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The use of SHM in the evaluation of a railway bridge

Baidar Bakht[1], Shamim Sheikh[2], Aftab Mufti[3], and Gamil Tadros[1]

[1] JMBT Structures Research Inc., Canada
[2] University of Toronto, Canada
[3] ISIS Canada, Canada

ABSTRACT: The paper relates to a Canadian single-track, 6-span railway bridge, in which each 26 m long simply supported span comprises two steel plate girders supporting transverse built up steel floor beams that in turn support longitudinal rolled steel stringers. In the analytical evaluation of the load carrying capacity of the bridge, in which the girders, floor beams and stringers were assumed to be simply supported and acting in isolation, it was found that the stresses in the main components of the bridge due to the Cooper E80 design loading were excessive.

For evaluation by structural health monitoring (SHM), the main components of one span of the bridge were installed with sensors to measure strains along the axes of the components, and the sensors were monitored under five different trains, two of which ran at a slow speed and the other three at the normal high speed. Peak stresses calculated from the observed strains were extrapolated to the stresses due to the design loading. In light of these extrapolated stresses due to the design loading, the following conclusions were reached.

While the maximum stresses in the top flanges of the girders were comparable to those obtained by analytical evaluation, the actual peak stresses in the bottom flanges were about 42% smaller due to the interaction between the floor system of the bridge and the bottom flanges of the girders. The peak stresses in the top and bottom flanges of the floor beams due to design loading, including impact, were found to be 31 and 43% smaller than those obtained by the analytical evaluation, respectively. Peak stresses due to design loading in both the top and bottom flanges of the stringers, including impact, were 58% smaller than those obtained by analytical evaluation. It was also found that the values of the impact factors used in the analytical evaluation were over-estimated.

1 DESCRIPTION OF BRIGE

The Canadian Pacific (CP) railway bridge under consideration is in Belleville, Ontario, Canada; it has six simply supported spans, each comprising two built-up steel plate girders, connected with 10 transverse built-up steel floor beams, each pair of which is connected with four longitudinal rolled steel stringers. The cross-section of the bridge is shown in Fig. 1, in which it can be seen that the timber ties rest on the stringers. The rail tracks rest on the ties, midway between two adjacent stringers. Since the top surfaces of the floor beams are slightly higher than the top surfaces of the stringers, the timber ties are not placed above the floor beams. A partial plan of the bridge is shown in Fig. 2. As can be seen in Fig. 3, the rail track is near the bottom of the girders

The rolled steel stringers can be seen in Fig. 4 framing into a built-up floor beam; timber ties above the stringers can also be seen in this figure along with the hangers for the scaffolding that was erected for the installation of the sensors.

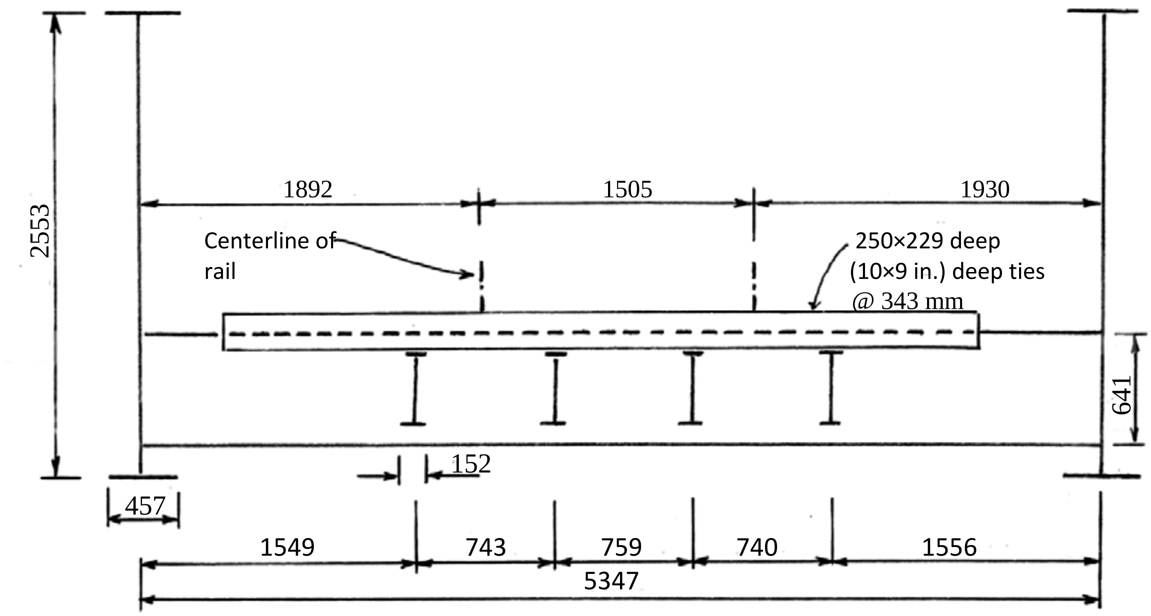

Figure 1. Cross-section of the railway bridge (all dimensions in mm)

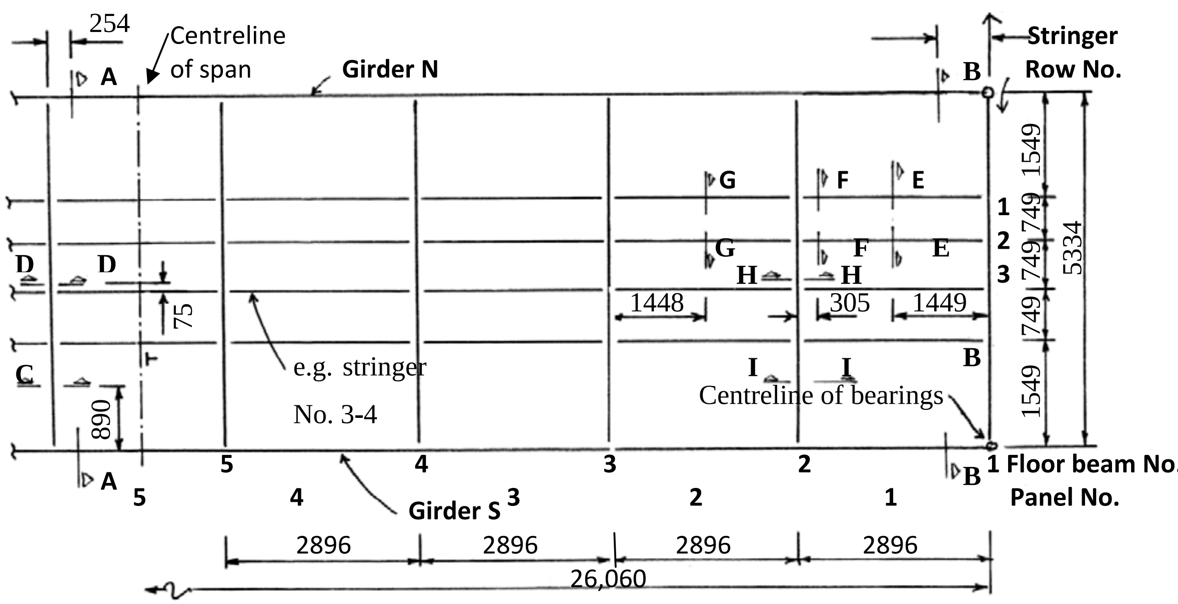

Figure 2. Partial plan of the bridge showing instrumented sections (all dimensions in mm)

The eastern-most span of the bridge, shown in plan in Fig. 2, was required by the CP to be monitored under rail traffic. As can be seen in Fig. 2, the length of the apparently simply supported girder span is about 26 m; this figure also labels the various components of the instrumented span. The two girders are labeled as North and South Girders; starting from the east end, the floor beams are labeled sequentially from 1 to 6, it being noted that the portion of the span west of Floor Beam 6 was not monitored; the stringers are labeled by two numbers separated by a hyphen, the first number refers to the stringer row number and the second to the panel number, both of which are identified in Fig. 2.

For analytical evaluation, the following values of the impact factor *I*, also known as the dynamic amplification factor, were assumed. For girders, floor beams and stringers for trains running at 72 km/h, *I* was taken as 30.6, 42.3 and 57.3%, respectively. The bridge was evaluated for the Cooper E80 design loading which is conceptually illustrated in Fig. 5, it being noted that the design loading, specified in Imperial units, has been soft-converted into metric units. The axles in each group of closely spaced axles carry equal weights.

Figure 3. Photograph of the railway bridge in Belleville, Ontario, Canada

Figure 4. Photograph showing stringers connected to the floor beams

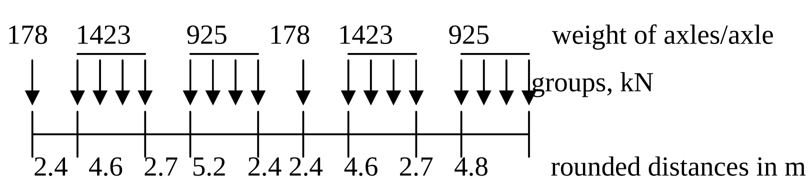

Figure 5. Cooper E80 design load with a total weight of 5,053 kN and a base length of 33.2 m.

According to the analytical evaluation, the maximum dead and live load stresses in the three main components of the bridge are as listed in Table 1.

Table 1 Maximum dead and live load stresses in various components of the instrumented span

Component	Girder		Floor beam		Stringer	
	Top flange	Bottom flange	Top flange	Bottom flange	Top flange	Bottom flange
Dead load stress, MPa	20.7	25.1	3.4	4.1	2.1	2.1
Live load stress, without impact, MPa	78.6	95.1	63.4	75.1	95.8	89.6
Live load stress with impact, MPa	102.7	124.1	90.3	106.9	151.0	151.0

The purpose of the evaluation of the bridge by SHM of the railway bridge was to determine maximum stresses in the three main components under different locomotives running at various speeds, and to extrapolate these stresses to the full Cooper E80 design loading. A comparison of these extrapolated stresses with those obtained by analytical evaluation, given in Table 1, was expected to revise the calculated load carrying capacity of the bridge.

2 INSTRUMENTATION AND LOADING

The three main components of the eastern most span of the bridge were instrumented at nine sections shown in Fig. 2 as A-A through I-I; the figure also shows the plan dimensions of the instrumented span and the numbering system for the floor beams and stringers. The instruments at all sections, except Section B-B, comprised uniaxial electrical resistance strain gauges placed at top and bottom flanges, measuring strains along the axes of the components. At Section B-B, the strain gauges were placed only at the bottom flanges of the two girders, again measuring strains along the axes of the girders. All data were collected at a rate of 100 samples per second.

The strain gauges on the bridge were monitored under five different trains. The 1st train which ran at 24 km/h, comprised a locomotive with a length of 21 m and a total weight of 1,748 kN, which is distributed over two 3-axle groups. By using the technique of equivalent base length, described by Mufti et al. (2008), Bakht et al. (2009) have shown that load effects induced by the locomotive of the 1st train are about 0.41 times those induced by the Cooper design loading.

The locomotive of the second train, travelling at 8 km/h, induced about 44% of the load effects due to the design loading. The 3rd, 4th and 5th trains ran at a speed of about 72 km/h, and the load effects due to their locomotives induced, respectively, about 44, 41 and 44% of the load effects due to the design loading.

3 OBSERVED DATA

The stresses in the various instrumented components of the bridge were calculated by multiplying the strains with the modulus of elasticity of steel, which is assumed to be 200,000 MPa. The impact factor could not be calculated directly from the observed data because of not being able to run the same train at two speeds, one of which should have been the crawling speed and the other the normal speed. However, since the peak values of the strains were used to calculate the stresses, the effect of impact is implicit in the calculated peak stresses.

3.1 Data for stringers

A sample of strains plotted against time is presented in Fig. 6 for Stringers 1-1 and 2-1 at Section E-E due to the 1st train. It was found that in Stringer 2-1, which is the middle stringer, the strains at top and bottom flanges were nearly the same in magnitude, thus indicating an absence of axial forces in the stringer. The magnitude of tensile strains in the bottom flange of Stringer 1-1, which is closer to Girder N, was greater than the magnitude of compressive strains, indicating the presence of tensile axial strains in the bottom flange, thus indicating that the stringers closer to the girders are taking some of the tensile forces due to overall longitudinal moments in the span. It was also found that the connections of the stringers to the floor beams offered some rotational restraint, because of which the stringers were not truly simply supported at their junctions with the floor beams. Mainly because of the change in the support conditions, the maximum stresses in both the top and bottom flanges of the stringers extrapolated from the test results to the design loading were about 58% smaller than the corresponding stresses obtained by analytical evaluation.

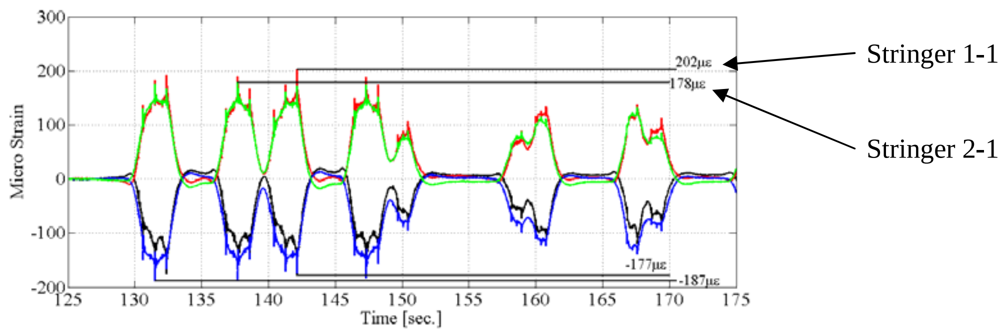

Figure 6. Strains in stringers due to the 1st train

3.2 Data for girders

Since the theoretical $S_{top,\ net}$ for girders is about 21% greater than its theoretical $S_{bott,\ net}$, the live load stresses in the bottom flanges of the girders should also have been about 21% greater than those at the top flanges. However, it was found that the magnitude of compressive stresses calculated from observed girder strains near the top flanges were larger than the magnitude of tensile stresses at the bottom. The strains at the top and bottom of the two girders collected under the 1st train are plotted against time in Fig. 7, in which it can be seen that in the North Girder the peak strains at the top and bottom are nearly -147 (compressive) and +106 (tensile) mm^{-6}/mm. Since the strain gauges near the top and bottom of the girder were 203 mm from the top and bottom surfaces of the top and bottom flanges, it can be easily calculated that the strains at the top and bottom flanges of the south girder are -174 and +130 micro-strains. Using a value 200 GPa for the modulus of elasticity of steel, these strains correspond to 34.8 and 26.8 MPa compressive and tensile stresses, respectively.

From the ratio of load effects induced by locomotive of the 1st train and those by the design load, being 0.41, it can be concluded that the maximum stresses in the top and bottom flanges of the North Girder due to the design load should be nearly -84.1 MPa (compressive) and 62.7 MPa (tensile), respectively. The corresponding stresses in the South Girder were found to be -102.0 and 76.5 MPa, respectively.

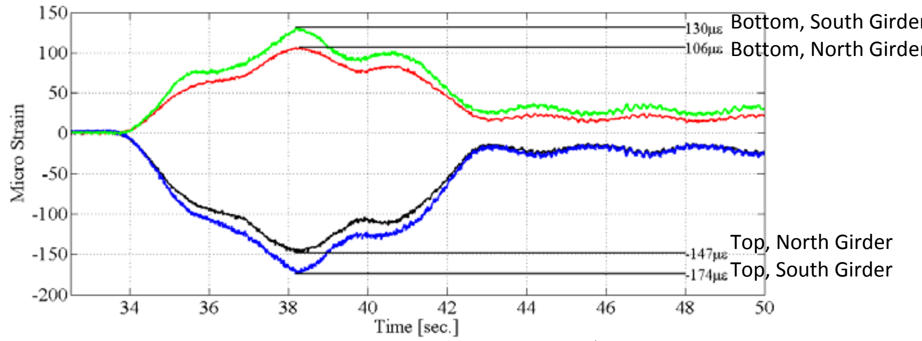

Figure 7. Observed strains in girders near mid-span due to the 1st train

The observed peak strains near the top and bottom of the South Girder due to Train No. 1 are -207 and 157 micro-strains, the corresponding stresses due to the design loading are calculated to be nearly -100.6 and 77.5 MPa, respectively. It was concluded that the actual stresses in the bottom flanges of the girders were much smaller than those obtained by analytical evaluation mainly because of the interaction of the floor system with the bottom flanges of the girders.

3.3 Data for floor beams

At its mid-span, floor beam No. 6 had two strain gauges on the top flange, ESG 5 and 6, to study if the proximity of bolts makes a difference in the readings. The strain on the bottom flange of this floor beam was designated as ESG 7. Strains in Floor beam No. 6 due to the first train are plotted in Fig. 8 against time; this plot also contains strains in gauge ESG 8, which is on the bottom flange of the same beam but closer to Girder S at Section C-C (Fig. 5).

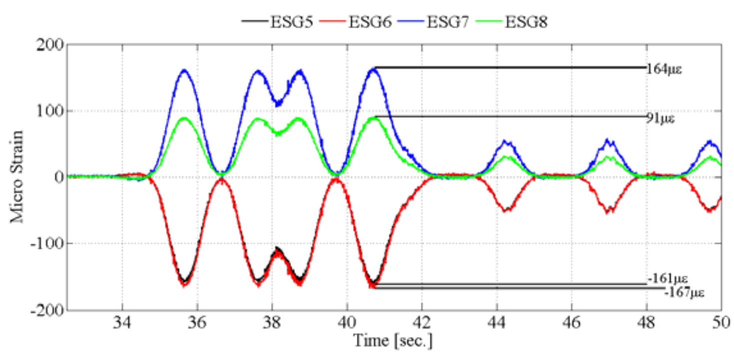

Figure 8. Strains due to the 1st train in floor beam No. 6

It can be seen in Fig. 8 that at the mid-span the magnitudes of strains in the top and bottom flanges are nearly the same, thus confirming that the floor beams are in pure flexure. The transition of strains from the mid-span to the location of ESG 8 at Section C-C (Fig. 5) showed that these beams are also simply supported. Despite the confirmation of the basic assumptions, it was found that the maximum stresses in the top and bottom flanges of the floor beams, calculated from the observed data and extrapolated to the design load, were respectively 31 and 41% smaller than the corresponding stresses obtained from analytical evaluation. The main reason for this observation is that a load applied to a floor beam also gets distributed to the other floor beams by the stringers and possibly the rails, it being noted that in the analytical evaluation, it was assumed that loads applied over a floor beam are all taken by the same floor beam.

3.4 Impact factor

A qualitative measure of the impact factor in various components can be obtained by comparing the observed data corresponding to the 2nd and 5th trains, each one of which had locomotives weighing 1846 kN; they travelled at speeds of 8 and 72 km/h, respectively. The observed peak values of strains in a girder, floor beam and stringer due to these two trains are listed in Table 2. The impact factor *I* in the three components, also listed in Table 2, was calculated by dividing the difference of the peak strains due to the two trains by the peak strains due to the 2nd train. It can be seen in Table 2 that the impact factor in the girders is negligble and those in the floor beams and stringers are considerably smaller than assumed in the analytical evaluation, which for trains running at 72 km/h are 30.46 and 57.3% for floor beams and stringers, respectively. While the values of *I* given in Table 2 are not valid when a wheel might become flat or there are irregulaities in the tracks, it is noted that the maximum load effects in all the main components are caused by several wheels, because of which the irregularity of a single wheel, or a single wheel going over an irregularity in the track, is likely to have relatively small effect on the impact factor. It appears that the impact factors assumed in the analytical evaluation are conservative. However, it must be emphasized that the determination of the values of the impact facor used for the 80 year design life of the bridge require extensive testing under different loads.

Table 2. Peak values of strains (in micro-strains) and corresponding impact factors

Component	Strain due to 2nd train @ 8 km/h	Strain due to 5th train @ 72 km/h	Impact factor
Girder	-210	-211	0%
Floor beam	+198	+223	13%
Stringer	-253	-261	3%

4 CONCLUSIONS

The evaluation of the bridge under consideration has shown that the actual maximum stresses in its critical components, including impact, are much smaller than those predicted by the analytical evaluation. It is thus concluded that the evaluation of the load carrying capacity of the bridge through SHM permits a better use of the bridge than that resulting from analytical evaluation.

5 ACKNOWLEDGMENT

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