MOCKUP. If the Von Mises stresses for a model with shear studs was similar to the stress level for the model MOCKUP, then the stress level in the connectors for the model under consideration was deemed acceptable. The maximum stresses in the models 2_100 and 2_75 were comparable to the maximum stresses in the model MOCKUP. The Von Mises stresses in the models 3_R and 3_L were close to the nominal ultimate strength of 70 ksi for the shear studs and the stresses were considerably greater than the maximum stresses in the model MOCKUP.

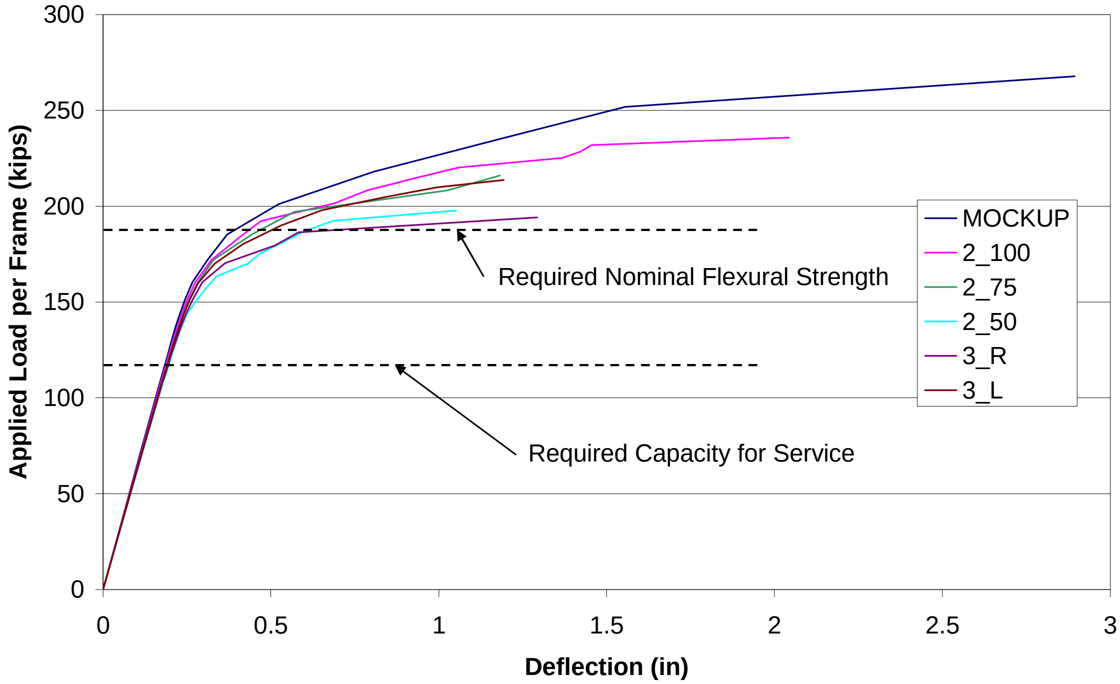


Figure 12 Comparison of Load vs. Deflection Response for Different Hooked Reinforcing Bar Quantities with 4 ft Pocket Spacing at the Inside Load Point

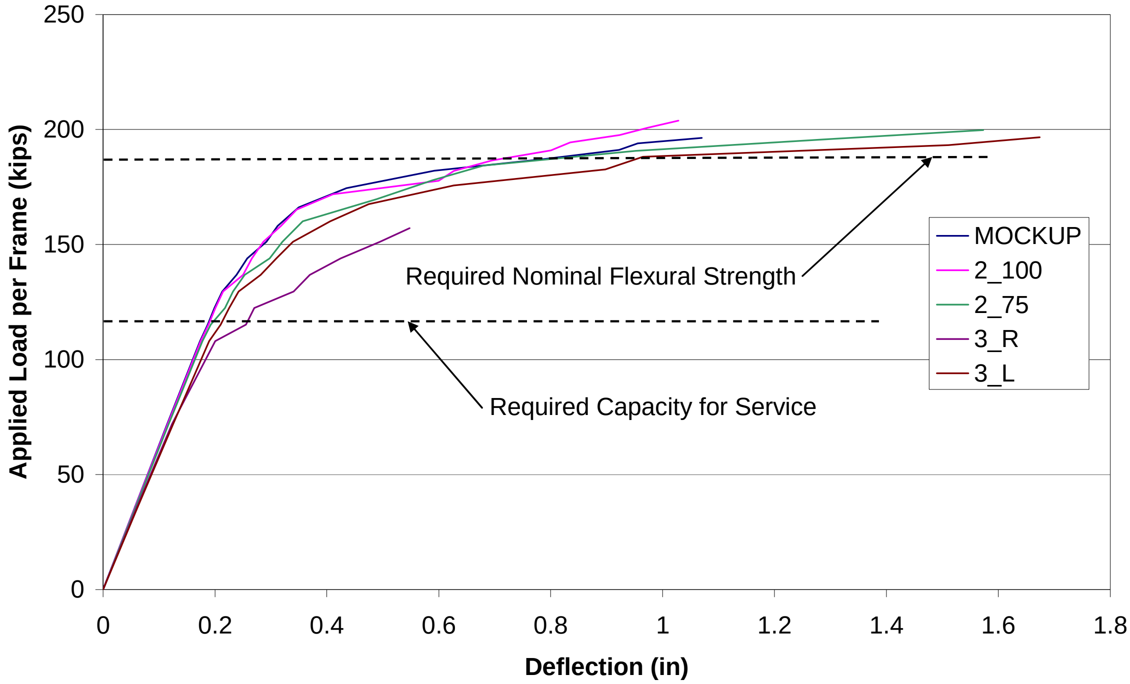


Figure 13 Comparison of Load vs. Deflection Response for Different Shear Stud Quantities with 4 ft Pocket Spacing at the Inside Load Point

Table 2 Maximum Von Mises Stresses in Shear Connectors at the Required Flexural Strength

Model Name	Hooked Reinforcing Bar Connectors		Shear Stud Connectors	
	4 ft Pocket Spacing	2 ft Pocket Spacing	4 ft Pocket Spacing	2 ft Pocket Spacing
MOCKUP	51.5	52.8	59.7	54.1
2_100	56.7	54.2	57.6	51.7
2_75	59.3	59.6	61.0	62.2
2_50	69.9	75.4	x	x
3_R	76.7	68.4	67.9 *	68.5 **
3_L	62.3	59.2	68.4	67.7

* Occurred at 87.8% of M_{n_req}.
** Occurred at 98.3% of M_{n_req}.

The finite element results show that both the 2 ft pocket spacing and 4 ft pocket spacing performs well for hooked reinforcing bars as shear connectors. The results for the hooked reinforcing bars as shear connectors indicate that the number of connectors required per pocket may be decreased. Two ft pocket spacing performs the best when shear studs are used as the shear connectors. However, when the minimum number of shear studs is provided from the AASHTO LRFD shear friction equation, 4 ft pocket spacing is an acceptable alternative to 2 ft pocket spacing.

CONCLUSIONS

Both types of shear connectors worked well based on the live load test results. The hooked reinforcing bars and the new shear stud detail had axial strains less than 50% of the nominal yield strain. The results show the Von Mises stresses were approximately equal to the yield stress of the shear connectors even when the number of shear connectors was reduced by 25%.

Equation 2 should be used to select the number of shear connectors instead of Equation 3. This was the case not only for hooked reinforcing bars used as shear connectors, but also for shear studs used as shear connectors. For the new shear stud detail, the spacing of the shear connectors embedded in the top flange of the girder can be determined from the equation

$$s = \frac{\mu(2A_{stud} f_y + P_c)}{\dfrac{V_u}{d_e \phi} - cb_v} \qquad \text{Eqn. 4}$$

where, A_{stud} = cross sectional area of one shear connector. The remaining terms for Equation 4 are defined with Equation 2. The spacing of the shear studs is determined assuming 2 shear studs will be placed in each row.

From this research program, 2 ft pocket spacing and 4 ft pocket spacing performed well. The results were based on a girder depth of 36 in. In general, the pocket spacing should be determined using the following equation:

$$s_{pocket} \le d_v \cot(\theta) \qquad \text{Eqn. 5}$$

where,

θ = angle at which the shear cracks form at (according to modified compression field theory).

d_v = distance between the centroid of the steel in the tension side of the girder to the resultant center of the compressive force in the deck.

The calculation for obtaining θ is outlined in AASHTO LRFD[1,2]. The angle, θ, can be conservatively taken as 45°. The pocket spacing determined from Equation 5 should not be greater than 4 ft.

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