Evaluation of a timber column bent substructure after more than 60 years in-service.

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Keywords – timber, bridge, pier, bent, condition, analysis, performance.

ABSTRACT

This paper describes both the field evaluation and laboratory testing of two timber-column-bent bridge substructures. These substructures served as intermediate pier supports for the East Deer Park Drive Bridge located in Gaithersburg, Maryland. A field evaluation of the bridge substructure was conducted in September 2008. Nondestructive testing was performed with a moisture meter, a stress-wave timer, and a resistance micro-drilling tool. In addition, several core samples were removed for further laboratory evaluation. Live load testing of the bridge superstructure, which consists of steel girders with a glulam timber deck, was conducted concurrently using a single test vehicle. Based on subsequent load rating analysis performed on the entire bridge structural system, it was determined that the timber substructures would be replaced and were salvaged for further testing. Nondestructive tests were conducted to detect internal decay in each of the timber columns using a stress wave scanning technique. The timber-column bridge bents were then reassembled for laboratory testing. Loadings were applied at the cap beam girder locations and were based upon the test truck position which yielded the maximum support reaction forces. Additional loads are planned based on the full American Association of State Highway Transportation Officials (AASHTO) HS20-44 design load for the bridge. A comparison of the field and laboratory testing results will yield valuable information about the load response of these structural systems. The results from this work will also assist engineers more reliably assign a safe load carrying capacity to this timber bridge structural system.

1. Introduction

The East Deer Park Drive Bridge is located in Montgomery County, in the city of Gaithersburg, within the state of Maryland. The bridge was built in 1945 and crosses over a railroad corridor as shown in Fig. 1. The bridge crossing consists of three simply-supported spans which measure 7.32–10.97–7.32 m (24–36–24 ft) from center-to-center of bearing supports. The bridge superstructure consists of steel girders supporting a panelized timber deck system and bituminous asphalt wearing surface. The timber deck consists of 171.5 mm (6.75 in.) deep glued-laminated timber (glulam) panels that measure 4.22 m (4 ft) in the traffic direction and extend the entire 6.7 m (22 ft) bridge width. A total of 5 steel girders per span are spaced at 1.42 m (56 in.) intervals with W18X50 girder section for the end spans and W18X76 girder section for the middle span. The bridge substructure consists of concrete abutments and two intermediate support piers, which are timber column-bents resting on reinforced-concrete footings. The bridge roadway is 6.10 m (20 ft) wide for a single lane of traffic, which is controlled by traffic signals positioned near each bridge end. In addition, the vertical alignment is not flat with the end spans grades at approximately 12 percent, and the middle span having a dramatic vertical curve. The load capacity of the bridge has been reduced recently based upon inspection and load rating analyses. The posted truck load limit was 35.6 kN (8,000 lb) at the time of this field investigation.

Figure 1 – View of East Deer Park Drive Bridge.

Figure 2 – Timber column-bent support pier.

In 2008, the Montgomery County Transportation Department contacted the USDA Forest Products Laboratory (FPL) for technical assistance in performing field evaluation of the East Deer Park Drive Bridge. In-depth inspections were planned for the concrete abutments and footings, timber-column bents, and the steel girder systems. The county expressed uncertainty about the condition and load carrying capacity of the heavy timber components in the timber-column bent support piers. They had concerns about their internal integrity and whether internal decay had reduced their compressive strength. They also expressed concern about deep checking present in many of the columns, and if they had any strength-reducing effects. Subsequently, FPL entered into a cooperative research agreement with the Montgomery County and an engineering consulting firm to conduct an extensive field evaluation of the East Deer Park Drive Bridge.

The condition of both heavy timber-column bents was the main focus of our in-depth field inspection. All timbers included in the timber bent structure were creosote-treated, Douglas fir, measuring 305 X 305 mm (12 x 12 in.) in cross section. The columns measured approximately 3.66 m (12 ft) in height, while the base (sill) beam and cap beams measured 7.62 m (25 ft) and 6.71 m (22 ft) long, respectively. Connections were made with 19 mm (0.75 in.) diameter steel drift pins at top and bottom of the columns. Diagonal bracing measuring 102 X 254mm (4 X 10 in.) was attached to each column with 19 mm (0.75 in.) through-bolts and malleable iron washers.

Based on the inspection recommendations and a finite element analysis of the overall bridge structure, the county decided to replace the timber column-bent piers entirely in 2009. During their rehabilitation they were able to salvage all of the heavy timber components and ship them to FPL for further laboratory evaluation. This paper will summarize the field evaluations of the timber-column bents and include preliminary laboratory test results for those tests completed to date.

2. Field evaluation

An extensive field evaluation of the East Deer Park Drive Bridge was performed during the period of Sept. 5-6, 2008. A nondestructive assessment was performed on the timber-column bent piers to determine their structural integrity. In addition, live load testing was performed to measure the overall system performance of the bridge.

Figure 3 – Nondestructive assessment included resistance micro-drilling (top) and stress-wave timing (bottom) in the columns and cap beams.

2.1.1. Inspection equipment

The specialized test equipment we utilized for this field investigation included an electrical-resistance type moisture meter, a stress-wave timer unit, and a resistance micro-drilling tool. The moisture meter (Delmhorst, Inc.) was used in conjunction with a hammer-probe and 76mm (3in.) long, insulated probe pins. Any wet zones detected with moisture content readings above 20 percent are susceptible to internal decay activity. The stress wave timer device (Fakopp, Inc.) was used to measure the speed of sound waves traveling through the timbers in the transverse-to-grain orientation. Accelerometers are pre-mounted on hammer spikes for ease of use and a compression sound wave is generated with a small hammer. We also utilized the Resistograph (IML-USA Inc.) micro-drilling tool which generates a true-scale plot of the relative density of the wood using a very small diameter drill bit. Areas with very low or zero density are indicative of internal voids or decay. Additional information (Ross et al, 2005) on these nondestructive evaluation tools and their use for inspecting wood structures is currently available [2].

2.1.2. Nondestructive assessment

The key focus of our structural condition assessment of the heavy timber components in the substructure was (a) locating suspect areas of internal deterioration (b) determining the magnitude of checking in the timber columns, and (c) removing core samples for species identification and decay testing.

Identifying suspect decay areas was performed with a variety of inspection techniques and tools. Visual inspection was used primarily to identify any external signs of distress or damage. Since the glulam deck and asphalt wearing surface were designed to perform as a waterproof shelter over the substructure members, water staining would be indicative of water intrusion. Hammer sounding, a moisture content probe, and a stress wave timer were also used to locate suspected areas. Areas that exhibit external signs of distress or have elevated moisture contents which enables decay activity, are suspected of internal decay and warrant further investigation. Stress wave transmission rates (micro-seconds/member dimension) higher than established thresholds for sound wood are indicative of internal decay pockets or internal checking. The extent of internal deterioration at a given cross-section can easily be further defined using a resistance micro-drilling tool.

Checking was visually inspected and physically mapped out for all heavy timber columns. Opening-width and length of the side checking was measured for all four faces (sides) of the columns. The depth of the checks was estimated by inserting metal feeler gauges at 305mm (1ft) intervals along each check. And in some cases, the resistance micro-drill unit was utilized to verify check depths by successively drilling in a perpendicular orientation.

Core samples were obtained with a specialized coring bit and portable drill. The size of the removed cores was 19 mm (0.75 in.) diameter by 51 mm (2 in.) long. To prevent future deterioration, treated wood plugs were inserted into all holes using waterproof adhesives. Density determination and microscopic examination of the cores were subsequently performed in the laboratory.

Our comprehensive inspection data indicated that column 5 in the East Pier contained a thru-split approximately 610 mm (2 ft) above the (sill) base beam, column 4 of the west pier contained a 610 mm (2 in.) deep decay pocket near its center approximately 1.2 m (4 ft) above the (sill) base beam, and extensive internal deterioration present in both cap beam members. Replacement of both cap members in conjunction with reinforcement techniques for the split column was recommended to the bridge owner.

2.1.3. Load testing

Federal law requires that the condition of USA highway bridges be inspected at two year intervals. The results from these routine inspections are entered into the Federal Highway Administration's National Bridge Inventory database [1] and form the basis for periodic bridge load rating calculations. In some cases, the bridge condition warrants further field investigation including the live load testing which can help to reduce the uncertainties of theoretical analyses.

In accordance with the *AASHTO Manual for Condition Evaluation* (AASHTO, 2008), a diagnostic load test was performed in order to further understand the behavior of the East Deer Park Drive Bridge. Strain and deflection measurements were obtained from the superstructure elements using a data acquisition system (Bridge Diagnostics Inc.). Strain transducers were attached to the top and bottom flanges of the steel girders to characterize the transverse load distribution. String potentiometers were attached to the underside of the girder flanges to measure deflection. Response envelope data was collected as a 3-axle test truck with a gross weight of 205 kN (46,100 lb) crossed the bridge at a crawl speed. Both concentric and eccentric test truck paths were run over several load cases. Fig. 5 illustrates the test truck configuration including axle weights and spacing.

The live load strain response data from the September 2008 load test is shown in table 1. For this concentric load case, the test truck was aligned with the roadway centerline, as it crossed over the bridge at a crawl speed. Maximum tensile strains in the end span steel girders totaled 203 micro-strains ($\mu\varepsilon$), with the both rear axles positioned at the bridge midspan and the front axle off the bridge. The strain distribution percentage for each individual girder was calculated as the girder strain divided by the cumulative total strain. These individual girder strain percentages defined the lateral load distribution characteristics of the superstructure and were later utilized in laboratory testing. Additional details about the load testing are summarized in a field report (Zhou, 2008).

Figure 4 – Load testing completed in Sept. 2008.

A detailed finite-element analysis of the complete bridge structure was performed by the engineering consultant after completion of load testing. Load ratings were recalculated based on the results from the field inspection and load testing. Controlling members in the bridge structure were the timber cap beams in the column-bent piers and the interior steel girders used at each of the end spans.

The bridge owner subsequently decided to completely replace both timber-column bent pier supports, as the

Beam	1	2	3	4	5
Strain ($\mu\varepsilon$)	28.0	38.9	59.2	61.1	15.8
Percentage	13.8	19.2	29.2	30.1	7.8

Table 1 – Maximum steel girder tensile strains obtained during field load testing.

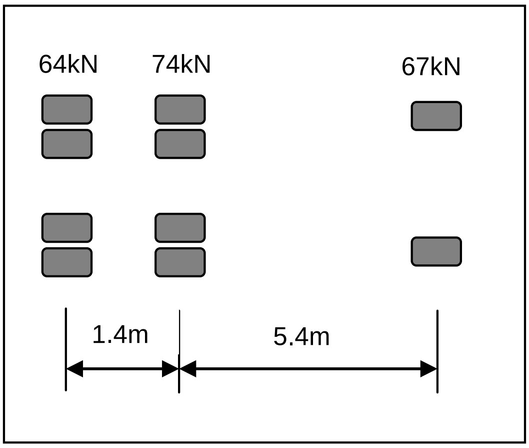

Figure 5 – Truck axle weights and spacings.

best option for extending the bridge service life. All the heavy timber bridge components were salvaged during bridge rehabilitation efforts and then shipped to the FPL for additional evaluation.

3. Laboratory evaluation

Approximately a year after the field assessment and load testing was completed, the salvaged bridge timbers were shipped to the FPL for further study. Prior to reassembly, the timbers (including columns, base "sill" beams, and cap beams) were each individually scanned using a stress-wave timer to characterize internal deterioration. After reassembly, the timber column-bent pier full-scale structures are currently being tested under various service load conditions in the laboratory. Additionally, column compression testing to determine ultimate load capacity is being planned.

3.1.1. Nondestructive scanning

Stress wave timing was performed on the individual timber components using the same nondestructive equipment but at a higher intensity than in the field. Data was collected along three lines and in top-bottom and side-side orientations at 310 mm (1 ft) intervals along their length. Results from scanning the cap beam from east column-bent pier are provided in Fig. 6. The transmission rate results are color-coded contour maps showing areas of internal deterioration (blue-moderate; red-advanced). It clearly shows a large area of internal deterioration above the interior columns, and a small area near its end. This confirms what was suspected in the field assessment. At the completion of full-scale testing, the cap beams will be sawed open at the decayed zones to visually inspect the level of internal deterioration present.

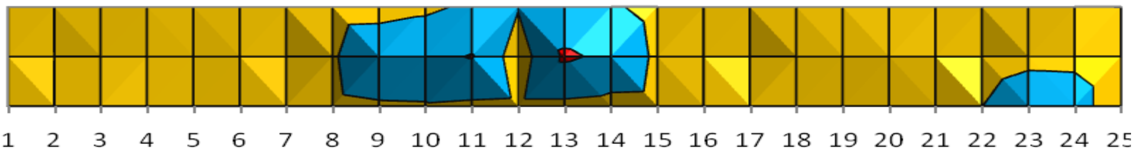

Figure 6 – Contour plot showing internal decayed zones (shown in blue & red) cap beam.

3.1.2. Full-scale structural testing

Structural testing of the reassembled timber-column bent piers involved the design and erection of a load frame and data acquisition system (National Instruments Inc.). The applied loads were calculated based on the test truck (or theoretical design trucks) and its longitudinal position on the bridge for maximum pier support reaction force.

A load frame consisting of steel I-section beams was erected around the column bent pier assembly as shown in Fig. 7. Loads were applied with a hydraulic system (Enerpac, Inc.) consisting of five 267 kN (30 Ton) capacity cylinders suspended from the load beam. Loads were measured with steel, hollow-core load cells custom manufactured at FPL. Load cells were placed on steel bearing plates that were sized to duplicate those used to anchor the steel girders in the bridge. Strains were measured with strain transducers (Bridge Diagnostics Inc.) attached with self-tapping wood screws to opposing column faces at approximately mid-height of each timber column. An additional strain transducer was placed in a vertical orientation (perpendicular-to-grain) on the cap beam directly above column 3. Lateral movements were monitored with string potentiometers attached to the cap beam above each column. All instrumentation was calibrated and checked for accuracy prior to test initiation.

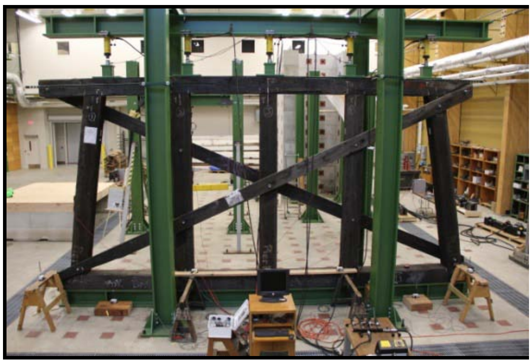

Figure 7 – Column-bent substructure re-assembled for laboratory evaluation.

Applied column compression loads corresponding to the single load test truck case were calculated from a combination of dead load and live loads. The dead load of the superstructure was estimated at approximately 46.3 kN (10.4 Kips) applied at each steel girder bearing location. The live load was based upon the loaded test truck positioned longitudinally to generate the maximum pier support reaction forces, and occurred when the 74 kN (16.6 Kip) forward rear axle was directly over the support. The remaining axles were positioned 1.4 m (4.5 ft) onto the end span and 5.4 m (17.7 ft) onto the middle span. For the truck axle located directly over the pier support, each truck wheel line was distributed to each adjacent column using beam theory. For the truck axles away from the pier support, lateral load distribution was based on the field measured strain distribution as listed in Table 1. The target compression load (including dead load plus live load) used for Test 1 and applied each load (stringer) position are summarized in Table 2. Applied loads were within plus/minus five percent of the target load, except for Column 5 which was slightly outside this range.

Test 1	Column 1		Column 2		Column 3		Column 4		Column 5	
	kN	Kip	kN	Kip	kN	Kip	kN	Kip	kN	Kip
target load	63.61	14.30	96.97	21.80	108.98	24.50	110.76	24.90	56.05	12.60
applied load	60.20	13.53	91.81	20.64	109.63	24.65	116.42	26.17	48.82	10.98

Table 2 – Comparision of target and applied compression loads for test 1.

The maximum strain measured in each timber column is provided in Table 3. The total column strain represents an average value from transducer measurements obtained from opposing sides (north and south) of each member at approximately mid-height. A cumulative total of 220 $\mu\varepsilon$ was measured from all the columns. Columns 2 and 4 exhibited the highest strain amounts near 60 $\mu\varepsilon$. Percentages are derived from taking the average strain response divided by the cumulative total strain.

Test 1	Column 1	Column 2	Column 3	Column 4	Column 5	Total
strain ($\mu\varepsilon$)	-26.3	-63.2	-43.8	-57.9	-33.1	-224.4
percent of total	11.7	28.2	19.5	25.8	14.8	100.0

Table 3 – Maximum strain response of timber columns for test 1.

The theoretical strain levels were computed based on the modulus of elasticity (E), applied load, and cross-sectional area of the columns. Stress wave velocity and density measurements from each column were used to derive the modulus of elasticity values in the parallel-to-grain direction. These computed theoretical strains ranged from 40-60 percent higher than the measured column strains. The large differences between measured and theoretical strains have prompted a review of the strain measurement system for the heavy timber columns. The measured strains observed with strain transducers attached with self-tapping screws may likely be underestimating the actual strains due to interference at connection points. Alternative attachment methods for the strain transducers are currently being examined in order to maximize the accuracy of column strain measurements.

4. Final remarks

Two timber-column bent support piers from the East Deer Park Drive Bridge were recently evaluated in the field, removed from service, and are currently being tested in the laboratory.

Field evaluations, using various nondestructive inspection tools, indicated one column (number 5 in the East Pier) contained a thru-split approximately 610 mm (2 ft) above the base beam, one column (column 4 of the west pier) contains a 51 mm (2 in.) deep decay pocket near its center approximately 1220 mm (4 ft) above the base beam, and extensive internal deterioration present in both cap beam members. Replacement of both cap members in conjunction with reinforcement techniques for the split columns was recommended. The bridge owner, however, decided to replace both timber column-bent piers entirely in 2009 which provided the opportunity for further laboratory evaluations.

Preliminary laboratory testing indicated the column and cap beam measured strains were significantly lower than theoretical strains for test run 1. The accuracy of the strain measurements on large sawn timbers is currently being re-evaluated in the laboratory. Additional load cases based upon the design truck loading (AASHTO HS20-44) and other heavy truck load configurations are being planned.

Future work is also planned to determine the ultimate load capacity of the timber columns in compression. These destructive tests should provide some indications of the effect of deep checks and through-splits on their load capacity.

Acknowledgements

The authors wish to thank the following individuals for their support of this research: Ed Zhou of URS Engineers for providing live load testing data and photographs; Gwo-Ruey Hwang of Montgomery County (Maryland) Transportation Department for supplying the salvaged timber bridge components for laboratory testing; Shan Gao of Northeast Forestry University (China) for assistance with NDE scanning of salvaged bridge timbers; and Marc Joyal and Dave Simpson of the FPL Engineering Mechanics and Remote Sensing Laboratory for calibrating instrumentation and conducting laboratory tests.

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Wacker, J.P. ; Wang, X. ; Rammer, D. R. ; Nelson, W. J. 2011. Evaluation of a timber column bent substructure after more than 60 years in-service. In: CIMAD 11 – 1º Congresso Ibero-LatinoAmericano da Madeira na Construção, 7-9/06/2011, Coimbra, Portugal. [Electronic resource] [S.l. : s.n.], 2011. 1 flash drive. 7 p.