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INVESTIGATION OF FATIGUE-INDUCED CRACK WITH K-TYPE BRACING SYSTEM

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ABSTRACT

Fatigue-induced cracking is a common failure mode in many steel bridges reaching their original design life. These aging bridge structures have experienced increasing traffic volume and weight, deteriorating components, as well as a large number of stress cycles. This paper presents a case study of fatigue assessment of an interstate highway steel bridge through real time monitoring under traffic. The bridge is a single-span, composite steel I-girder structure with K-type cross frame diaphragms. The study also included numerical analysis using 3D global finite element models. Based on the simulated traffic flow, statistical dynamic responses such as displacements and stress of bridge girders were studied for the cause of fatigue cracks that occurred in some cross frame connections. Meanwhile, long-term field monitoring has also been conducted. Furthermore, the influence of connection plate configuration and bracing system configuration was discussed using a series of controlled finite element tests. Based on the information from field tests, simulated numerical analytical results were verified. Thus, the performance of highway bridges under truck load can be predicted in a more realistic way to estimate the fatigue performance of highway bridges.

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ABSTRACT

Fatigue-induced cracking is a common failure mode in many steel bridges reaching their original design life. These aging bridge structures have experienced increasing traffic volume and weight, deteriorating components as well as a large number of stress cycles. This paper presents a case study of fatigue assessment of an interstate highway steel bridge through real time monitoring under traffic. The bridge is a single-span, composite steel I-girder structure with K-type cross frame diaphragms. The study also included numerical analysis using 3D global finite element models. Based on the simulated traffic flow, statistical dynamic responses such as displacements and stress-ranges of bridge girders were studied for the cause of fatigue cracks that occurred in some cross frame connections. Meanwhile, long-term field monitoring has also been conducted. Furthermore, the influence of connection plate configuration and bracing system configuration was discussed using a series of controlled finite element tests. Based on the information from field tests, simulated numerical analytical results were verified. Thus, the performance of highway bridges under truck load can be predicted in a more realistic way to estimate the fatigue performance of highway bridges.

Introduction

Intermediate cross-frame diaphragms of composite steel girder bridges can serve two distinct functions. They can (1) brace the girders' compression flanges, and (2) distribute loads among the girders. The truly essential function of traditional cross-frame diaphragms is to stabilize the girders' compression flanges, especially during construction. Many different configurations of cross-frame diaphragms have been employed in the construction of steel plate-girder bridges [1]. Generally speaking, there are four (4) types of cross-frames used in steel girder bridges:

1. X-frame without top chord (X-frame w/o top)
2. X-frame with top and bottom chords (X-frame w/top)
3. K-frame without top chord (K-frame w/o top)
4. K-frame with top chord (K-frame w/top)

The aspect ratio, i.e., girder spacing / girder depth, is the key factor in choosing economical cross frame configuration. In general, the following rules-of-thumb are available:

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- X-frames good for aspect ratios < 1
- K-frames good for aspect ratios > 1.5
- 1 < Aspect ratio < 1.5 - more subjective - owners' standard details or preferences may control selection of frame type

Transverse connection plates for cross-frames, or transverse stiffeners used as connection plates, based on AASHTO LRFD Specification (2012) [2], must be welded or bolted to both the compression and tension flanges of the plate girders for distortion-induced fatigue-cracking considerations. Usually, the connection of the cross-frame members with the plate girder is considered a working point as a truss joint, with the lines of action of the cross-frame members coincident with the junction of the flange and the web.

Fatigue Cracks and Bridge Testing

The Middlebrook Bridge is a simple span structure consisting of 17 welded steel plate girders and carries I-270 with three traffic lanes in the southbound roadway and five traffic lanes in the Northbound roadway (Figure 1 for photo views and Figure 2 for plan view of the bridge with the instrumentation plan). Four fatigue cracks were reported in the June 2011 Bridge Inspection Report, all in the connection weld between the lower end of the cross frame connection plate and the girder bottom flange. Figure 3 shows two of the four crack locations at G3B2D3 (Girder 3 Bay 2 Diaphragm 3) and G4B3D3 (Girder 4 Bay 3 Diaphragm 3), and their sensor placement locations.

A research project sponsored by the US Department of Transportation's Research and Innovative Technology Administration (RITA), under The Commercial Remote Sensing and Spatial Information (CRS&SI) Technologies Program required a pilot testing bridge to develop and field test a Wireless Integrated Structural Health Monitoring (ISHM) System and the Middlebrook Road Bridge with active fatigue cracks was selected. Complete pilot testing was performed by using acoustic emission (AE), accelerometer, deflection, and strain sensors for bridge information collection.

Figure 1. Elevation and close-up views of the Middlebrook Bridge.

Cracks always occur in the direction essentially perpendicular to the direction of principal tensile stress. In order to assess the driving force of the fatigue cracks in the connection welds, strain gages were placed vertically on the connection plate just beyond the tip of the existing crack. Strain gages were also placed longitudinally on the girder flanges to correlate with the occurrence of vehicular loads. Maximum top flange stress range measured right above the diaphragm G4B3D3 in the longitudinal direction was approximately 1.6 ksi due to regular traffic, which is low comparatively. Girder displacement and stress range records due to truck traffic were also part of field measurements in this study.

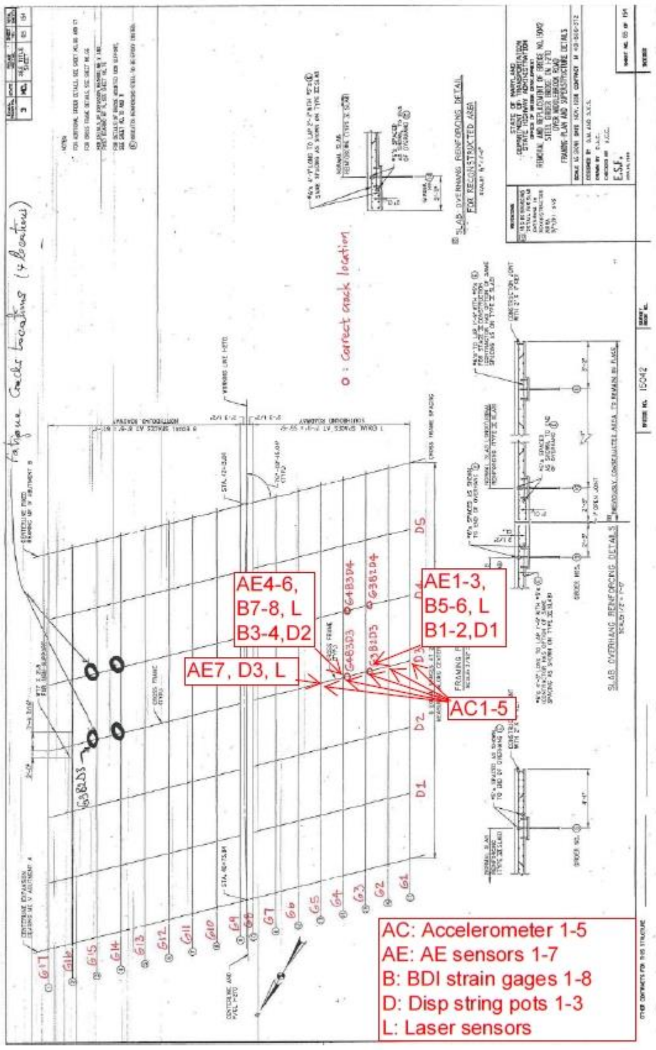

Figure 2.   Crack locations and sensor placement on the framing plan.

Figure 3.   Crack at G3B2D3 (Girder 3 Bay 2 Diaphragm 3, left) and G4B3D3 (Girder 4 Bay 3 Diaphragm 3, right) and sensor locations.

Field Test Results

In bridge fatigue evaluation, one key component is to accurately determine the live load-induced stress range. Compared with analytical methods, field test is the most accurate method since no assumptions need to be made for uncertainties in load distribution such as unintended composite action between structural components, contribution of nonstructural members, stiffness of various connections, and behavior of concrete deck in tension. The actual strain histories experienced by bridge components are directly measured by strain gages at the areas of concern. The effects of varying vehicle weights and their random combinations in multiple lanes are also reflected in the measured strains [3].

Bridge Deflection Monitoring

Both laser and ultrasonic distance sensors were used to measure the dynamic vertical deflections of the girder bottom flange. Only one laser sensor and one ultrasonic distance sensor were used each time. The data from laser sensor is shown in Figure 4.

Table 1. Maximum deflection measured by laser sensor.

Girder Number	**MaxD***(m)
3	0.0066
4	0.0069
5	0.0063

*MaxD= average(Displacement)-minimum(Displacement)

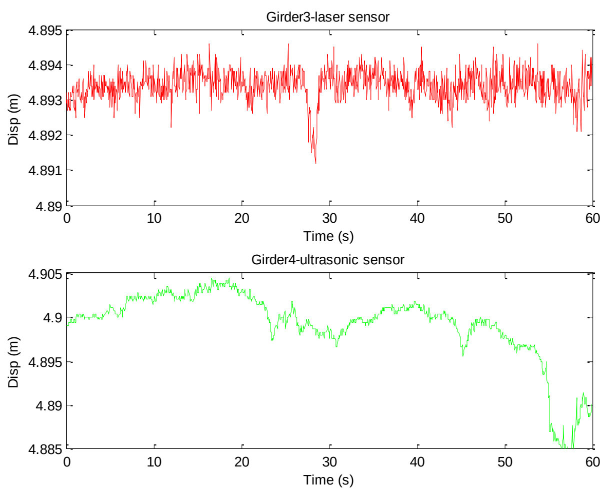

Figure 4. Bridge deflection data by laser sensor (upper) and ultrasonic sensor (lower). (The measured value is the distance between the sensor and girder bottom surface)

String Pots

String pots were placed on Girders 3 and 4, synchronized with strain and acoustic emission results (Figure 5). The maximum measurements within the testing period are 0.231” on Girder 3 and 0.205” on Girder 4, respectively, which are very close to the laser results, though laser was independently measured. (This short-term measurement is lower than previously measured up to 0.5" or 0.75".)

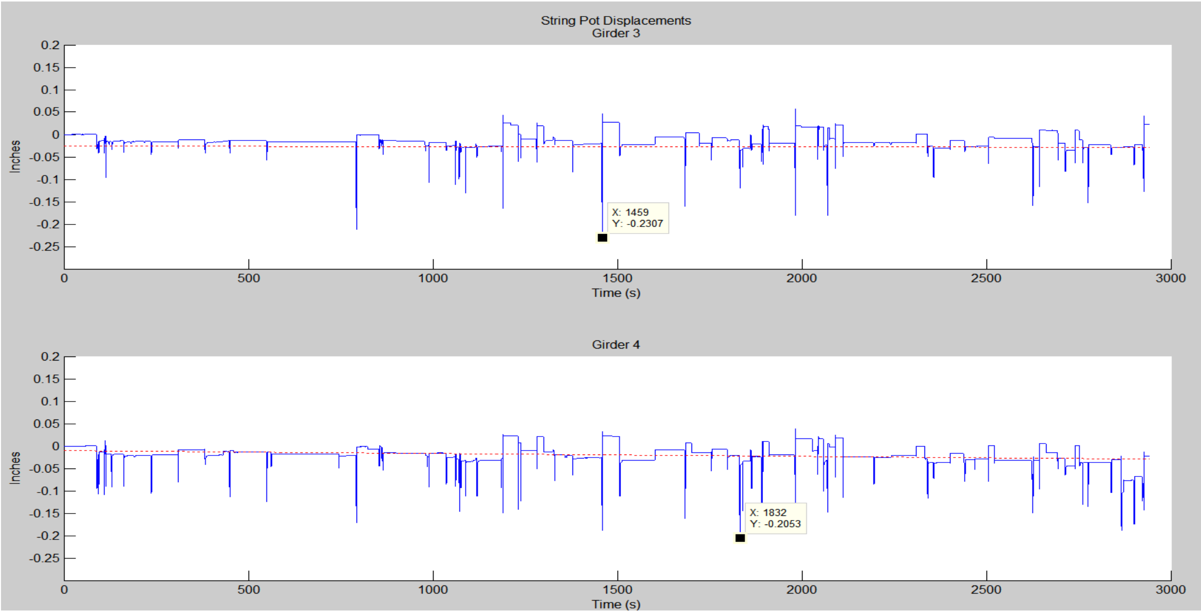

Figure 5. String pot deflection results on Girders 3 and 4.

Strain Transducers

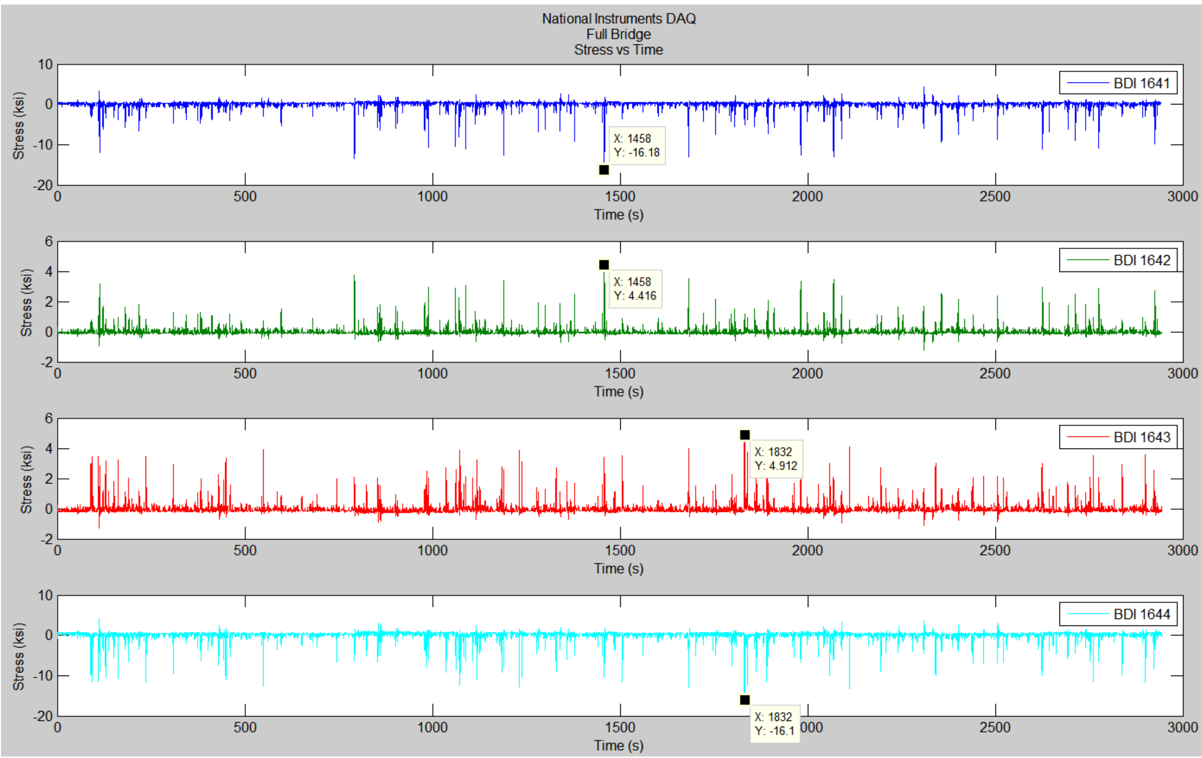

Figure 6. BDI strain transducer connection plate measurements. (Positive indicates compression; BDI1641 on G3 cracked side; BDI-1642 on G3 uncracked side, BDI-1643 on G4 cracked side and BDI 1644 G4 uncracked side)

BDI 1-4 strain transducers of approximately 4" gage length were placed in the vertical direction on the connection plate on both sides of girder web while BDI 5-8 were placed in the longitudinal direction on the top and bottom flanges of Girders 3 and 4. Figure 6 shows the recorded stresses in the connection plates. As for the connection plates, the maximum measured stresses are 16.18 ksi in tension for BDI 1641 on Girder 3 and 16.1 ksi in tension for BDI 1644 on Girder 4. In comparison, the maximum stress measured on the girder bottom flange is 1.604 ksi in tension for BDI 3215 on the bottom flange of Girder 3.

Finite Element Model (FEM) Simulation

Traffic Loading

The traffic data that was used to simulate traffic flow is the time varying vehicle count data from the Internet Traffic Monitoring System operated by Maryland Department of Transportation State Highway Administration [4]. The simulation procedure could be summarized in four steps. (1) Build the simulation network in TSIS 5.1 [5] around the MD Bridge No. 1504200 I-270 over Middlebrook Road based on the background map obtained from Google Map. (2) Use the time varying vehicle count data collected from nearby detectors as the input data for the simulation model. The truck count data is converted to truck percentage. (3) Install three loop detectors at the bridge in the created simulation network, one for each lane in order to record the speed, type and passage time of the detected vehicles. (4) Run the simulation. The passage time, speed and lane occurred of trucks could be recorded, just like virtual WIM data.

Finite Element Model

To investigate the fatigue performance of the bridge, a three-dimensional (3D) finite element model was developed by using the CSiBridge™ [6] for linear-elastic structural analyses. Boundary conditions represent actual characteristics of support and continuity. The model of the Southbound consists of eight I-girders as shown in Figure 7. The concrete deck, the eight I-girders, and connection plates which connecting diaphragms and girder webs were modeled by shell elements, while all the diaphragms were modeled by truss element. The translations of x-, y-, z-directions are fixed at the abutments. In order to locate the hot spot, a global model refined meshed around the hot spots was built for analysis.

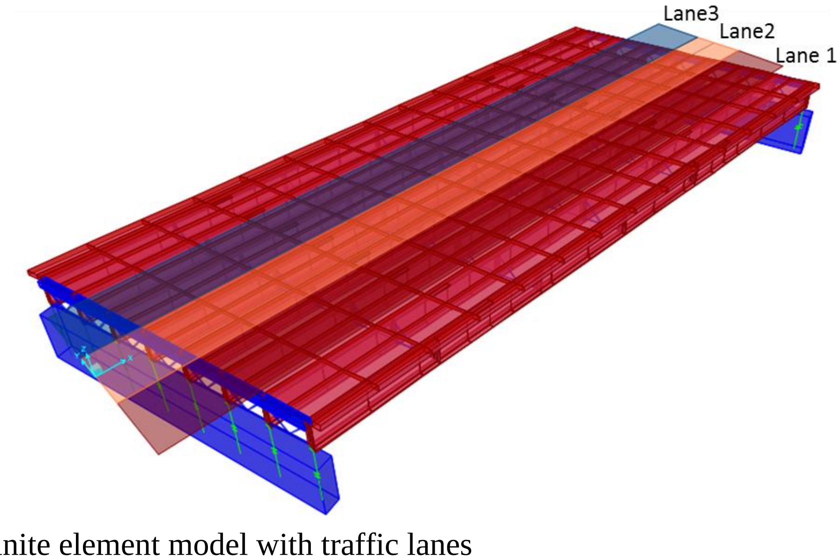

Figure 7. Finite element model with traffic lanes

Once the truck information is collected, it can be converted to truck loading and will be simulated to the bridge model by the CSiBridge™ program. As part of the results, Figure 8 shows the time history curves of vertical stress at the lower end of cross frame connection plate, located at Girder 3 Diaphragm 3. Shell element 252 is at the lower end of connection plate on the G3 cracked side, and shell element 250 is on G3 uncracked side. Both of them are on the same face. As an example of the graphic results, Figure 9 shows a zoom-in vertical stress contour of connection plates on Girder 4 Diaphragm 3 at T=283 second.

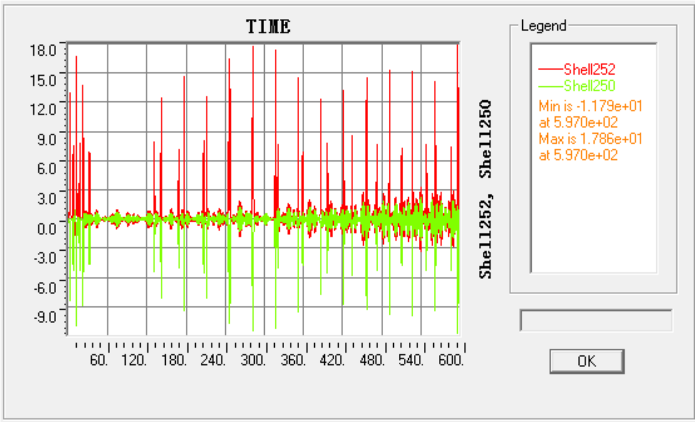

Figure 8. Vertical stress at lower end of connection plate (shell252-G3cracked side, shell250-G3uncracked side), unit-ksi.

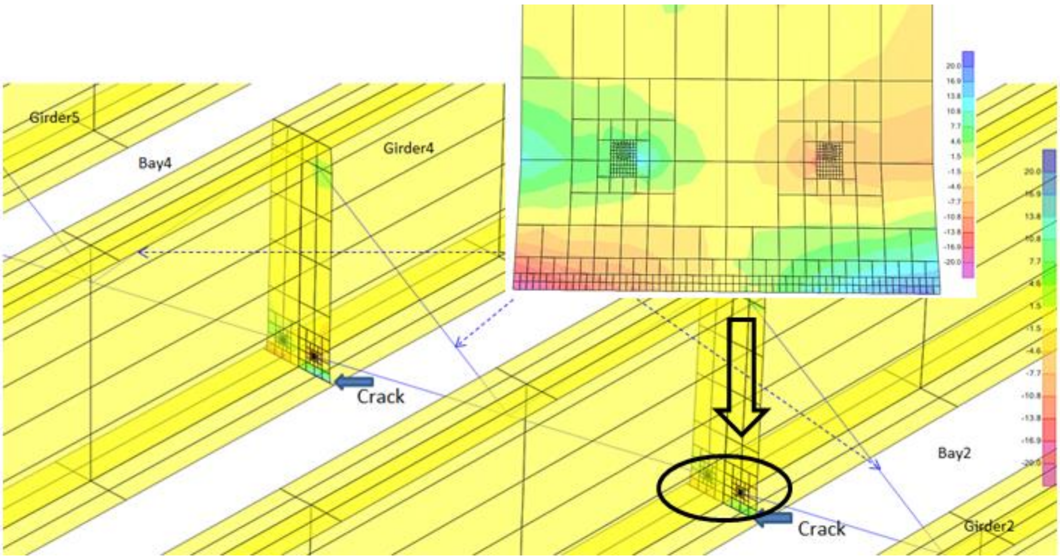

Figure 9. Zoom-in vertical stress contour of connection plate (Girder 3 Diaphragm 3) at T=597second

Connection Plates and Bracing System Configuration

The results of finite element analysis match with the filed test data; all the cracks located in western side of the connection plates. The vertical stress near the welded edges of connection plates follows the same pattern; the western sides of the connection plates are under tension, and the eastern sides of the connection plates are under compression. To further discuss the cause of this phenomenon, a series of controlled FEM tests were established for comparison study.

Connection Plates Configuration

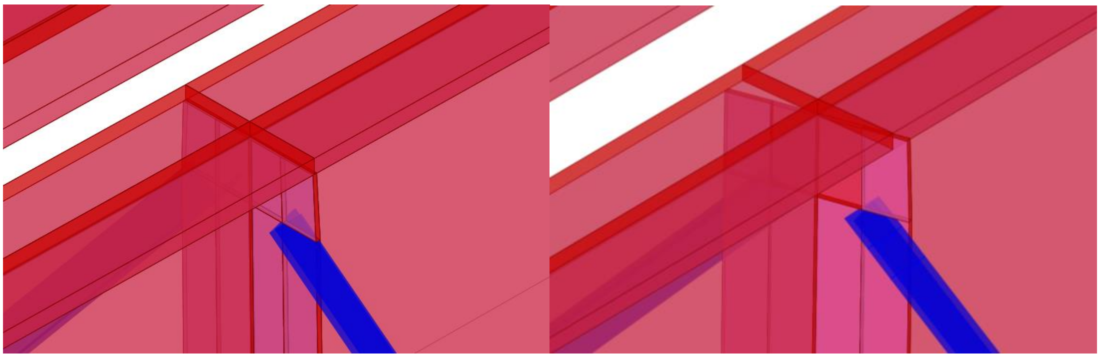

Figure 10. Skew (right) and non-skew (left) connection plates

According to the design drawing, cross frame connection plates and bearing stiffeners shall be normal to stringer and gusset plates shall be bent as required. Therefore, for the original FE model, all the connection plates were normal to the girders and the cross frames were parallel to two abutments with the angel of 76º. For the controlled model, all the connection plates were parallel to the cross frames with the same angel of 76º.

Bracing System Configuration

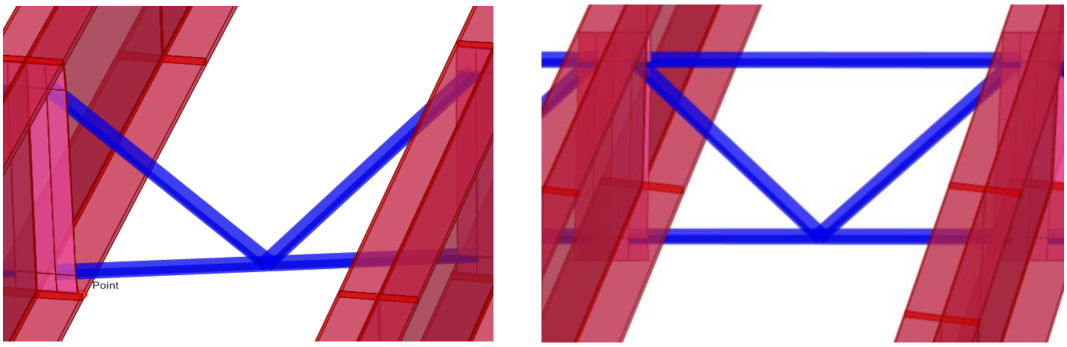

Figure 11. K-frame without top chord (left) and K-frame with top chord (right)

The K-type bracing system was modeled for studying the influence of bracing system configuration on the stress distribution of connection plates. The cross section of diagonal and bottom chords was employed for the added top chords.

All the models were subjected to the same live load case. The live load case was defined as HS20 on lane 1 passing through the bridge from north to south at the limited speed of 55 miles/hour. The vertical stress at the crack location (Girder 3 Diaphragm 3), and the axial forces of top chord located at Diaphragm 3 Bay2, directly connecting with the crack side were analyzed and shown in Table 2. The maximum vertical stresses of non-skew models are much higher than those of skew models. The maximum axial forces of the models during the time history analysis were quiet small; the values were only 3.47 kip and 1.12 kip. The values of maximum vertical stresses do not change much due to the added top chord. It is represented that the connection plates configuration has a significant influence on the stress distribution of the connection plates, while the top chord of K-type bracing plays an negligible role in this situation.

Table 2. Maximum vertical stress and axial force.

Connection plates configuration	Bracing system configuration	Max axial force (kip)	Max vertical stress of crack location (ksi)
Non-skew connection plats	K-frame without top chord	-	13.50
	K-frame with top chord	3.47	12.66
Skew connection plats	K-frame without top chord	-	0.33
	K-frame with top chord	1.12	0.30

Conclusions

Differential displacements between girders cause one diagonal in tension and one in compression. Since the working point of the diagonal is not at the junction of girder web and top flange plus no help from the top chord, one side of the connection plate will be under tension and one under compression under live load. Measured vertical tensile stresses up to 16.1 ksi in the connection plate explains why fatigue cracks have occurred at their connections to the girder bottom flange. Girders 3 and 4 are under the slow moving lane where most heavy trucks are using while Girders 1 and 2 support a shoulder and thus large differential deflections occurred between Girders 2 and 3 (with up to 0.5” to 0.75” vertical deflections due to live load observed). The connection plate configuration is a key factor for the stress distribution of the connection plates.

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