Dynamic Evaluation of Timber Bridges

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Abstract

The dynamic response of both glued-laminated timber (glulam) girder bridges and stress-laminated (stress-lam) deck bridges was determined from field test results using heavily loaded trucks. Deflections were measured for various vehicle speeds and transverse positions at the bridge midspan and were recorded using a high speed data acquisition system. A dynamic amplification factor (DAF) was computed from these data. These field tests are part of a larger research study that will also include analytical research. Only some of the experimental data are described in this paper. The overall objective of the larger study is to determine the dynamic behavior of timber bridges so that reliable design specifications can be developed.

Key Words: Timber, Bridge, Dynamic, Glued laminated, Girder, Stress laminated, Deck

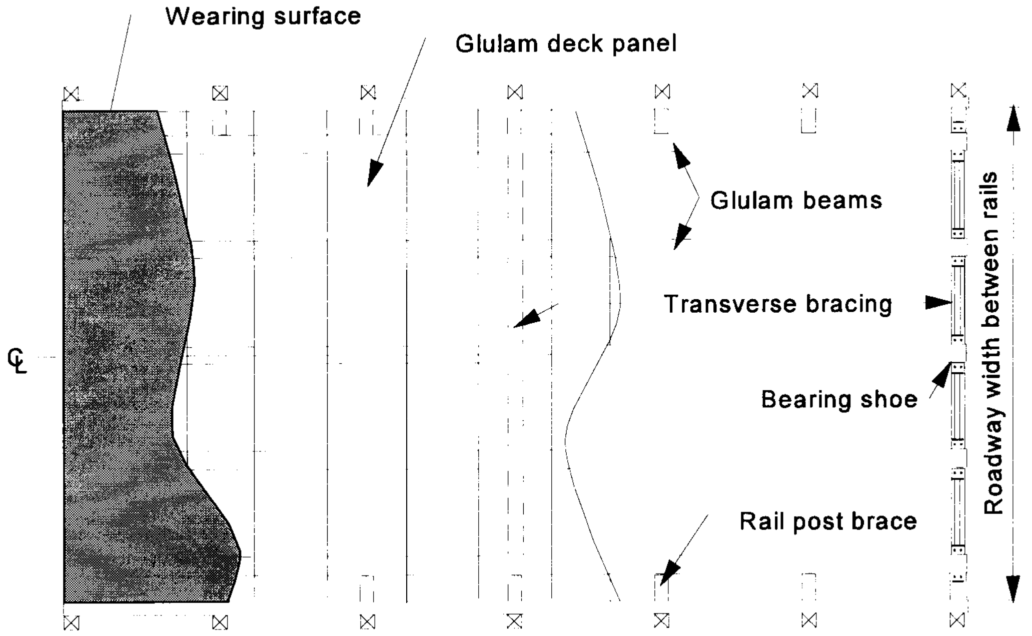
Introduction

Wood has been used as a bridge material in the United States for hundreds of years. Despite the exclusive use of wood bridges during much of the 19th century, the 20th century brought a significant decline in the percentage of wood bridges relative to those of other materials. At the present time, approximately 8% of the bridges listed in the National Bridge Inventory are wood (FHWA 1995). Recently, there has been a renewed interest in wood as a bridge material, and several national programs have been implemented to further develop wood bridge systems. As a result of the Timber Bridge Initiative and the Intermodal Surface Transportation Efficiency Act, passed by Congress in 1988 and 1991, respectively, funding has been made available for timber bridge research (Duwadi and Ritter 1994).

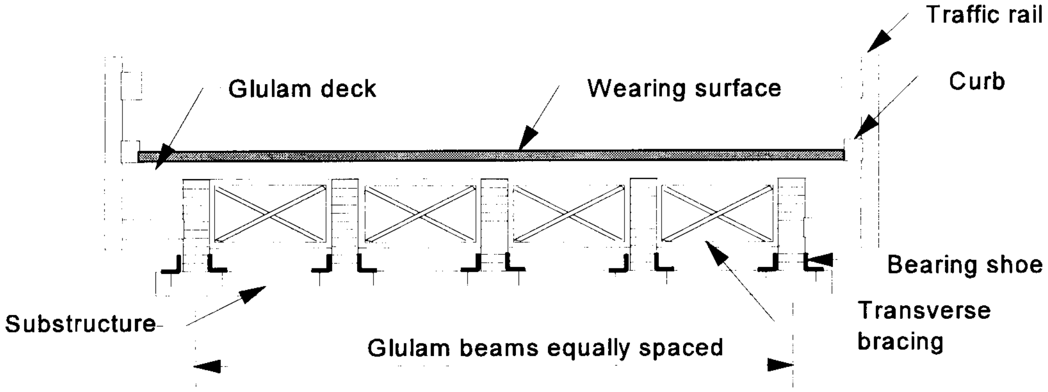
A portion of the timber bridge research is aimed at refining and developing design criteria for wood bridge systems. This project, to investigate the dynamic characteristics of wood bridges, is part of that program and involves a cooperative research study between Iowa State University; the USDA Forest Service, Forest Products Laboratory; and the Federal Highway Administration. The first phase of the project addressed the dynamic performance of stress-laminated timber bridge decks (Ritter et al. 1995). The second phase of the project assessed the dynamic characteristics of glulam timber girder bridges (Wipf et al. 1996). The third phase of the project will assess the dynamic characteristics of longitudinal glulam timber deck bridges.

General Research Program

Field tests for this program were designed to observe bridge deflections and vertical accelerations and test vehicle vertical accelerations under both static and



Cutaway plan



Roadway section

Figure 1 -- Layout of a typical glulam girder bridge

dynamic loading. Vertical deflections were measured for several vehicle velocities for two different bridge entrance conditions: the in situ condition and that due to an artificial bump at the entrance, which was used to simulate a potential rough bridge entrance condition that might occur in the field. Dynamic deflection data were compared to static deflections to quantify a dynamic amplification factor (DAF) for each test.

Description of Bridges

Glulam timber girder bridges typically consist of a series of longitudinal glulam beams that support transverse glulam deck panels (Fig. 1). The girders are available in standard nominal widths, ranging from 10.1 to 40.6 cm (4 to 16 in.) with girder depth limited only by transportation and pressure treating restrictions. Deck panels are usually 12.7 to 17.1 cm (5 to 6 3/4 in.) thick, 1.2 m (4 ft) wide, and are continuous across the bridge width. Lateral support and alignment of the girders are provided by transverse bracing at the bearings and at intermediate locations along the span. Glulam girder bridges are feasible for spans ranging from 6 to 42 m (20 to 140 ft), although most are in the span range of 7 to 24 m (25 to 80 ft).

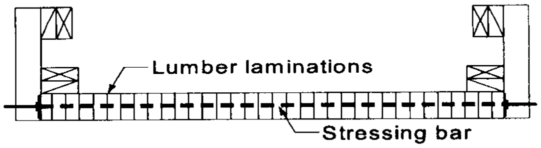


Figure 2 -- Layout of typical stress-lam timber deck bridge

Stress-laminated timber bridge decks (see Fig. 2) consist of a series of wood laminations that are placed edgewise between supports and stressed together with high strength steel bars (Ritter 1990). The bar force, which typically ranges from 111.2 to 355.8 kn (25,000 to 80,000 lb), squeezes the laminations together so that the stressed deck acts as a solid wood plate. The concept of stress laminating was originally developed in Ontario, Canada, in 1976 as a means of rehabilitating existing nail-laminated lumber decks that delaminated as a result of cyclic loading and wood moisture content variations (Taylor and Csagoly 1979; Taylor et al. 1983). In the 1980s, the concept was adapted for the construction of new bridges and numerous structures were successfully built or rehabilitated in Ontario using the stress-laminating concept. The first stress-laminated bridges in the United States were built in the late 1980s. Since that time several hundred stress-laminated timber bridges have been constructed, primarily on low volume roads.

The bridges tested in this program involve both glulam girder bridges and stress-laminated deck bridges. Table 1 summarizes information about the 12 bridges presented in this paper.

Table 1 -- Bridge description (1 m = 3.28 ft)

Bridge	Type	Span C-C bearings (m)	Width (m)	Bridge and bridge approach pavement type
Trout	Stress-lam deck	13.99	7.83	Asphalt
Little Salmon	Stress-lam deck	7.62	4.72	Unpaved
Lampeter	Stress-lam deck	6.77	9.02	Asphalt
Capitola	Stress-lam deck	7.13	10.85	Asphalt
Olean	Stress-lam deck	7.68	9.36	Asphalt
Schuylkill	Stress-lam deck	10.64	7.92	Chip Seal
Wadesboro	Stress-lam deck	6.92	11.19	Asphalt
Teal	Stress-lam deck	9.60	7.25	Asphalt
Chambers County	Glulam girder	16.18	8.23	Asphalt
Mud Creek	Glulam girder	12.74	6.80	Chip Seal
Wittson - Span 1	Glulam girder	15.70	4.63	Asphalt
Wittson - Span 3	Glulam girder	31.09	4.63	Asphalt

Instrumentation

The dynamic response of each bridge was recorded during the passage of the three axle trucks traveling at constant velocity. Deflections were measured at midspan and quarterspan of each bridge span using Celesco potentiometer transducers (DCPT). Accelerometers were also mounted at several locations on the bridge at midspan and quarterspan. Details of the complete instrumentation can be found in Ritter et al. (1995).

Acceleration data were also collected on the vehicle simultaneously with the bridge DCPT data. The accelerometers were mounted on the vehicle frame over the rear axles and on the rear tandem axle.

Test Procedure

The dynamic load behavior of the bridge was evaluated for several vehicle velocities for in situ and artificially rough approach conditions at the bridge entrance. For the two-lane bridges, two different transverse vehicle positions were used: (1) eccentric, with the left wheel line (driver side) 0.61 m (2 ft) to the right of centerline and (2) concentric, with the axle of the truck centered on the bridge (i.e., straddling the centerline). For each bridge, the test vehicles used were tandem axle dump trucks with steel leaf rear suspensions. The range of gross vehicle weights was 244.6 to 275.8 kn (55 to 62 kips).

To obtain a basis by which the dynamic load effects could be compared, crawl tests were performed for each loading position. During these crawl tests, the vehicle velocity was approximately 8 kmh (5 mph). Deflections at higher velocities were then obtained with velocities ranging from 16 kmh (10 mph) to safe upper limit speeds based on bridge alignments.

Data Processing

A plot of bridge deflection versus vehicle position along the bridge length (using the vehicle front axle as a reference) was made for each DCPT location at midspan. A typical example of the dynamic versus crawl deflections is given in Figure 3. The maximum deflection obtained for crawl speed is δ_{stat}. The maximum dynamic deflection is δ_{dyn}.

A dynamic amplification factor (DAF) was computed for each bridge. Each DCPT location was scanned to find the maximum absolute crawl deflection, and this data point was then used as the reference point for calculation of the DAF. As per recommendations by Bahkt and Pinjarkar (1989), this approach yields the most useful design information. It should be noted that the data point with the highest crawl deflection typically had the highest dynamic response. The DAF was computed as

$$\mathrm{DAF} = 1 + \left(\frac{\delta_{dyn} - \delta_{stat}}{\delta_{stat}} \right) \qquad (1)$$

where

DAF = dynamic amplification factor,

δ_{dyn} = maximum deflection under the vehicle traveling at designated speed, and

δ_{stat} = maximum deflection under the vehicle traveling at crawl speed.

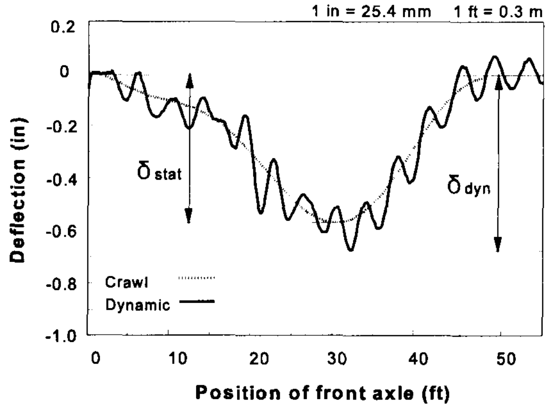


Figure 3 -- Typical bridge dynamic deflection data for bridge tests

Plots of the dynamic amplification factor are shown in Figure 4. In this paper, only the DAF data associated with the in situ conditions are presented (i.e., no artificial rough entrance conditions are given). A summary of the maximum DAF and the experimental fundamental frequency for each bridge is shown in Table 2.

Discussion of Results

From observation of the DAF data presented here and more detailed analysis not presented in this paper, it is clear that DAF data from a finite number of experimental tests provides only limited information about the complex dynamic behavior of the vehicle/bridge system (Ritter et al. 1995, Wipf et al. 1996). The DAF data do provide some limited information about the possible amplification of deflections that can be expected. The highest DAF recorded was 1.60 for the Little Salmon bridge.

Table 2 -- Summary of the experimental field data

Bridge	Maximum DAF	Experimental fundamental frequencies (Hz)
Trout	1.23[a]	3.9
Little Salmon	1.60[b]	8.6
Lampeter	1.10	10.6
Capitola	1.04	9.0
Olean	1.18	10.7
Schuylkill	1.08	5.9
Wadesboro	1.10	9.6
Teal	1.18	7.4
Chambers County	1.12	6.4
Mud Creek	1.38[a]	8.9
Wittson - Span 1	1.15	5.9
Wittson - Span 3	1.09	2.8

[a] in situ conditions included rough entrance.
[b] in situ condition included both rough approach and rough entrance.

However, it should be noted that the approach conditions for this bridge, which was an unpaved gravel road, were extreme, and much worse than the others, and definitely contributed to the high DAF value. Mud Creek bridge had the next highest DAF of 1.38, but it also had a rough in situ bridge entrance condition. The next highest DAF was 1.23 for Trout bridge, which also had a rough in situ bridge entrance condition. In general, the DAF is magnified when the vehicle's initial conditions (i.e., bounce and pitch motion) are magnified by either the approach conditions of the roadway or the entrance conditions at the bridge. For bridges with smooth entrance conditions, the highest DAF was 1.18 for both Olean and Teal stress-lam bridges. For the glulam stringer bridges, the highest DAF for smooth entrance conditions was 1.15 for span 1 of the Wittson bridge.

In summary, although DAF data are not exclusive indicators of dynamic behavior, the experimental values represent actual field performance under the test conditions. As this study progresses and more data are available, further analysis and recommendations will be forthcoming.

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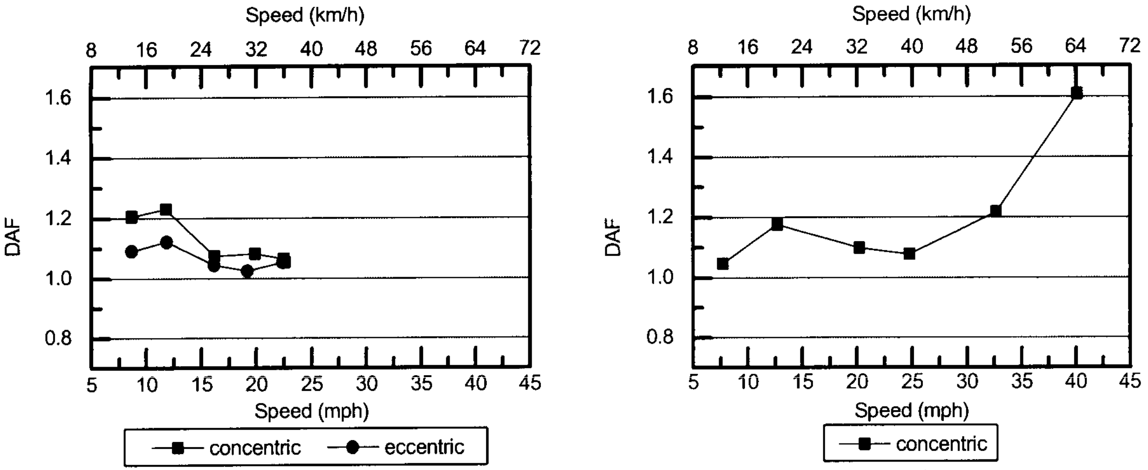
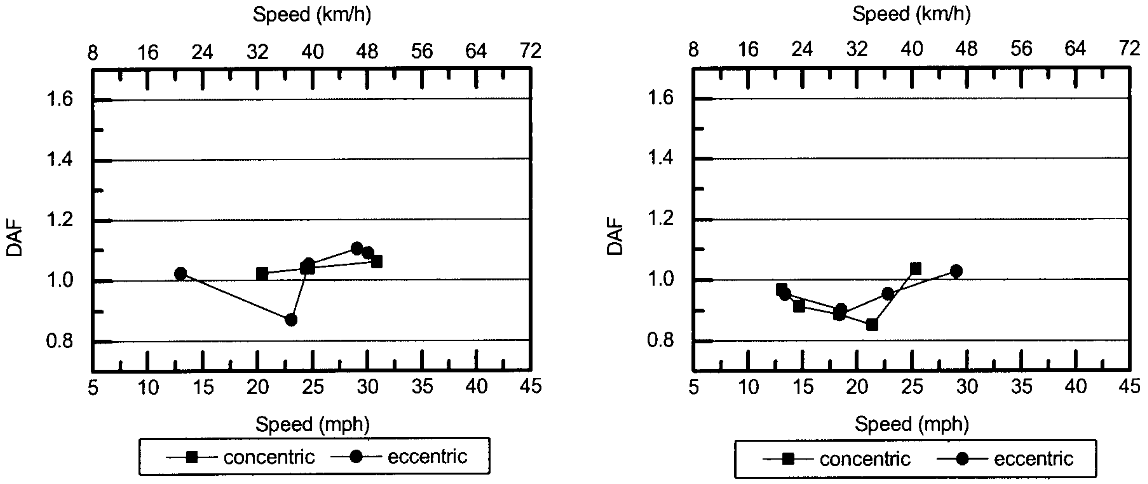


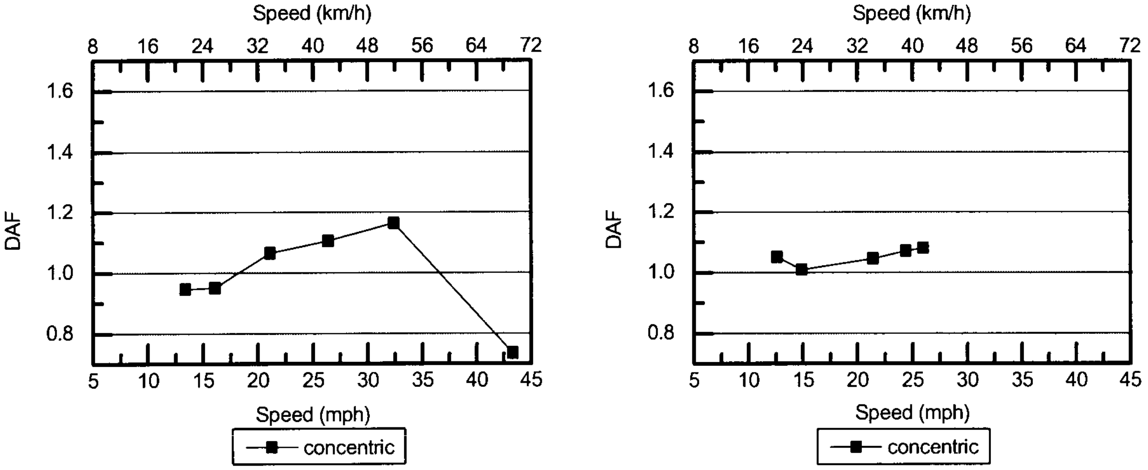
a. Trout

b. Little Salmon



c. Lampeter

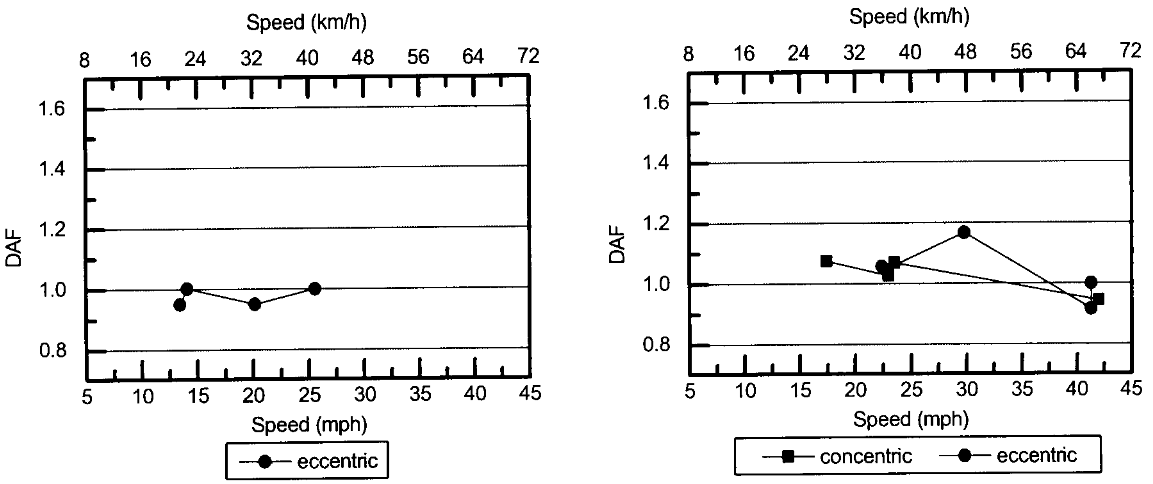
d. Capitola



e. Olean

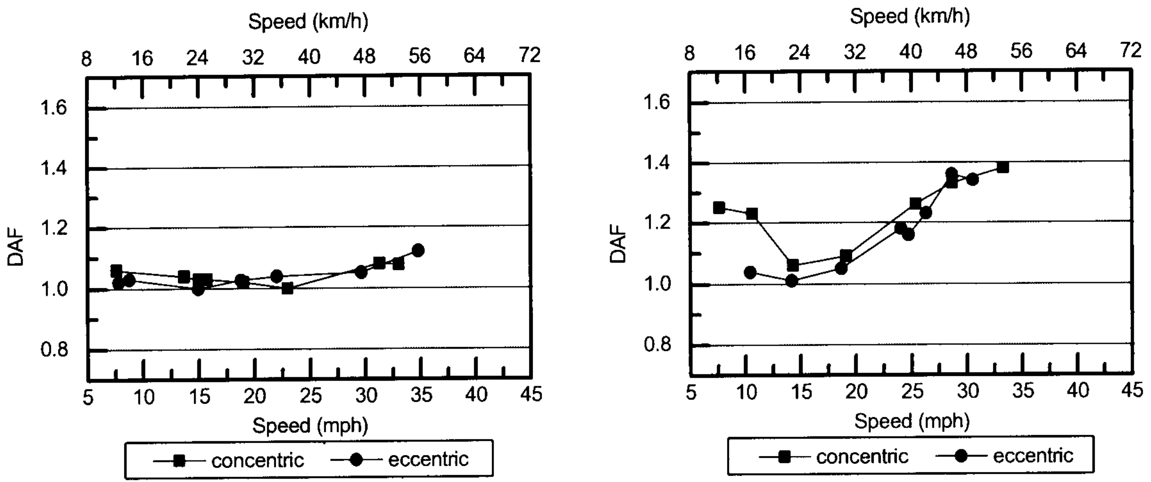
f. Schuylkill

Figure 4 -- Plots of in situ DAF



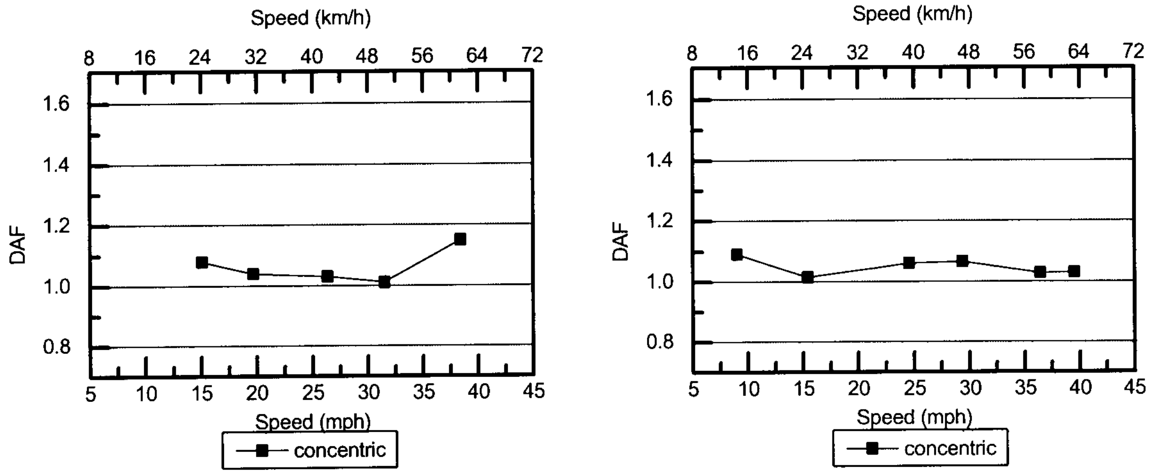
g. Wadesboro

h. Teal



i. Chambers County

j. Mud Creek



k. Wittson - Span 1

l. Wittson - Span 3

Figure 4 -- Plots in situ DAF (continued)

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