



# RESILIENT INFRASTRUCTURE

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## RE-TESTING OF A FIRE-DAMAGED BRIDGE

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### ABSTRACT

A proof load test was performed on a fire-damaged bridge in October 2008 and its load capacity was confirmed. The bridge was then re-opened for full traffic shortly after the load test. It was further repaired in 2009 which included concrete patching and carbon fibre reinforced polymer (CFRP) warping of 6 girders in the main span. The bridge is located at one of the busiest highways of the country with many heavy trucks passing daily. Also, reinforced concrete could deteriorate much faster than usual after the fire accident. In 2014, the Bridge Office was requested to re-test the bridge in order to re-confirm its performance. The re-testing of the bridge was carried out in November 2014, 6 years after the first load test. The objective of this test was to re-confirm the bridge performance. Therefore, strain gauges and linear voltage displacement transducers (LVDT) were installed on the bridge at the locations almost identical to that installed in 2008's load test. The applied loads were also similar to that from the original load test. This second load test was successfully completed and the load effects from both load tests were compared. It was observed that the bridge performances in 2008 and 2014 were similar. Therefore, the conclusions from the 2008 load test are still applicable.

Keywords: load test, fire-damaged bridge.

### 1. INTRODUCTION

Highway 401 westbound Bridge #7 over Highway 401 Ramp to Highway 404 northbound/Don Valley Parkway southbound (see Figure 1(a)) is a three-span (23.2m-29.5m-23.2m) semi-continuous slab-on-girder bridge (continuous for live load). Figure 1(b) shows a picture of the bridge. The bridge deck consists originally a 175 mm thick concrete deck slab, cast compositely on top of 9 rows of 1350 mm depth CPCI girders. A 40 mm thick concrete overlay was added to the deck in 1981, which has an exposed concrete riding surface. The superstructure is supported on abutments and two multi-column pier systems. The bridge, more than 60 degree skew, has a roadway width of 16.5 m, which carries three lanes of Highway 401 WB Collector traffic over the ramp from Hwy 401 Express to Highway 404 Northbound and Don Valley Parkway Southbound. The highway ramp below provides two lanes of traffic.



Figure 1(a): Bridge Site at Highway 401 and Highway 404



Figure 1(b): South Elevation of the Bridge



Figure 2: Girders with CFRP Wraps



Figure 3: Bridge Conditions

The bridge structure was damaged by a fire created by a truck accident in the early morning of September 4, 2008. The damage was severe and was confined to the south-west portion of the main span. Two repair works were applied to the bridge separately in 2008 and in 2009. The first repair work was an emergency deck soffit repair and massive transverse diaphragms were constructed between girders. After the first repair in September, a load test (Au, Lam and Tharmabala 2010) was carried out in October 2008. It was concluded that the bridge was able to carry the Canadian Highway Bridge Design Code (CHBDC) design load. The bridge was then re-opened for full traffic shortly after the load test. The first repair did not cover minor damages. The second repair in 2009 included concrete patching and CFRP wrapping of 6 girders in the main span. The effect of the CFRP on the bridge stiffness and its behaviour would be small, and it would be expected to become significant at higher loads approaching bridge's ultimate capacity (Cerullo et. al. 2013). Figure 2 shows the girders with the CFRP wrapping. Behaviour load test was performed on 23 November 2014 to re-assure the bridge's performance. The bridge was inspected before the load test (see Figure 3). It was found that the girders were in a reasonable good condition (see Figures 1 (b) to 3).

## 2. APPLIED LOADS

Two test trucks loaded incrementally with concrete blocks to a maximum total gross vehicle weight of 650 kN and 668 kN respectively were moved together across the structure in 15 pre-determined steps. Three load levels were considered (i.e. 24, 36 and 48 concrete blocks per test truck). Each concrete block is about 1 tonne. The test trucks were moved along load line 5 that maximized the load effects on girder #3 (from south). Figure 4 indicates the transverse locations of the test trucks and Figure 5 shows the 15 load steps. 9 out of 15 steps were identical to that applied in the 2008 load test. 6 more steps were considered in the second test in order to generate a broader range of behaviour. The maximum load will generate about 65% of the load effect caused by CHBDC (CSA 2006) design live load and 118% of the CHBDC (CSA 2006) SLS load. Because the bridge is carrying Highway 401 westbound collector traffic and there is a hospital nearby, it was decided not to close the bridge completely and to leave one lane opened during the load test. Although leaving one lane opened during bridge test is unusual, the following mitigations were used to minimize the influence of the data for girder 3: i) the test was performed in the early hours

on Sunday morning when the traffic volume was low; ii) only lane 3 was opened to keep live load traffic away from girder 3; and iii) test data were recorded when there was no other commercial truck on the bridge.

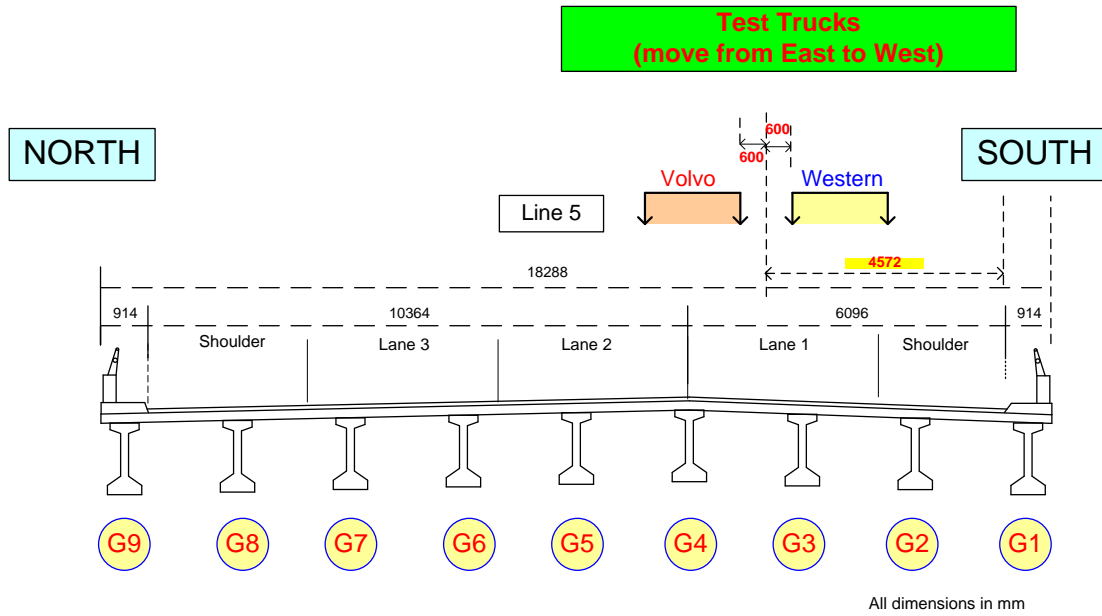


Figure 4: Load Line 5 on the Bridge

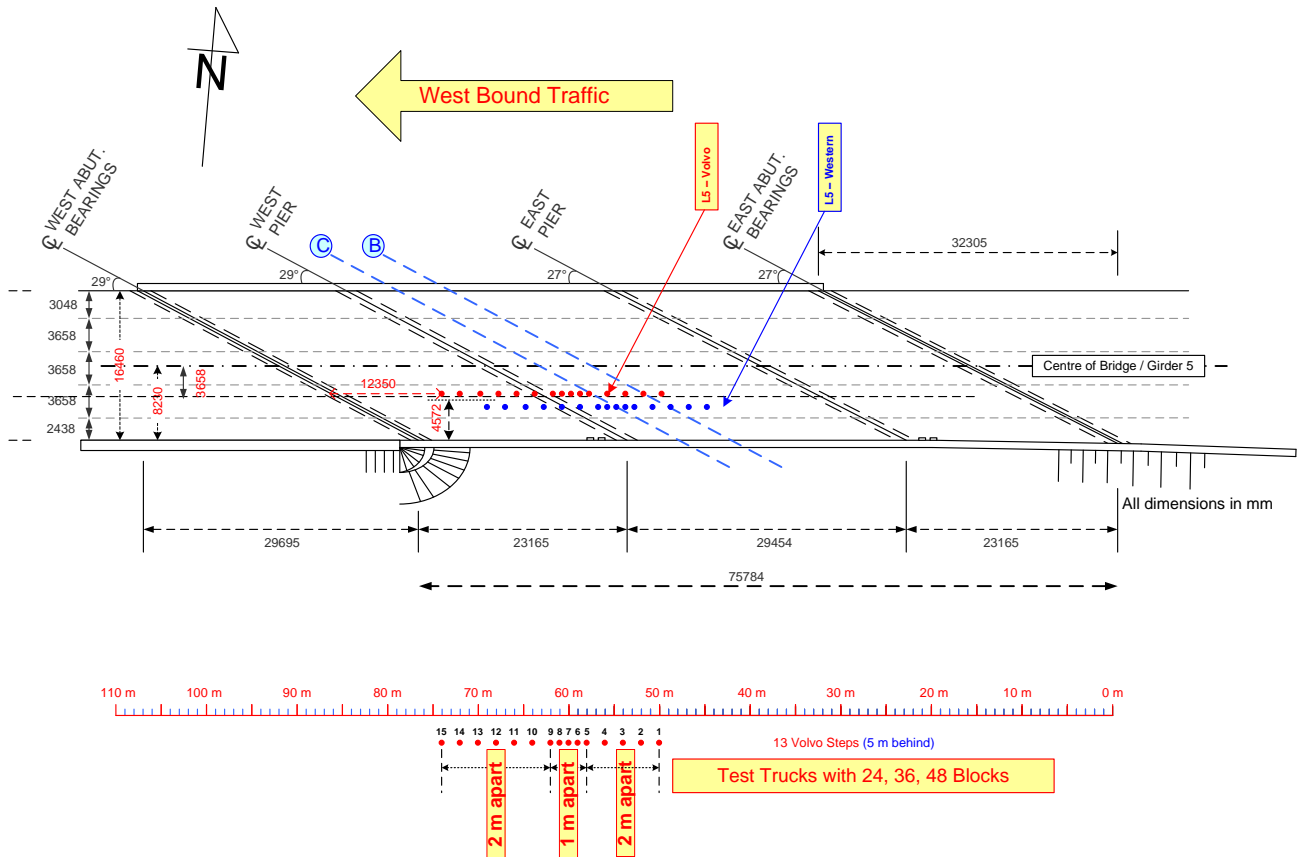


Figure 5: 15 Step Locations for the Test Trucks

### 3. INSTRUMENTATION

From the experience of the load test in 2008, 8 locations on two sections B and C from girders 2 to 5 were monitored. There were 3 strain gauges and 1 LVDT on each location. A total of 32 gauges were installed. Figures 6 and 7 indicate the instrumentation locations. Figure 7 shows the channel numbers identifying 8 LVDTs and 24 strain gauges. Figure 8 shows typical LVDT and strain gauges. LVDTs were installed directly on the concrete surface to avoid the CFRP sheet. All strain gauges were installed directly on the top of the CFRP sheet. It was assumed a perfect bonding between strain gauges and CFRP warps (see Cerullo et. al. 2013). The instrumentation was carried out in the early hours on two separated Sunday mornings (November 9 and 16, 2014). Due to the repair of the girders and the installation of CFRP wraps, it was difficult to install the gauges exactly at the same locations as in the load test of 2008. Therefore, the gauge locations in this test were installed as close as possible to that installed in the 2008 load test. The performance of the strain gauges was a concern because instrumentation was carried out in November under wet conditions and near zero temperature. On the test day, one of the LVDTs and some strain gauges were found to be malfunctioned. In order not to further delay the test, these gauges were not fixed and were ignored.

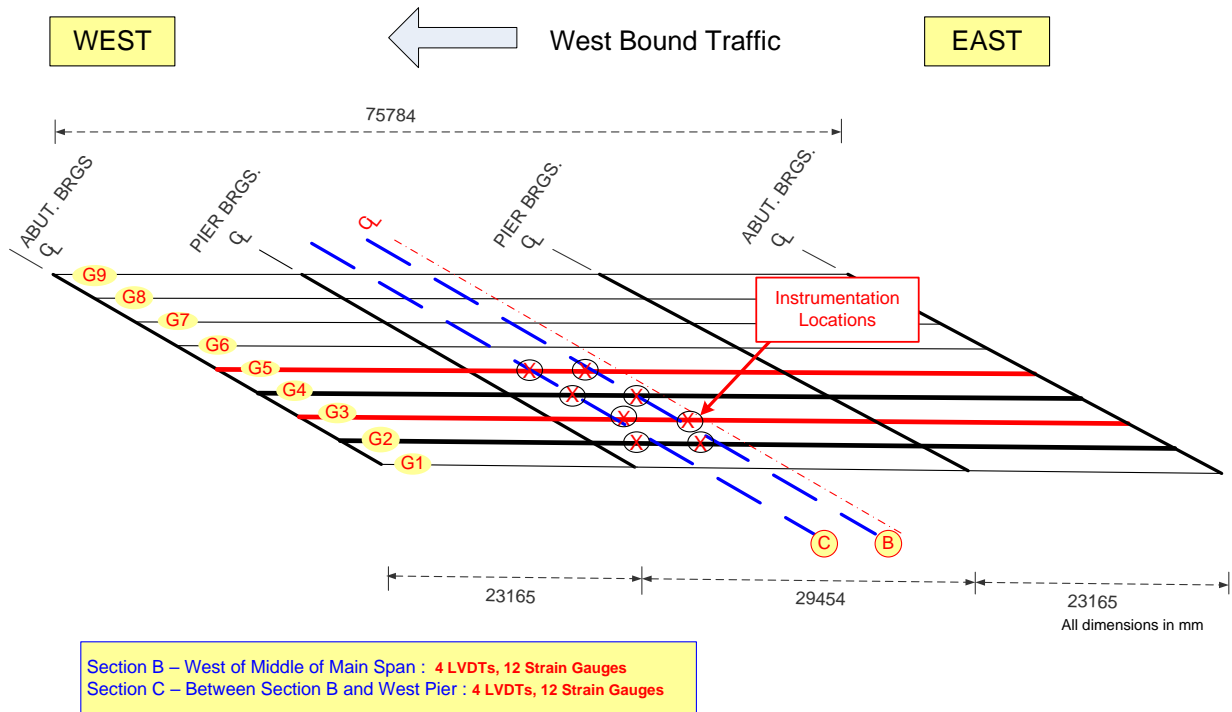


Figure 6: 8 Instrumentation Locations

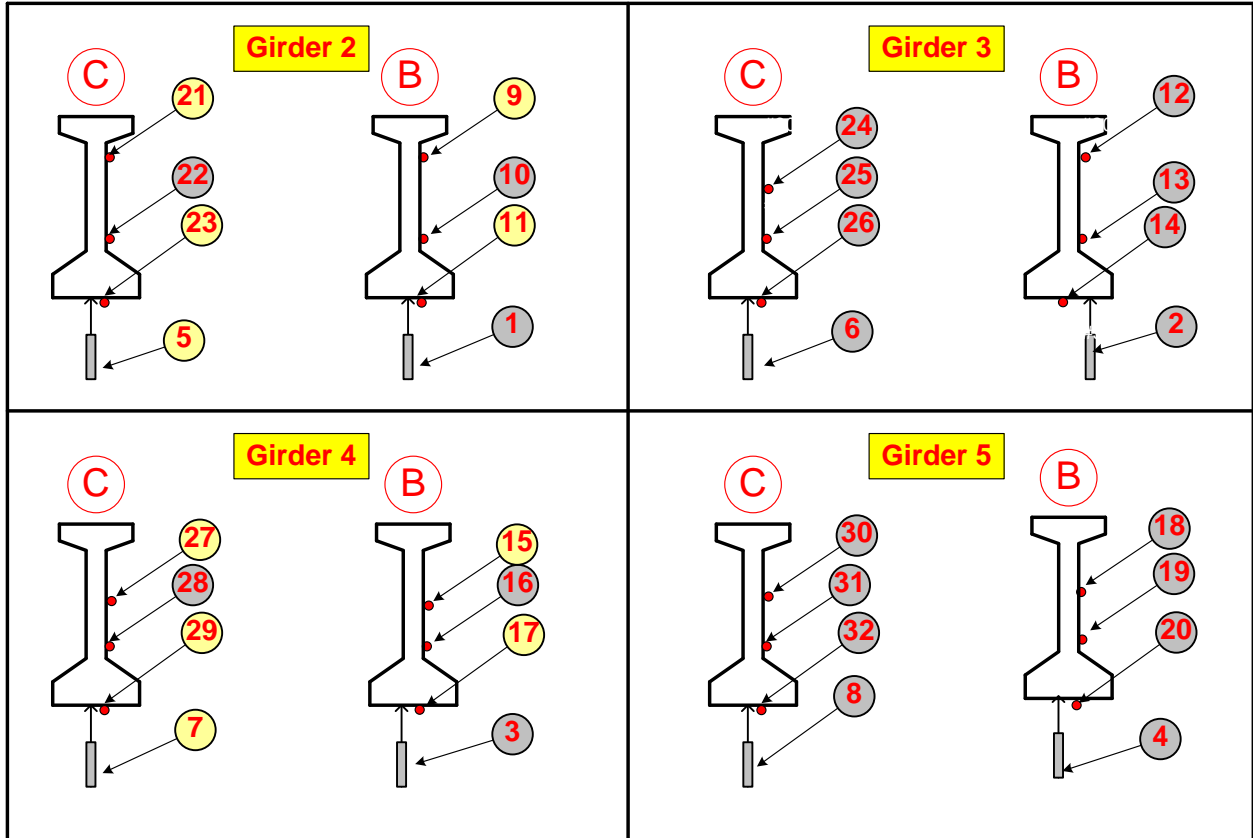


Figure 7: Strain Gauge and LVDT Locations at Sections B and C of Girders 2 to 5

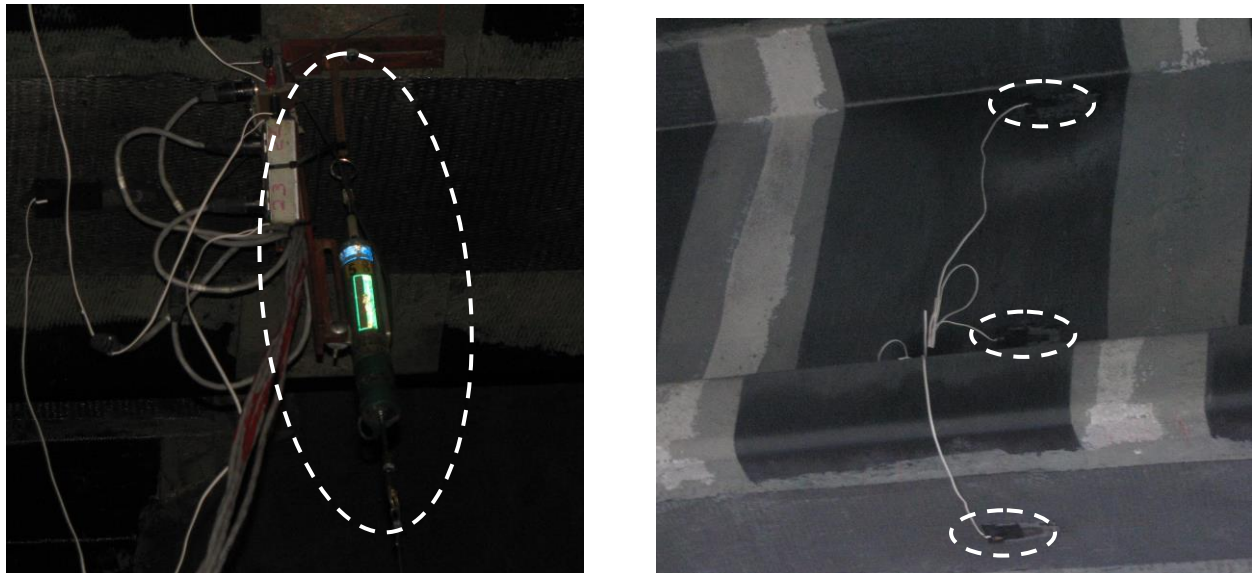


Figure 8: Typical LVDT on Concrete and Typical Strain Gauges on CFRP

#### 4. RESULTS

Based on the theoretical analysis and results from the load test in 2008, Girder 3 was known to experience the maximum response due to test trucks positioned transversely at load line 5 as shown in Figure 4. The second load test was carried out on November 23, 2014. The load test in 2008 was carried out in early October. The average temperatures at the site were different for both tests. However, the data obtained was processed after the test so that temperature effects were eliminated. Therefore, the resulting data were mainly due to live loads. For both tests, identical live loads were applied and so, the results from both tests were directly comparable.

In Figures 9 and 10, the deflections of Girder 3 measured at sections B and C were found to be similar to that recorded from 2008 test and also behaved linearly. Figure 11 shows the transverse distribution of the deflections at section B. It indicates that the deflection distribution compared very well with the result from the 2008 load test. Load line 5 produced the highest deflection on Girder 3. The transverse deflection distribution is a reflection of the transverse load distribution of the bridge. Therefore, the lateral load distribution of the bridge is similar to that from the 2008 load test.

The largest strain was observed at strain gauge 14 (Girder 3 and Section B). Figure 12 shows a comparison of the strain measured from 2008 and from this load test. The current measurement was smaller than the strain measured in 2008, and the distributions over different load steps were similar with respect to the test truck locations. However, the comparison of strain measurement was not as good as the deflection measurement. This discrepancy was mainly due to the slightly different locations of the strain gauges in both load tests. Such discrepancy could be reproduced from a theoretical analysis.

The linear relationship of the strain with respect to the applied load was confirmed in Figures 13 (a) and (b) for Sections B and C respectively. Both figures show a lower rate of increase in strain with loads and the strain was linear up to 48 concrete blocks per test truck.

Figure 14 shows the vertical strain distributions at Section C of Girder 3. It was based on the strain gauges located at mid-web, lower web and girder soffit, under three different load levels at 24, 36 and 48 blocks per truck. The location of the neutral axis matches the one obtained from the theoretical analysis. It means that the bridge composite action remains unaffected.

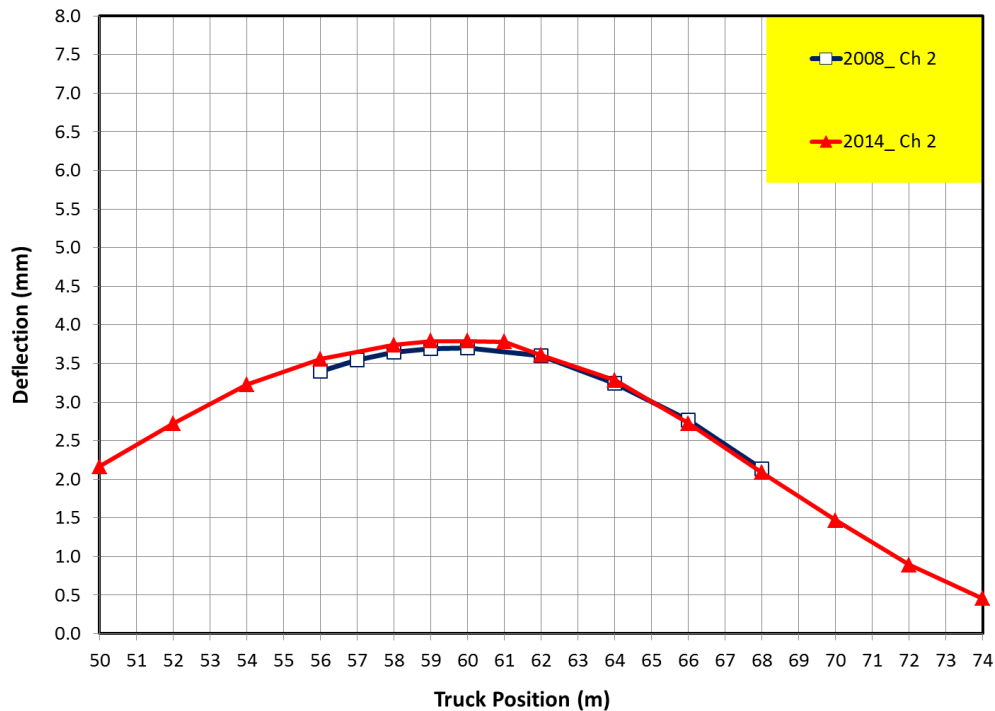
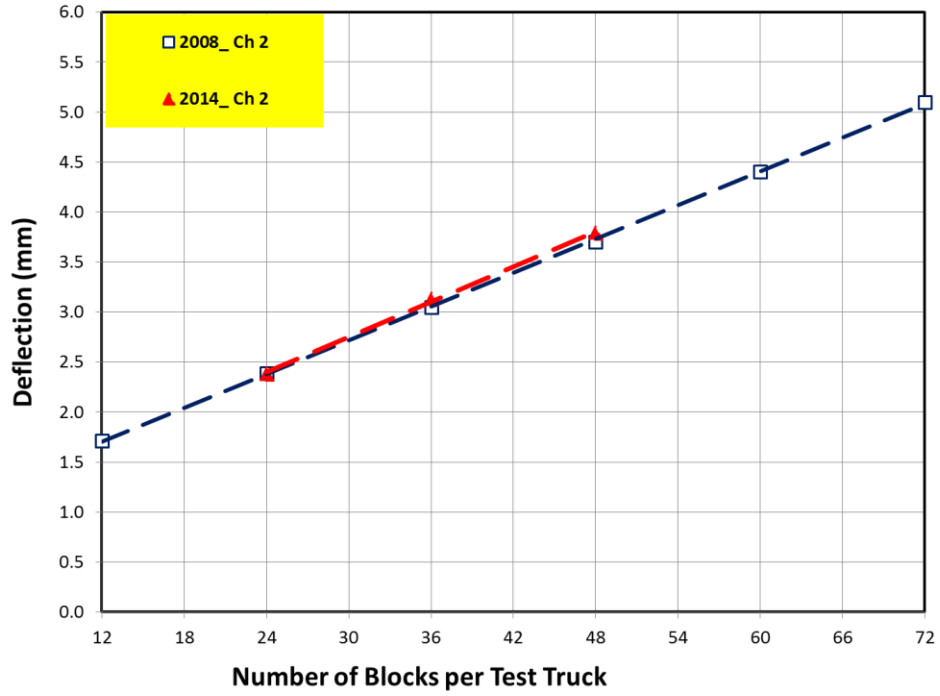
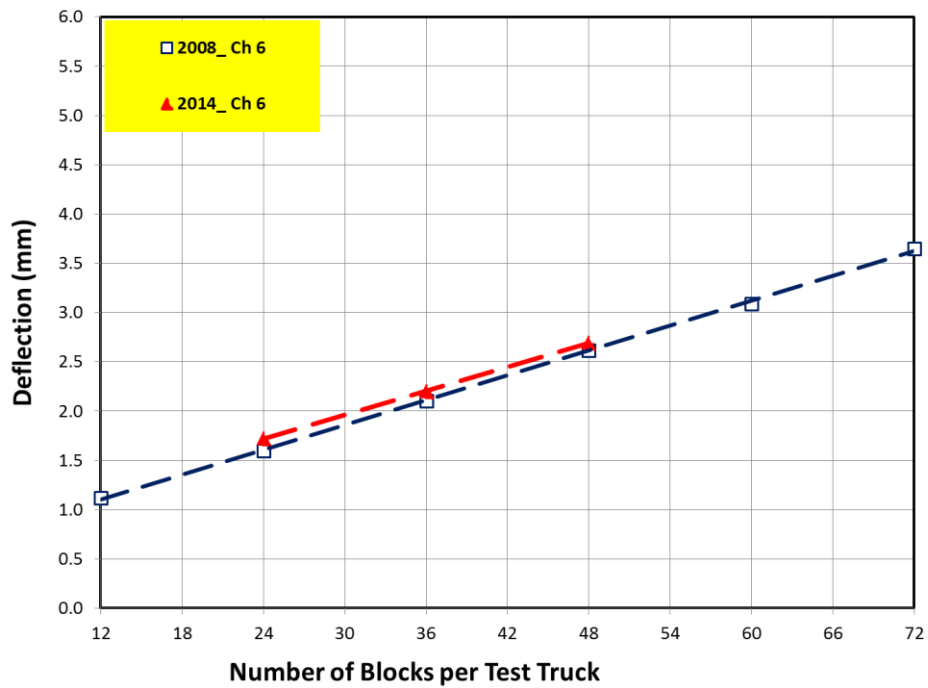


Figure 9: Deflections of Girder 3 at Section B due to Two 48 Blocks Test Trucks



(a) Girder 3 Section B



(b) Girder 3 Section C

Figure 10: Maximum Deflections of Girder 3 due to Two Test Trucks with Different Load Levels

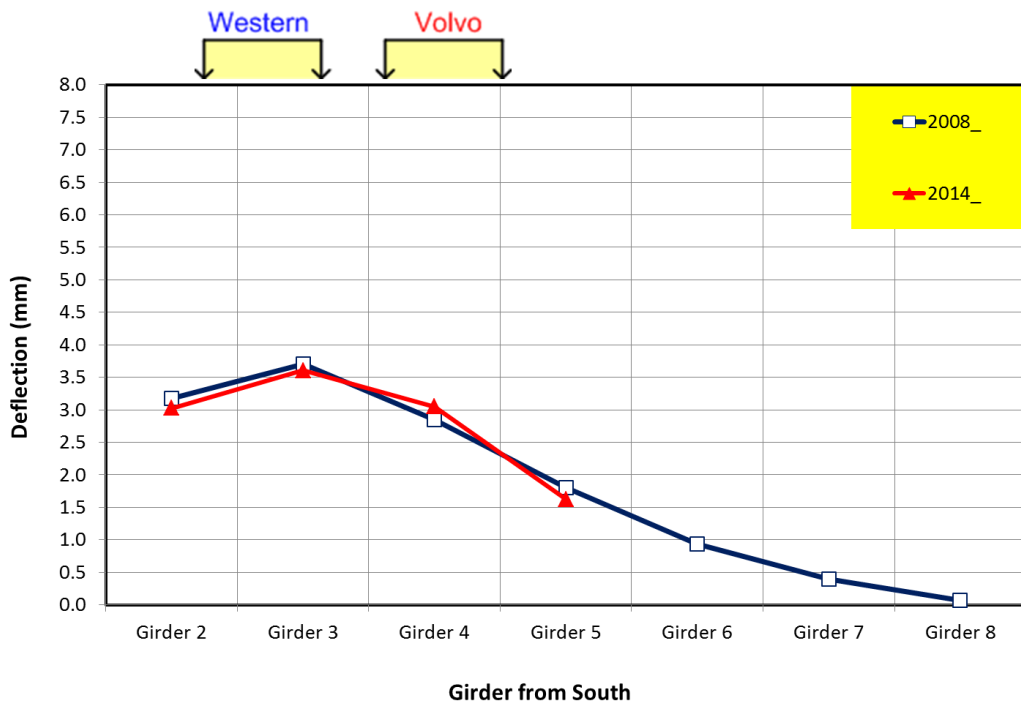


Figure 11: Transverse Deflection Distribution of Different Girders at Maximum Load Level

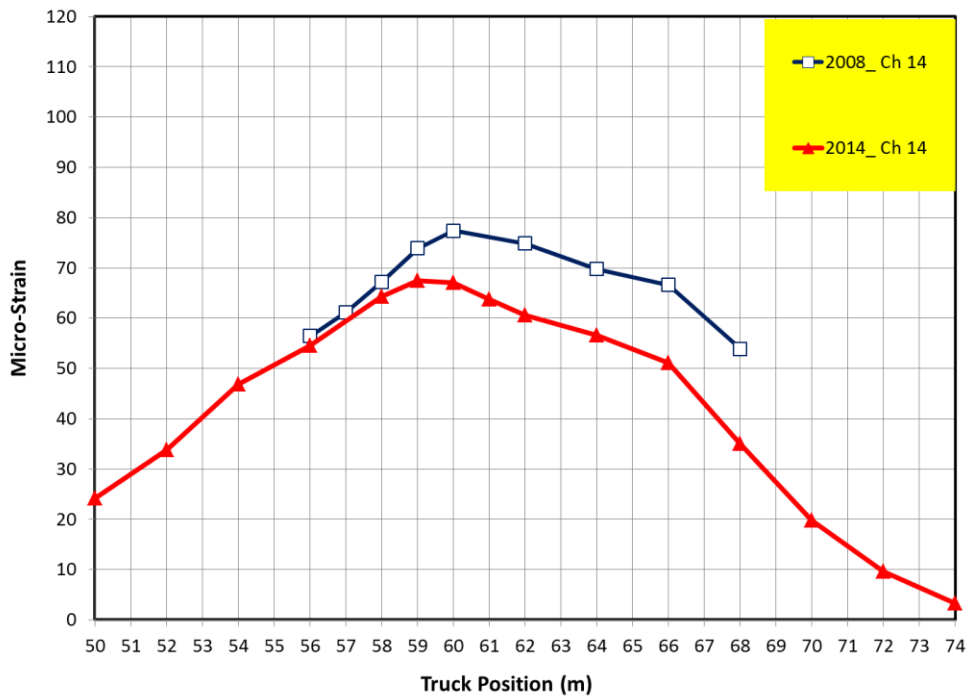
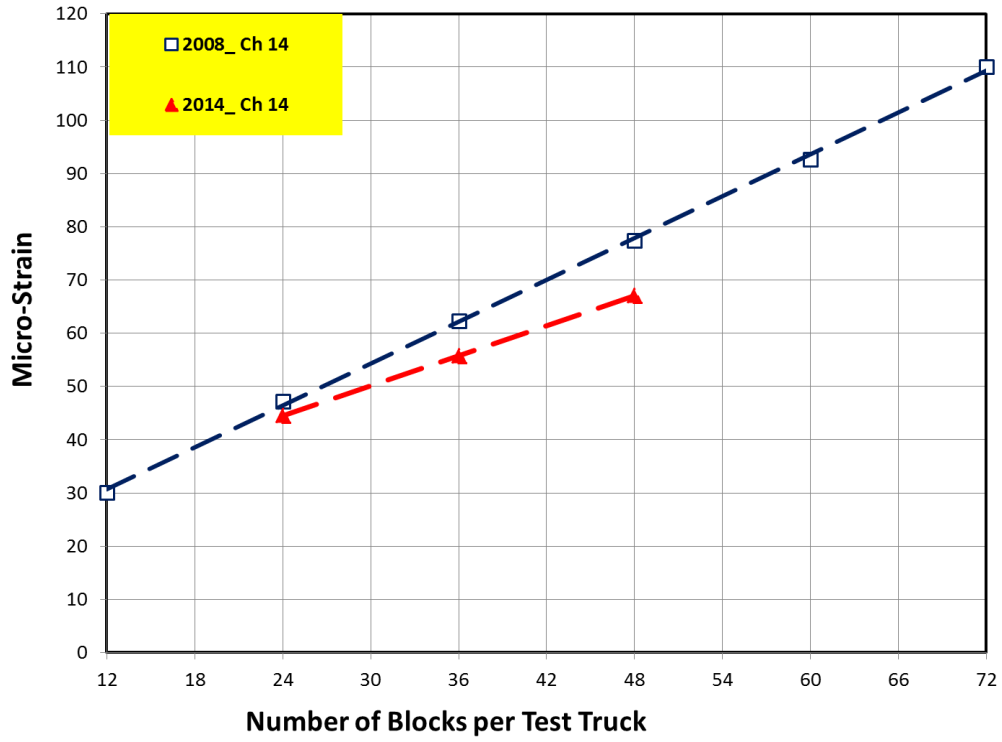
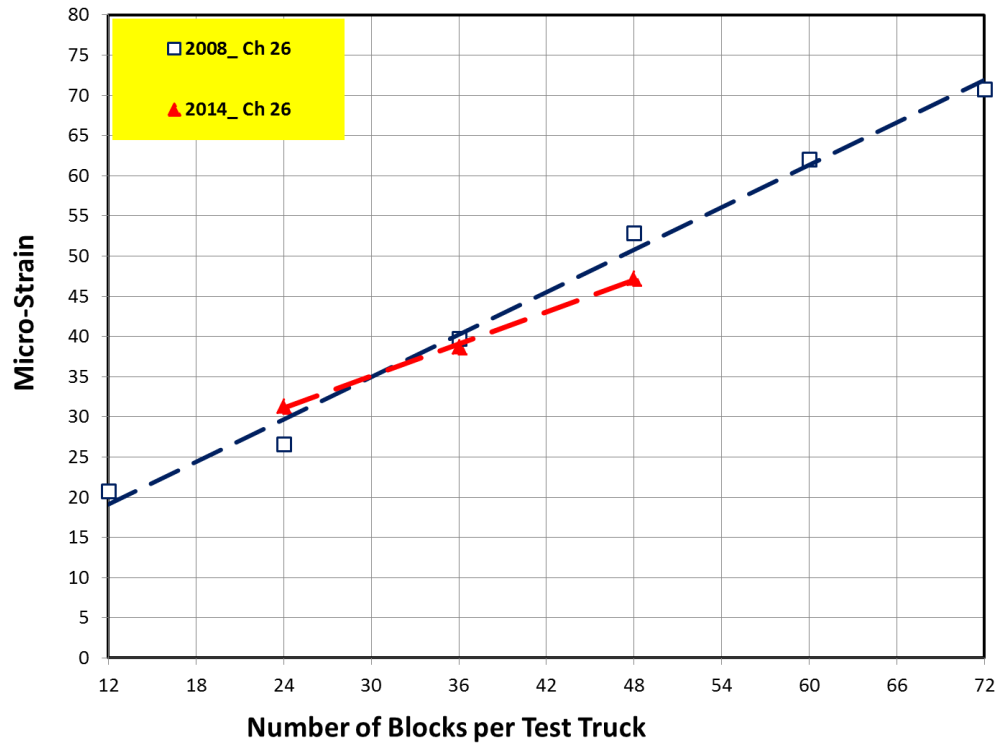


Figure 12: Comparison of Strains at Girder 3 at 48 Blocks per Test Truck





(a) Girder 3 Section B



(b) Girder 3 Section C

Figure 13: Strains Measured at Soffits of Girder 3 at Sections B and C due to Different Load Levels

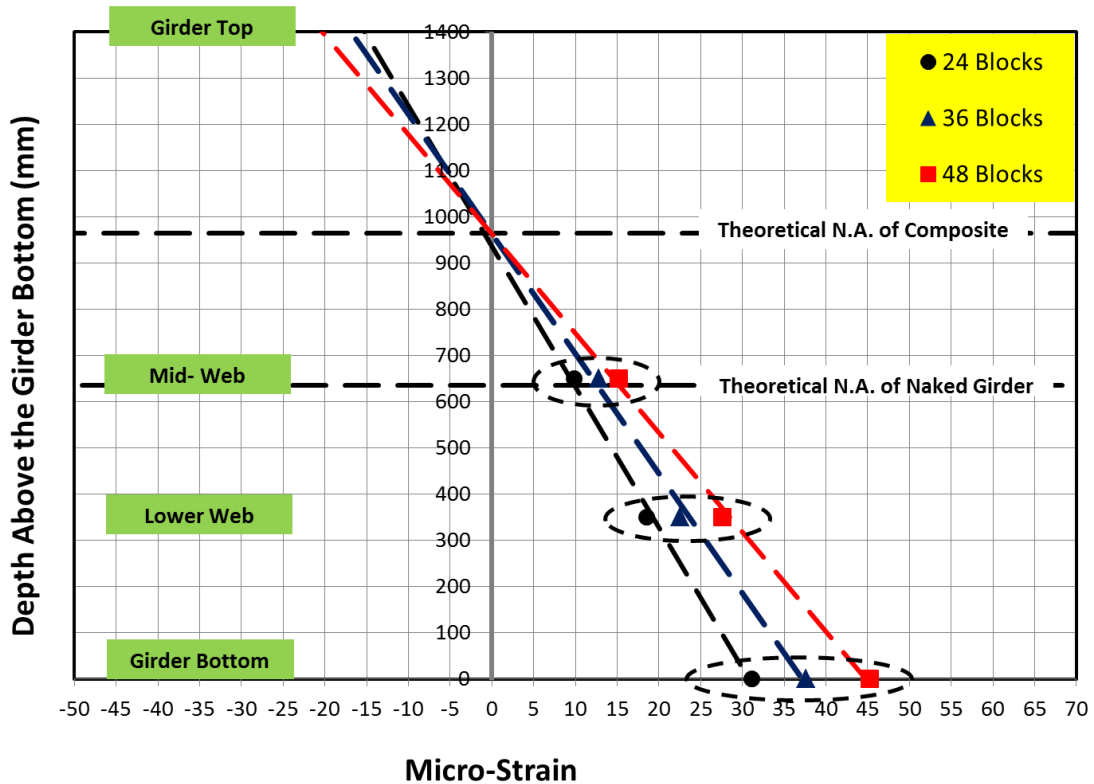


Figure 14: Strain Measured at Section C of Girder 3 (Channels 24, 25 and 26)

## 5. CONCLUSIONS

The following conclusions could be drawn from the re-testing of this bridge. The vertical deflections were similar to that measured in the 2008 load test and were linear with increasing applied loads. The measured strains were comparable to that measured in 2008 and the measured strains were also linear with the applied load. Similar to the result in the 2008 load test, the measured deflections and the measured strains were smaller than the corresponding values given by the theory. From the transverse distribution of the deflections, the bridge is capable to distribute load transversely similar to that in the 2008 load test. Composite action remains unaffected possibly due to the attribution of the repair. Based on the above observations, the bridge is performing in a similar manner showed in the 2008 load test. Therefore, the conclusions from the 2008 load test are still applicable and the bridge should be capable of carrying the CHBDC design load.

## REFERENCES

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