DIAPHRAGMS IN THE DECKED BULB-TEE GIRDER BRIDGES

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ABSTRACT

This paper presents the results of a study to determine the effect of intermediate diaphragms on the behavior of decked bulb-tee bridges. The research studied five bridge models with different types of intermediate diaphragms (steel and concrete), as well as differing number of the diaphragms. Intermediate diaphragms affect the behavior of DPPCG bridges. Bridges with diaphragms tended to reduce maximum forces at the joint, deflection and strain at mid-span when compared with bridges without any intermediate diaphragms. However, the effect was dependent on the loading positions in the transverse direction. If the loading was located at the interior girder, the effect was significant. If the loading was at the exterior girder, the effect could be ignored. For the bridge with steel diaphragms, decreasing the number of or the stiffness of the steel diaphragms increased the load distribution factor. However the influence could be ignored as long as there was one diaphragm at the midspan of the bridge. A concrete diaphragm at the midspan of the bridge could reduce the load distribution factor compared with the steel diaphragms.

Keywords: Intermediate Diaphragm, Decked Bulb-Tee, Precast Prestressed Concrete Girder Bridges, Field Test, 3D Finite Elements.

INTRODUCTION

The speed of construction, especially for the case of bridge replacement and repair projects, has become an increasingly critical issue. A strong momentum exists for the spread of precast construction for bridges as well as for pushing the limits of long-span bridges. One of the promising systems for precast bridge construction involves using decked precast, prestressed concrete girders (DPPCG) for the bridge superstructure[1]. The DPPCG bridges do not need a cast-in-place deck, or any wearing course, and by having the deck integral with the girder (decked bulb tee), several benefits are obtained. These benefits include: an accelerated time frame for construction, high quality plant produced concrete members, and an entire prestressed cross section which enhances section properties and durability.

Despite several major benefits, the construction of this type of bridge has not shown the growth it deserves and has been mostly limited to the Pacific Northwest states of Alaska, Idaho, Oregon, and Washington. The reason is two-fold: the concerns and limitations in design and construction using DPPC girders, and the lack of understanding due to limited research in this area. These issues include live load distribution[2], connections between adjacent units as well as the use of intermediate diaphragms[3], and other factors such as deck replaceability. In 2004, NCHRP initiated project 12-69 titled *Design and Construction Guidelines for Long-Span Decked Precast, Prestressed Concrete Girder Bridges*. The objective of that research was to develop design and construction guidelines for long-span DPPCG bridges including deck replaceability features.

(a) Diaphragm with Steel

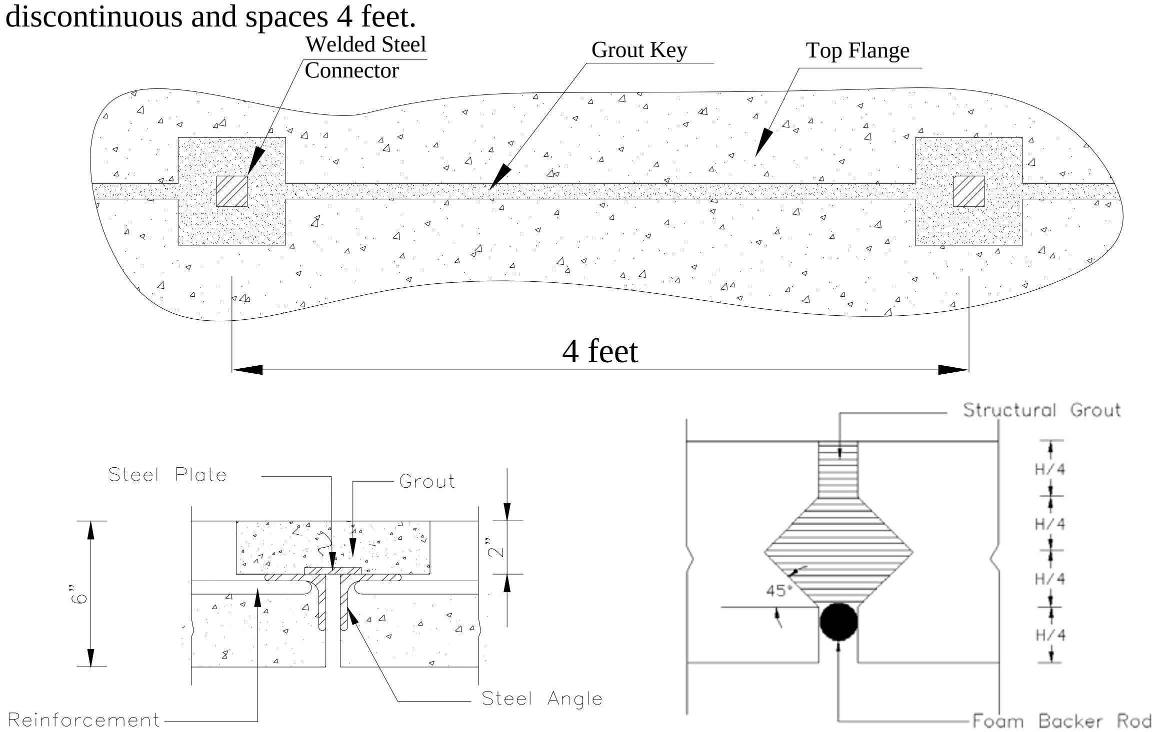
(b) Diaphragm with Concrete

Fig. 1 Diaphragms Used in Alaska DPPCG Bridges

The most predominant use of this type of system is by the Alaska Department of Transportation (AKDOT)[4]. In Alaska, they used to use intermediate steel diaphragms. When intermediate steel diaphragms were used in the past, they were spaced not to exceed 25 ft as was required for steel girder bridges in the AASHTO Standard Specifications for Highway Bridges of that time. Based on a field testing program of DPPCG bridges with steel diaphragms sponsored by AKDOT[4,3], it was found that it was not necessary to require a steel diaphragm every 15 or 20 ft. However, when comparing the moment distribution factors of bridges without any intermediate diaphragm and bridges with only one diaphragm at the midspan, the distribution factor can be reduced from 0.40 to 0.26. One intermediate diaphragm can also reduce the shear connector forces along the longitudinal joints in DPPCG bridges[3,4]. Steel diaphragms have resulted in construction problems in some of the DPPCG bridges in Alaska (Fig. 1a). For the last eight years or so, the AKDOT has provided only one intermediate cast-in-place concrete diaphragm at mid-span of DPPCG bridges (Fig. 1b). According to the research conducted by Zokaie et al[5], the benefits of diaphragms are controversial among different states and the policy in practice is not uniform because of the number, spacing and layout of diaphragms depending on bridge type. A survey of the information on the use of intermediate diaphragms prestressed concrete girder bridges was conducted in the 50 state DOTs [6]. A total of 95 percent of the responding agencies reported the use of intermediate diaphragms, of which about 10 percent of the agencies put diaphragms at quarter points along the span, 30 percent put diaphragms at the one-third points, and 50 percent put diaphragms at the middle span. The objective of this paper is to study the impact of reducing the number of intermediate steel diaphragms and/or replacing steel diaphragms with concrete diaphragms for DPPCG bridges.

CONNECTIONS IN DPPCG BRIDGES

Currently, the connection between top flanges of the DPPCG is provided by welded steel connectors and a grouted shear key, as shown in Fig. 2. The welded steel connector is discontinuous and spaces 4 feet.



Welded Steel Connector Grout Key

Fig. 2 Connections between DPPCG Flanges

Besides the connection between the top flanges by connectors and a shear key, the adjacent webs and bottom flanges of DPPCGs are connected by intermediate diaphragms, such as a "K" brace steel diaphragm shown in Fig. 3. The "K" brace steel diaphragm consisted of three L-shape steel members: two inclined and one horizontal.

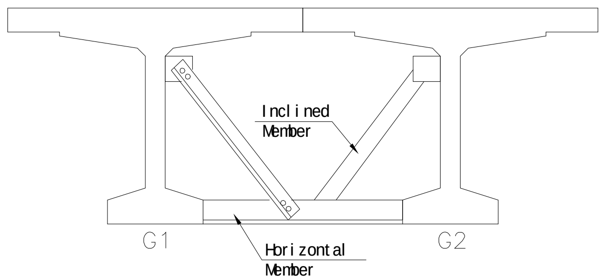


Fig. 3 A Steel Diaphragm between Two DPPCGs

A FIELD TESTED DPPCG BRIDGE WITH STEEL DIAPHRAGMS

A field testing program on DPPCG bridges with intermediate steel diaphragms was reported elsewhere[3]. In order to demonstrate the calibration of the 3D FE models presented below, one of the tested DPPCG bridges is summarized here. The bridge, W 100th bridge located in Anchorage, Alaska, was a simply supported DPPCG bridge with intermediate steel diaphragms and concrete end diaphragms at each end. Table 1 shows the geometry of the bridge.

Table 1 Field Tested Bridge

NAME	Bridge Geometry			Girder	
	Span(ft)	Width(ft)	Skew(^{o})	Spacing(in.)	Depth(in.)
W 100th Bridge	113.75	37	0	88.8	54

The bridge consisted of five DPPCGs. The top flanges of two adjacent girders were connected by a total of 28 welded steel connectors and a grout key along the length of the bridge. The spacing of the weld steel connectors was four feet. Intermediate steel diaphragms were used at 18 ft 10 in centers along the length of the bridge, for a total of 20 intermediate steel diaphragms. For the "K" brace steel diaphragm used in the bridge, the inclined member (IM) had a cross-sectional area of 3.27 in^2 while the horizontal member (HM) 4.4 in^2. A cross-section of the bridge can be seen in Fig. 4. Please note that the metal railing is not shown in the cross-section drawing.

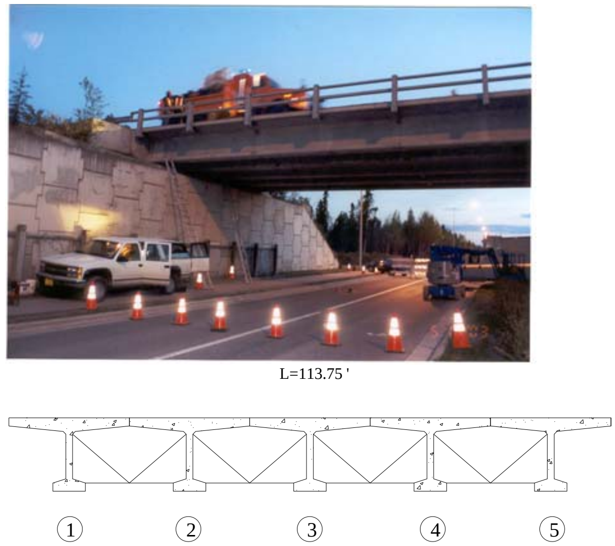


Fig. 4 W 100th Bridge

The bridge was field tested using a loaded AKDOT end dump truck with three axles. During the loading, the vehicle slowly traveled across the bridge in the longitudinal direction at various locations along the bridge cross-section. The vehicle was stopped in such a manner that the middle axle was located at the mid-span of the bridge, and a minimum of 30 seconds of data was recorded as the vehicle remained stationary. The vehicle stop position and load is shown in Fig. 5.

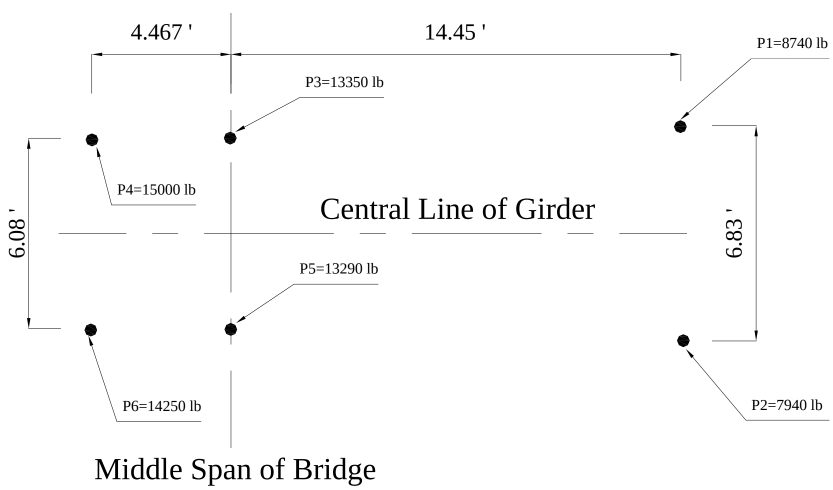


Fig. 5 Dimension and Load of Vehicle

In the transverse direction, there were five different loading positions: Loading G1 to Loading G5. For loading G2, G3 and G4 (interior girder 2, 3 and 4), the vehicle was positioned at the center of that girder; for loading G1 and G5 (exterior girder 1 and 5), the vehicle was positioned with its outside wheel line two feet from the edge of the bridge, as shown in Fig. 6.

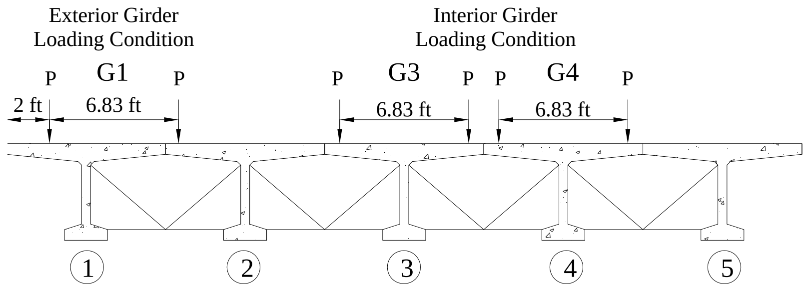


Fig. 6 Transverse Vehicle Positions

Strain transducers fabricated by Bridge Diagnostics Inc were attached to the center of the bottom flange of each girder at the middle span (Fig. 7). The strains of the bottom flanges in the longitudinal direction were measured using MEGADAC 5414 Series data acquisition system by OPTIM Electronics. This system was connected to a laptop utilizing TCS for Windows Version 3.4.

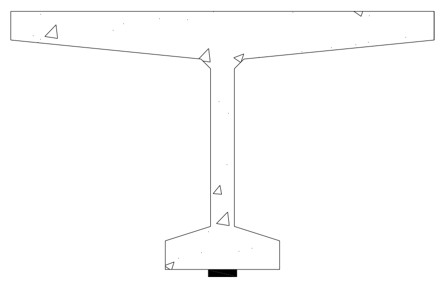
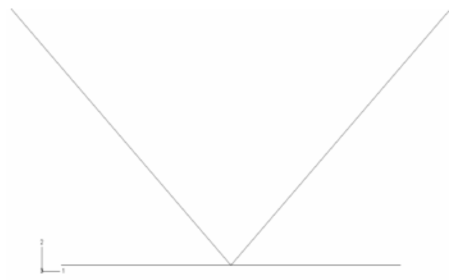


Fig. 7 Strain Gauge Position

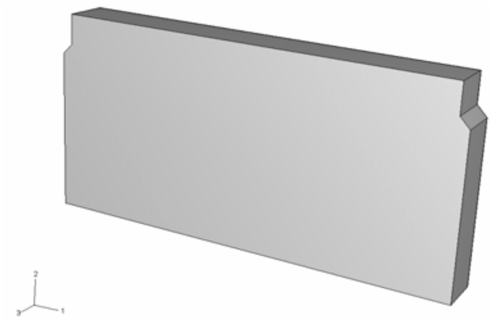
FINITE ELEMENT MODELING

The finite element method is an effective tool for predicting the behavior of structures such as bridges and bridge components. Using the general purpose finite element program ABAQUS 6.4.1, a 3D FE model was calibrated using the field testing data from the W 100th bridge. In the calibration process, four types of elements were selected in the model: 3D solid elements, 3D shell elements, 3D truss elements and 3D beam elements. The material properties from the design drawings of the tested bridge were used in the model. The Young's modulus for the concrete and steel were 4645 ksi and 29000 ksi respectively, and the Poisson's ratios were 0.18 and 0.3 respectively.

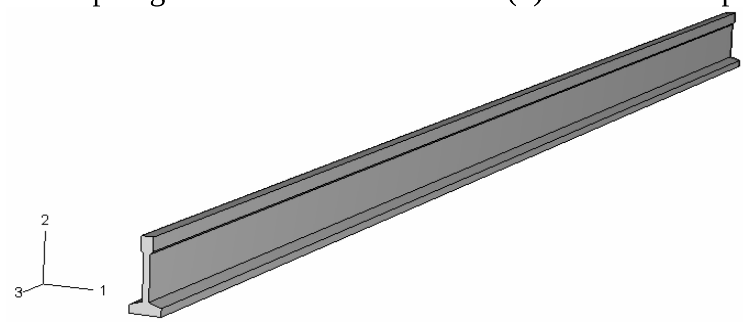
The finite element model for the W100th Bridge consisted of three main components: intermediate diaphragm, concrete beam and top flange (Fig. 8). Two different kinds of intermediate diaphragms were developed: steel and concrete. The inclined member of steel diaphragm was modeled using three-dimensional, two-node truss elements (T3D2); the horizontal member of steel diaphragm was modeled using three-dimensional, two-node beam elements (B31). The concrete diaphragm was modeled using three-dimensional, twenty-node solid elements (C3D20). The concrete beam was modeled using three-dimensional, twenty-node solid elements (C3D20). The maximum aspect ratio for the 20-node elements was about 3. The concrete top flange was modeled using three-dimensional, eight-node shell elements (S8R).



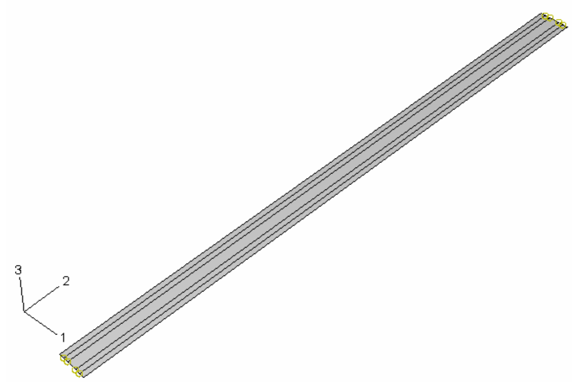
(a) Steel Diaphragm



(b) Concrete Diaphragm



(c) Concrete Beam



(d) Concrete Top Flange

Fig. 8 Bridge Components by 3D FE Models

Since the thickness of shell elements is constant and the actual top flange was tapered from 10.0 in. to 6.0 in., a stepped-top flange was decided based on a parametric study (Fig. 9).

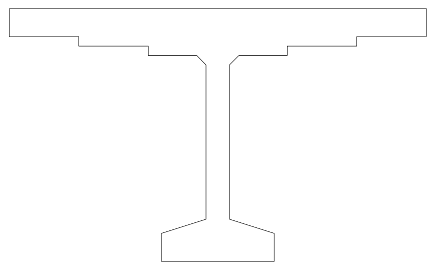


Fig. 9 Stepped Top Flange Used in the 3D FE Model

The pinned-roller boundary condition was applied in the longitudinal and vertical directions of the bridge while the movement in transverse direction of the bridge was fixed at both ends to consider the effect of the concrete diaphragm. A sufficiently refined mesh was used to ensure the resulting accuracy. Fig. 10 shows an example of the refined mesh.

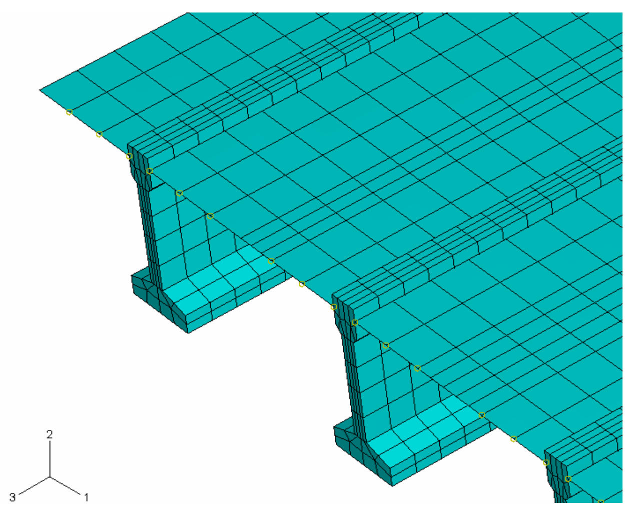


Fig. 10 Refined Mesh

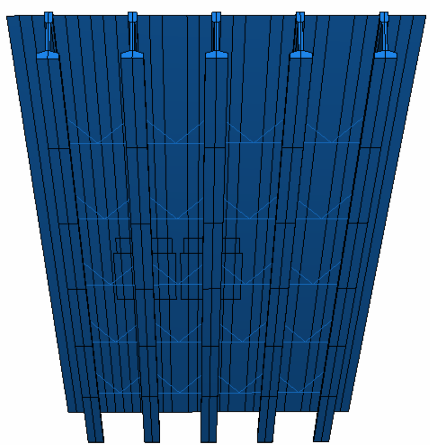
IMPACT OF INTERMEDIATE DIAPHRAGMS

Five different bridge models were developed as summarized in Table 2.

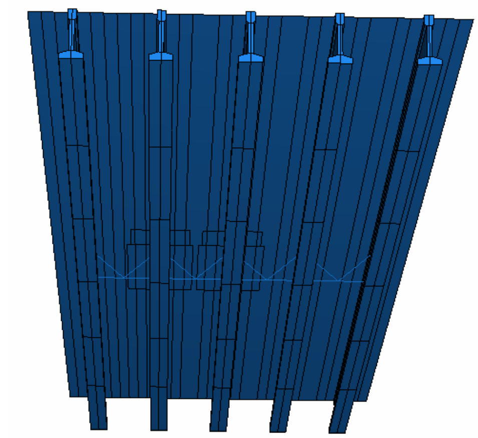
Table 2 Bridge Models

Bridge Model	Intermediate Diaphragms		
	Type	Number	Area of Steel (As) or Compressive Strength of Concrete (f'_c)
1	Steel	20	As (IM)=3.27 in^2 As (HM)=4.40 in^2
2	Steel	4	
3	Steel	4	As (IM and HM)=2.11 in^2
4	Concrete	4	f'_c=4 ksi
5	None Diaphragm		

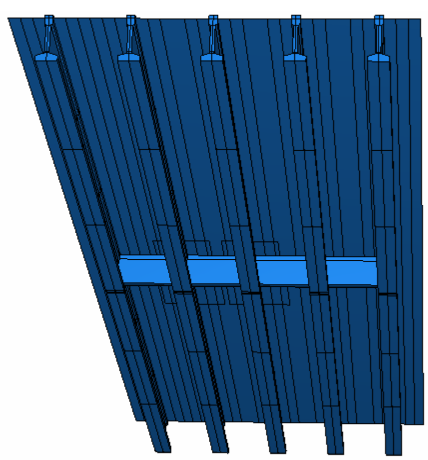
Bridge models 1, 2, 4 and 5 are shown in Fig. 11.



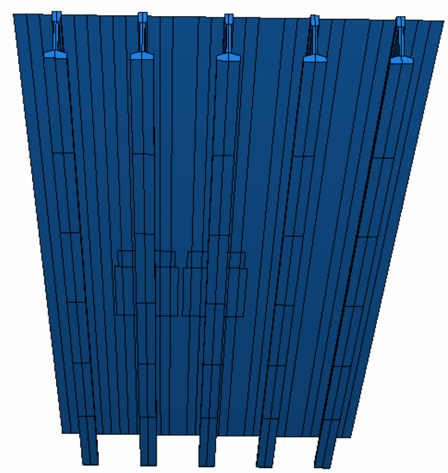
(a) Model 1



(b) Model 2



(c) Model 4



(d) Model 5

Fig. 11 Bridge Models

The following labeling system was used to define the model loading positions: Model Number-Girder Number. For example, “Model 1-G1” represents the loading was over the exterior girder (G1 in Fig. 6) in the bridge Model 1. For the field tested bridge, the labeling “SB or NB-Girder Number” was used. For example, “SB-G1” means that the loading truck was driven over the exterior girder (G1) from the southbound (SB) direction.

The results of the strain at the position shown in Fig. 7 when the truck was loaded over the mid girder of the tested bridge are presented in Fig. 12. It shows the good correlation between the developed 3D FE model and field testing data.

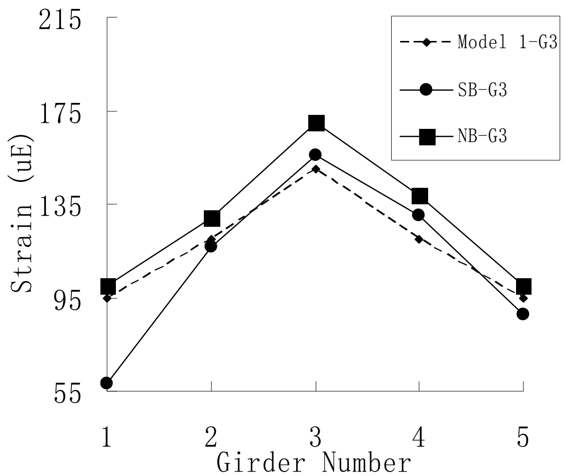
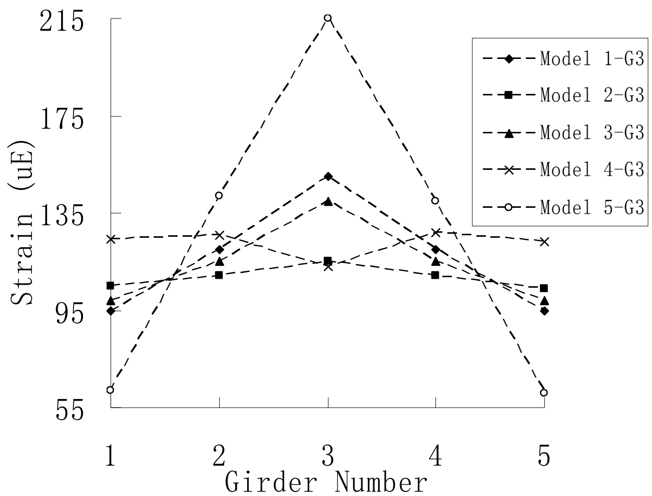
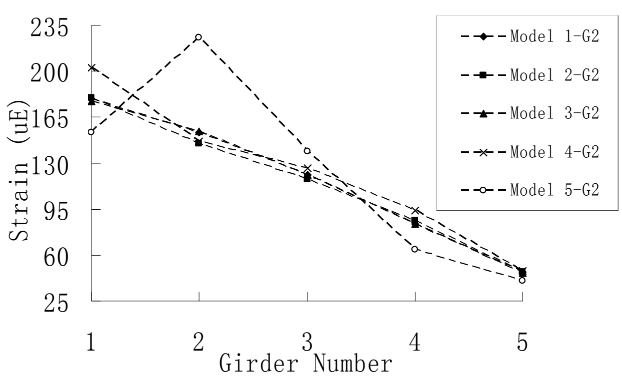


Fig. 12 Calibration of the FE Model

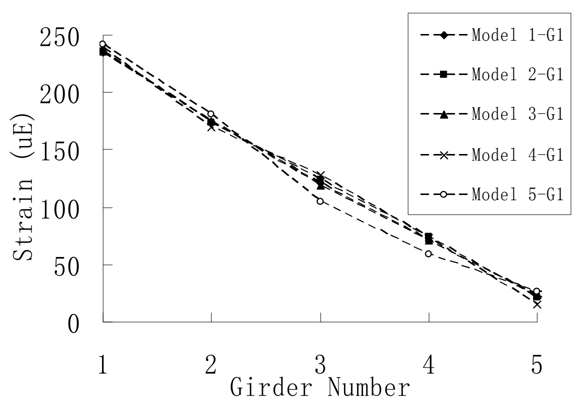
The influence of intermediate diaphragms on the bridge behavior was studied and the results of the strain at mid-span of each girder under different loadings are presented in Fig. 13. From the results of mid-span strains, it can be seen that the intermediate diaphragms affect the bridge behavior. The maximum strains in bridge Model five (without intermediate diaphragm) were the largest among those in bridge models with diaphragms (Model 1 to Model 4). This effect varied with the loading positions in transverse direction. When the vehicle was located on the girder 3, the maximum strain in Model 5 was much larger than the corresponding strains in Models 1 through 4. When the concrete diaphragm was used in Model 4, however, the maximum strain occurred in adjacent girders instead of the loaded girder. When the vehicle was located on the top of girder 1, the maximum strain in Model 5 was almost the same as the corresponding strains in Models 1 through 4. The effect decreased with the loading moving from the middle to the edge in the transverse direction.



(a) Loading G3



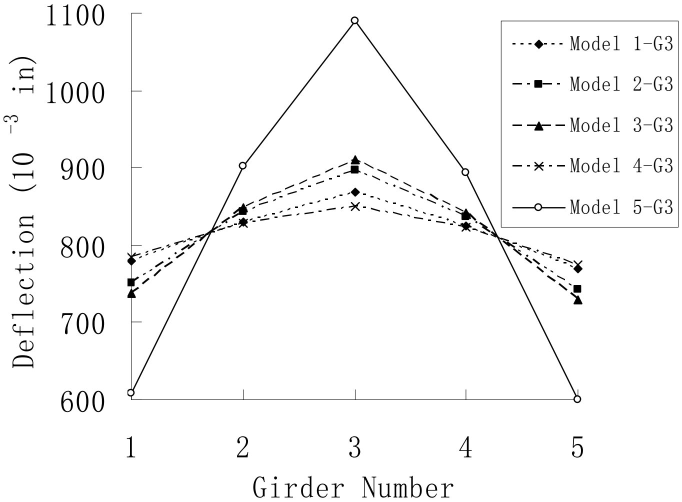
(b) Loading G2



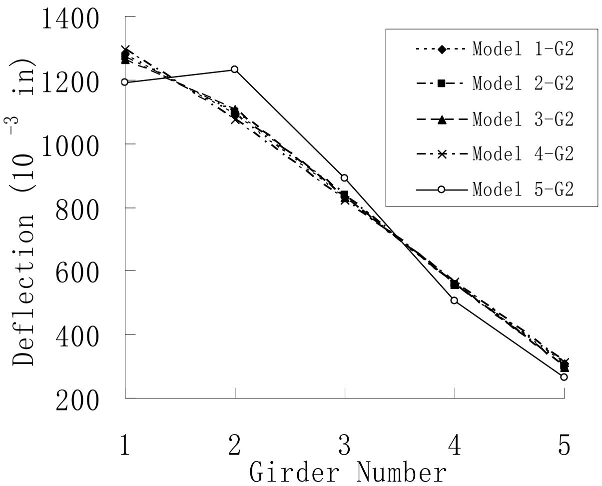
(c) Loading G1

Fig. 13 Strains under Different Loadings

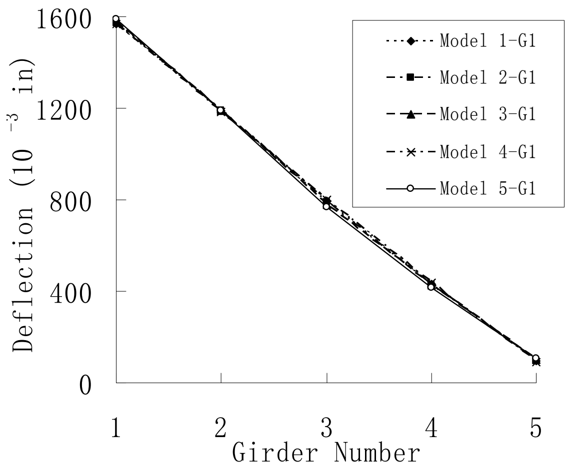
The deflections at the middle span of each girder under different loadings are compared in Fig. 14.



(a) Loading G3



(b) Loading G2



(c) Loading G1

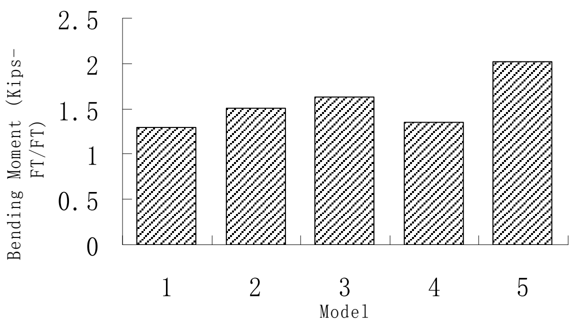
Fig. 14 Deflections under Different Loadings

From the mid-span deflection results, it can be seen that the maximum deflection in Model 5 (without diaphragm) was the largest among those in Models 1 through 4 (with diaphragms). The difference decreased as the loading position moved from girder 3 to girder 1. This confirms the assumption that intermediate have more influence on bridge behavior when the load is at the middle of the bridge in the transverse direction (on girder 3) than if the load was located at the edge (on girder 1).

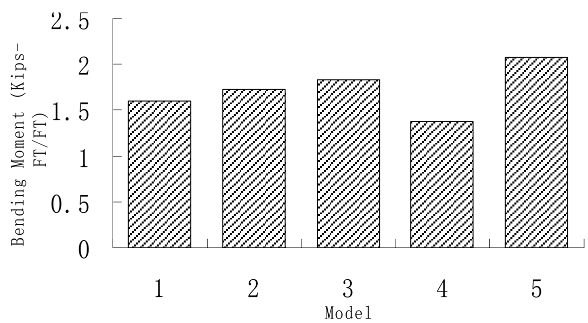
The variation of strains and deflections in Model 4 (concrete diaphragm) were smaller than that in Models 1 through 3 (steel diaphragm). It can be seen that the load distribution factor could be reduced with the concrete diaphragms compared with the steel diaphragms. Steel diaphragms were used in Models 1 through 3. From Fig. 13(a), Model 2 (4 steel diaphragms) had the minimum strain compared with Model 1 and 3. However, from Fig. 14(a), Model 1

(20 steel diaphragms) had the minimum deflection compared with Model 2 and 3. Overall, the effect of reducing the number of or the stiffness of steel diaphragms on the behavior of bridge is not significant as long as there is one at the mid-span.

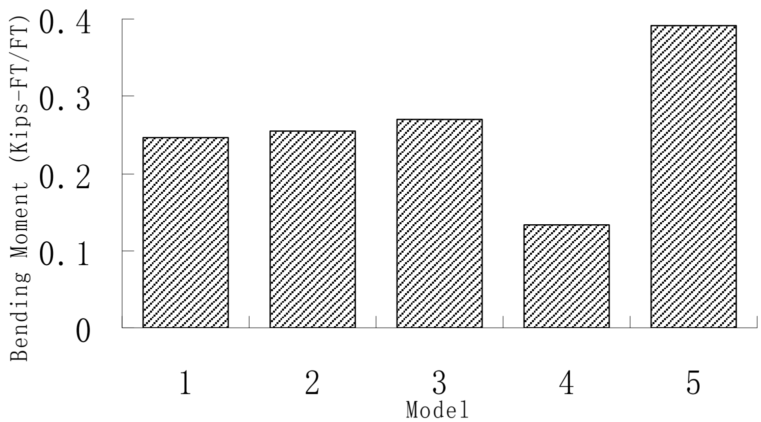
At the longitudinal joint between two adjacent DPPCGs, the maximum bending moment and the maximum vertical shear force under loadings G3, G2 and G1 were shown in Figs. 15 and 16.



(a) Loading G3

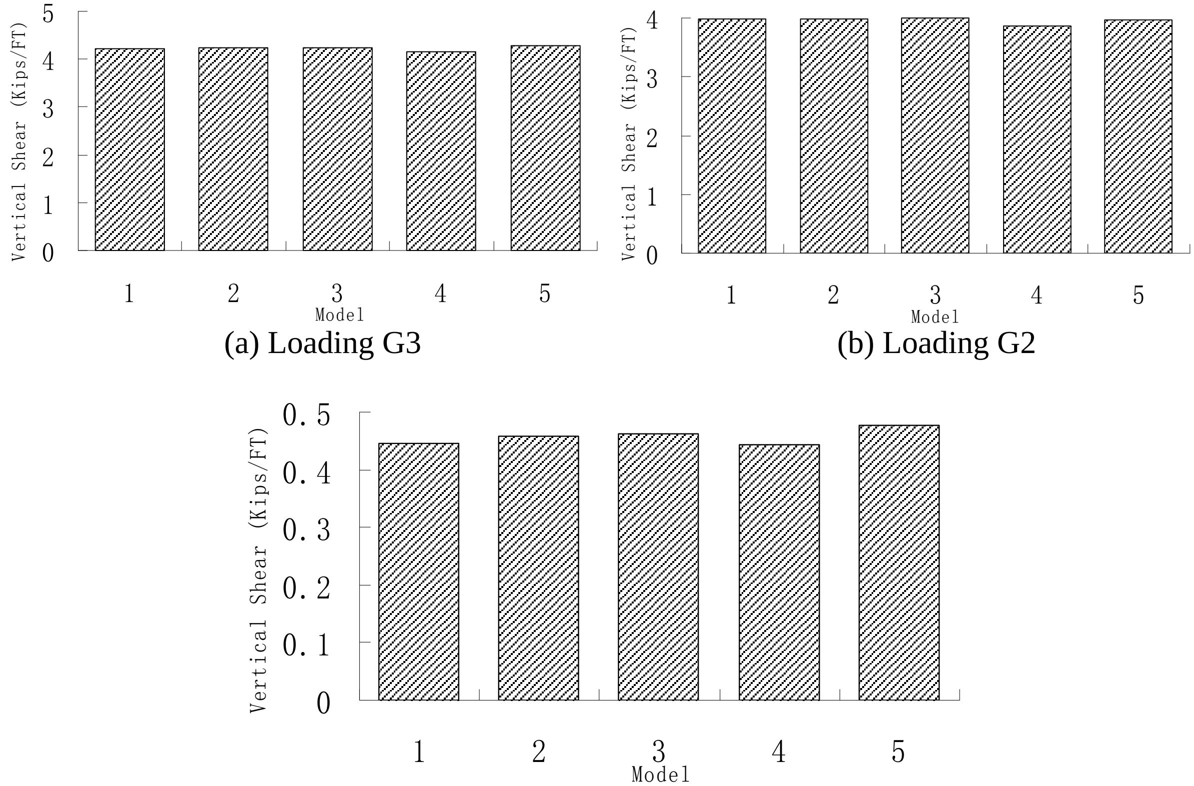


(b) Loading G2



(c) Loading G1

Fig. 15 Maximum Bending Moments at Joint



(a) Loading G3 (b) Loading G2

(c) Loading G1

Fig. 16 Maximum Vertical Shear Force at Joint

It can be seen that the maximum bending moment and vertical shear in Model 5 (without diaphragm) were larger than those in the models with diaphragms. This was especially true for the maximum moment, which was consistent with the results of strains and deflections discussed earlier, although the maximum shear force was about the same in all cases. Model 4 (concrete diaphragm) had the smallest bending moment and vertical shear in the joint. For the Models 1 to 3 with steel diaphragms, the maximum bending moment and vertical shear increased with decreasing the number of or the stiffness of the steel diaphragms.

CONCLUSION

Based on the finite element analysis and the field testing of the W 100th bridge, the following conclusions can be drawn:

1. Intermediate diaphragms affect the behavior of DPPCG bridges. Bridges with diaphragms tended to reduce maximum forces at the joint, deflection and strain at mid-span when compared with bridges without any intermediate diaphragms. However, the effect was dependent on the loading positions in the transverse direction. If the loading was located at

the interior girder, the effect was significant. If the loading was at the exterior girder, the effect could be ignored.

2. For the bridge with steel diaphragms, decreasing the number of or the stiffness of the steel diaphragms increased the load distribution factor. However the influence was very small as long as there was one diaphragm at the midspan of the bridge.

3. A concrete diaphragm at the midspan of the bridge could reduce the load distribution factor compared with the steel diaphragms.

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