SERVICE LOAD INVESTIGATION OF THE COMPOSITE BEHAVIOR
OF A BALLASTED THROUGH PLATE GIRDER SPAN

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ABSTRACT

Canadian Pacific Railway’s Eagle River Bridge at Mile 31.10 Shuswap Subdivision, near Revelstoke, British Columbia is a 157’-4” single track, Ballasted deck Through Plate Girder (BTPG). Constructed in 1977, the BTPG span consists of welded plate girders, transverse floorbeams and a reinforced concrete deck. The concrete deck was made composite with the floorbeams using shear studs.

Shortly after construction, transverse cracking of the concrete deck was noticed near mid-span. Over the 27 years that the bridge has been in service, the cracking has progressed and caused loss of the composite action between the deck and the floorbeams. Attempts have been made to seal the cracks, additional lateral bracing and longitudinal supports to the slabs have been added, and some sections of slab have been removed and replaced.

Preliminary evaluations indicated that a loss of composite action may have compromised the strength of the floor system. In order to better understand the behavior of the floor system and the level of live load stresses in the floor system, UMA Engineering Ltd. was engaged to design and conduct a monitoring program. Forty-eight strain gauges were installed on selected steel members and data was collected over three days of monitoring.

This paper describes the history and condition of the bridge, and details the field instrumentation and monitoring program. The results of the structural analysis are compared to the collected data, and rehabilitation options being considered by Canadian Pacific Railway are presented.

(Key Words: TPG, Composite, Strain Gauge)
INTRODUCTION

In 1977, Canadian Pacific Railway (CPR) replaced a seventy-three year old 157-foot long through truss span which carried rail traffic over the Eagle River, at milepost 31.10 Shuswap Subdivision, in central British Columbia. The through truss span was replaced due to an advanced state of corrosion, cracks in the floorbeams and low rated strength of the floor system. The replacement span was a new ballasted deck Through Plate Girder span (BTPG).

CPR typically designs long span BTPG bridges using steel deck plate construction. However, the 1977 design for 31.10 Shuswap used a reinforced concrete deck. This deck is continuous in the longitudinal direction and uses mechanical shear connection studs to act compositely with the transverse floorbeams in the lateral direction (Figure 1).

The BTPG span was placed into service in late 1977. Since then, the behavior of the deck has been less than acceptable, with significant transverse cracking and separation of the deck from the transverse steel floorbeams.

HISTORY OF DECK BEHAVIOR SINCE CONSTRUCTION

In the spring of 1979, it was reported that the reinforced concrete deck exhibited cracking and spalling at the curbs adjacent to knee braces near the ends of the BTPG span. These were thought to result from poor construction details at the concrete curb to steel knee brace interfaces. Random cracking was also reported in the tops of the curbs over intermediate floorbeams; which was attributed to relatively high tension in the concrete curbs caused by the negative longitudinal flexure over the floorbeams.
Later in 1979, transverse hairline cracks were discovered that extended for the full width of the curbs and the deck. Most transverse cracks occurred near the center of the BTPG span. Otherwise, the span was observed to be performing well. It was surmised that the transverse cracking was due to the cold weather in-situ concrete deck construction in late 1977. It was recommended to seal the cracks against water intrusion and to monitor crack behavior. It was also noted, at that time, that the concrete deck may be subjected to greater tensile strain than was anticipated during the design.

In 1980, the Canadian Portland Cement Association was retained to assess deck performance and concluded that;

• it was likely that improper curing, particularly at relatively thin sections near larger steel components, created conditions that promoted cracking at the deck curbs;
• the transverse cracks through the deck are shrinkage cracks and should not be detrimental to structural integrity;
• the segmental curb design details may perform poorly due to differential thermal expansion between the steel girders and the concrete deck (i.e. exterior of girder heated by sunlight with deck shaded); and
• the cantilevered deck segments at the end of BTPG span are difficult to cure and may be subjected to considerable bending moment because of size and geometry relative to the concrete deck.

Following this assessment, it was recommended that the curb and deck transverse cracks less than 1/32” wide (hairline) be injected with penetrating sealer and that wider cracks up to 1/8” be repaired with an epoxy compound. To allow moisture to escape, it was recommended to seal
only the tops and sides of the cracks. It was also recommended to caulk the interfaces between the steel knee brace and the concrete deck.

By the middle of 1981, prior to performing the planned concrete deck repairs, the number of cracks in the deck and curbs were observed to have increased significantly and that some cracks had propagated through, to the underside of the slab. Many of the curb cracks occurred in the negative moment regions over floorbeams.

By this time, spalling and cracking of the curbs at the knee brace locations was observed to have increased, particularly near the ends of the spans. This pattern of cracking occurs due to differential strains between the steel girders (and attached knee braces) and the concrete deck. These differential strains are most pronounced at the ends of the bridge, apply a thrust to the concrete curbs directed toward the end of the bridge, and cause concrete spalling at the curb to knee brace interfaces.

Crack injection repairs were completed in 1982.

The bridge was reported to be very lively (both vertically and laterally) under heavier live loads such as unit coal trains. Detailed deflection measurements were taken. Lateral displacements of the girders of up to 3/8" and vertical deformations of up to 1-1/4" (span/1500) were recorded under revenue traffic. It was recommended to install a lateral bracing system in an attempt to increase overall lateral stiffness of the bridge. This lateral stiffening was also expected assist in halting further cracking and spalling of the curbs that might be related to diaphragm action of the deck.
In 1986, it was noted that the repairs to the transverse cracks had failed. Many had re-opened and, in some cases, propagated further. Concrete deterioration (particularly at the edges of the deck) and reinforcing bar rust staining from moisture ingress were also noted.

At this juncture, a three-step approach for the bridge rehabilitation was proposed:

1) Install lateral bracing to stiffen the structure against lateral displacements.
2) Patch the deteriorated concrete and install deck waterproofing to delay concrete deterioration and reinforcement corrosion.
3) If required, install additional supports under the concrete deck to arrest transverse deck cracking.

Also in 1986, separation of the concrete deck from the transverse floorbeams was noted at the ends of many of the floorbeams.

By 1989 (prior to planned rehabilitation) the transverse cracks appeared to have propagated further. Linear potentiometer crack monitors were installed but readings were not taken at sufficient intervals to determine definitive propagation trends or rates. Nonetheless, the readings did show that considerable crack opening and closing was occurring under live load.

Lateral bracing and waterproofing were completed in 1989. Lateral girder deflections decreased to about 50% of those measured prior to bracing installation.

During 1988 and 1989, separation at the concrete deck to floorbeam interface was noted to be propagating (slowly) from the ends of the floorbeams toward the center. By 1990 the length of separation measured up to 85” on some floorbeams, about 50% of the length that is intended to be composite. Loss of floorbeam strength became a concern.
The loss of shear connection is partly due to embedment and/or splitting cracking at the mechanical shear connectors in concrete tension zones created by span continuity or end rigidity of the floor beams (i.e. at knee brace locations) (Figure 2). This phenomenon, in conjunction with the longitudinal tensions in the slab from participation with the main girders in flexure, creates bi-axial tension in the concrete. It is little wonder that the shear studs were unable to maintain proper mechanical connection.

AREMA (AREA) design procedures are based on full shear connection; in which case partial interaction (i.e. slip strains) can be safely ignored in the design. However, the reduction in shear connection stiffness associated with partial shear connection may compromise service load strength of the floorbeams if the slip strains are large enough. Service load partial shear connection analysis is complex, but techniques based on experimentation and theory have been developed (Reference 1). The elastic strain distribution through the floorbeam section with full and partial shear interaction is shown in Figure 3.

The floorbeams were rated as simply supported beams (per the original design assumption). Partial composite action due to mechanical shear connectors was considered. The floor beams were found to be marginally sufficient with an assumed 50% loss of composite connection at the concrete to steel interface. This reduction in the composite action might compromise service load strength (fatigue life) of the floor beams due to the reduced flexural stiffness.

Additionally, the flexural strength of the deck in the longitudinal direction was found to be inadequate. In 1991, temporary support was recommended, using longitudinal stringers to reinforce the structure until the deck or bridge could be replaced. The longitudinal stringers would provide direct support to the concrete deck beneath the rails between floor beams. The
stringers would also strengthen the overall flexural capacity of the bridge in the longitudinal
direction by carrying some proportion of the flexural tensile stresses of the main girders.

In 1993, only minor hairline propagation of some transverse deck cracks had been reported, but
by 1997 some further propagation was observed (Figure 4). In 1998, longitudinal stringers were
installed to provide vertical support to the deck slab as recommended in 1991 (Figure 5).

The cumulative rehabilitation measures performed over a sixteen-year period, prior to 1998,
were observed to have reduced crack propagation rates related to longitudinal behavior of the
deck slab. The life of the concrete deck, in terms of longitudinal behavior, had been extended.
However, the integrity of transverse composite action remained compromised so there was
continuing concern about the fatigue life of the floor beams.

A detailed inspection in 2003 noted increased separation and “pumping” under live load at the
concrete deck to floorbeam interface. A structural analysis and rating indicated that complete
loss of composite behavior (no shear connection) would over-stress transverse floorbeams and
promote further cracking and deterioration of the deck. To protect the safety of ongoing
operations, rehabilitation or replacement of the BTPG span was recommended.

Initial investigations revealed that rehabilitation under traffic (up to 40 trains per day on single
track) would have a major and costly impact on train operations. Because it was felt that the
relatively complex nature of the shear loss behaviour was not fully understood, and in light of the
expensive capital cost of complete span replacement, it was deemed prudent to conduct field
measurement of stresses under service conditions to more thoroughly understand the longitudinal
and transverse stress states and composite behavior of the span.
FIELD INVESTIGATION, AND ANALYSIS

Scope of Work

UMA Engineering Ltd., was retained by Canadian Pacific Railway to install strain gauges at selected locations on the main girders, floorbeams and stringers; and to record the stresses seen by the structure under three days of revenue traffic. The purpose was to determine the distribution of stresses and to evaluate the composite participation of the concrete deck slab with the steel structure.

While it would have been desirable to measure the stresses in the slabs, concrete is an extremely non-homogenous material (strain measurements are locally affected by the proximity to rebar, cracks, and other non-homogeneous features) making it difficult to obtain reliable and useful information. It was concluded that only the steel should be strain gauged, and that the stresses obtained could be used to calculate the average stresses in the concrete.

48 strain gauges were installed on the bridge, between the East end (Floorbeam 1) and mid-span (Floorbeam 14). Four gauges were installed on each of the following members to capture the strains across the entire cross section of the member (See figure 6):

- Midspan of the left and right girders
- Midspan of the left and right stringers between Floorbeams 13 and 14.
- Both ends of Floorbeam 14, 6 inches from the end of the slab
- Mid-span of Floorbeams 1, 8, 12, 13 & 14

Analysis of Results
Strain readings were recorded over a four-day period during daylight hours under a variety of unit trains, including 286,000 lb coal and sulfur as well as Intermodal and mixed freight. The data acquisition equipment was able to simultaneously record 16 channels of data. Various combinations of strain gauge locations were paired at different times depending on the information sought. For example, to determine the participation of the slab in carrying longitudinal bending at mid-span, it was necessary to simultaneously record the stresses in both main girders and both stringers at mid-span.

The recorded data was mathematically processed to convert the strain readings into stresses which were extrapolated to determine the extreme fibre stresses, assuming that all strains were linearly elastic across the cross section. A typical printout showing the data and analysis is given in Figure 7.

**Summary of Results**

The analysis showed that the percent of composite action (as compared to the theoretical design slab width that should participate with the floorbeams in accordance with the design guidelines in section 15.5.1.2c of AREMA) was between 2% and 16% at mid-span of the floorbeams (See Figure 8). If there was full composite action (as designed), the Neutral Axis of the floorbeams should have been near the top flange of the beams. The actual location of the neutral axis was much closer to mid depth. As a result, floor beam flexural stresses were between 25% and 103% higher than if the intended composite behavior were occurring.

It must be noted that strain gauges showed that the flexure in floorbeam 14, near the outer edges of the slabs, was actually negative - resulting in transverse tension and loss of shear connection.
This behavior was not properly addressed in the design, which assumed simple support conditions for the floor beams.

The analysis of the main girder flexural stresses at mid-span of the bridge showed that the neutral axis of the main girder was roughly six inches below mid-depth of the doubly symmetric girders, indicating significant composite contribution from other structural components below mid-depth (Figure 9). Strain gauges on the stringers permitted calculation of longitudinal live load axial tension so a simple summation of longitudinal forces to maintain equilibrium showed that the concrete deck carries about 34 to 179 kips tension, depending on the type of train (loaded unit train vs. empties). The neutral axis position confirmed significant longitudinal composite action of the concrete deck, which must be carried across cracks, primarily by the longitudinal reinforcing steel.

**Other interesting observations**

Figure 10 shows that at Floorbeam 14, the East and West top flange compressive stresses track almost parallel to each other except that the West side compressive stresses clip at approximately 60% of the peak East side compressive stresses. The authors speculate that this clipping effect may be due to “wear” or “slack” between the shear studs on that side of the floorbeam flange which allows slip until the gaps and the shear studs bear on, and transfer load to, the concrete deck slab; thereby increasing partial composite behavior at high loadings.

Figure 11 shows the stresses in the main girders during passage of a unit coal train. Notable are the cyclic stresses in the top flange of the girder, which are clearly evident. When the North side of the girder sees its peak compressive stress, the South side sees a corresponding opposite
reduction. The top flange of the girder is, therefore, bending in the transverse direction. This was visually observed during the passage of all trains, particularly heavily loaded unit trains. The wavelength of this transverse bending was approximately equal to the centre to centre of coupled trucks in the unit coal train. This implied that the flexing of the floorbeams, connected via the stiff knee braces, was rotating the girders inward. The magnitude of flexural stresses was approximately 5 ksi at the strain gauges (equivalent to 7.5 ksi at the extreme inner and outer fibers of the top flange). This was a significant level of cyclic stress, particularly since it occurred once with the passage of each car.

RECOMMENDATIONS FOR REHABILITATION OR RECONSTRUCTION

The field investigation confirmed the significant loss of composite action between the concrete deck and steel floorbeams; and that continued degradation of the deck and composite behavior is likely.

Rehabilitation of the BTPG bridge by removal of the concrete deck and shear connectors; and subsequent installation of additional transverse floorbeams, knee braces and steel deck plate was investigated in detail. This would provide a rehabilitated BTPG bridge with a steel deck that would perform well in longitudinal flexure, and would create a stiffer floor system, with increased resistance to lateral-torsional buckling of the girder compression flanges and a reduced level of cyclic transverse flexing of the top flanges of the girders imposed by floor beam behavior.

Preliminary planning and design of the rehabilitation revealed that over 200 tons of steel combined with field drilling of up to 5000 holes in plate girder webs would be required. This
work is well suited to CPR’s Bridge and Building forces. Following ballast removal and blocking of the skeletonized track, it was estimated that up to 25 track blocks, with durations between 4 and 8 hours, would be required to stage the rehabilitation work. While this was an attractive option from a capital cost perspective, the interruption to railroad traffic was not acceptable.

Alternatives for span replacement on rehabilitated abutments were investigated for a steel deck BTPG or steel through truss. The use of two ballasted concrete deck plate girder spans (BDPG) or two steel deck BTPG spans on the existing abutments and a new central pier were also reviewed. The latter was the most cost effective alternative from a construction and operations perspective.

In early 2004, it was proposed to further develop this option and seek the required regulatory approvals for construction.

Later in 2004, CPR began planning for capacity improvement across its heavily traveled lines in Western Canada. Fortuitously, the construction of a second bridge beside the existing crossing of the Eagle River at milepost 31.10 Shuswap Subdivision became a part of CPR’s double tracking initiatives in central British Columbia.

Therefore, CPR is planning to construct a new bridge adjacent to the existing bridge in 2006 and to operate traffic over that bridge while rehabilitating the existing BTPG span, without interruption to railroad traffic. The timing of the rehabilitation will depend on overall scheduling of the many related capacity improvement projects on CPR’s Western lines.
Various proposals, generally involving construction of a three span bridge, are being developed for consideration. It is expected that, following regulatory approval of a construction general arrangement, site investigations and detailed design will proceed in 2005.

CONCLUSIONS AND LESSONS LEARNED

1) If concrete is used as the deck in a TPG span, it should be engineered to address participation with the longitudinal bending of the main girders. Options range from longitudinal post-tensioning of the concrete (to remain in compression under live load), or the use of control joints or other means to prevent unwanted composite participation.

2) On the Eagle River Bridge, participation of the concrete deck with the longitudinal flexure of the main girders was not addressed in the initial design. The concrete deck was intended to be transversely composite with the floorbeams in accordance with AREMA (AREA) chapters 8 and 15 but no consideration was given to the tensile stresses in the longitudinal direction. The failure of the concrete deck was not immediate nor was it catastrophic. It manifested itself as long-term growth of transverse tension cracks to relieve the longitudinal tensile stresses. Unfortunately, many of these cracks formed at the transverse floorbeams where composite action in the transverse direction was intended by the design. This lead to a general loss of composite connection between the slab and the floor beams and a corresponding loss of flexural stiffness. Nonetheless, with significant maintenance expenditures to date, 28 years of safe and useful service have been provided by this structure.

3) An unexpected finding of the strain gauge investigation was that the main girder flanges experience large transverse stress cycles, at levels in the order of 7.5 ksi in the top flange.
These stress cycles are caused by elastic flexure of the floorbeams under each set of coupled trucks which, through the stiff knee braces, rotate the top flange inwards. The magnitude and frequency of these stress cycles is not currently contemplated in the conventional design of TPG girders. The number of these stress cycles is two orders of magnitude higher (once per car) than the number of stress cycles for longitudinal flexure (once or twice per train). The magnitude of these stress cycles will depend on the flexibility of the floorbeams, the depth of the girders, and the stiffness of the knee braces connecting the two. While most TPG spans have shallower girders and stiffer floorbeams, this phenomenon should be understood by railway bridge designers to assist in the selection of appropriately stiff floor systems when designing deep girders. It is possible that, in extreme cases, the magnitude and frequency of these stress cycles could affect the fatigue life of the structure.

4) Floorbeams exhibited significant negative moments at the knee braces. While it is conservative to size floorbeams as if they are simply supported, designers of composite decks must recognize that negative moments can and will occur in the vicinity of stiff end connections (particularly at knee braces). The negative moments generate tensile stresses in the concrete in the transverse direction, which in this case were superimposed on unintended tensile stresses in the longitudinal direction (due to composite participation with flexure of the main girders). The design of similar structures should adequately consider the cyclical biaxial tensions in the concrete at the ends of the floor beams.
REFERENCES

(1) AREMA chapter 15 Section 15.5.1.2c (2005)


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### Analysis of Floor Beam Composite Action

**W24x76 Floor Beam:**
- depth \( d = 23.96 \) in
- Area \( A_{bcf} = 22.15 \) in²
- Moment of Inertia \( I_{bcf} = 2072 \) in⁴

**Concrete Slab:**
- thickness \( t = 10 \) in
- Design Width \( b = 42 \) in
- Modulus Ratio \( n = E_b/E_c = 7.2 \)

**Composite Section Properties, as Designed:**
- \( b_s = b \times n = 5.833 \) in
- \( y_s = 24.29 \) in
- \( k_c = 7188.32075 \) in⁴
- \( S_b = 225.814 \) in⁴

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<th>Selected Data for Analysis (Seconds since Start of Test)</th>
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<th>Peak Bottom Flange Stress (ksi)</th>
<th>Equivalent Steel Width ( b_s ) (in)</th>
<th>Equivalent Concrete Width ( b_e ) (in)</th>
<th>Effective Composite Behaviour</th>
<th>Effective Moment of Inertia (in⁴)</th>
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<th>Increase in Peak Steel Stress due to Loss of Composite Action</th>
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