

Evaluation and Field Load Testing of Timber Railroad Bridge

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Several spans of a 60-year-old open-deck timber railroad bridge on the Southern Pacific Railroad Line (now the Union Pacific) in Southwest Texas were field tested. The tests were conducted with the sponsorship and cooperation of the Association of American Railroads to determine the vertical live load distribution characteristics of the superstructure. The bridge was originally constructed with Douglas-fir larch solid sawn stringers but was rehabilitated on several occasions to allow comparisons to be made with respect to different rehabilitation options, including the use of a helper stringer and the use of glued laminated timber (glulam) stringers. The test spans measured approximately 4.1 m (13.5 ft) center-to-center of supports and included two closely "packed" chords, each consisting of four timber stringers (one test span included an additional helper stringer added to one chord). One chord was made up of glulam timber and the other was made up of solid sawn timber. The bridge superstructure was generally in satisfactory condition, with some stringer horizontal splitting noted over the bents. The bents were in reasonably good condition, but chord bearing was uneven on bent caps. Static and dynamic deflection load test data were obtained using a special test train. The test results indicate that the glulam chord performed better than the older sawn stringer chord, even when a helper stringer was added. Individual stringers within a chord did not always share the load equally.

A recent study by Wipf et al. (1) performed in cooperation with the Association of American Railroads (AAR) addressed the increasing loads carried by modern freight trains to provide a fundamental understanding of bridge structural behavior and load distribution characteristics. The study was the first phase of a two-phase study with an ultimate objective of investigating cost-effective and efficient new designs. The specific tasks of the first-phase study dealt with measuring and quantifying the static and dynamic behavior of two open-deck timber bridges. The scope of the study was restricted to experimental testing in which superstructure deflections and accelerations were measured. These two open-deck bridges had been in service for more than 60 years and only in recent years had shown significant deterioration. The bridges were constructed of solid sawn stringers that were placed within two four-stringer chords "packed" closely together. The recent applications of frequent heavy axle loads (HALs) were the primary cause of the deterioration. The tests showed that stringers within a chord acted independently to resist the applied loading from a special test train and revenue traffic. Experimental results also showed that there was little longitudinal continuity of deflection between chords of adjacent spans.

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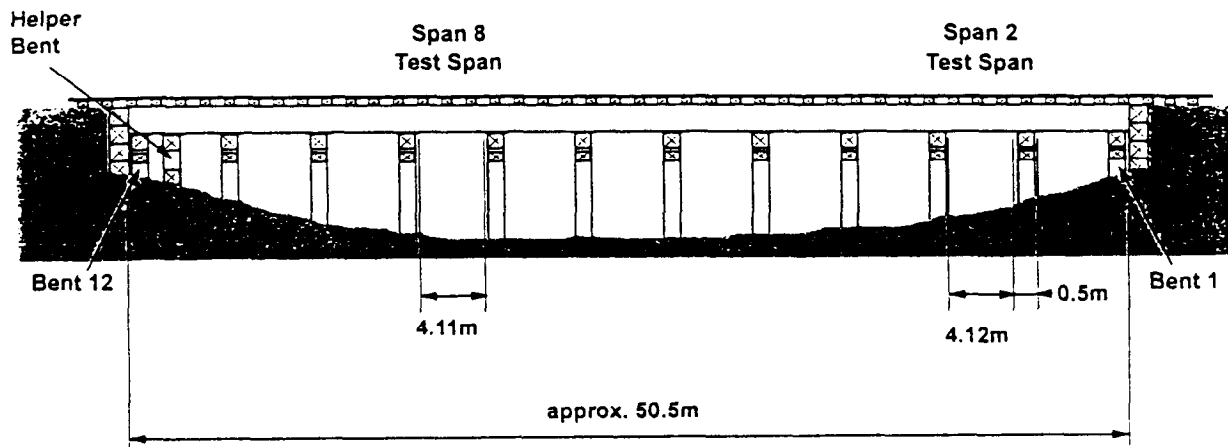
As with the timber bridges just described, many timber railroad bridges have been in service approximately 50 years or more and are at, or near, the end of their service life. Particularly with the recent increase in train loads, a need exists to investigate the effectiveness of rehabilitation alternatives, which includes new construction. This rehabilitation will enable timber railroad bridges to continue to perform satisfactorily in meeting the demands of the increasing loads. This need was recognized by the AAR and the Southern Pacific (SP) Lines, now merged with the Union Pacific (UP) Line, in their initiation of the study presented in this paper to assess the static and dynamic load distribution characteristics of several rehabilitated timber railroad bridges (2). The SP Line was chosen for these tests because it (a) possesses a large number of timber bridges, (b) carries large volumes of HAL traffic, and (c) has an aggressive inspection and monitoring program that has included significant replacement and rehabilitation work.

OBJECTIVE AND SCOPE

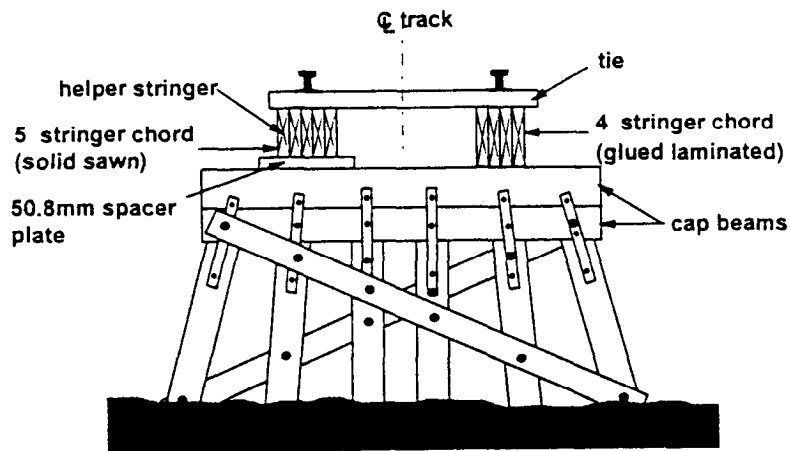
The objective of the research presented in this paper was to measure and quantify the static and dynamic load distribution characteristics of the D'Hanis bridge, a recently rehabilitated timber railroad bridge. The scope of the study was primarily related to field assessment and testing, although an analytical correlation was made with the field deflection results.

DESCRIPTION OF BRIDGE

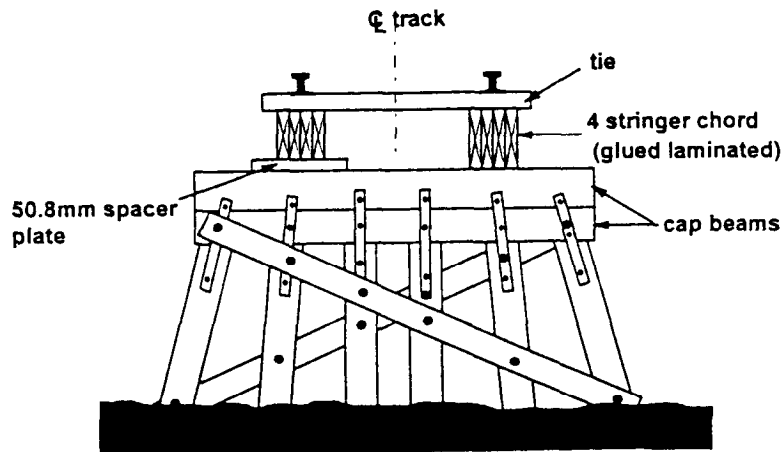
The D'Hanis bridge is located approximately 3.2 km (2 mi) west of D'Hanis, Texas, on the Del Rio District of the SP southern main line. It is an 11-span open-deck timber bridge supported by timber pile bents (Figure 1). The bridge is approximately 50.5 m (165 ft 8 in.) long and 2.1 m (7 ft) high and was built approximately 60 years ago using creosote-treated Douglas-fir larch. The north chord was replaced approximately 10 years ago with treated glued laminated (glulam) stringers as an informal study by the SP to investigate the potential use of glulam as a replacement material. The south chord remained as solid sawn stringers. In 1996 additional modifications were made to the superstructure as part of the research study presented in this paper. Helper stringers were placed in Spans 1, 2, 3, and 4 of the solid sawn chord, resulting in a five-stringer chord for these four spans. The helper stringers appeared to have been previously used on another bridge and were placed on the outside (south side) of the south chord. The resulting five-stringer chord was centered so that the rail aligned with the center of the five-stringer group. This modification was completed so that a load test comparison could be made between the glulam north chord and the solid sawn lumber south chord.



a. elevation - looking north



b. cross section - intermediate bent - span 2



c. cross section - intermediate bent - span 8

FIGURE 1 Schematic views of D'Hanis bridge.

Each span consisted of the rails, ties, and bridge chords. The rails were continuous welded rail and were anchored at each tie. The ties were supported by the two longitudinal chords; each span of the north chord consisted of four 175-mm (6¾-in.) wide by 457-mm (18-in.) deep glulam timber stringers. On the south chord, in Spans 1 through 4, the ties were supported by five 197-mm (7¾-in.) wide by 419-mm (16½-in.) deep sawn lumber stringers (including the helper stringer). In Spans 5 through 11, the ties were supported by four 197-mm (7¾-in.) wide by 419-mm (16½-in.) deep sawn lumber stringers. The majority of the stringers were continuous over two spans and were alternately continuous over the intermediate bents.

The timber bents consisted of two cap beams, one on top of the other, supported by six piles at intermediate bents and five piles at the abutments. The bottom cap beam was 343 mm (13 ½ in.) wide by 292 mm (11 ½ in.) deep. The top cap beam was nominally 356 mm (14 in.) square. The two cap beams were connected with splice plates at several locations. On the north chord, two steel plates, typically 13 mm (½ in.) thick, were placed on the cap beam to provide bearing. At several locations three steel plates of various thicknesses were used. The south chord bearing conditions included a 51-mm (2-in.) deep lumber spacer with steel plates of various thicknesses on the top of the caps, to provide matching elevations with the north chord. The typical steel bearing plate configuration utilized a single plate 10 mm (¾ in.) thick. However, at several locations two or three steel plates of thickness ranging from 6 mm (¼ in.) to 10 mm (¾ in.) were used. The bearing length on the continuous stringers was approximately 483 mm (19 in.) and that for the discontinuous stringers was approximately 241 mm (9½ in.).

The approach track was in good condition and contained a full section of rock ballast material. The track was tangent and the grade was approximately level.

RESEARCH METHODS

The research methods for this study focused on load application to the bridge superstructure and documentation and analysis of the bridge response by a combination of visual observations and electronic instrumentation. A discussion of the general research methods including test procedures and loading, condition assessment, instrumentation, and data analysis follows.

Test Procedures and Loading

The general test procedures for this study called for the application of a known loading on the bridge and measurement of the bridge response. To apply a known load, a special test train consisting of one locomotive and three hopper cars was provided by the SP (Figure 2). Before the testing was performed, the geometry of the test train was measured and the three hopper cars (not including the locomotive) were weighed on a scale.

In order to assess the dynamic response of the bridge, load tests with the test train were completed at crawl speed [approximately 3.2 km/h (2 mph)] and at velocities of approximately 24, 48, and 64 km/h (15, 30, and 40 mph). These velocities were selected in order to evaluate the change in dynamic bridge response under the

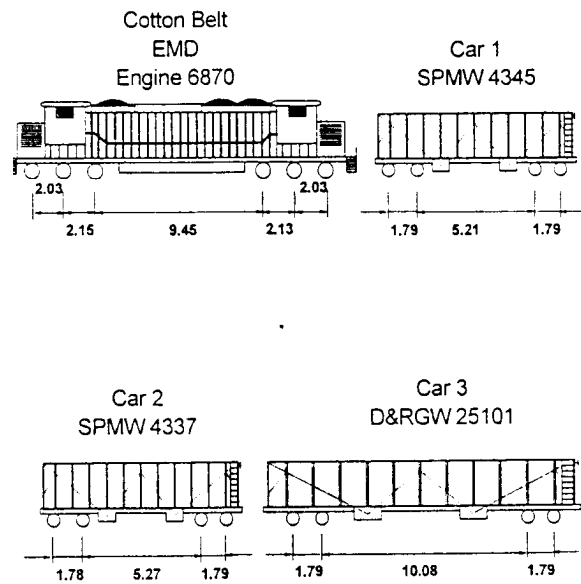
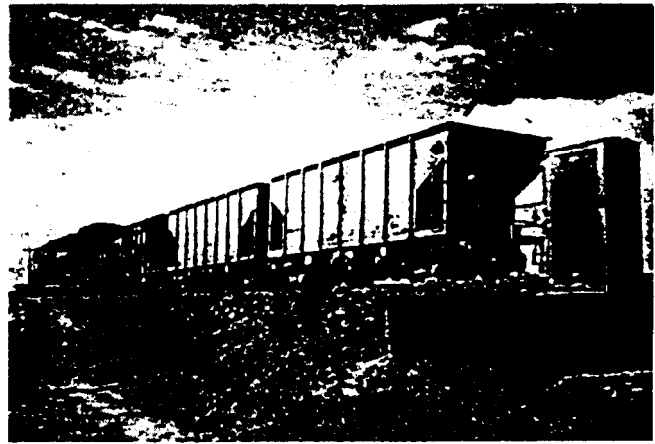


FIGURE 2 Test train.

same loading. Because of speed restrictions on the hopper cars, the test train velocity was limited to 64 km/h (40 mph).

Condition Assessment

Before load testing, a condition assessment was completed on the bridge. The assessment consisted of bridge configuration measurements, intense visual inspections, photographic and video documentation, and moisture content measurements.

Instrumentation

The bridge was instrumented to measure bridge deflections and accelerations as well as wheel forces on the rails. Three types of instrumentation were employed: displacement transducers, accelerometers, and rail load circuits (2,3). Data from the instrumentation were collected, processed, and stored using an electronic data acquisition

system (2). Figure 3 shows the layout of the instrumentation on the two test spans.

Data Analysis

After field data were collected, they were stored on magnetic media and transported to the laboratory, where they were reduced and plotted. Comparative data plots were generated using commercial software so that an evaluation of bridge response could be made. Areas of specific interest included (a) magnitude of rail loading and the comparative loading on each chord at midspan and at the bents, (b) individual stringer deflections at midspan, (c) comparative chord deflections for adjacent spans, (d) chord deflections at bents, and (e) vertical midspan chord accelerations. For vertical deflections at midspan, both absolute and relative values were plotted. Absolute values were measured during testing and include the effects of stringer and chord movement at the supports. Relative deflections were calculated by subtracting the average of the absolute stringer deflections at each support (which assumes that these deflections represent rigid body vertical movement) from the absolute deflections at midspan.

To quantify the dynamic effects of test train speed on load magnitude and midspan deflections, dynamic amplification factors were calculated. For test train wheel loads, the dynamic load factor (DLF) was calculated by the following equation:

$$DLF = \frac{\text{maximum dynamic wheel load}}{\text{maximum static (crawl) wheel load}}$$

The dynamic amplification factor (DAF) for midspan was based on the following equation:

$$DAF = \frac{\text{maximum dynamic deflection}}{\text{maximum static (crawl) deflection}}$$

Analytical Model

An analytical model of the bridge was developed to further study the static load behavior. The model was used to calculate deflections that were compared with field deflections. The following assumptions and design simplifications were used in the development of the model: (a) the packed chords were modeled as single beam elements with the combined properties of the stringers in the chords, (b) support conditions of the chords at the interior bents were considered to be both simply supported and continuous, (c) the wheel loads were assumed to be directly applied to the chords and the rail and ties were ignored, and (d) the loads were stepped across the bridge in small increments to simulate a moving train; however, the wheel load values were constant, and thus no dynamic component was considered.

RESULTS

Condition Assessment

Rail and Ties

Rail condition was good, and transitions from the approach sections to the bridge were smooth with no major vertical elevation differences. The bridge ties were in fair condition with moderate damage noted in most cases. It appears that the ties had been replaced during the earlier rehabilitation of the glulam north chord.

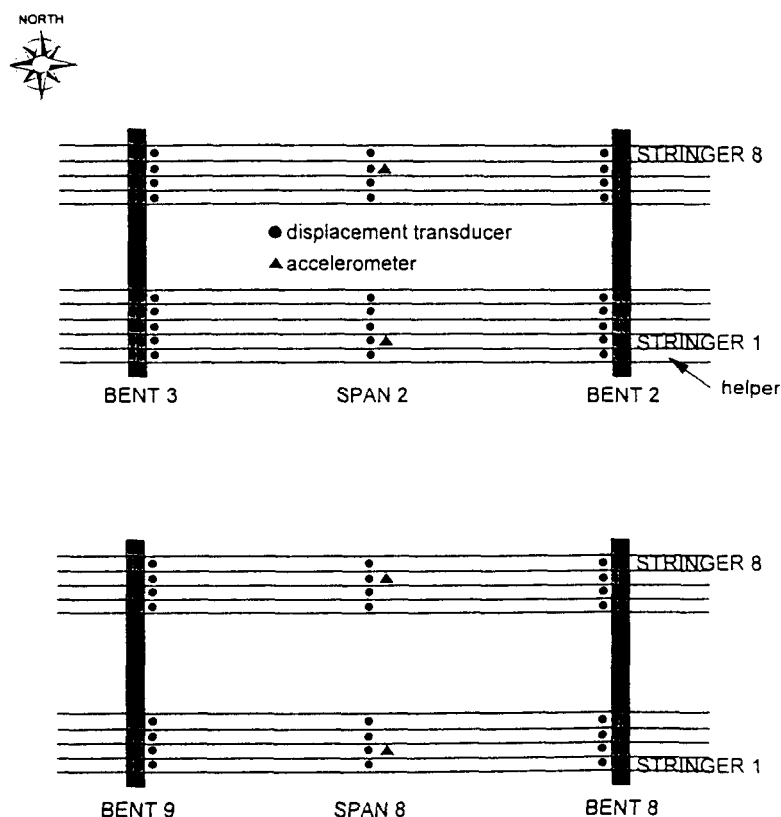


FIGURE 3 Chord instrumentation layout of D'Hanis bridge.

In general, the bearing between the ties and the top of the chords appeared to be uniform as viewed from the edge stringers. However, in Spans 1 and 2 of the south chord there were several locations where the tie bearing was not uniform and gaps were noted between the ties and the helper stringer (this was particularly true near Bents 1 and 3). These gaps occurred at locations where the helper stringer was chiseled, most likely during previous use before installation in the D'Hanis bridge. Plywood shims had been added at several locations in an attempt to create a more uniform bearing surface between the ties and the stringer.

Chords

The south chords (solid sawn stringers) of test Spans 1, 2, and 8 were in adequate condition given the service life of the stringers. The most obvious signs of distress were longitudinal splits in the stringers near the supports and near the midspan, which likely resulted from horizontal shear failures near the stringer neutral axis. The splitting was most pronounced in the vicinity of the bearings for both the simple-span and continuous-span supports. The length of the splits ranged from 0.3 m (1 ft) to more than approximately 1 m (several feet) in members continuous over two spans. Several members were split in multiple locations over the stringer depth. Because of the packed chord configurations, observations were limited to the outside stringers; however, it is suspected that the interior stringer condition was similar to that of the outside stringers.

The helper stringer in Spans 1 and 2 appeared to be in worse condition than the helper stringer in Spans 3 and 4. This stringer had significant splitting at the ends (Bents 1 and 3) and near the midspan of Span 2. As was previously discussed, this stringer also had chiseled regions at three locations. The other helper stringer (over Spans 3 and 4) was in similar condition to the existing stringers noted above in this chord, and some noticeable splitting was evident near the bearings.

Although the newer glulam north chord was in better condition than the solid sawn south chord, noticeable checks were also developing in the region of several of the bearings. The length of the checks was approximately 0.3 m (1 ft) to approximately 1 m (several ft), and they were noted at both simple-span and continuous-bearing locations.

Stringer moisture content measurements indicated an average moisture content of 9.5 to 11.0 percent.

Chord Bearings

South chord bearing was on lumber shims that were placed to compensate for elevation differences with the north chord caused by a shallower stringer depth. Steel plates of varying thicknesses were also present at the bearings to compensate for minor elevation differences between the chords and the spacers. Chord attachment to the substructure was with steel angles that were bolted to the chords and to the shims and cap. The north chord bridge bearings consisted of steel plates of varying thicknesses.

In general, the condition of the bearings on both chords was poor. Under tram loading there was some apparent differential movement between the stringers, and few of the chords appeared to be uniformly supported at the bents. The nonuniform bearing was primarily a problem because of the gaps in the multiple bearing plates that were used. As the loads passed, differential compression was evi-

dent in the shim and bearing plates and the cap beam and the chord bearing plates.

Substructure

The condition of the timber piles and the pile cap was generally acceptable. There was some evidence of pile decay at the ground line in several locations. Checking of the timber was apparent but did not appear to be excessive or to have an adverse effect on the substructure performance.

Load Test Results

Rail Loading

A comparison of the vertical load circuit scale weights with the calculated weights for the four test train cars at crawl velocity at the midspan of Spans 2 and 8 and at grade on the bridge approach is shown in Figure 4. The dynamic amplification of the measured test train locomotive wheel loads based on a calculated DLF relative to the crawl velocity is shown in Table 1 for velocities of 24, 48, and 64 km/h (15, 30, and 40 mph). The data are for the midspan of Spans 2 and 8 and at grade on the bridge approach. The maximum DLF at the midspan load circuit is 1.17 for a velocity of 48 km/h (30 mph). At the approach load circuit, the maximum DLF is 1.32 for a velocity of 64 km/h (40 mph).

Stringer Midspan Deflection

Relative deflections at the north chord midspan of Span 2 at each of the four test train velocities are shown in Figure 5. As shown, the results for all four velocities are similar. The largest difference in relative deflection is approximately 1 mm (0.03 in.) between Stringers 6 and 8. Similar results were obtained for the north chord of Span 8.

Relative test train deflections at the south chord midspan of Spans 2 and 8 are given in Figures 6 and 7. Figure 6 shows the relative midspan deflection for Span 2 for each of the four test train velocities. As shown, all stringers deflect a different amount, with the largest difference of approximately 3 mm (0.13 in.) occurring between Stringers 1 and 3 at crawl velocity. As the test train velocity increases above crawl, the relative deflection difference between stringers decreases slightly. For Span 8, Figure 7 shows that the relative stringer deflection trends are similar for all four test vehicle velocities. The maximum relative deflection difference is approximately 4 mm (0.14 in.) between Stringers 3 and 4.

Stringer Dynamic Amplification

The dynamic amplification of the test train absolute and relative midspan deflections was computed using a DAF for 24, 48, and 64 km/h (15, 30, and 40 mph) (see Table 2). From the Span 2 absolute deflections for the north chord, the DAF decreases with increasing train velocity. For Span 8 the maximum DAF of 1.09 was at 48 km/h (30 mph). The maximum DAF for Span 2 was 1.16 at a velocity of 64 km/h (40 mph). The same trends were noted for the relative deflections, with a maximum DAF for Span 2 of 1.22 and for Span 8 of 1.14.

Train Vehicle	Scale Weight (kN)	Span 8 Load Circuit (kN)	Span 2 Load Circuit (kN)	Approach Load Circuit (kN)	Difference Relative to Scale Weights		
					Span 8 (%)	Span 2 (%)	Approach (%)
Locomotive	¹	1828	1801	1810	-	-	-
SPMW 4337	1130	1154	1138	1131	2.1	0.7	0.1
DRGW 25101	1166	1156	1156	1133	0.8	0.9	2.8
SPMW 4345	1146	1186	1169	1152	3.5	2.0	0.5

¹Scale weight data for the locomotive was not available and is based on estimated values

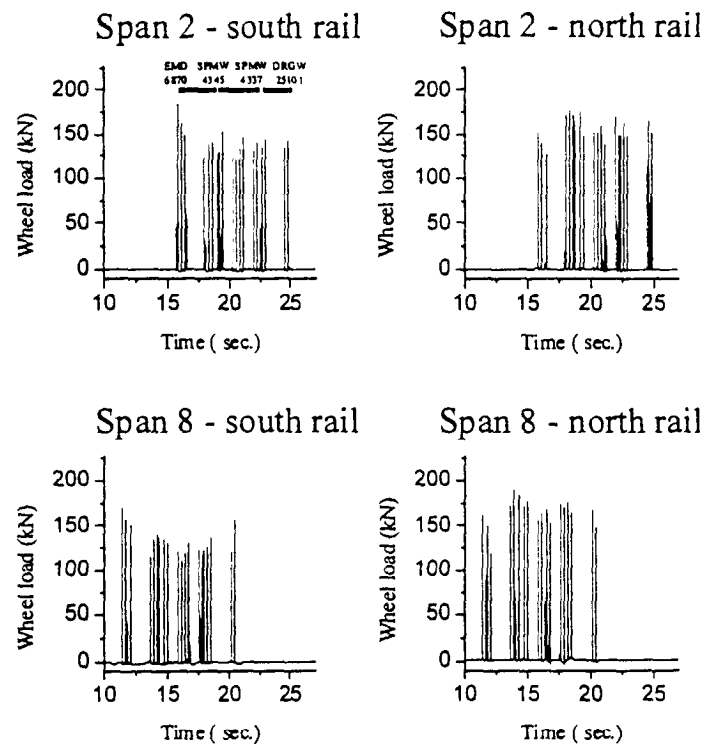


FIGURE 4 Measured D'Hanis bridge test train wheel loads from vertical load circuits at crawl velocity.

TABLE 1 D'Hanis Bridge Dynamic Load Factors Based on Maximum Test Train Locomotive Dynamic Wheel Loads Measured at Load Circuits

Velocity (km/h)	Maximum Wheel Load in Span 8 (kN)	Maximum Wheel Load in Span 2 (kN)	Maximum Wheel Load on Approach (kN)	Span 8 DLF	Span 2 DLF	Approach DLF
crawl	182	178	178	-	-	-
24	185	181	177	1.02	1.02	1.00
48	185	208	190	1.01	1.17	1.06
64	182	181	236	1.00	1.02	1.32

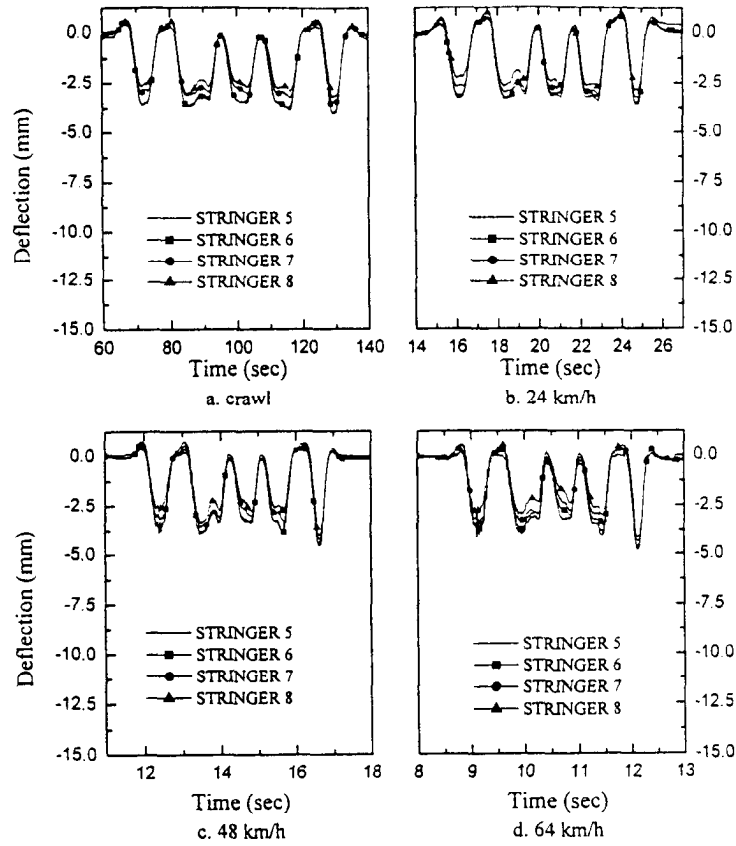


FIGURE 5 D'Hanis bridge Span 2 north chord relative midspan stringer deflections due to test train loading.

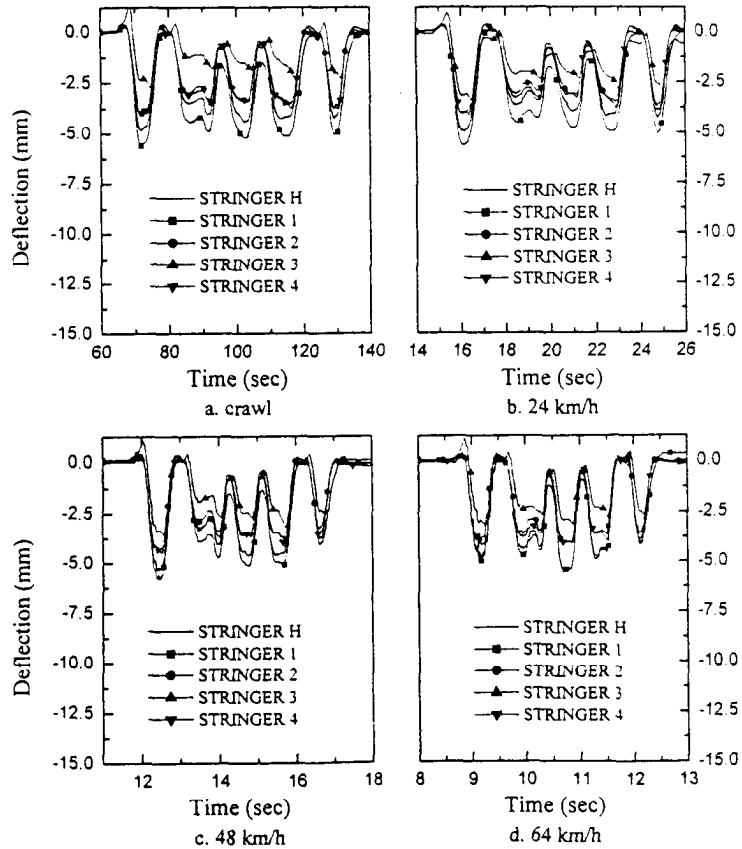


FIGURE 6 D'Hanis bridge Span 2 south chord relative midspan stringer deflections due to test train loading.

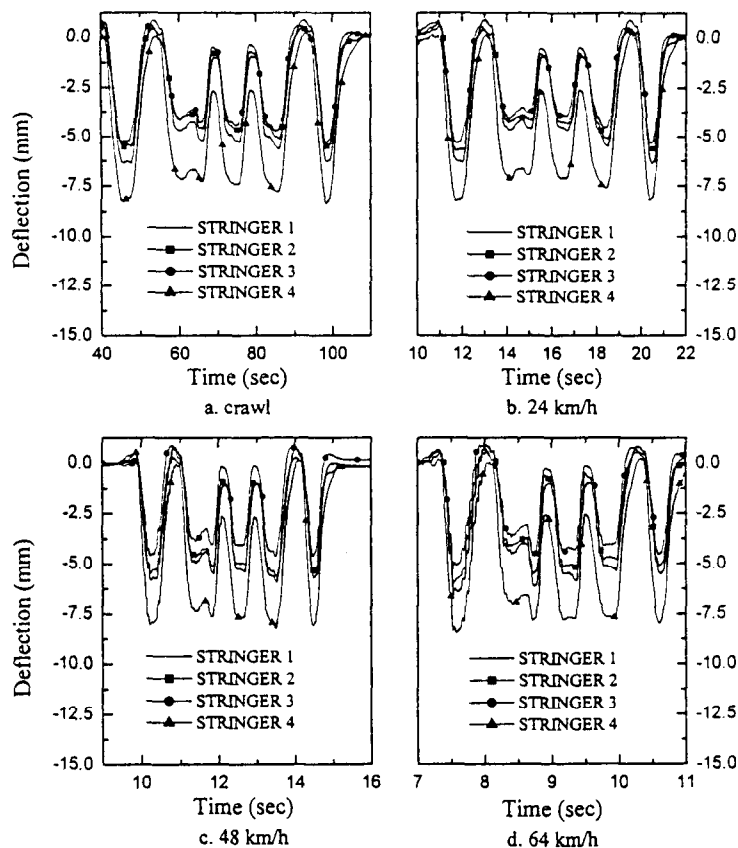


FIGURE 7 D'Hanis bridge Span 8 south chord relative midspan stringer deflections due to test train loading.

For the south chord, the trend for both absolute and relative deflections in both Spans 2 and 8 was decreasing DAF for increasing train velocity. The maximum DAFs for absolute and relative deflections for Span 2 were 1.21 and 1.06, respectively. For Span 8, the maximum DAFs were 1.03 and 1.07 for absolute and relative deflections, respectively.

Chord Midspan Lateral Distribution

Using the Span 2 and Span 8 absolute deflection data, a general assessment of load sharing between the north and south chords can be made based on the assumption that the maximum stringer deflection in both the north (glulam) and south (solid sawn) chords repre-

sents the actual vertical deflection of the tie and rail. Figure 8 shows a comparison of the Span 8 north and south chord absolute midspan deflections for crawl and 64-km/h (40-mph) test tram velocities. The results shown are similar, as they were for the other two test tram velocities not shown. As shown in Figure 8, the south chord deflects considerably more than the north chord, the difference being approximately 6 mm (0.25 in.). The test vehicle loads shown in Figure 4 for crawl velocity indicate that the measured wheel loads are actually higher at the north chord rail.

Figure 9 shows a comparison of Span 2 north and south chord absolute midspan deflections for crawl and 64-km/h (40-mph) test tram velocities. It should be recalled that in Span 2, the south chord has one more stringer (a helper) than the north chord and that the north chord is glulam timber and the south chord is solid sawn

TABLE 2 D'Hanis Bridge Maximum Midspan Stringer Dynamic Amplification Factors

Test Train Velocity (km/h)	Absolute DAF				Relative DAF			
	Span 2		Span 8		Span 2		Span 8	
	North Chord	South Chord	North Chord	South Chord	North Chord	South Chord	North Chord	South Chord
24	1.00	1.02	1.05	1.01	1.00	1.05	1.00	1.00
48	1.11	1.21	1.09	1.03	1.15	1.05	1.07	1.01
64	1.16	1.15	1.04	1.03	1.22	1.06	1.14	1.07

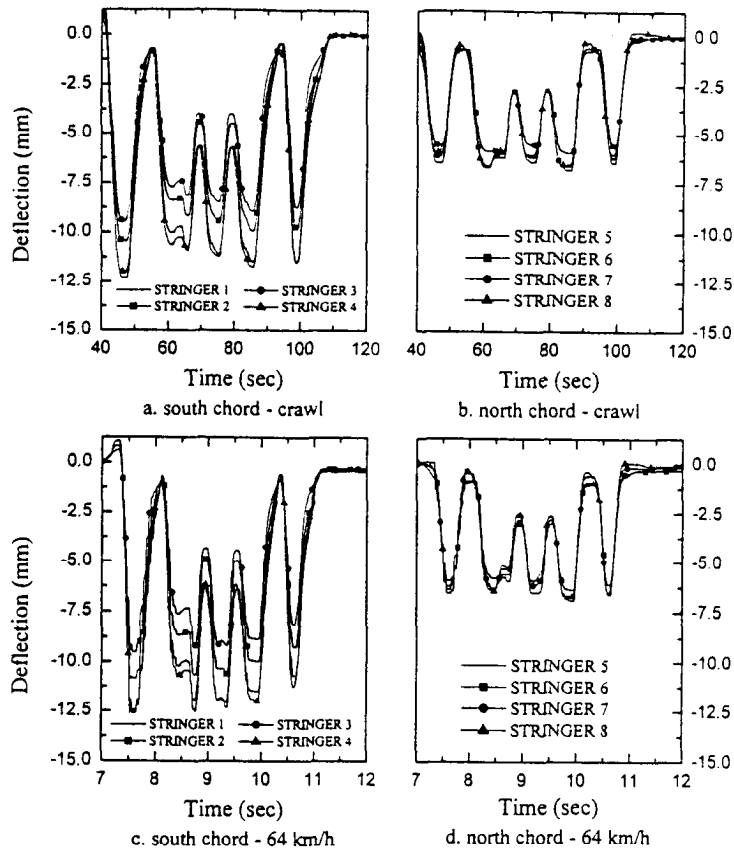


FIGURE 8 Comparison of D'Hanis bridge Span 8 test train absolute midspan deflections for north and south chords.

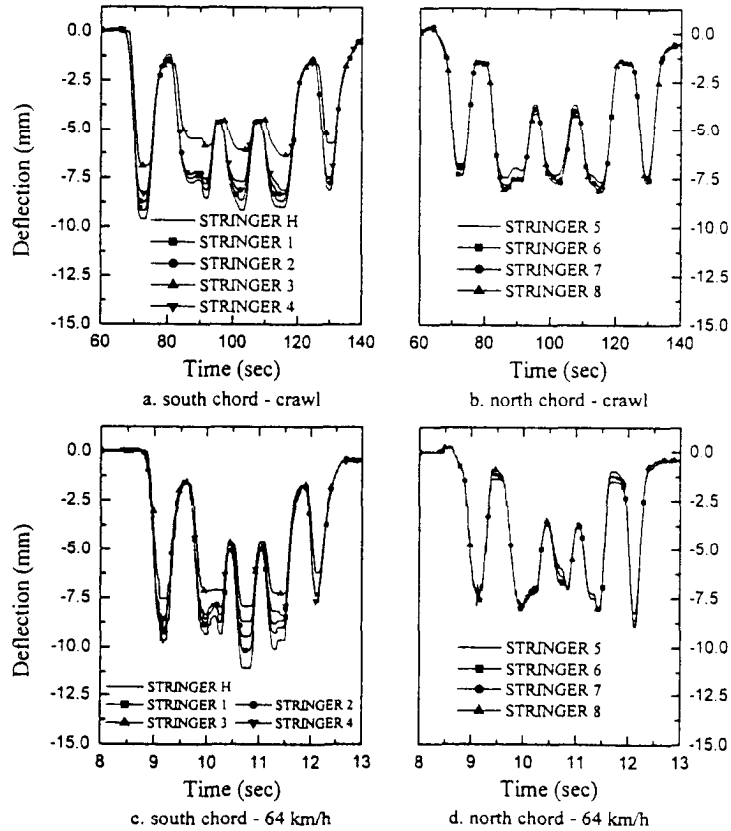


FIGURE 9 Comparison of D'Hanis bridge Span 2 north and south chord absolute midspan stringer deflections due to test train loading.

timber. The results shown are similar, as they were for the other two test train velocities not shown. In general, the south chord (with the helper stringer) deflects a little more than the north glulam chord. At crawl velocity, as noted in Figure 4, the test train loads are similar on both chords. Note also that the helper stringer has the largest deflection in the south chord for both velocities (this behavior was also noted for the other velocities not shown).

Chord Behavior at Bents

Absolute deflections of the chord stringers at the face of Bents 2 and 3 for the north chord are shown in Figure 10 for crawl and 64-km/h (40-mph) velocities. At both locations, there are significant absolute deflections as well as relative deflections between individual stringers in a chord. The data are almost identical for both crawl and 64-km/h (40-mph) velocities. The results at other bents and for the south chord were similar.

Analytical Results

Figure 11 shows the north chord deflections at midspan in Span 8 based on the analytical model. Data are shown for the field results of Stringer 5 of the four stringers in the north chord and for simple and continuous analytical support conditions at the interior bents. As shown, Stringer 5 field data fall between the data for the two sup-

port conditions, although correlation is closer with the simple support data. The correlation of the field and analytical results is good for the given support conditions. Future work will include additional modeling to evaluate the bridge design performance.

SUMMARY AND CONCLUDING REMARKS

Two spans of an existing open-deck timber railroad bridge were tested after superstructure rehabilitation to determine dynamic characteristics and the comparative performance of four- and five-stringer sawn lumber chords and four-stringer glulam chords. On the basis of the results from this testing, the following conclusions are drawn:

- The 60-year service life provided by the solid sawn bridge is evidence of the durability of timber for use in railroad bridges.
- The solid sawn stringers were in moderately good condition, with significant horizontal splitting over the bents in many locations. The condition of the glulam stringers was generally good except for horizontal checking over bents.
- The maximum measured dynamic load factor was 1.32 and occurred at a test train velocity of 64 km/h (40 mph).
- Individual stringers in the sawn lumber chords are not acting as a unit as designed. This is true for both spans that were instrumented, including the span with the helper stringer. Incomplete and uneven chord bearing on the bents and uneven tie bearing on the

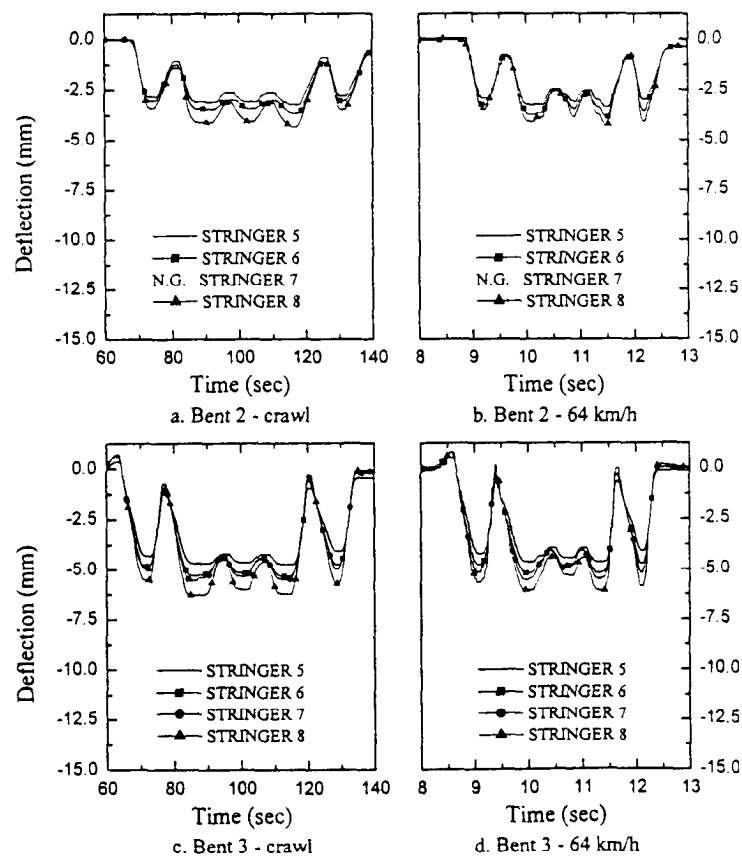


FIGURE 10 D'Hanis bridge absolute stringer deflections at face of bents in north chord due to test train loading: Bents 2 and 3.

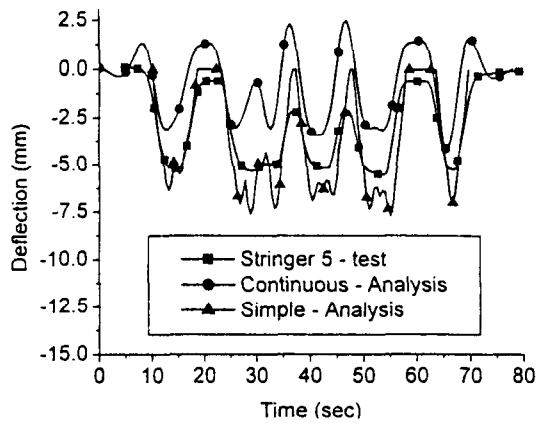


FIGURE 11 Comparison between D'Hanis field test and analytical results.

chords contribute significantly to this observed behavior. The data also showed that the individual stringers in the glulam chords are deflecting approximately equal amounts and are acting together as a unit to resist the applied loads.

- The maximum measured dynamic amplification factor was 1.22 and occurred at a test train velocity of 64 km/h (40 mph).
- Significant absolute stringer deflections were measured at the face of all bents, although some were much larger than others. The worst cases occurred on the sawn lumber chord.
- Although the helper stringer contributed to an increased load capacity in the sawn lumber chords, the distribution of loads within the individual stringers in the chord was not uniform.

Overall, this study shows that glulam stringers provide performance that equals or exceeds that of traditional sawn lumber. It should be noted, however, that the condition and performance of both glulam and the sawn lumber spans would have been signifi-

cantly improved if the rehabilitation had included substructure components where bearing between the pile caps and the stringers was irregular and uneven. The uneven bearing was a contributing factor in the unequal load distribution to stringers and the failure of sawn lumber stringers to act together as a unit. It should also be noted that the bridge tested in this study was typical of designs that have been historically used for railroad bridges for over 60 years. To fully develop the potential of glulam in railroad applications, new designs that take advantage of the improved strength, stiffness, and size of glulam members should be developed.

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