FULL-SCALE LABORATORY TESTING OF A

TIMBER TRESTLE RAILROAD BRIDGE CHORD

(Phase 1)

Richard M. Gutkowski, Professor

Research Assistants:

Kimberly R. Doyle
Jeno Balogh

Colorado State University

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Acknowledgements

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Disclaimer

ABSTRACT

A laboratory test program was conducted on full-scale specimens replicating the main elements of a timber trestle railroad bridge chord. Previous field load testing had been done and was complicated by differing site-specific conditions. The purpose of the ongoing laboratory study described herein is to compare field test results with controlled laboratory tests. A three-span bridge chord similar to that of the field bridge was constructed. It was then disassembled and reassembled into a two-span, and then into a one-span, specimen. Load testing was conducted on each specimen to examine the service load behavior of the specimens. Single-point and two-point loadings were used to simulate single and double axles of trains. Comparisons were made between the laboratory results, elementary analytical beam models, and a semi-continuous beam model. Unintended uplift occurred at the supports in the laboratory test set-up. Analytical modeling was modified to reflect this and improve predictions of the response. Additional laboratory tests are recommended on a specimen with support conditions modified to eliminate the uplift.

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EXECUTIVE SUMMARY

Many timber trestle railroad bridges in the United States have been in service for 40-100 years. Due to dramatic increases in train loads over the decades, strengthening is needed in many of them. Imminent code changes for increased load requirements led to the railroad industry's interest in load test programs for timber trestle bridges. A need exists to examine effectiveness of the methods being used by the railroads to accomplish strengthening and repair of its timber trestle bridges.

A laboratory test program was conducted on full-scale specimens replicating the main elements of a timber trestle railroad bridge chord. Previous field load testing had been done and was complicated by differing site-specific conditions. The purpose of the ongoing laboratory study described herein is to compare field tests results with controlled laboratory tests. A three-span bridge chord similar to that of the field bridge was constructed. It was then disassembled and reassembled into a two-span, and then into a one-span, specimen. Load testing was conducted on each specimen to examine the service load behavior of the specimens. Single-Point and two-point loadings were used to simulate single and double axles of trains. Comparisons were made between the laboratory results, elementary analytical beam models, and a semi-continuous beam model. Unintended uplift occurred at the supports in the laboratory test set-up. The analytical modeling was modified to reflect this and improve predictions of the response. Additional laboratory tests are recommended on a specimen with support conditions modified to eliminate the uplift.

Also, field expediency and site conditions can lead to deviations from intended repairs. This project examined some of the effects of some simpler ones and showed modest effects on load sharing.

INTRODUCTION

State of Timber Railroad Bridges

In the United States there are more than 100,000 railroad bridges. About one-third of these use wood or timber construction. Most of the present timber railroad bridges were built in the early- to mid-twentieth century. Development of the railroad system prior to the advent of steel is one factor in many older railroad bridges of timber construction, and much of that tradition continued even later. Byers et al. (1996) report that service life of existing bridges ranges between 35 and 95 years, typically about 70 years. Over time, many have settled, started to decay and require regular maintenance.

Of equal importance, trains in use today are carrying much heavier loads than the bridges were originally designed to withstand. Early railroad bridges were designed for a Cooper E-40 loading in which the maximum single axle load was 40,000 lbs. In the 1960s, a Cooper E-60 loading was adopted in which the maximum single axle load was 60,000 lbs. Presently, it is reported that railroad lines must now actually carry cars with 78,000 lbs per axle (Oommen and Sweeney 1996). Thus, older railroad bridges are being inspected, analyzed and evaluated for adequacy to carry increased loads. However, analytical methods for timber trestle bridges in the American Railroad Engineering Association (AREA) design manual (AREA 1995) are based on dated approaches, i.e prior to the advent of computer-based structural modeling and analysis methods. Designs are based on generalized material properties and standard loadings. Assumptions in regard to distribution of the loads and load paths through the bridge are involved.

Rehabilitation and replacements of railroad bridges is occurring. One complication is the increased difficulty of obtaining the large size timbers used in the existing bridges. Consequently, efforts have been undertaken by the Association of American Railroads (AAR) to better understand field performance, examine effects of field upgrades, and improve structural modeling. One potential outcome is the avoidance of unnecessary upgrades. Experimentally investigating the response of a bridge to actual loads is a very useful tool in understanding field performance. Comparisons of analytical predictions with measured response provides a bench mark by which to refine analytical and design modeling assumptions.

Load tests also provide better understanding of load distribution and load paths. Axle loads are applied to the steel rails by the wheels and then propagate to the ties, chords and substructure system. In situ bridges have imperfect connections between members, gaps between members, non-parallel bearing surfaces etc. With wear, connections loosen, repairs and modifications alter the original configuration and support settlements may occur. Thus, distribution of loads to individual components is affected, for example, multiple plies of a bridge chord do not carry the same proportion of load, so determining the maximum stressed stringer ply is critical to assessing load capacity.

BACKGROUND

The study reported here was undertaken as part of a comprehensive investigation of timber trestle railroad bridges. In 1995, researchers from Colorado State University (CSU) and the AAR jointly conducted extensive field load tests of three timber trestle railroad bridges (Gutkowski, et al., 1998, 1999). Numerous static and moving train loads, as well as ramp and sinusoidal actuator loads, were applied. The main motivation for the testing was to examine the load sharing aspects in the chords. The outcome of the field tests have been published in the literature (Gutkowski, et al. 1997, 1998, 1999) and in a M.S. thesis (Robinson, 1998). Related recent investigations related to timber railroad bridges have been conducted by others and are summarized in an AAR publication (AAR 2002)

As noted, consideration is being given to increase required axle loads in the AREA design manual (AREA 1995) by 30 percent. Thus, a possible need to strengthen old timber trestle bridges is a parallel concern to the AAR. Related to that concern, one of the initially field-tested bridges was strengthened by the addition of one stringer ply in each chord. The bridge was then retested in 1996. Useful findings resulted, but were site-specific (Gutkowski et al. 1999). Consequently, to conform to more idealized (design basis) behavior, full-scale load tests were conducted in the laboratory at CSU. The laboratory specimen was nearly a replication of the field bridge geometry. Some aspects of the field testing and the laboratory study have been reported (Gutkowski, et al. 1999) and those and other findings were published in a M.S. thesis by the second author (Doyle, 2000). This report presents the primary findings of the first phase of the study.

OBJECTIVE AND GOALS

Predominately, existing timber railroad bridges in the U.S. are of the standard open-deck, timber trestle bridge configuration. Consequently, the focus of this study was on that type of bridge. The objective was to conduct laboratory load testing of typical open-deck, timber trestle railroad bridge chords to compare measured response vs. analytical models and field performance. The specimen used essentially replicated an existing bridge that was field load tested before and after strengthening. In addition, effects of various field repair and rehabilitation methods were to be examined in controlled laboratory tests.

The main goals of the investigation were as follows:

1. Construct various full-scale bridge chord laboratory specimens.
2. Conduct load tests on each specimen.
3. Develop a realistic analytical model to predict the responses of the specimens.
4. Compare measured deflections with analytical models.
5. Improve the model based on experimental results.
6. Investigate the effects of adding plies, removing ply tie rods, centering vs. not centering plies under the steel rail, and differential bearing of plies.

These goals were developed in with input from technical staff of the AAR's Transportation Technology Center, Inc.

OPEN-DECK TIMBER TRESTLE RAILROAD BRIDGES

Standard Configuration

Figure 1.1 is a schematic illustration of a standard open-deck, timber trestle railroad bridge. A longitudinal "chord" consisting of a three to five "plies of stringers" (four are shown) is centered below each steel rail of the track. The two rails are supported by timber cross-ties spanning the chords. Interior span chords are supported by pile bents, each comprised of a solid sawn timber cap and round wood piles. End piers are similar to the pile bents and have a timber retaining wall. Plies of the chords are either “packed” tightly together side by side or “spaced” with gaps of about 1”-4” between adjacent plies. In some cases, steel ply tie rods are used to laterally connect the plies at selected locations. Details of the design and construction of these bridges are available in the applicable design code for railroad bridges (AREA 1995).

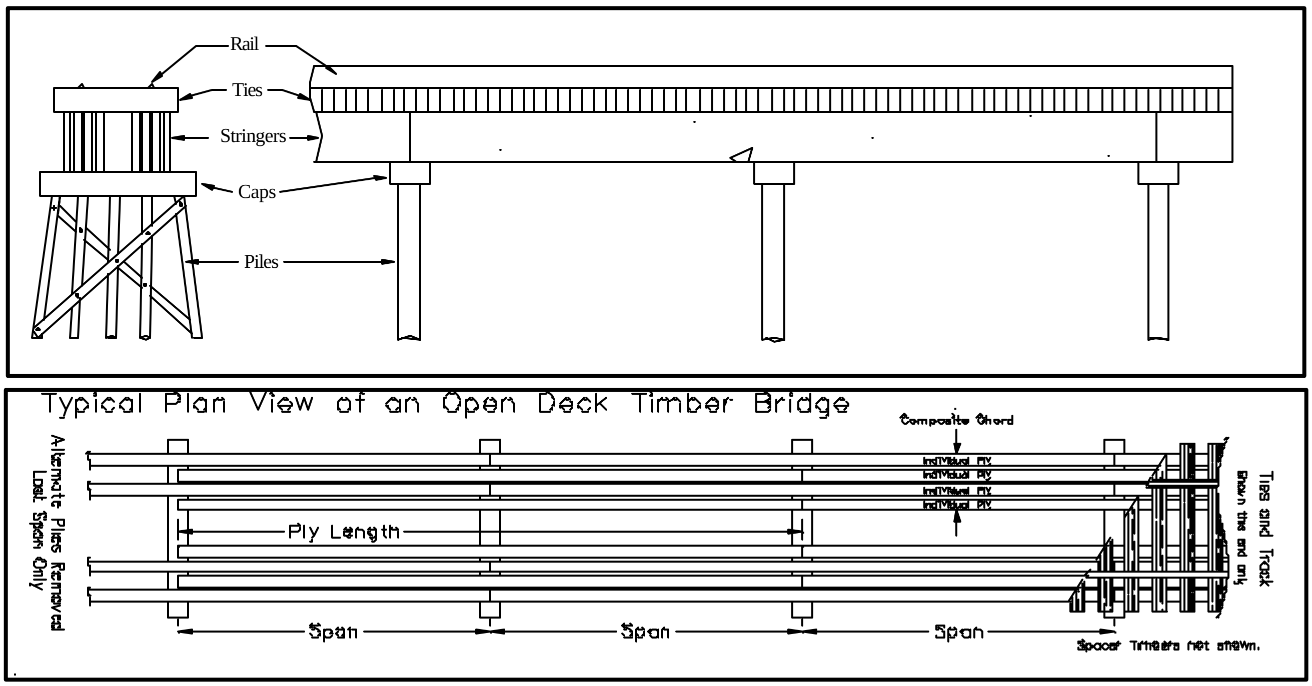


Figure 1.1: General Configuration of an Open-Deck Timber Trestle Railroad Bridge

Bridge No. 101 (from an AAR numbering system) was one of the bridges that was field load tested in 1995. The main laboratory specimen used in the research described herein was a three-span bridge chord configured to be similar to those in Bridge No 101. Bridge No. 101 is depicted in Figure 1.2. It is a right bridge and has three spans of 13', 14' and 13'. The bridge supports a slightly curved rail. Main components are creosote treated Douglas fir timbers. Before the strengthening, each chord had four packed plies of stringers in a staggered, two-span continuous pattern. In the end spans, alternate plies were simply supported. Plies of stringers were tied together laterally by steel tie rods located near the caps. A plank walkway existed on each side. The caps were solid-sawn timbers supported by five round piles. Caps were lag screwed to the piles and the ends of the chord plies were through-bolted to the caps. Piles were X-braced by two wood members bolted to each pile.

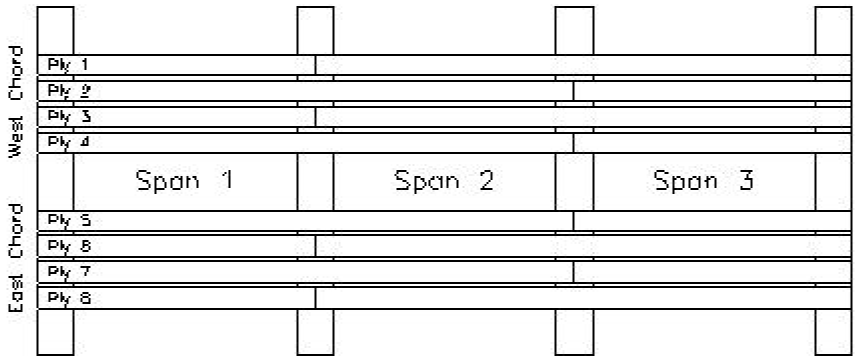


Figure 1.2: Bridge No. 101

In 1996, Bridge 101 was strengthened by the addition of a stringer ply to each chord in each span. Added plies were the same size as the existing plies. They were installed by force fitting them between the existing cross-ties and pier caps. All plies were placed so as to be consistent with the existing lapped pattern of one and two span plies. The plies of the chord were not re-centered under the steel rail. This is preferred by some railroad owners as it makes the retrofit easier, and thus faster and more costly. Lateral ply tie rods were removed and replaced with longer ones.

LABORATORY SPECIMENS

Each rail of a track carries a line of train wheels. In a right bridge each chord carries essentially the same side-by-side load pattern. Due to variability of materials, a slight difference in resistance of each bridge chord is exhibited and the two wheels of any axle can transfer slightly different loads, but a right bridge essentially is symmetrically loaded about its centerline. This symmetry allowed researchers to economize on materials. Specifically, each laboratory test specimen was only one bridge chord plus other associated structural components. The laboratory specimens were made from a common set of main materials provided by the AAR. From the provided material, it was decided to build a three-span bridge chord, a two-span bridge chord, and a one-span bridge chord.

The specimens were built with either four or five plies. To describe the number of spans and plies in a specific chord specimen the key "Chord S/P" is used where S = the number of spans (either 1, 2, or 3 for one span, two spans or three spans, respectively), and P = the number of plies in the bridge chord (either 4 or 5 for four plies or five plies, respectively). For example, the three-span chord with four plies is Chord 3/4.

Specimen 3S

Specimen 3S was a three-span, semi-continuous bridge chord. Each span was 172" center to center of supports (see Figure 1.3). It was comprised of four single-span and four double-span plies laid out in the pattern shown in Figure 1.3. The single span plies measured 8" wide x 16" deep. The double span plies measured 8" wide x 6-3/4" deep. The single-span plies were shimmed at their ends to raise them to the same heights as the double-span plies. Ply tie rods (3/4" diameter threaded steel rods) were placed through 1" diameter holes. Timber railroad ties ("cross-ties") were placed on top of the chord at a spacing of 3"-4" along the specimen. They were 9" wide x 7" deep, with some minor variability in dimensions.

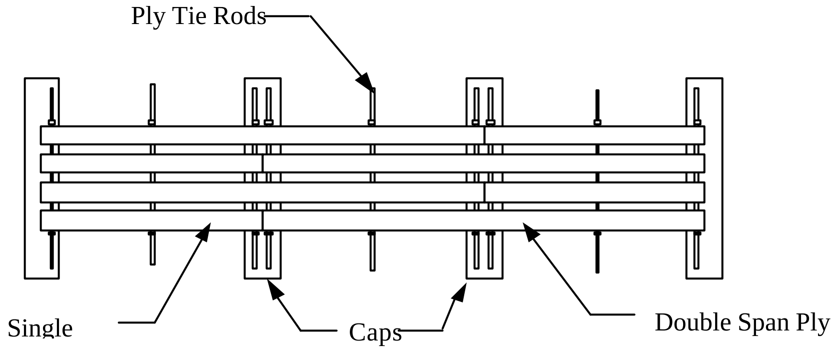


Figure 1.3: 4-Ply Configuration for the 3-Span Bridge Chord

The steel rail was placed on top of the timber cross-ties and centered about the four plies of stringers. It was comprised of either three, two or one segment(s); each 15' in length; as applicable. Thus the rail slightly overhung the ends in each specimen. Because of the need to disassemble the specimen to reuse materials, the rail was not spiked to each cross-tie, but instead tied down at selected locations using a fabricated fixture (see Figure 1.4). These were located at each exterior support, at mid-span of each non-loaded span, and at each interior support.

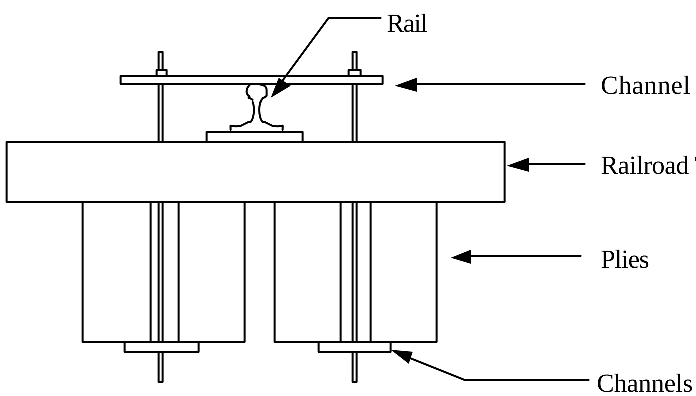


Figure 1.4: Cross-Sectional Schematic of Rail Tie Down Configuration

A solid-sawn timber cap was used at each support. They were 14" wide x 12" deep and 5"-4" long. At the exterior support, the cap was supported on two 12" diameter pole stubs, each vertically placed atop a concrete-filled barrel. Interior supports were the same, except the pole stubs were supported on a large concrete pad. After testing the original specimen, a single-span ply was added to the outside of each span, without adjusting the rail position (i.e. it was left "off-center") and the specimen was load tested again (see Figure 1.5).

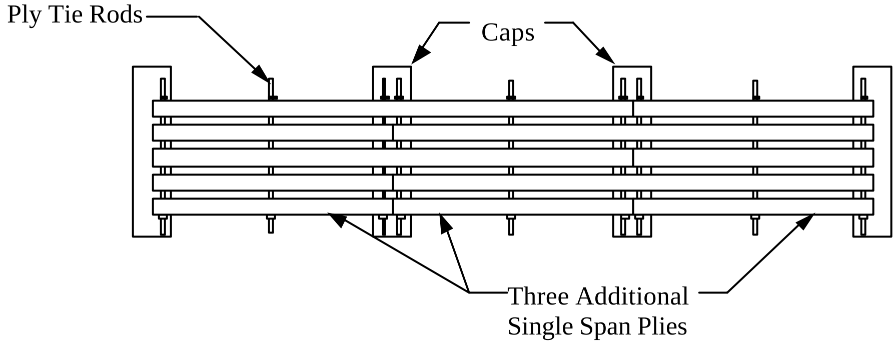


Figure 1.5: 5-Ply Configuration for the 3-Span Bridge Chord

Specimen 2S

Specimen 2S (see Figure 1.6) was similar to Specimen 3S, except it was a two-span (each 172" from center of support to center of. support) chord and the four plies were all two-span continuous. The plies were 8" wide by 16-3/4" deep and spaced 2" apart. One-ply tie rod was placed at each mid span and at mid-depth of the plies. Two-ply tie rods were placed over each exterior support and four-ply tie rods were centered over each interior support. Wood cross-ties were placed similarly to the arrangement in the three-span specimen. The steel rail, tie downs, caps sizes and exterior/interior pole supports were the same as used in Specimen 3S.

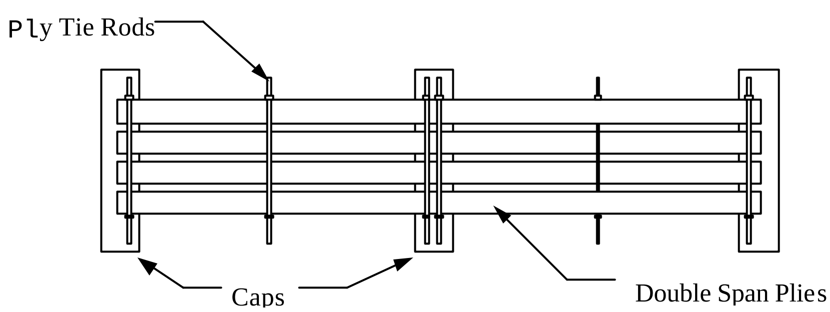


Figure 1.6: 4-Ply Configuration for the 2-Span Bridge Chord

Specimen 1S

Specimen 1S (see Figure 1.7) was a one-span chord consisting of four spaced (2" gap) plies. Each ply was 8" wide x 16" deep. Steel rail attachment and cap sizes and pole stubs were the same as in the prior specimens, and the pole stubs were supported on the concrete pad. Later, a retest was done after adding a fifth ply to the span. Specimens 3S, 2S, and 1S were assembled and tested in that order. Additional load tests were made on each specimen after either intentional misalignment of plies, creating differential end support levels, or removing steel tie rods, etc. to examine the effects of such field irregularities that occur over time.

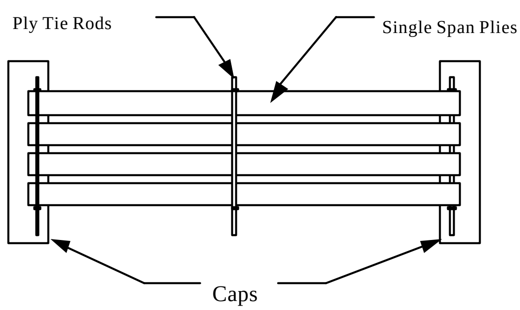


Figure 1.7: 4-Ply Configuration for the 1-Span Bridge Chord

MATERIAL PROPERTIES

The modulus of elasticity (MOE) values of the plies of stringers were determined by actual load tests of each ply. Details of the testing procedure and calculations are contained in Doyle's thesis (Doyle 2000). Loadings used were consistent with those studied in the laboratory testing and analytical models of the bridge chords. This was done to include shear deformation in a consistent manner. MOE values for member and in relation to the placement of the plies in each specimen are shown in Figures 1.8, 1.9 and 1.10.

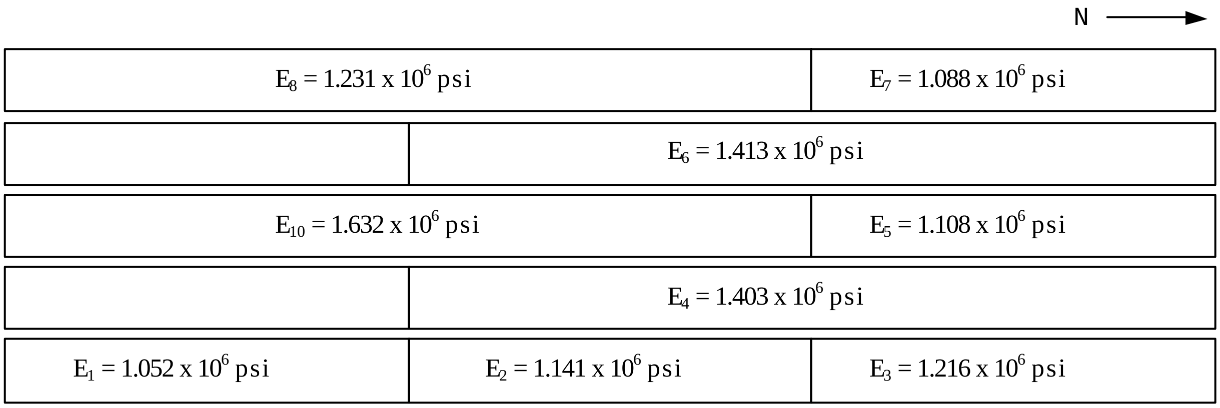


Figure 1.8: Measured Modulus of Elasticity Values for the 3-Span Bridge Chord, with 4- and 5-Plies

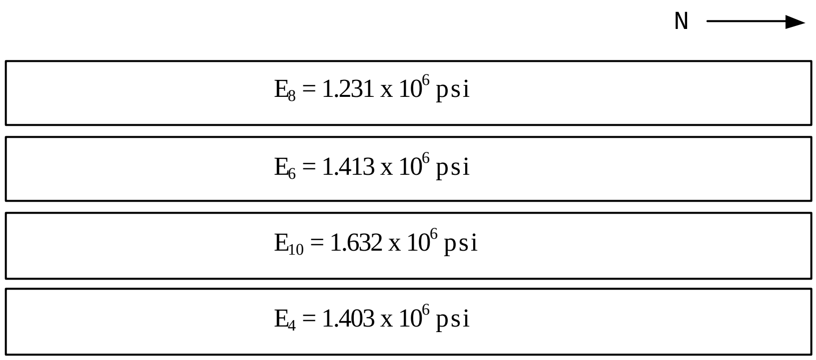


Figure 1.9: Measured Modulus of Elasticity Values for the 2-Span Bridge Chord

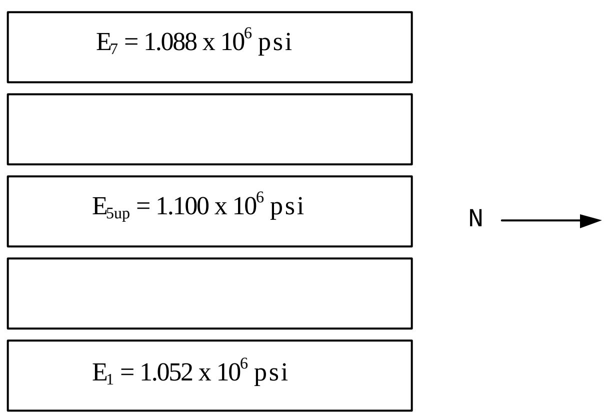


Figure 1.10: Measured Modulus of Elasticity Values for the 1-Span Bridge Chord, with 4- and 5-Plies

LOADING PROCEDURES

Loads were applied using a hydraulic actuator attached to a movable overhead steel frame. Either a single point load or a pair of point loads was then applied (along the centerline of the length of the chord), depending on the particular specimen. The two loadings for Specimen 1S were: 1) a single-point load was applied at mid-span, and 2) a pair of point loads (109" apart), centered longitudinally about mid-span. For specimen 2, a single-point load was applied at mid span of one of the end spans. A pair of point loads (109" apart), centered longitudinally about mid-span, was later applied to the same end span. For Specimen 3 a single-point load was applied at mid-span of the middle span. A pair of point loads (109" apart), centered longitudinally about mid-span, was later applied in the middle span.

Figures 1.11, 1.12 and 1.13 depict the loadings used in each chord specimen. The single-point load is comparable to the ramp loadings applied in the field load testing of Bridge No. 101. The two-point loading simulates a pair of train axles in the front or back of a freight car. For the single-point load, the actuator was incrementally ramped to approximately 70,000 lbs. For the two-point loading, both actuators were simultaneously incrementally ramped to that load level. After the target load was reached, the load was decreased to near zero and then reloaded in the same manner, twice. During a fourth loading cycle, the imposed load was held constant for about 30 seconds at each 10,000 lb increment.

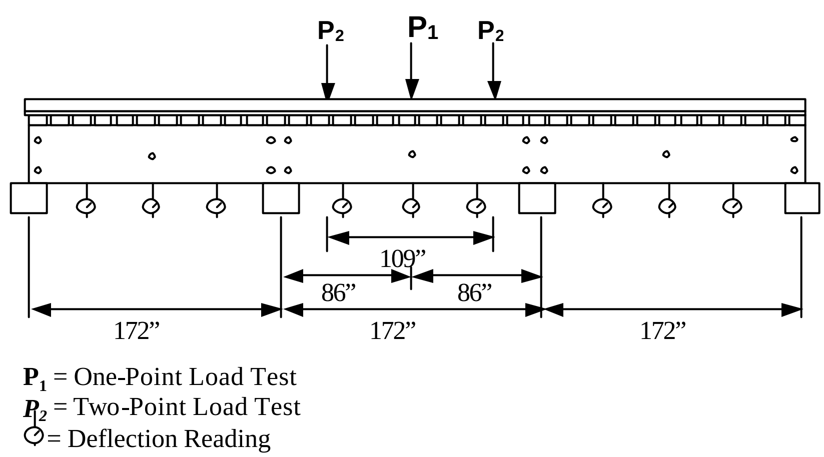
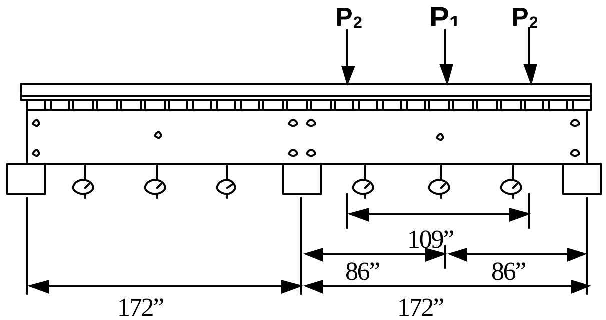
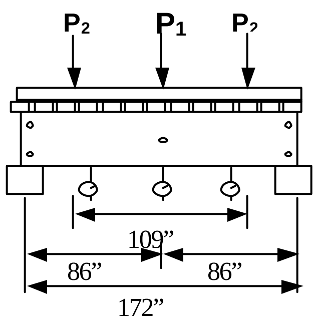


Figure 1.11: 3-Span Bridge Chord with 1- and 2-Point Loading Configurations



P_1 = One-Point Load Test
P_2 = Two-Point Load Test
= Deflection Reading

Figure 1.12: 2-Span Bridge Chord with 1- and 2-Point Loading Configurations



P_1 = One-Point Load Test
P_2 = Two-Point Load Test
= Deflection Reading

Figure 1.13: 1-Span Bridge Chord with 1- and 2-Point Loading Configurations

Several ply configurations were load tested as described below:

Three-Span Bridge Chord – One-Point Load

• 4 plies, with mid-span ply rods in place

• 4 plies with mid-span ply tie rods removed

• 5 plies, with mid-span ply rods in place

• 5 plies with mid-span ply tie rods removed

Three Span Bridge Chord: - Two-Point Loads

• 4 plies, with mid-span ply rods in place

• 4 plies with mid-span ply tie rods removed

• 5 plies, with mid-span ply rods in place

• 5 plies with mid-span ply tie rods removed

Two-Span Bridge Chord – One-Point Load

• 4 plies, with mid-span ply rods in place

• 4 plies with mid-span ply tie rods removed

Two-Span Bridge Chord: - Two-Point Loads

• 4 plies, with mid-span ply rods in place

• 4 plies with mid-span ply tie rods removed

One-Span Bridge Chord – One-Point Load

• 4 plies, with mid-span ply rods in place

• 4 plies with mid-span ply tie rods removed

• 5 plies, with mid-span ply rods in place

• 5 plies with mid-span ply tie rods removed

One-Span Bridge Chord: - Two-Point Loads

• 4 plies, with mid-span ply rods in place

• 4 plies with mid-span ply tie rods removed

• 5 plies, with mid-span ply rods in place

• 5 plies with mid-span ply tie rods removed

INSTRUMENTATION

Celesco cable-extension position transducers (potentiometers) were used to measure vertical displacement relative to the ground at mid-span and quarter points of each clear span. All plies were monitored for each load configuration. The potentiometers were placed from the underside of the plies and centered about their width. These positions are shown in the earlier Figures 1.11, 1.12 and 1.13.

EXPERIMENTAL RESULTS

Initial Load Tests

Extensive data were taken from the various load tests. Doyle's thesis includes comprehensive, detailed results (Doyle 2000). Selected results are provided in this report.

Chord 3/4

Figure 1.14 indicates the ply arrangement and monitored mid-span deflection locations for Chord 3/4. Load versus measured mid-span deflections for each ply of the spans are shown in Figures 1.15, 1.16 and 1.17 for the single-point loading with ply tie rods in place at each mid-span. Positive (negative) values represent upward (downward) displaced position. Positive (negative) slopes of the curves represent upward (downward) direction of movement. Plies in the center span all displaced downward and in a similar linear load-deflection pattern. Each ply in the end spans moved in a different

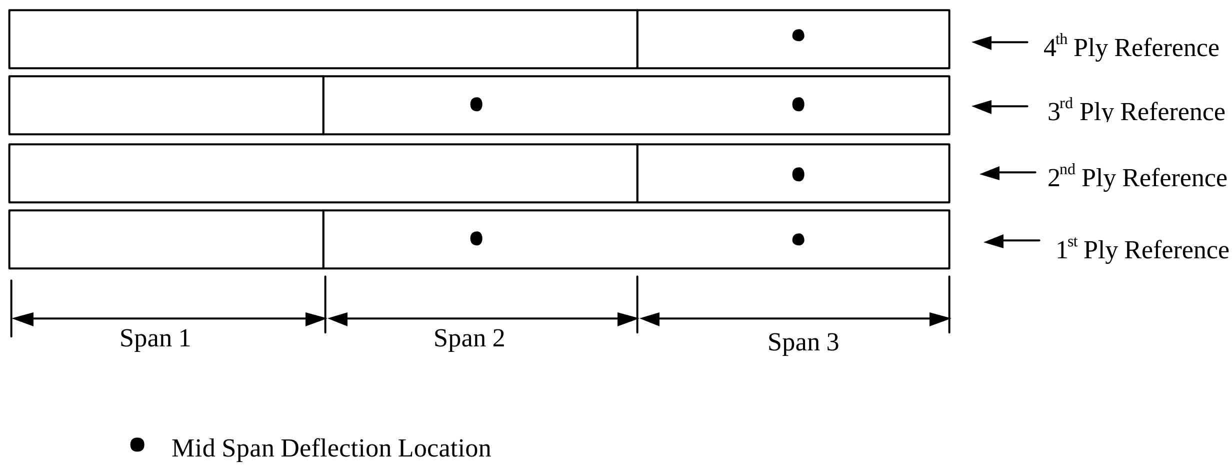


Figure 1.14: 3-Span, 4-Plies, Mid Span Deflection Locations

load-deflection manner, and some displaced upward, others downward. Magnitudes were much less than in the loaded span.

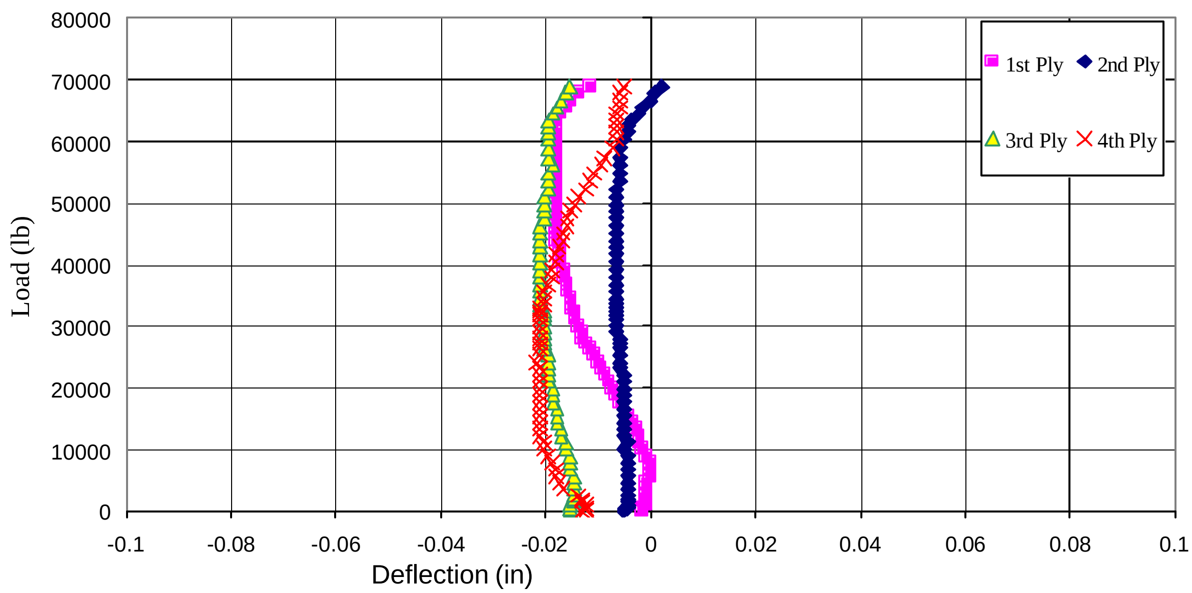


Figure 1.15: Chord 3/4 - 1-Point Load, Mid Span Deflections of Each Ply in Span 1

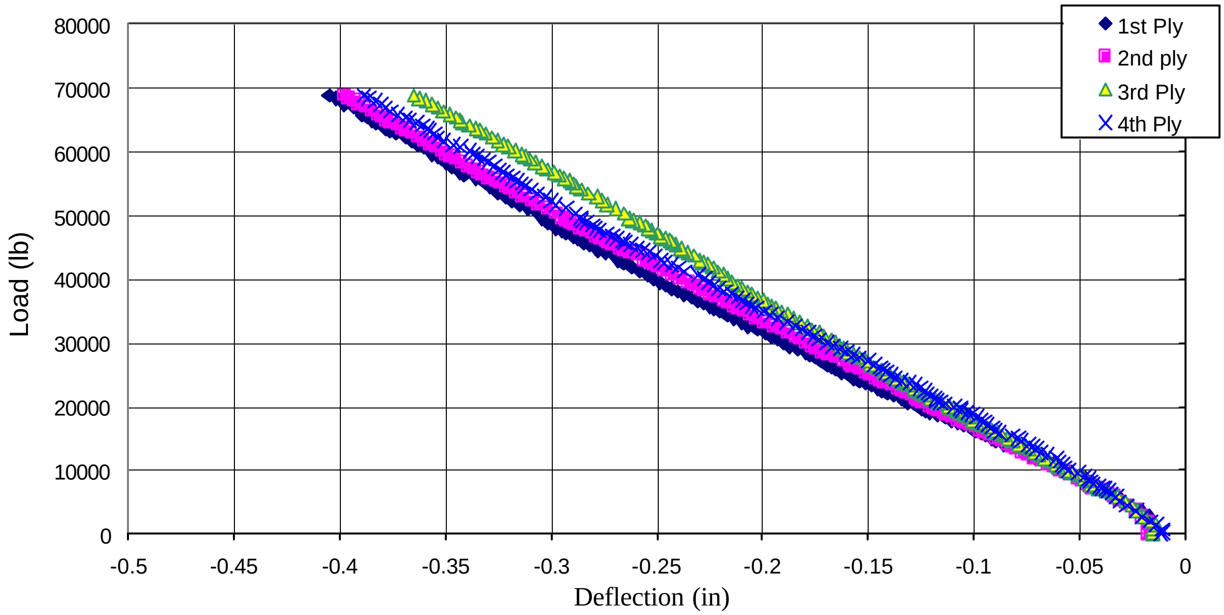


Figure 1.16: Chord 3/4 - 1-Point Load, Mid Span Deflections of Each Ply in Span 2

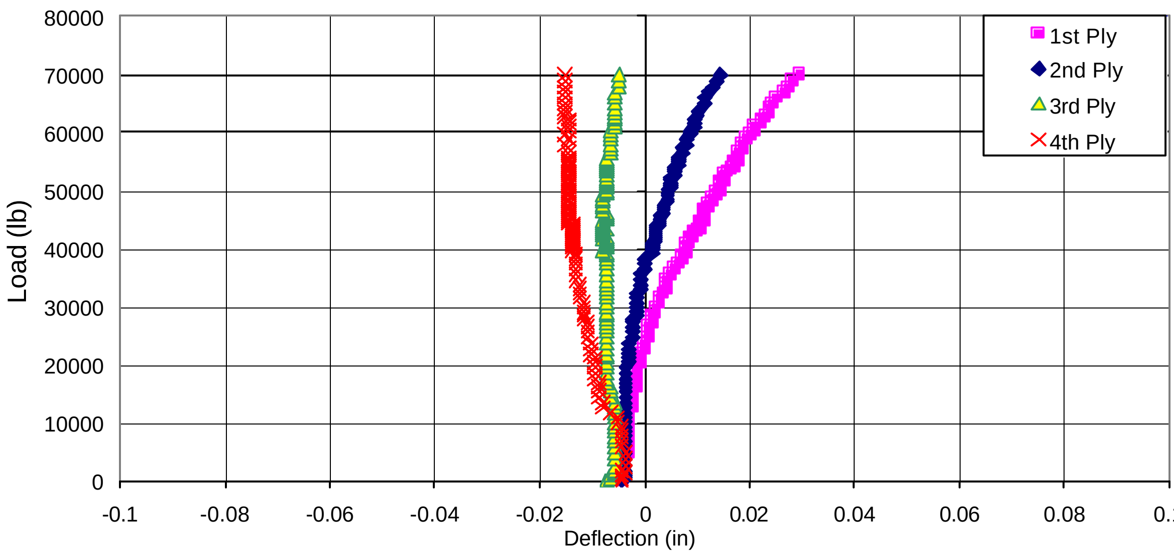


Figure 1.17: Chord 3/4 - 1-Point Load, Mid Span Deflections of Each Ply in Span 3

Figures 1.18, 1.19 and 1.20 show the mid span deflections for chord 3/4 for the two-point loading. Qualitatively, these are similar to those for the single point loading. It is evident from the center span, that despite doubling the load, the separation of the two loads away from mid-span greatly offsets the higher load, as the deflections only increased about 40 percent.

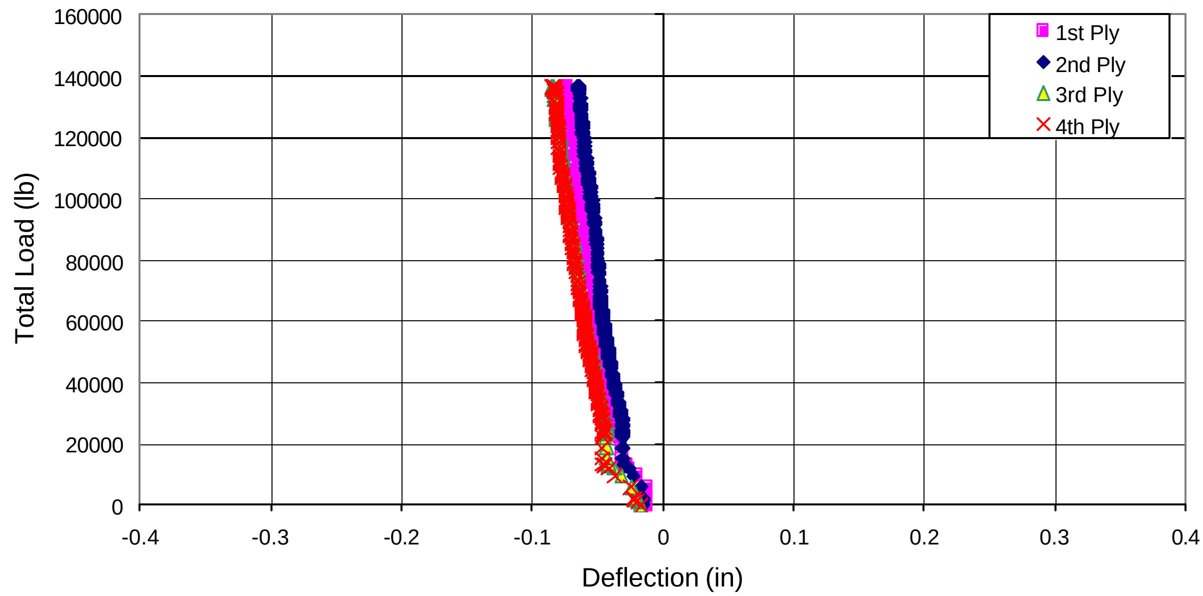


Figure 1.18: Chord 3/4 - 2-Point Load, Mid Span Deflections of Each Ply in Span 1

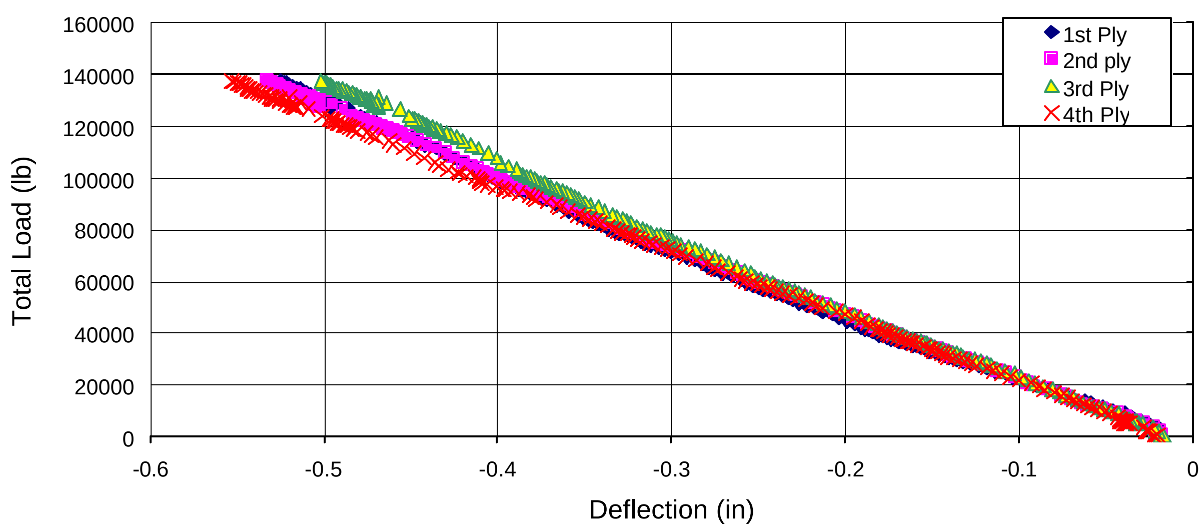


Figure 1.19: Chord 3/4 - 2-Point Load, Mid Span Deflections of Each Ply in Span 2

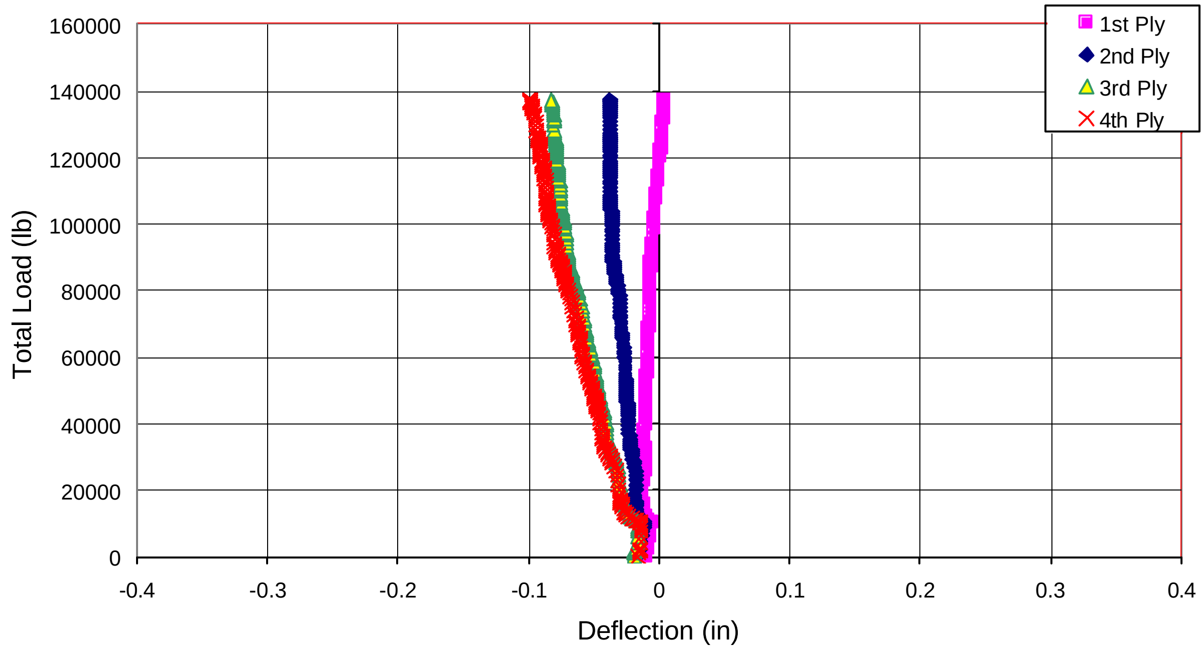


Figure 1.20: Chord 3/4 - 2 Point Load, Mid Span Deflections of Each Ply in Span 3

Chord 3/5

Figure 1.21 indicates the ply arrangement and monitored mid-span deflection locations for Chord 3/5. Load versus measured midspan deflections for each ply of the center span are shown in Figs. 1.22, 1.23 and 1.24 for the single point load (centered on the original four plies) case and ply tie rods in place at each mid-span. Plies 1, 2 and 3 experienced less deflection compared to chord 3/4 under the same loading. Ply 4 remained about the same and the added ply (a single span ply) deflected much less than the two-span plies. The former is attributed to an observed upward bow in ply 4, causing it to initially pick up load before the others and thus ultimately carrying more relative load share. The latter suggests the added ply was not fully effective and carried much less load than the others. It was also farthest from the load. Figures 1.25, 1.26, and 1.27 show the comparable results for the two-point loading. Similar observations are evident when compared to chord 3/4 under this loading.

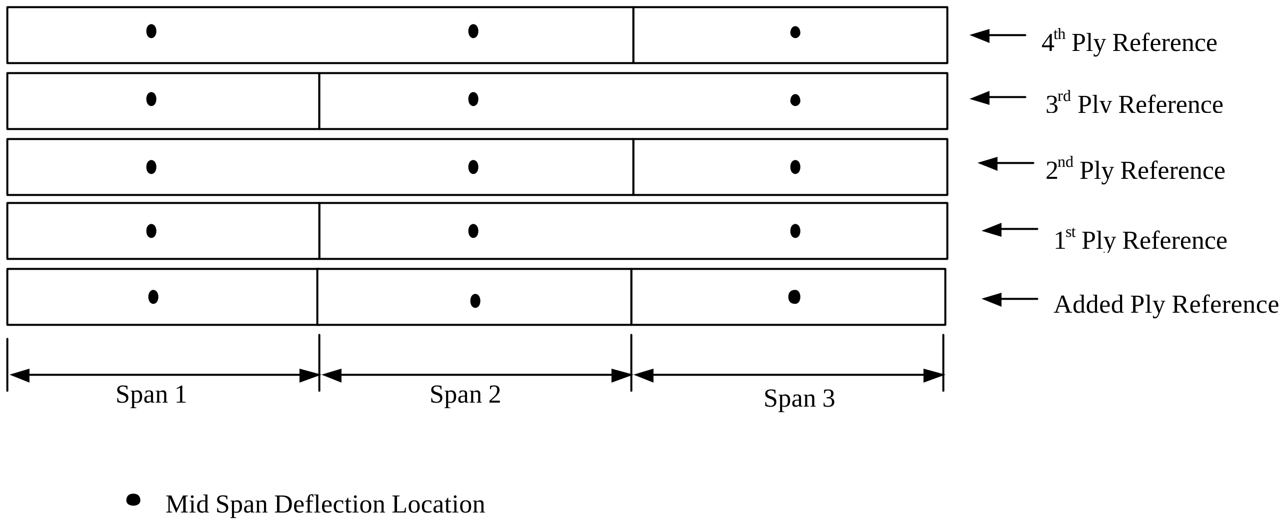


Figure 1.21: 3-Span, 5-Plies, Mid Span Deflection Locations

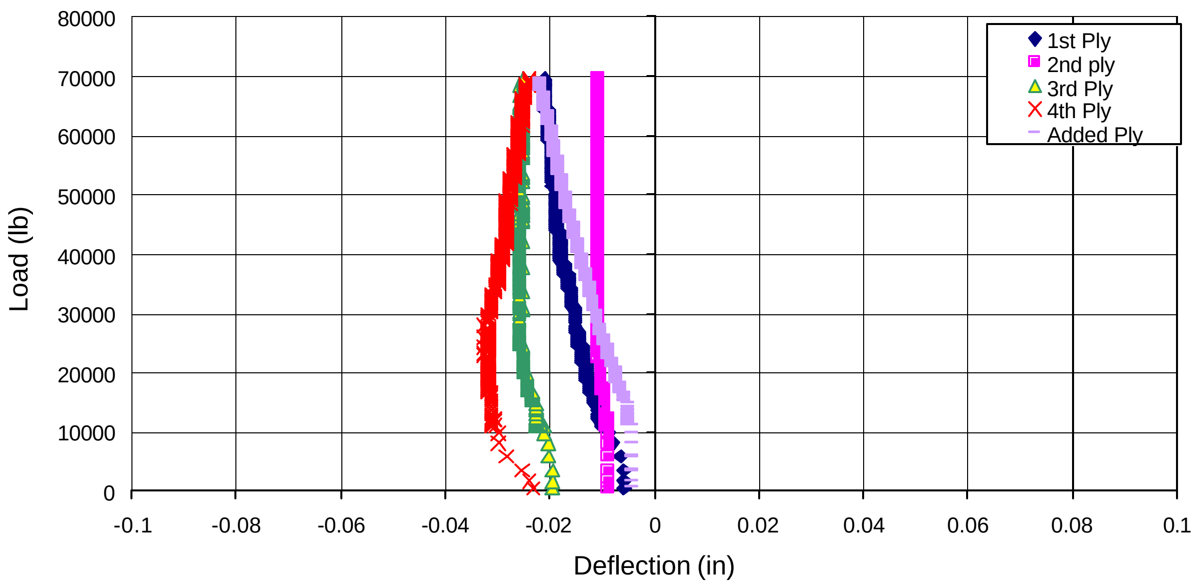


Figure 1.22: Chord 3/5 - 1-Point Load, Mid Span Deflections of Each Ply in Span 1

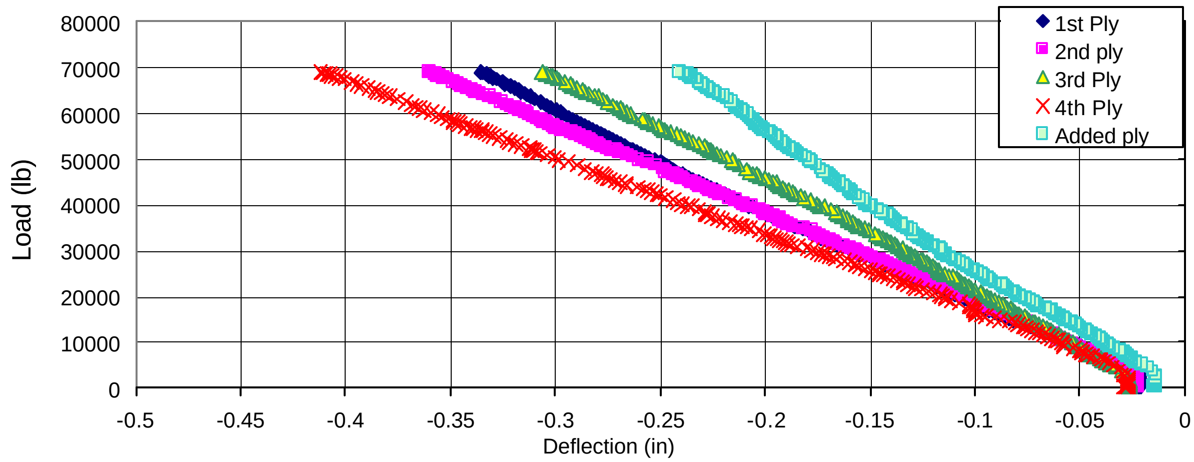


Figure 1.23: Chord 3/5 - 1-Point Load, Mid Span Deflections of Each Ply in Span 2

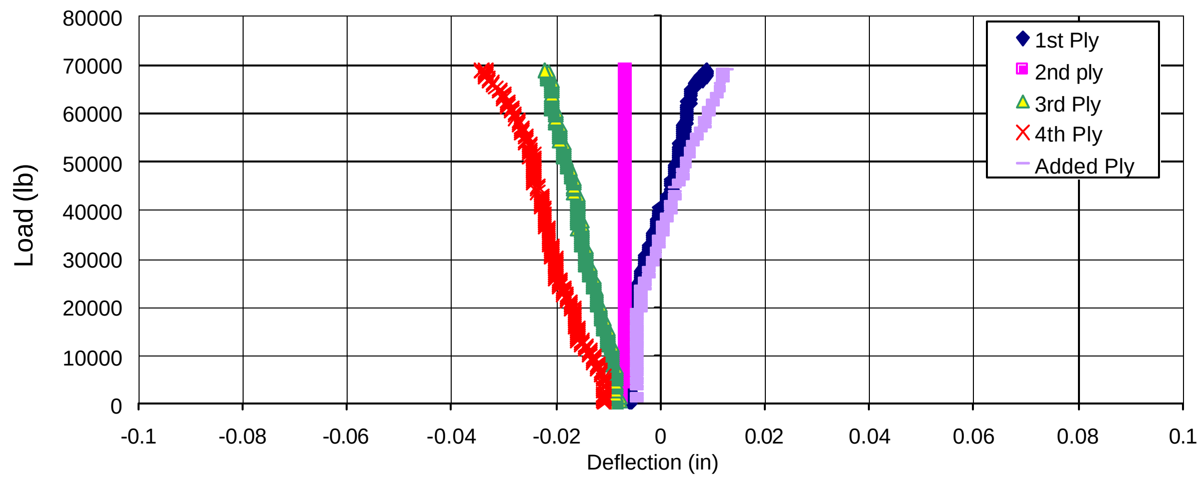


Figure 1.24: Chord 3/5 - 1-Point Load, Mid Span Deflections of Each Ply in Span 3

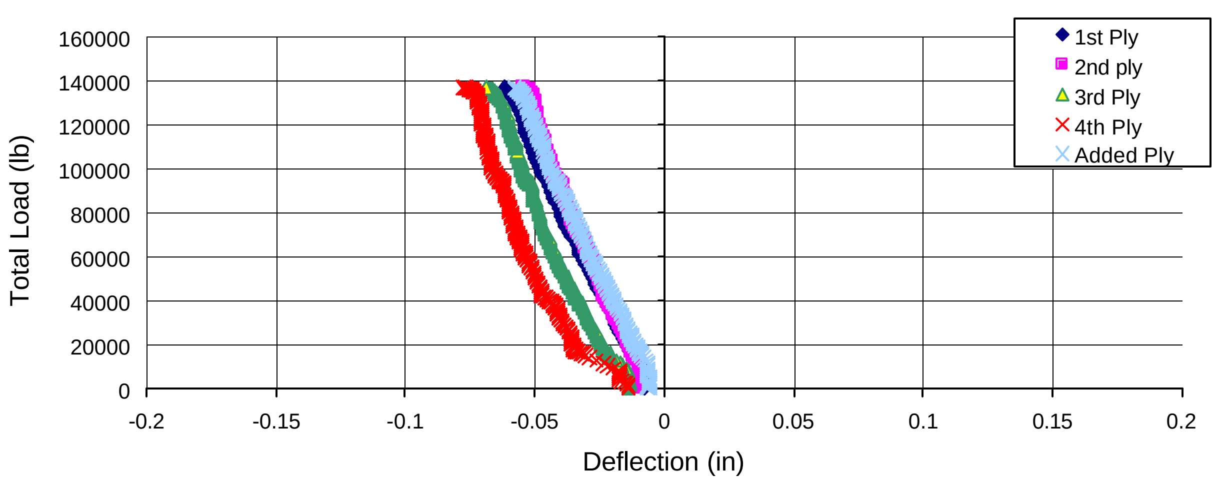


Figure 1.25: Chord 3/5 - 2-Point Load, Mid Span Deflections of Each Ply in Span 1

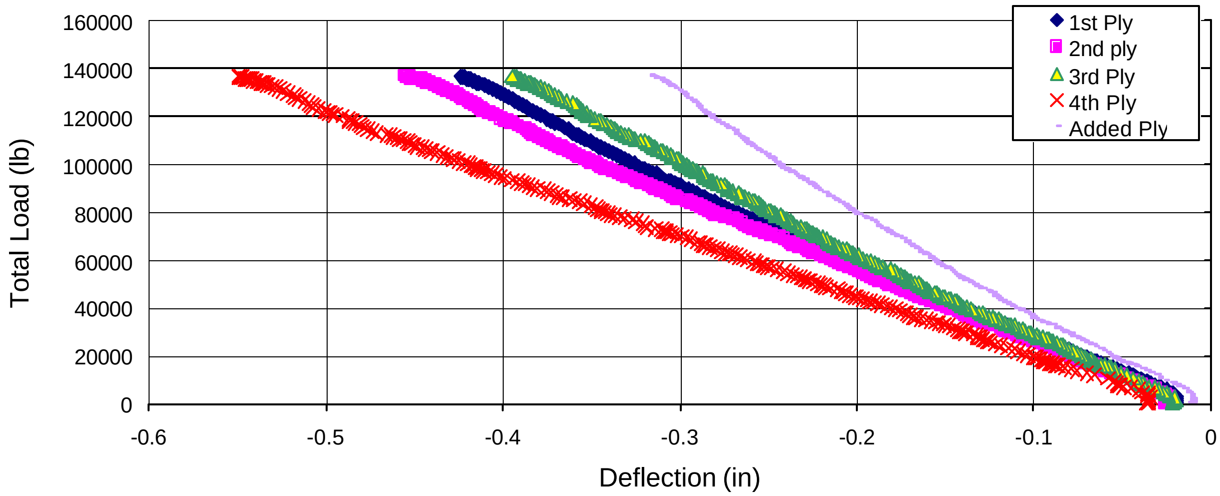


Figure 1.26: Chord 3/5 - 2-Point Load, Mid Span Deflections of Each Ply in Span 2

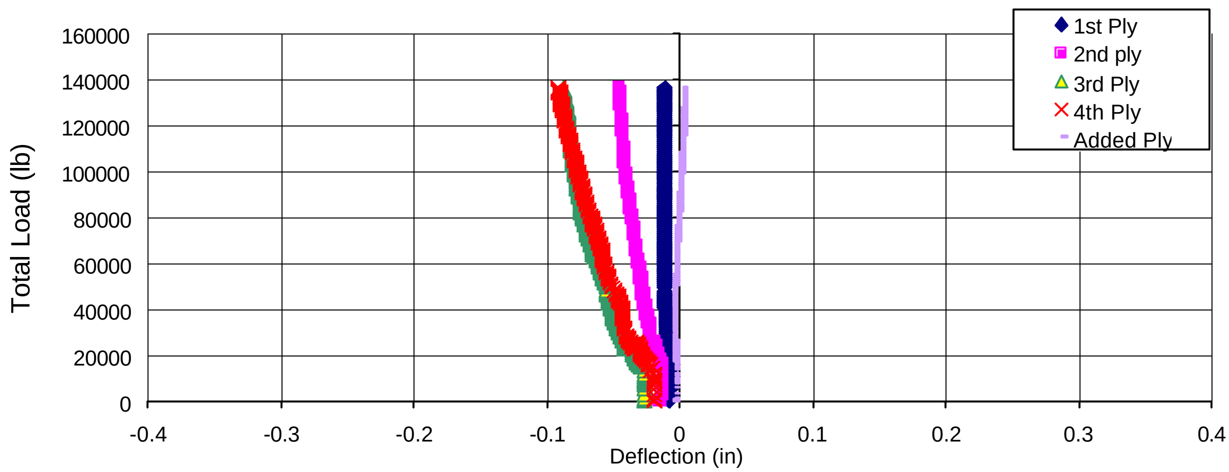


Figure 1.27: Chord 3/5 - 2-Point Load, Mid Span Deflections of Each Ply in Span 3

Chord 2/4

Figure 1.28 indicates the ply arrangement and monitored mid-span deflection locations for Chord 2/4. Ply tie rods were present at mid-span of each span. Load versus measured mid-span deflections for each ply of the non-loaded span are shown in Fig. 1.29 for the single point applied at mid-span of Span 2. The response essentially is linear. Plies in the loaded span all displaced downward and in a similar linear load-deflection pattern. Ply 4 displacements were noticeably greater than those of the other plies. At the highest load, the individual ply displacements varied between .38" and .43".

These compare with a range between .36" and .41" observed for Chord 3/4.

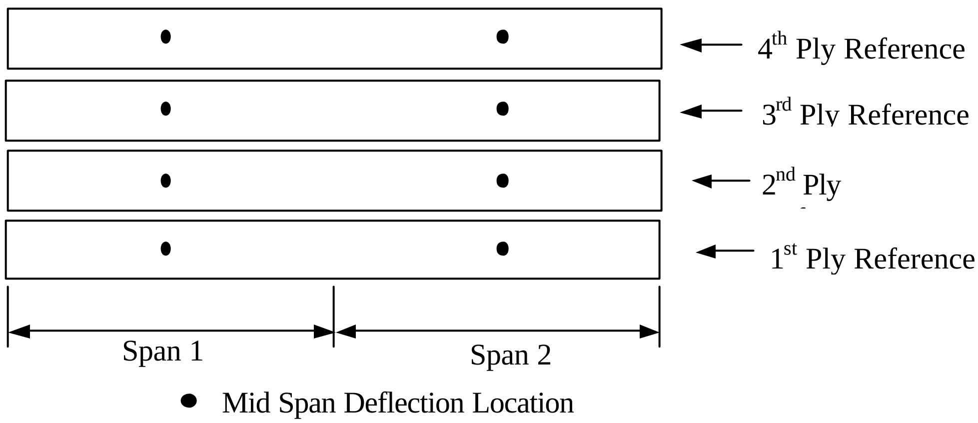


Figure 1.28: 3-Span, 4-Plies, Mid Span Deflection Locations

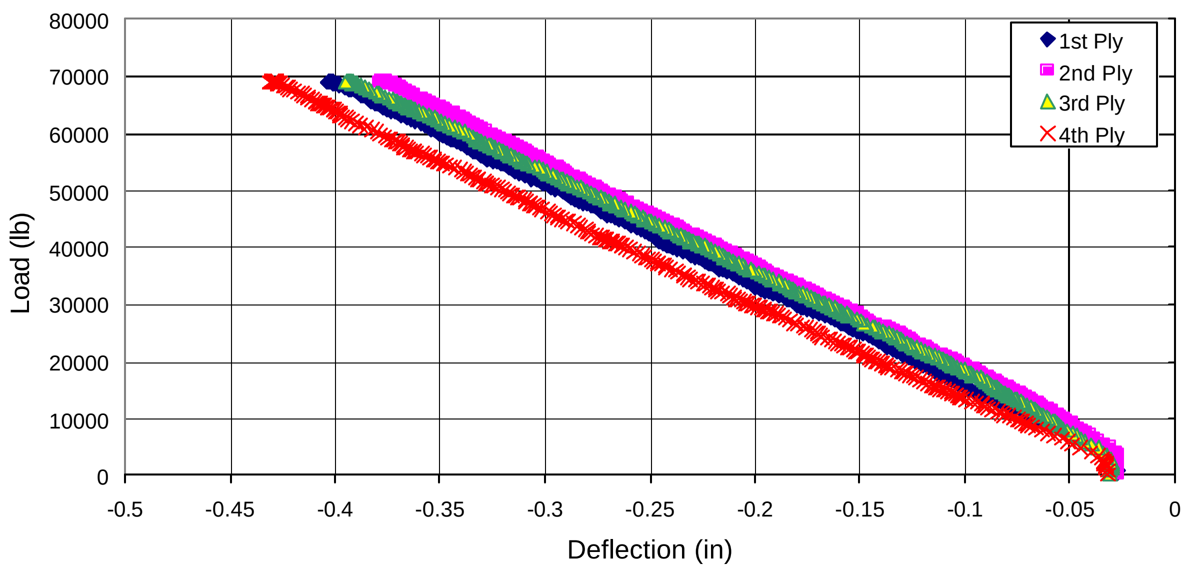


Figure 1.29: Chord 2/4 - 1-Point Load, Mid Span Deflections of Each Ply in Span 2

Figure 1.30 shows the response of the non-loaded span. Ply 1 deflected upward throughout the loading. The other plies initially had slight downward motion, then displaced upward for the remainder of the loading. Displacements are small in comparison to those of the loaded span.

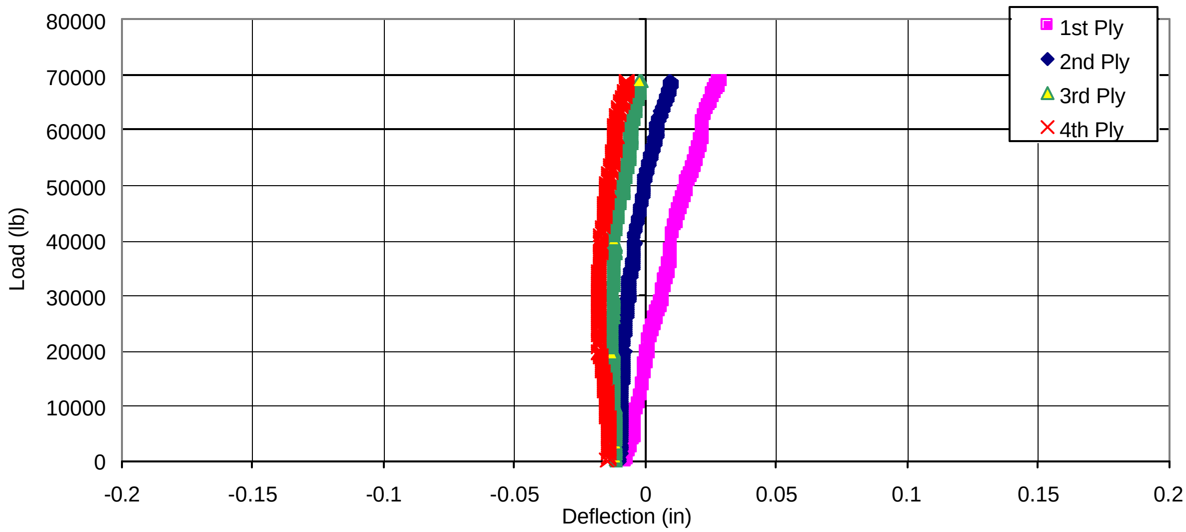


Figure 1.30: Chord 2/4 - 1-Point Load, Mid Span Deflections of Each Ply in Span 1

The corresponding responses under two-point loading are shown in Figures 1.31 (loaded span) and 1.32 (non-loaded span). All plies in the non-loaded span experienced a small downward motion at the maximum load level, 138,000 lbs. This suggests possible support displacement occurred at this higher total load. In the loaded span, at the highest load, all plies displacement downward and the range of maximum displacements was between .52" and .60". Again, ply 4 displaced noticeably more than the other plies.

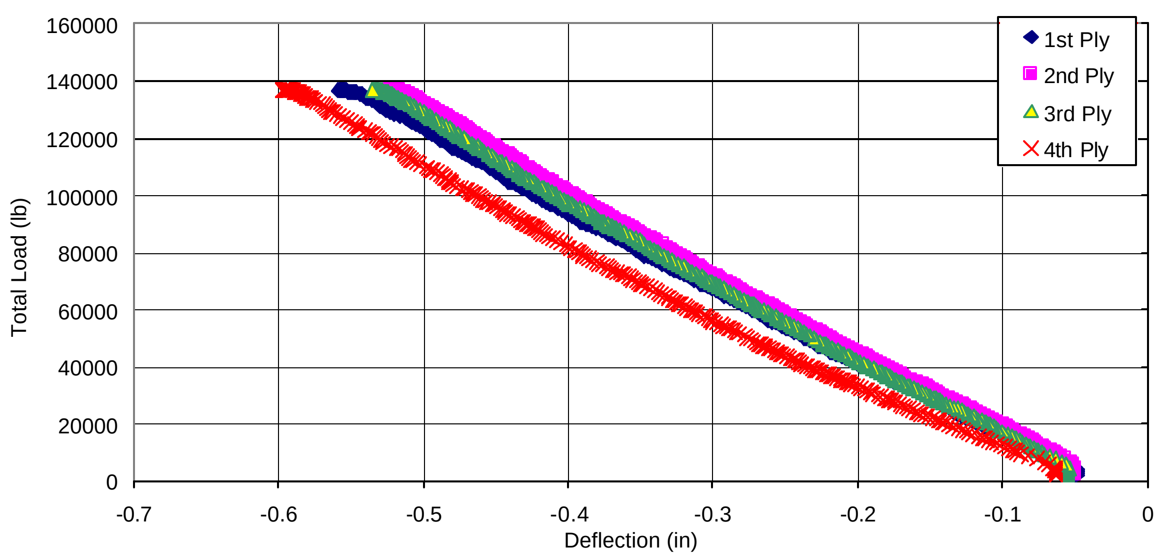


Figure 1.31: Chord 2/4 - 2-Point Load, Mid Span Deflections of Each Ply in Span 2

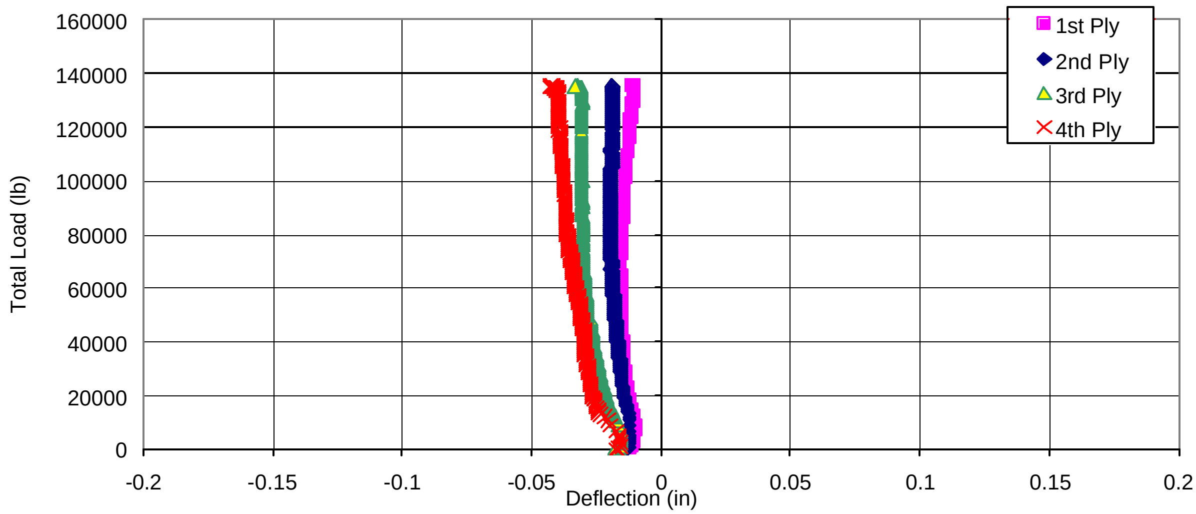


Figure 1.32: Chord 2/4 - 2-Point Load, Mid Span Deflections of Each Ply in Span 1

Chord 1/4

Figure 1.33 indicates the ply arrangement and monitored mid-span deflection locations for Chord 1/4 (and Chord 1/5). Ply tie rods were present at mid-span of each span. Load versus measured midspan deflections for each ply of the loaded span are shown in Figure 1.34 for the single point load applied at mid-span. The response essentially is linear beyond 10,000 lbs of load. All plies displaced downward and in similar linear load-deflection patterns. Ply 3 displacements were noticeably greater than those of the other plies. At the highest load (69,000 lbs), the individual ply displacements varied between .37" and .44". These compare with a range between .36" and .41" observed for Chord 3/4 and between 38" and .43" observed for Chord 2/4.

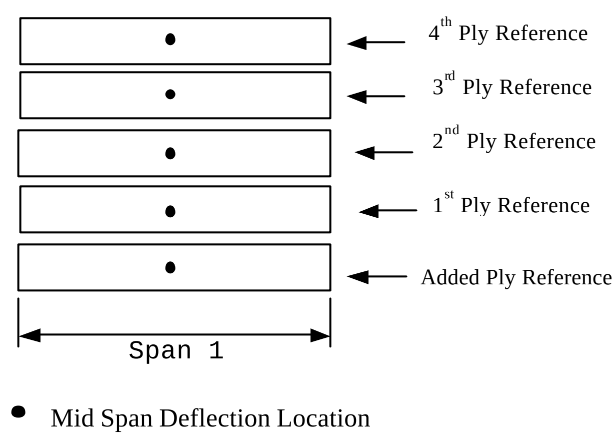


Figure 1.33: 1-Span, 4- and 5-Ply Deflection Locations for the One-Point Load Test

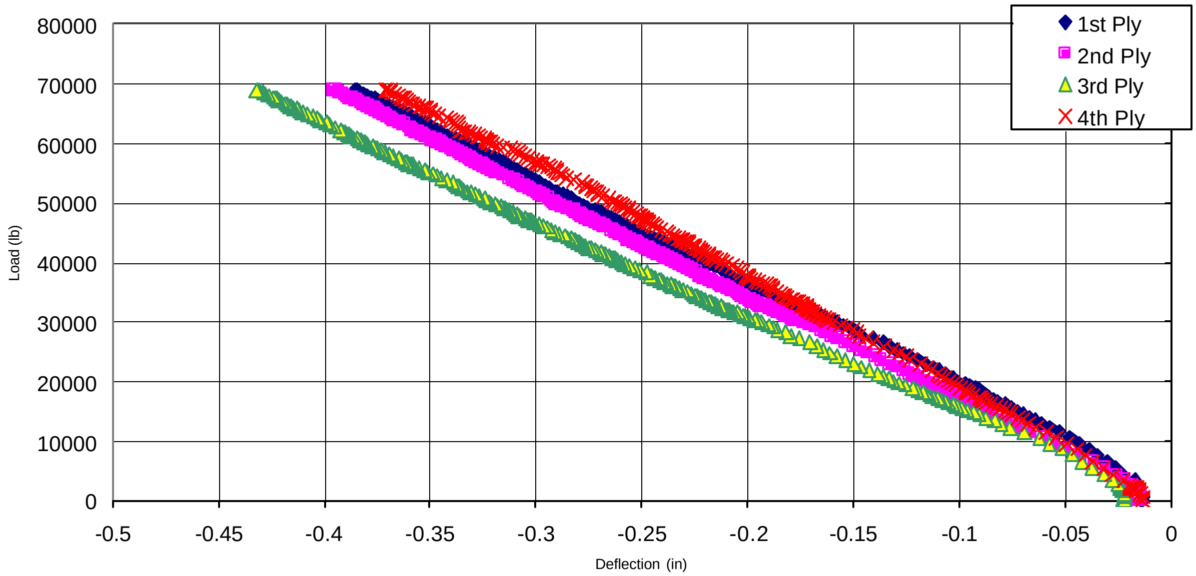


Figure 1.34: Chord 1/4 - 1 Point Load, Mid Span Deflections of Each Ply

The corresponding response under the two-point loading case is shown in Figure 1.35. All plies displaced linearly, essentially from the outset of loading. At the maximum load (138,000 lbs), the ply displacements ranged from .42" to .48", i.e. were higher than for the one single-point loading by about 9 percent.

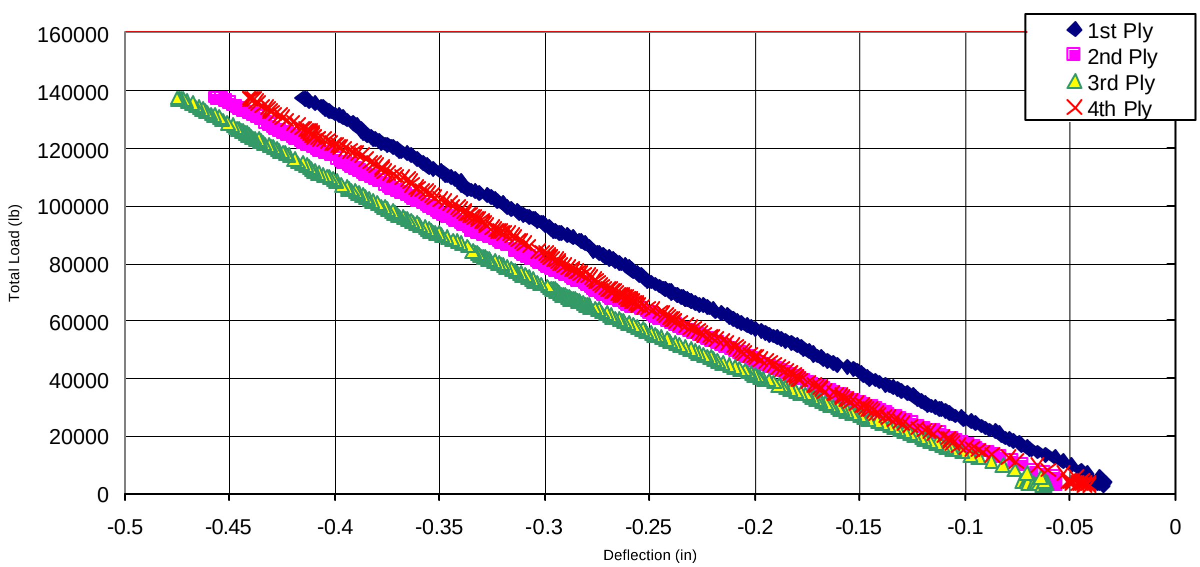


Figure 1.35: Chord 1/4 - 2-Point Load, Mid Span Deflections of Each Ply

Chord 1/5

For the five-ply arrangement in Chord 1/5, the steel rail was left centered over the original four plies. Load-displacement responses for each ply of the loaded span are shown in Fig. 1.36 for the single-point loading. The response essentially is linear beyond about 5,000 lbs of load. All plies displaced downward and in similar linear load-deflection patterns. Compared to Chord 1/4, the individual ply responses were more spread out. At the highest load (69,000 lbs), the individual ply displacements varied between .31" and .55". The added ply displaced about .28", which is the least overall.

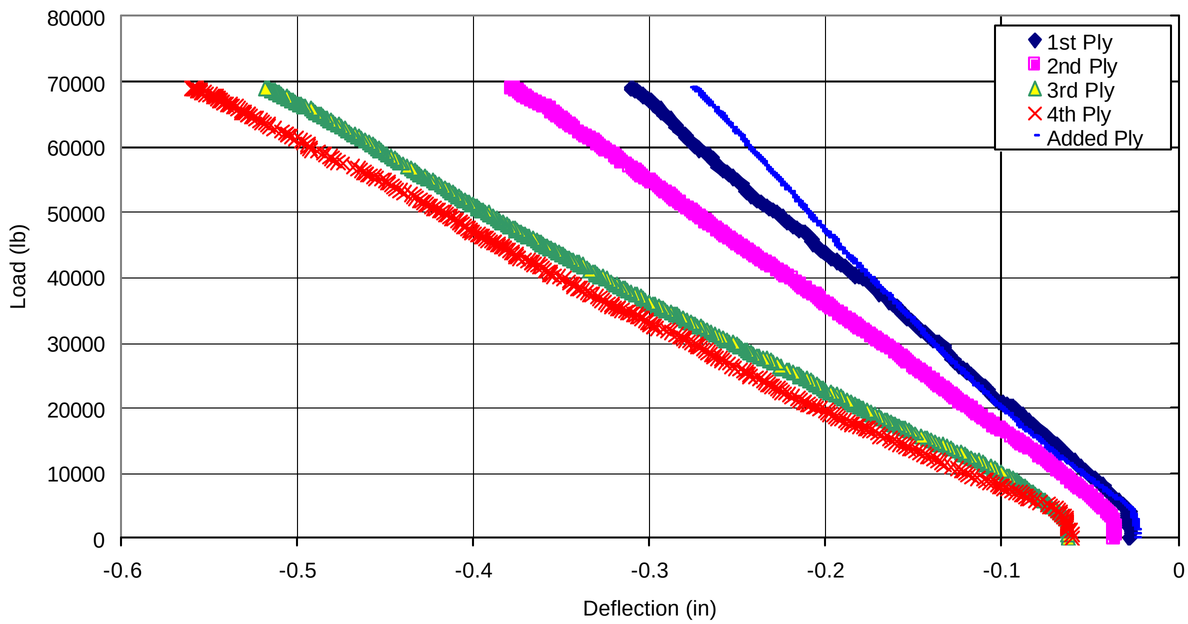


Figure 1.36: Chord 1/5 - 1-Point Load, Mid Span Deflections of Each Ply

The corresponding response under the two point loading case is shown in Figure 1.37. All plies displaced linearly, essentially from the outset of loading, but with varied responses. At the maximum load (138,000 lbs), the ply displacements ranged from .31" to .46", i.e. they were slightly lower on average than for the single-point load case. The added ply deflected .29", which is the least overall. Ply 1 displaced only slightly more than the added ply. Plies 3 and 4 displaced about the same and the highest amounts overall.

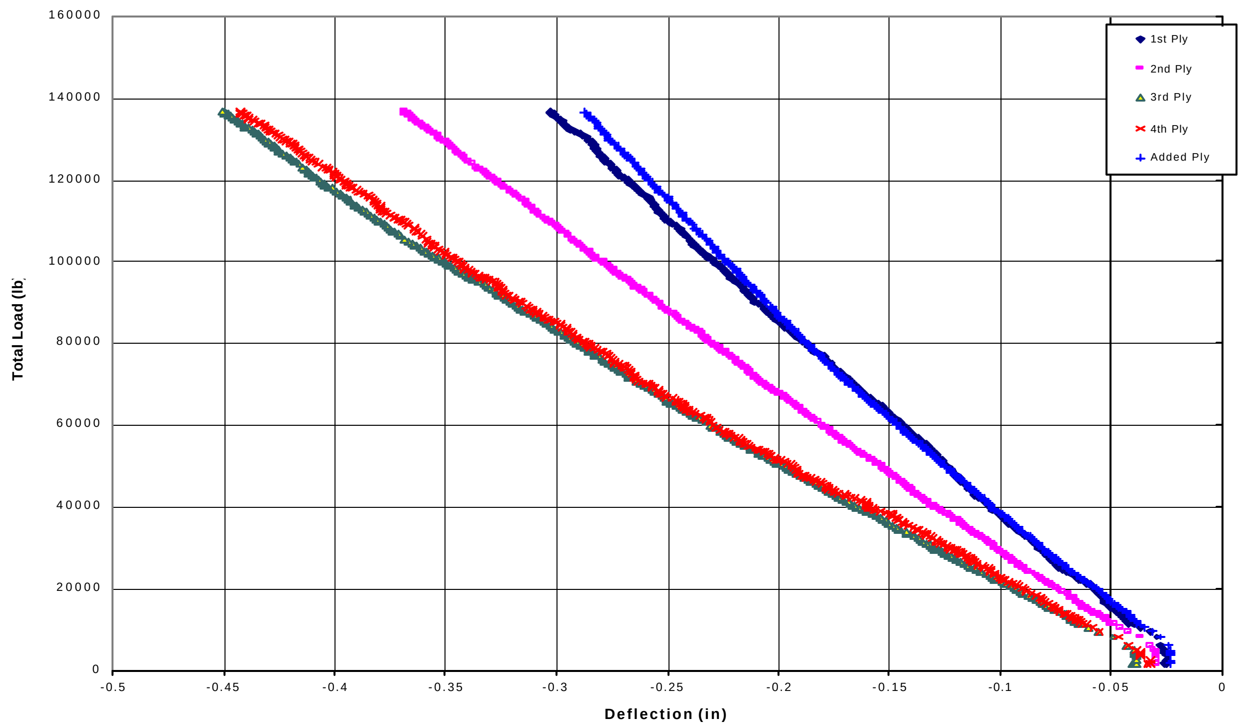


Figure 1.37: Chord 1/5 – 2-Point Load, Mid Span Deflections of Each Ply

Effect of Ply Tie Rods

Chord 3/4 and Chord 3/5 – Single-Point Loading

Figure 1.38 shows the average of the ply deflections at mid-span (Row 5) and the two-quarter points (Row 4 and Row 6) of the loaded span (Span 2) in Chord 3/4 for the single point load. The results at the 69,000 lbs. load level were .391" for Row 5 and .298" and .296 in. for Row 4 and Row 6, respectively. Fig, 1.39 shows the corresponding results with the mid-span ply tie rods removed in all spans. The results for Row 5, Row 4, and Row 6 are .405", .303" and .311", respectively. The differences are 3.5 percent, 1.7 percent and 5.1 percent increase in deflection, respectively, when ties were removed.

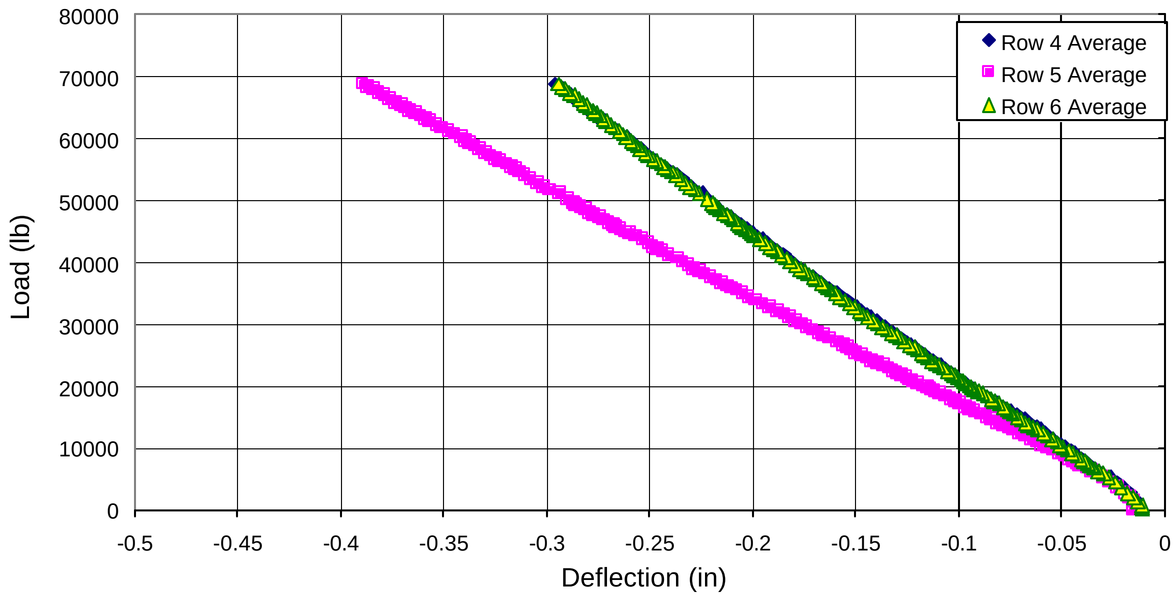


Figure 1.38: Chord 3/4 - 1-Point Load, Average Row Deflections in Span 2

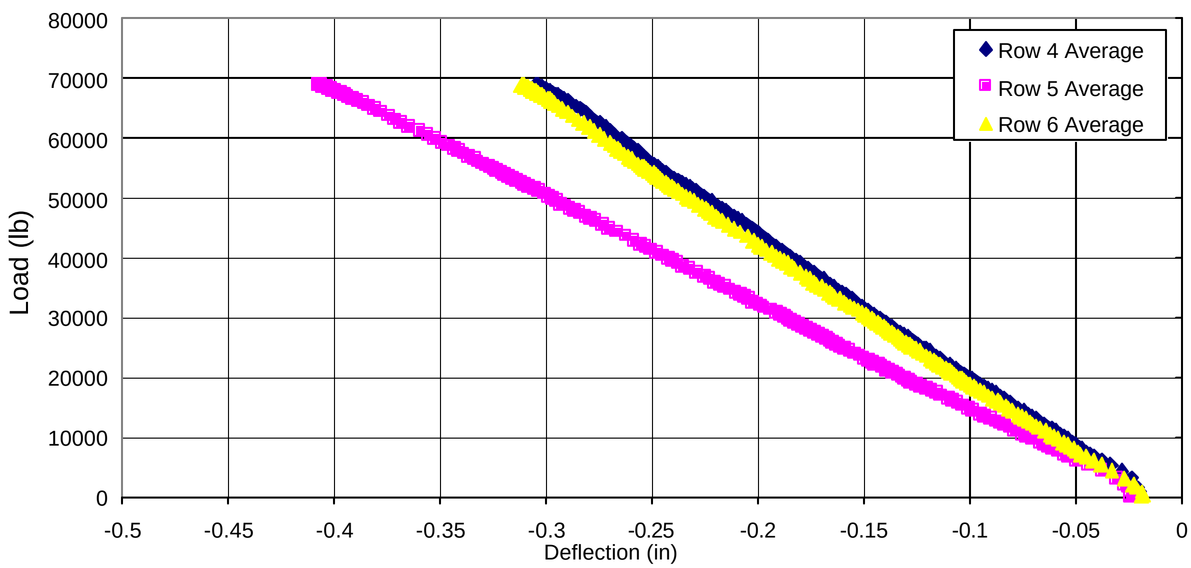


Figure 1.39: Chord 3/4 - 1-Point Load, Average Row Deflections in Span 2, No Mid Span Ply Tie Rods

For the Chord 3/5 under single point loading, the corresponding values with ply tie rods present (removed) were .330", .244" and .274" (.334", .267", and .275"), respectively. The differences are 1.2 percent, 9 percent and .3 percent increase in deflection, respectively, when ties were removed.

Chord 3/4 and Chord 3/5 – Two-Point Loading

Figures 1.40 shows the average of the ply deflections at mid-span (Row 5) and the two-quarter points (Row 4 and Row 6) of the loaded span (Span 2) in Chord 3/4 for the two-point load case. Results for the 138,000 lbs. load level were .531" for Row 5 and .445" and .435" for Row 4 and Row 6, respectively. Fig. 1.41 shows the corresponding results with the mid-span ply tie rods removed in all spans. The results for Row 5, Row 4, and Row 6 were .559", .467", and .457" respectively. The differences are 5.3%, 4.9% and 2.7% increase in deflection, respectively, when ties were removed.

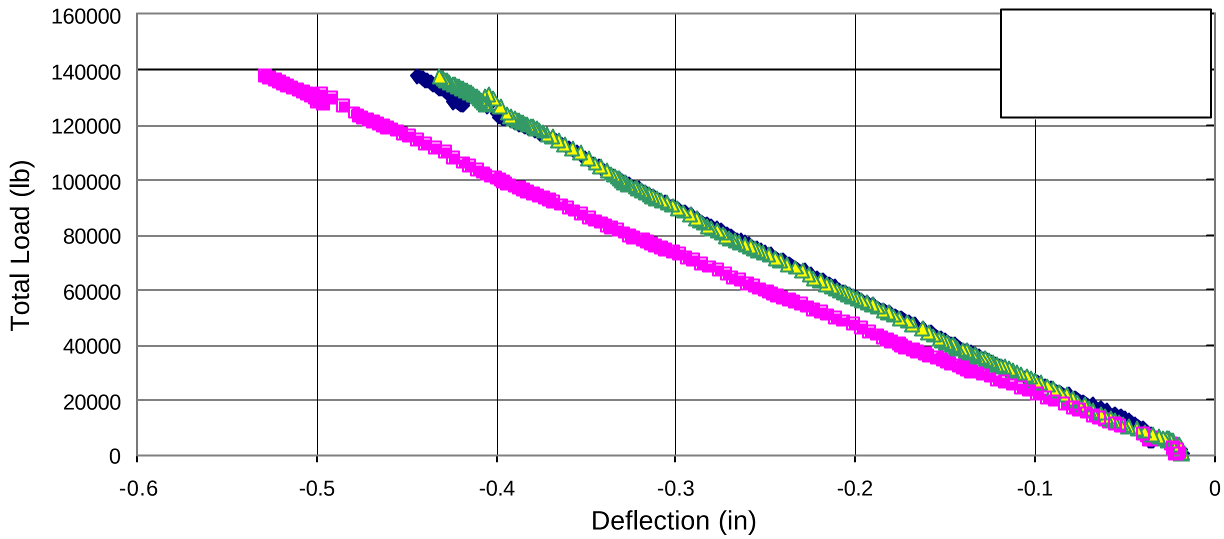


Figure 1.40: Chord 3/4 - 2-Point Load, Average Row Deflections in Span 2

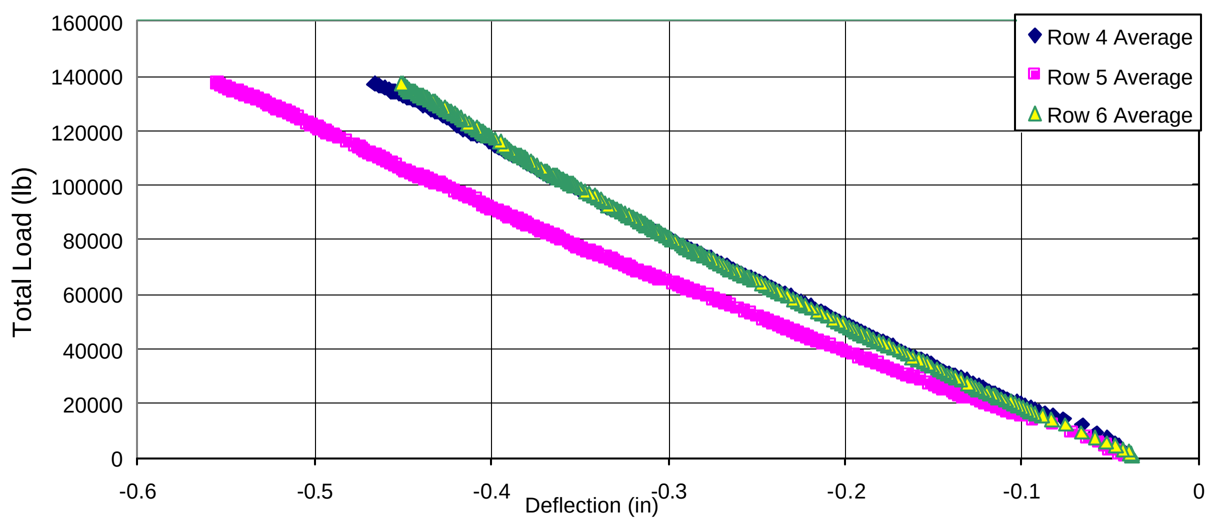


Figure 1.41: Chord 3/4 - 2-Point Load, Average Row Deflections in Span 2, No Mid Span Ply Tie Rods

For the Chord 3/5 under two point loading (138,000 lbs), the corresponding values with ply tie rods present (removed) were .431", 401" and .379" (.421", .382", and .379"), respectively. In this case, the spans actually deflected less than when the ply tie rods were removed, by an average of 2.3 percent.

Chord 2/4 - Single-Point Loading

Figure 1.42 shows the average of the ply deflections at mid-span (Row 5) and the two-quarter points (Row 4 and Row 6) of the loaded span (Span 2) in Chord 2/4 for the single point load. esults at the 69,000 lbs. load level were .400" for Row 5 and .288" and .343 in. for Row 4 And Row 6, respectively. Figure 1.43 shows the corresponding results with the mid-span ply tie rods removed in all spans. The results for Row 5, Row 4, and Row 6 are .400", .288", and .346", respectively. Virtually no change in deflection occurred when ply tie rods were removed.

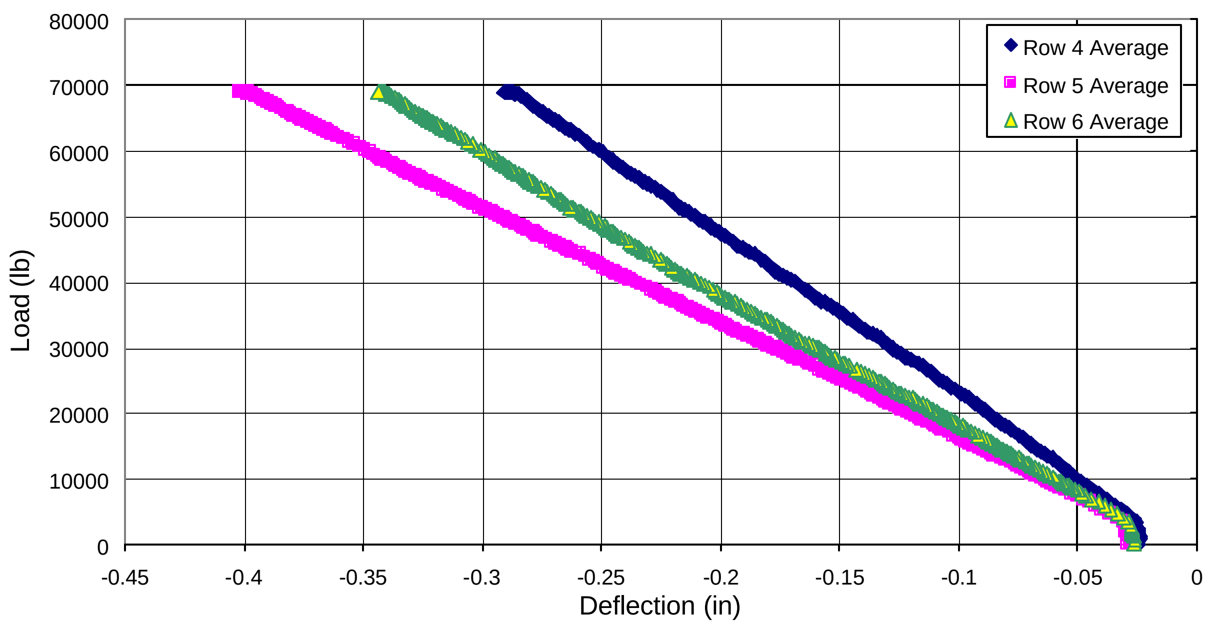


Figure 1.42: Chord 2/4 - 1-Point Load, Average Row Deflections in Span 2

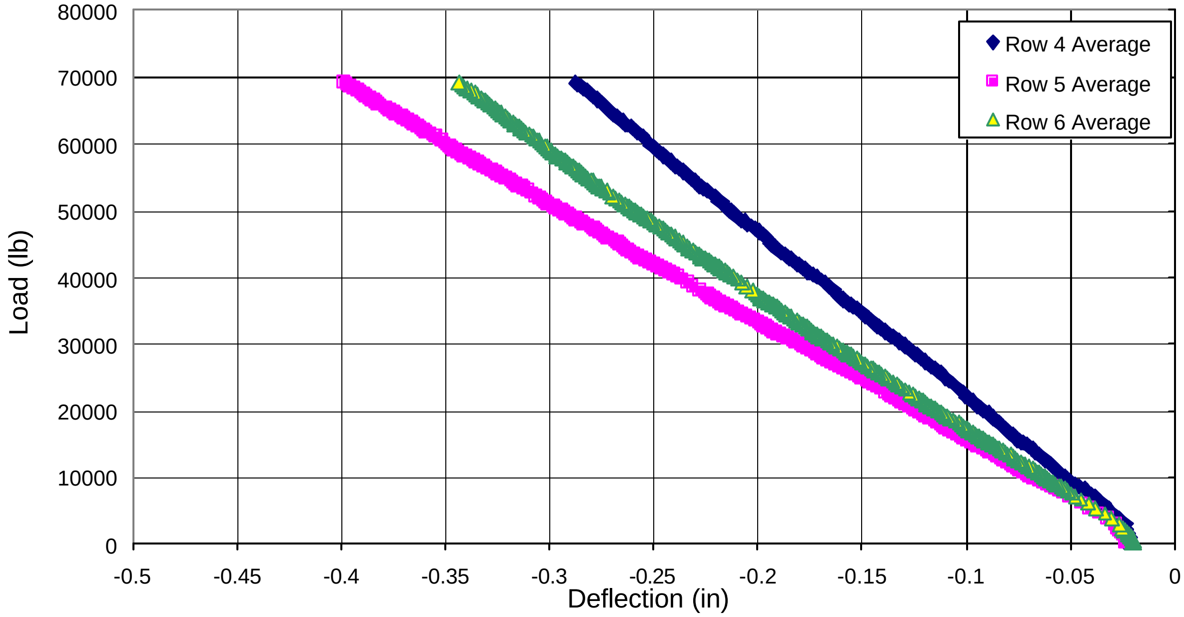


Figure 1.43: Chord 2/4 - 1-Point Load, Average Row Deflections in Span 2, No Ply Tie Rods

Chord 2/4 – Two-Point Loading

For the Chord 2/4 under two point loading (138,000 lbs), the corresponding values with ply tie rods present (removed) were .554", .428", and .496" (.510". 391", and .458"), respectively. In this case, the loaded span actually deflected measurably less (by an average of 8.1 percent) than when the ply tie rods were removed.

Chord 1/4 and Chord 1/5 – Single-Point Loading

Figure 1.44 shows the average of the ply deflections at mid-span (Row 2) and the two-quarter points (Row 1 and Row 3) in Chord 1/4 for the single point load. The results at the 69,000 lbs load level were .397" for Row 2 and .334" and .312 in. for Row 1 and Row 3, respectively. Figure 1.45 shows the corresponding results with the mid-span ply tie rods removed. The results for Row 5, Row 4, and Row 6 are .421", .350" and, .326", respectively. The differences are 6.0 percent, 4.7 percent and 4.5 percent increase in deflection when ties were removed. For Chord 1/5 the average mid span ply deflection was

.408". When the ply ties removed the average value was .392", which actually is a decrease of 4.0 percent.

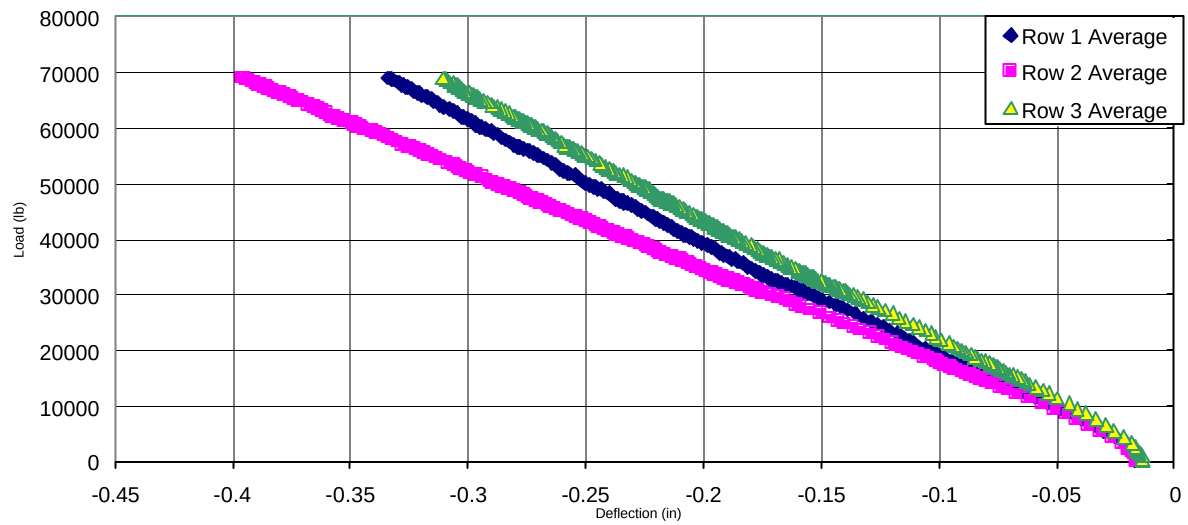


Figure 1.44: Chord 1/4 - 1-Point Load, Average Row Deflections

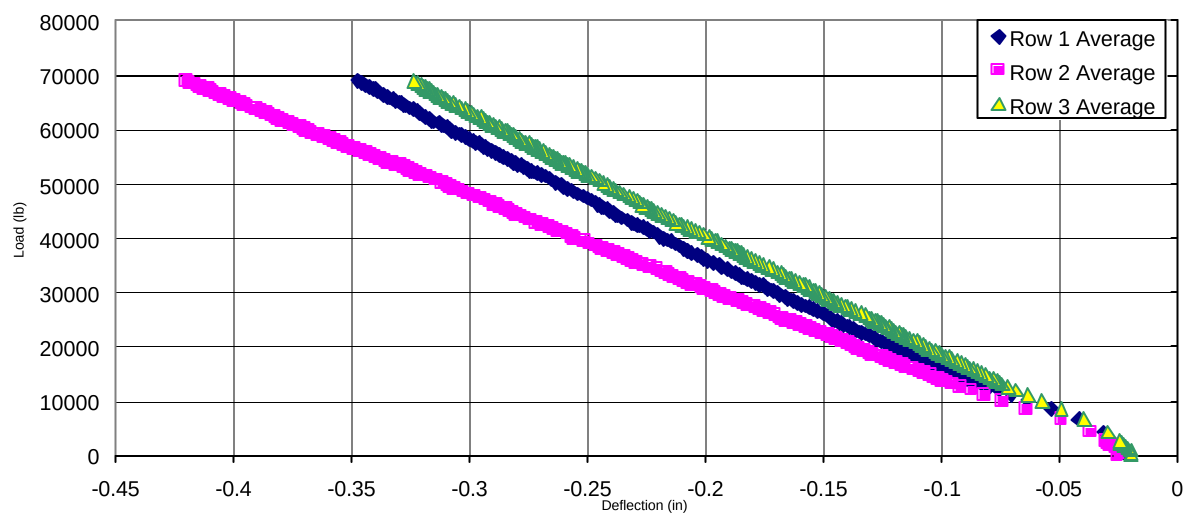
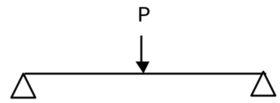
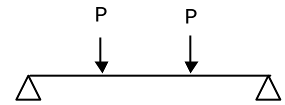
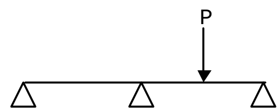
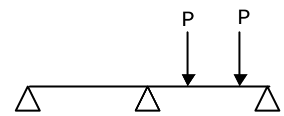
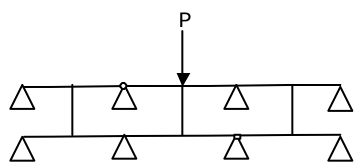
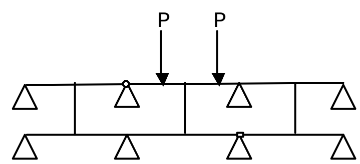
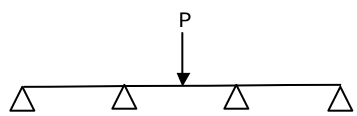
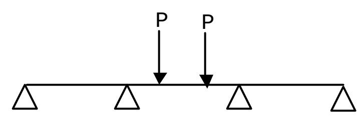
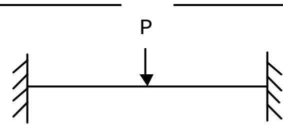
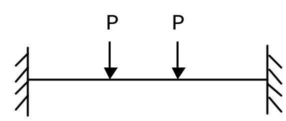


Figure 1.45: Chord 1/4 - 1-Point Load, Average Row Deflections, No Ply Tie Rods

ANALYTICAL MODELING

Initially, several ordinary structural modeling methods were used to predict deflections of the experimentally investigated bridge chords. The models and the loadings are depicted in Table 1.1. The following describes each of them in detail.

Table 1.1: Representative Beam Models

Beam Models	Pictured Illustrations	
	One-Point Load	**Two-Point Load**
Simply Supported Beam		
Two-Span Continuous Beam		
Three-Span Semi-Continuous Beam		
Three-Span Continuous Beam		
Fixed-Fixed Beam		

If support motion does not occur, then it is rational to anticipate deflection of the loaded span of a specimen to fall between two bounds. If all ply ends behaved like a pinned end (free to rotate), the response would be that of a single span, simply supported beam and produce an upper bound on deflection. If all ply ends behaved like a fixed end (no rotation possible), then the response would be that of a single span, fixed-end beam and produce a lower bound on deflection. As neither is the case, the actual response would be in between these bounds. Also, the individual plies in the chord span have different moduli of elasticity, which affects the overall chord behavior. Thus, even without support motion occurring in the specimens, these models still are approximate.

In the absence of support motion, it is also rational to expect the one-, two-, and three-span chord specimens to respond somewhat like one-, two- and three-span continuous beams, respectively. In the case of the one-span and two-span chord specimens, they are configured as one-span and two-span beams. However, the differing moduli elasticity of the individual plies in each span effects the combined behavior of the systems. The three-span chord has this factor as well as the staggered placement of the lapped two-span plies and presence of both single-span plies and two-span plies in the end spans. Hence, there actually is a "semi-continuous" configuration - at interior supports some plies have pinned ends and others are continuous over the support. Thus, even without support motion occurring in the specimens, these multi-span beam models also are approximate.

To effect the above ordinary beam models, the following primary assumptions were made:

1. As load sharing among plies in a span was neither known nor directly measured, the plies were assembled into a single member, having an EI value equal to the summed I values of the plies times their average MOE.
2. There were no gaps between the plies and the caps they rested on.
3. There was no support vertical motion.
4. The steel ply tie rods and timber cross-ties were not considered in the models.
5. The steel rail was not included because it was only fastened nominally at a few locations and was believed not to transfer significant interlayer shear. In effect, it was considered dead weight only.

For the three-span bridge chord (Specimen 3S), a "semi-continuous" beam model also was employed. Specimen 3S had lapped, two-span plies in the middle span and in-filled single-span plies in the end spans (see earlier Figure 1.3). The semi-continuous beam model used a stiffness matrix assembled for the system schematically depicted in Figure 1.46. Referring to that figure, over the length of the specimen a line of plies has either a configuration A (i.e that of the upper beam with a hinge at the third support), or a configuration B (i.e that of the lower beam with a hinge at the second support) of the schematic. Thus a stiffness model is assembled for each configuration, with the individual spans assigned an EI equal to the sum of the individual EI values of the plies that have the configuration of that line. The vertical rigid link at each mid-span is to force each span of the two configurations to be joined in resisting the single applied load. The hinges are used to reflect the discontinuity of the butted ends. The steel ply tie rods, timber cross-ties and steel rail were neglected.

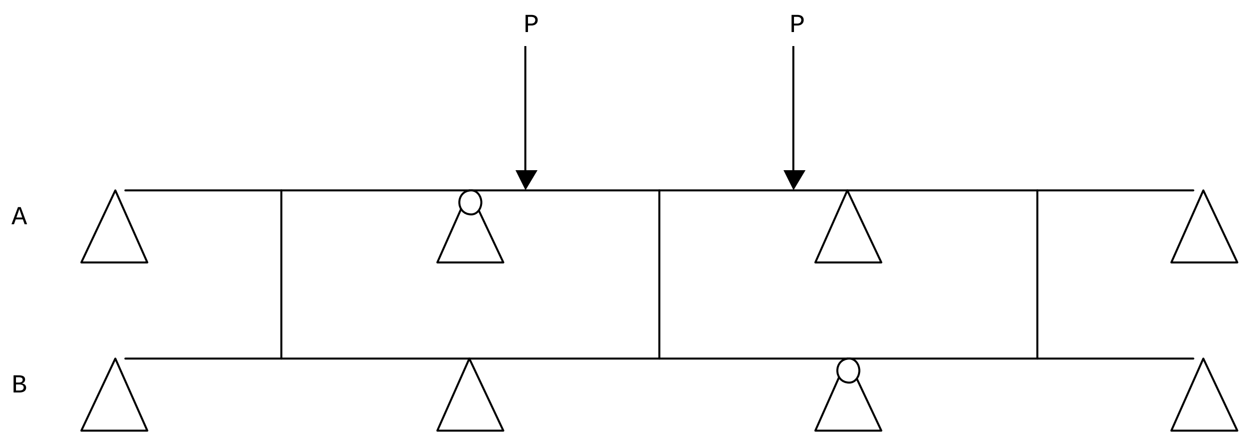


Figure 1.46 Semi-Continuous Beam Model

Table 1.2 lists the theoretical values expected from each of the above beam models. The results are expressed in terms of the variables involved. Each is in the form PL^3/nEI, where n is a numerical value, which differs for each case. Using the procedure described by Doyle (Doyle 2000) the corresponding values of "n" based on the measured data were determined. These are listed in Table 1.3. Comparing with the theoretical values, it is evident that the bridge chord specimens did not respond in magnitude of deflections consistent with these simplified models. The magnitude of the deflections of the two-span and three-span bridge chords were closest to the single-span, simply-supported beam. For the two-point loading, the measured result was outside the expected bounds. The single-span chord was stiffer than the single-span, simply-supported beam model. In fact, the magnitude was between those of the two-span and three-span continuous beam models.

Inaccuracies in the above ordinary models were initially attributed to downward support motion and relative displacements due to gaps between other surfaces. However, more detailed analysis indicated that the ends of the members experienced uplift at the end supports (Doyle 2000). This was possible because of the expedient connection details chosen to enable disassembly of each specimen for re-use of the materials. It also was understood that the steel rail contributed some resistance, at least that due to its centroidal moment of inertia, as an offsetting factor.

Table 1.2: Theoretical Deflection Models of the Representative Beam Models

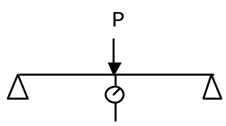
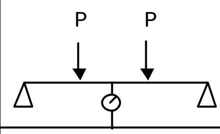
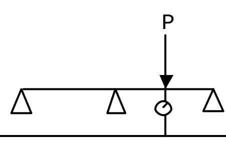
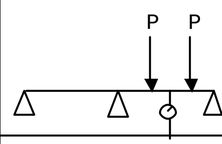
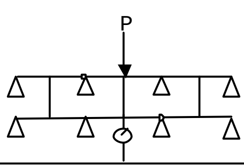
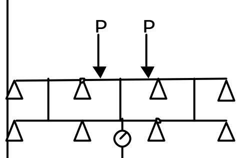
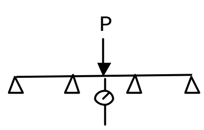
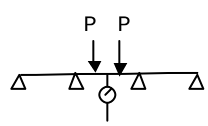
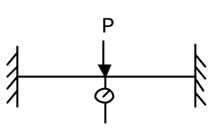
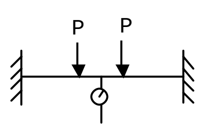
Beam Models	**Pictured Illustrations**		**Deflection Equations**	
	One-Point Load	**Two-Point Load [a]**	**One-Point Load**	**Two-Point Load**
Simply Supported Beam			$\Delta = \frac{PL^3}{48EI}$	$\Delta = \frac{PL^3}{46EI}$
Two-Span Continuous Beam			$\Delta = \frac{PL^3}{67EI}$	$\Delta = \frac{PL^3}{67EI}$
Three-Span Semi-Continuous Beam			$\Delta = \frac{PL^3}{73EI}$	$\Delta = \frac{PL^3}{76EI}$
Three-Span Continuous Beam			$\Delta = \frac{PL^3}{87EI}$	$\Delta = \frac{PL^3}{94EI}$
Fixed-Fixed Beam			$\Delta = \frac{PL^3}{316EI}$	$\Delta = \frac{PL^3}{316EI}$

Table 1.3: Calculated “n” Values for the 3-Span, 2-Span and 1-Span Bridge Chords

Bridge Chords	1-Point Load Test	2-Point Load Test
3-Span Bridge Chord	n = 51	n = 39
2-Span Bridge Chord	n = 50	n = 40
1-Span Bridge Chord	n = 75	n = 81

To proceed with the study, it was decided three steps were needed. One, the modeling should be modified in such a way to confirm results of the laboratory testing as currently known. Two, one or more of the specimens should be reconstructed so as to eliminate the uplift issue and retest it. Three, the modified model should be adapted to the conditions of the new test set-up and compare its predictions to the measured results of the reconstructed specimens. Step one actually was done subsequently as part of phase 1 of the work. Consequently, modified beam models were developed and are described below. The other two steps are to be done in phase 2 of the project and are not addressed in this report.

The modified models used to predict the behavior of the three-span bridge chord are shown in Figures 1.47-50. In each case, the top member represents the addition of the steel rail and the bottom member represents the timber chord. Vertical rods are employed to simulate the actual tied down locations from the steel rail to the chord. In Figures 1.47 and 1.48 the ties are modeled by frame elements, i.e. the connection is considered rigid at each end. In Figures 1.49 and 1.50 the rods are treated as hinged at each end. The actual condition is in between these limits. Each solid dot constitutes a normal node location and each open dot constitutes a hinge location where the rail steel rail sections are butted. Although shown at the top of the schematic, the loads are actually applied to the timber chord member, to be consistent with the prior model.

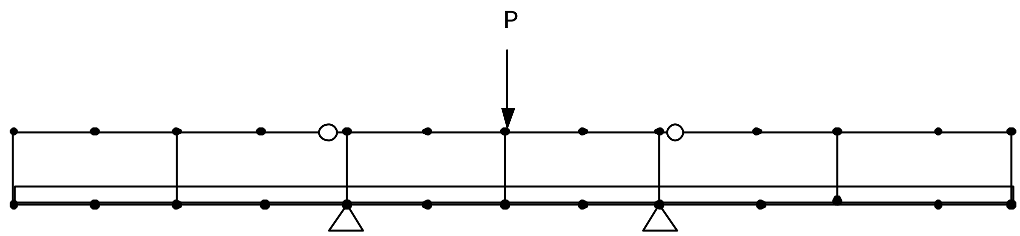


Figure 1.47: Illustration of 3-Span Bridge Chord, Rods are Frame Elements, 1-Point Load Beam Model

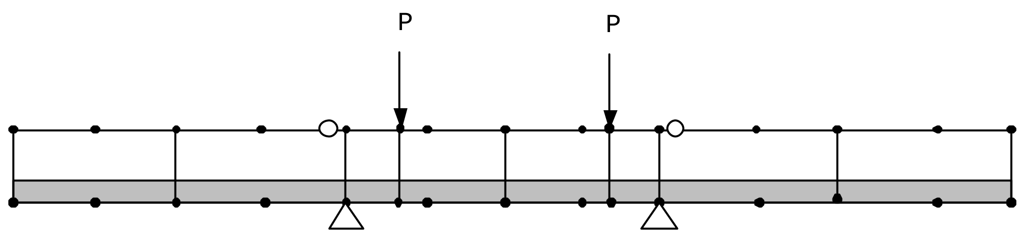


Figure 1.48: Illustration of 3-Span Bridge Chord, Rods are Frame Elements, 2-Point Load Beam Model

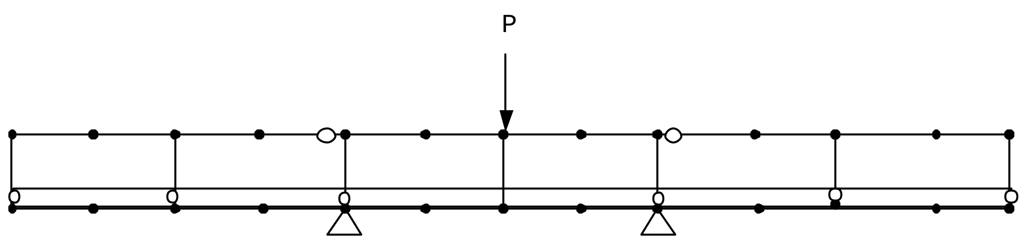


Figure 1.49: Illustration of 3-Span Bridge Chord, Rods are Hinge Elements, 1-Point Load Beam Model

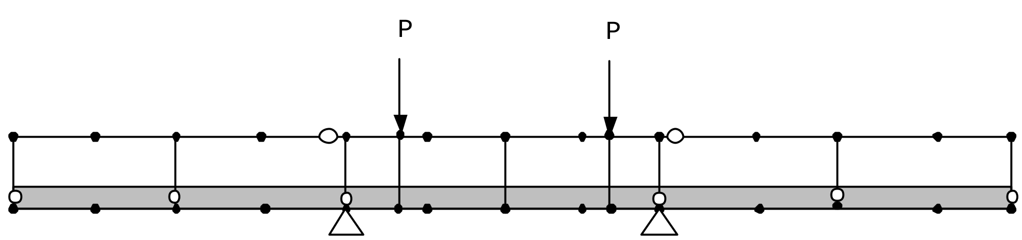


Figure 1.50: Illustration of 3-Span Bridge Chord, Rods are Hinge Elements, 2-Point Load Beam Model

The following assumptions were made:

1. An E value was obtained for each span of the chord member by averaging the values for the plies in that span. Thus, each span had a different value (in the initial models, all spans had the same E value).
2. The I value for each span was based on adding the I values of the plies in that span. Thus, each span had a different I value (in the initial models, all spans had the same I value).
3. Gaps between members were not considered.
4. Support motion was not considered.
5. The overhangs allowed for the end uplift to be included.
6. The steel rail was included as a separate member vertically connected to the timber chord at each support, the free ends, and each mid-span of the cantilevered ends. The vertical connector members were 3/4" diameter threaded steel rods.
7. The steel rail was hinged over each interior support.
8. At each load point, a vertical element with infinite axial stiffness was used to connect the steel rail to the chord.
9. The steel rail was schematically placed 18" above the schematic timber chord members, i.e. at the approximate distance between the two centroids.
10. The dead load of the system (estimated to be 369 lb/ft) was applied as a uniform load on the timber chord member.

Nodes were located to correspond with locations where deflection was measured in the corresponding laboratory specimen.

The modified models used to predict the behavior of the two-span bridge chord are shown in Figures 1.51 to 1.54 for each of the load cases. The modified models used to predict behavior of the single-span bridge chord are shown in Figures 1.55-1.58. The modeling aspects are the same as for the three-span model, except for the removal of the appropriate overhanging members.

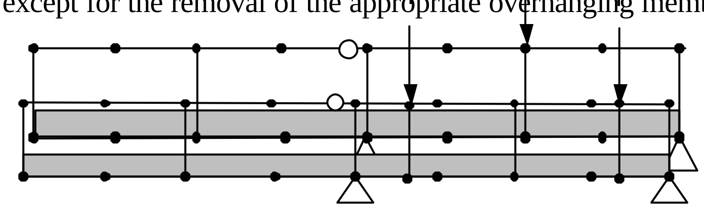


Figure 1.52: Illustration of 2-Span Bridge Chord, Rods are Frame Elements, 2-Point Load Beam Model

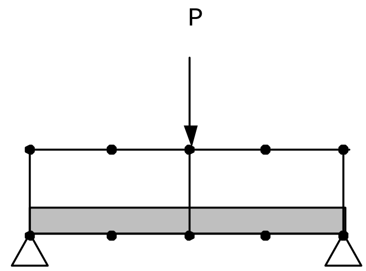


Figure 1.55: Illustration of 1-Span Bridge Chord, Rods are Frame Elements, 1-Point Load Beam Model

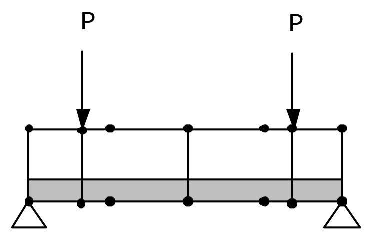


Figure 1.56: Illustration of 1-Span Bridge Chord, Rods are Frame Elements, 2-Point Load Beam Model

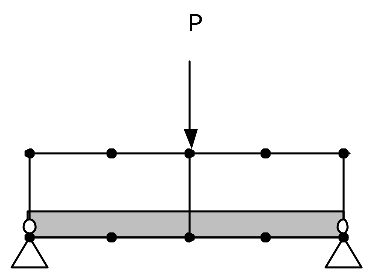


Figure 1.57: Illustration of 1-Span Bridge Chord, Rods are Hinge Elements, 1-Point Load Beam Model

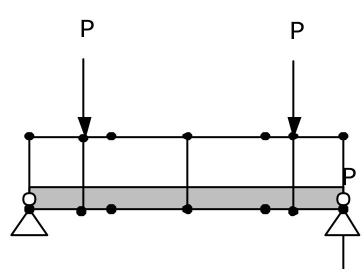


Figure 1.58: Illustration of 1-Span Bridge Chord, Rods are Hinge Elements, 2-Point Load Beam Model

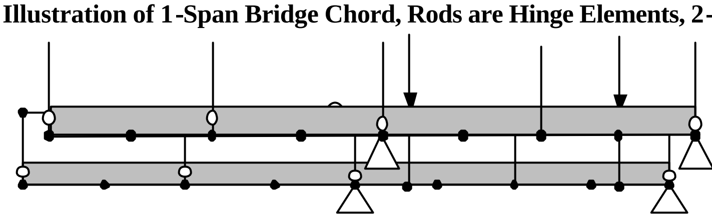


Figure 1.54: Illustration of 2-Span Bridge Chord, Rods are Hinge Elements,

COMPARISON OF ANALYTICAL AND EXPERIMENTAL RESULTS

As the above models were based on collapsing multiple plies into single member, the averaged measured deflections for the plies in a span were used as that span's deflection. Only the four-ply specimens were studied. The average measured deflections were based on linear regression fits to the average ply data taken at the quarter points and mid-spans. From these regression fits, deflection profiles were plotted for each specimen and loading. The deflection profiles were obtained by fitting a 4th order polynomial in each span to the quarter point and mid-span averaged values. The result is referred to as the "measured deflection profile" although only a few locations actually were measured.

Figure 1.59 shows the resulting measured deflection profile of Chord 3/4 for the single point load case. It is plotted with triangles. It can be observed that downward vertical support motion occurred at the interior supports, as non-zero displacements result from the polynomial fit. This motion is attributed to a combination of gaps between members closing and deformation of the cap and pole stub members. The raw data shows slight uplift at the overhanging ends. The inclined reference line passes through the actual displaced positions of the interior supports. Relative to this position (i.e. had the supports not displaced) the overhanging ends would have lifted by about .1” to .2". Figure 1.59 also shows the predicted deflection using an ideal three-span continuous beam without support motion or uplift and no steel rail, but different EI values for each span. That model clearly is inadequate.

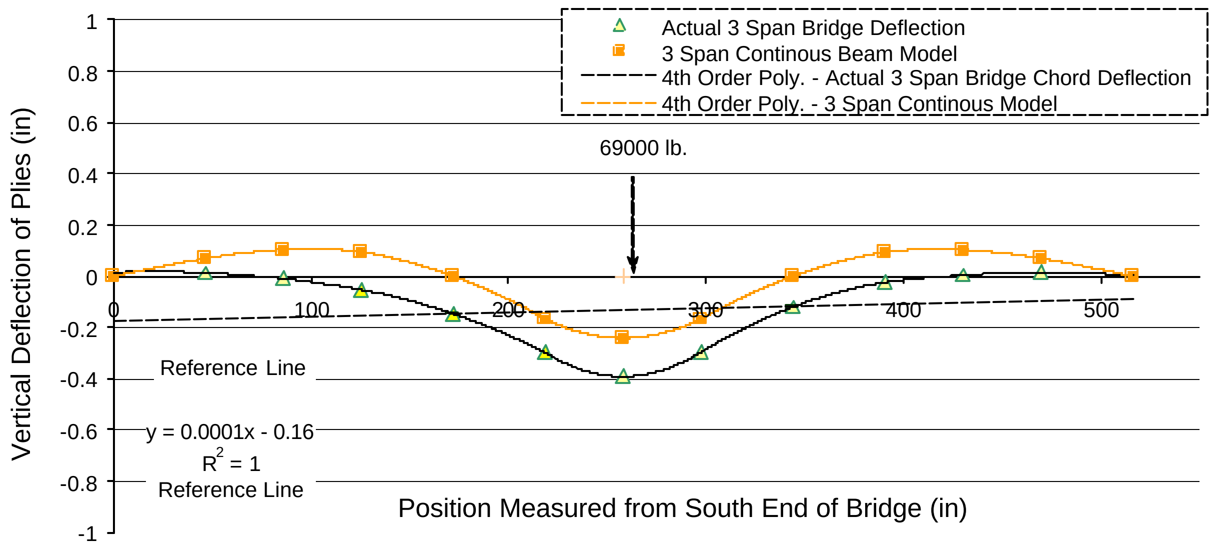


Figure 1.59: Chord 3/4 - 1-Point Load, Deflection Profile for 3-Span Continuous Beam Model

Figure 1.60 shows the measured deflection profile for Chord 3/4, reference line, and results of the modified beam analyses. One model (shown in diamonds) assumes the vertical ties are hinged elements. The other model (shown as X-X-X) assumes the vertical ties are frame elements. To make a comparison, one must remove the support motion from the measured deflection profile, i.e. one must shift the reference line to the horizontal orientation. If done, then within the loaded span the hinged tie model gives a close prediction, while the hinged tie model result is not close. Within the non-loaded spans the shifted measured deflection profile lies roughly 40 percent and 60 percent below (60 percent and 40 percent above) the hinged tie (framed tie) result at the left and right overhangs, respectively. "Below" and "above" infer the proportions of the distance between the two bounds of the predicted values.

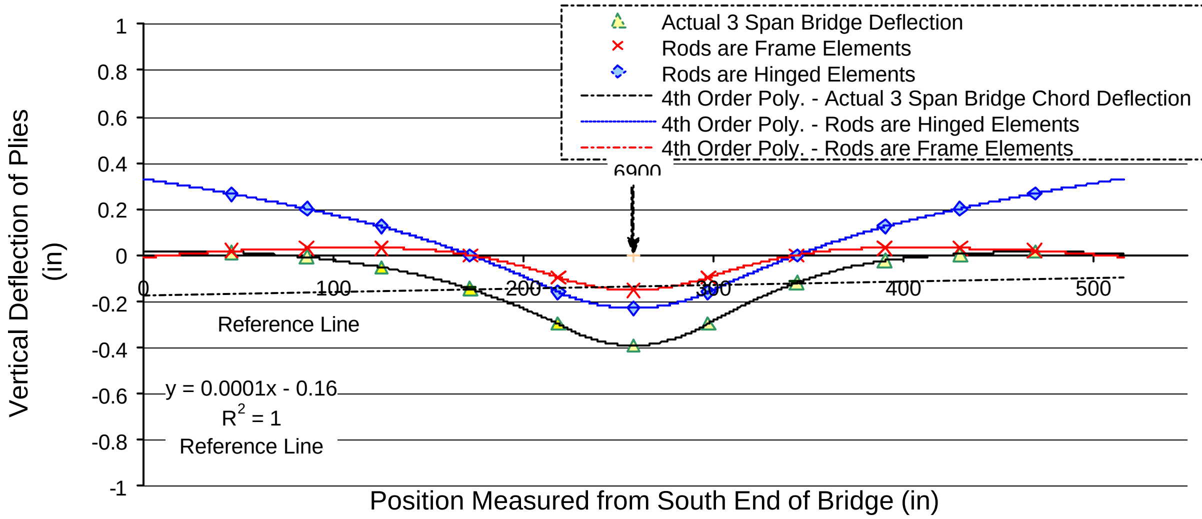


Figure 1.60: Chord 3/4 - 1-Point Load, Deflection Profile for Hinged and Framed Models

Figure 1.61 is the counterpart to Figure 1.60 for the two-point load case. In the loaded span, the hinged tie model gives a close prediction to the shifted measured result, while the hinged result is not close. In the non-loaded spans the shifted measured result lies roughly 25 percent and 60 percent below (75 percent – 40 percent above) the hinged tie (framed tie) result at the left and right overhangs, respectively.

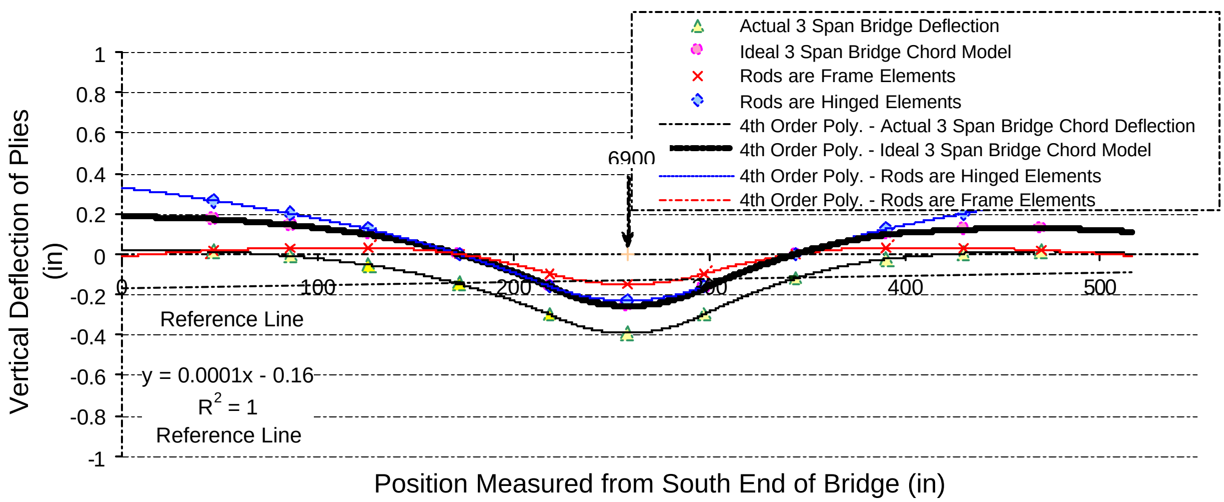


Figure 1.61: Chord 3/4 - 2-Point Load, Measured Deflection Profile

Figure 1.62 shows the resulting measured deflection profile of Chord 2/4 for the single-point load case. It is plotted with triangles. It can be observed that downward vertical support motion occurred at the interior support, as a non-zero displacements results from the polynomial fit. The reference line passes throughout the actual displaced positions of the interior supports. Relative to this position the overhanging end would have lifted by about .22". Figure 1.62 also shows the predicted deflection using an ideal two-span, continuous beam without support motion or uplift and no steel rail but different EI values for each span. That model clearly is inadequate.

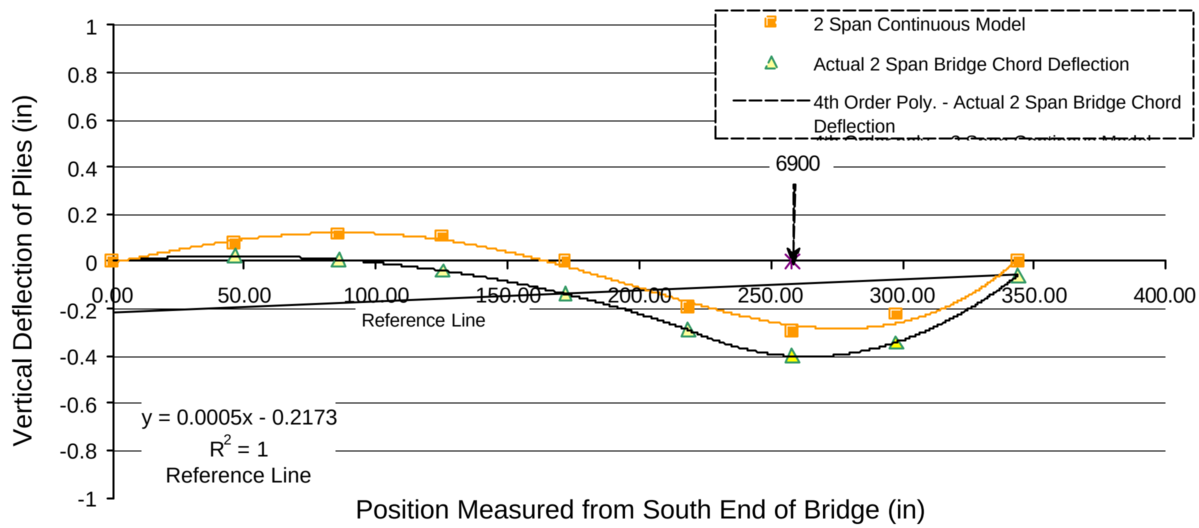


Figure 1.62: Chord 2/4 - 1-Point Load, Deflection Profile for 2-Span Continuous Beam Model

Figure 1.63 shows the measured deflection profile for Chord 2/4, shifted reference line, and results of the modified beam analyses. One model (shown in diamonds) assumes the vertical ties are hinged elements. If one shifts the reference line of the measured deflection profile to the horizontal position, then in the loaded span the hinged tie model gives a close prediction, while the hinged result is not close. In the non-loaded span the shifted measured result lies roughly and 60 percent below (40 percent above) the hinged tie (framed tie) result at the le ft and right overhangs, respectively.

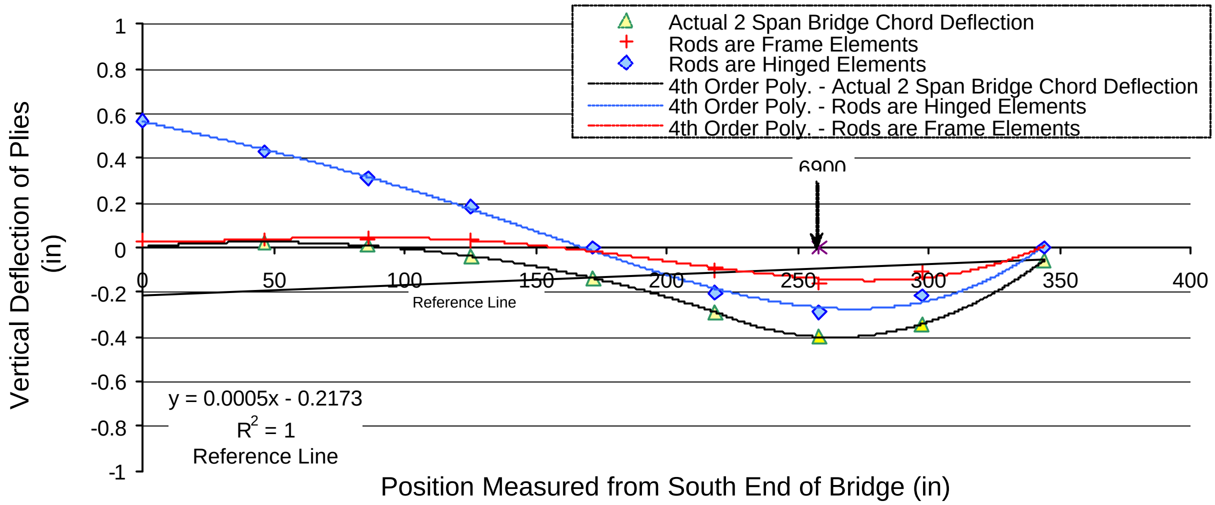


Figure 1.63: Chord 2/4 - 1-Point Load, Deflection Profile for Hinged and Framed Models

Figure 1.64 is the counterpart to Figure 1.63 for the two-point load case. In the loaded span the hinged tie model gives a close prediction to the shifted measured result, while the hinged result is not close. In the non-loaded span the shifted measured result lies roughly 55 percent below (45 percent above) the hinged tie (framed tie) result at the left overhang.

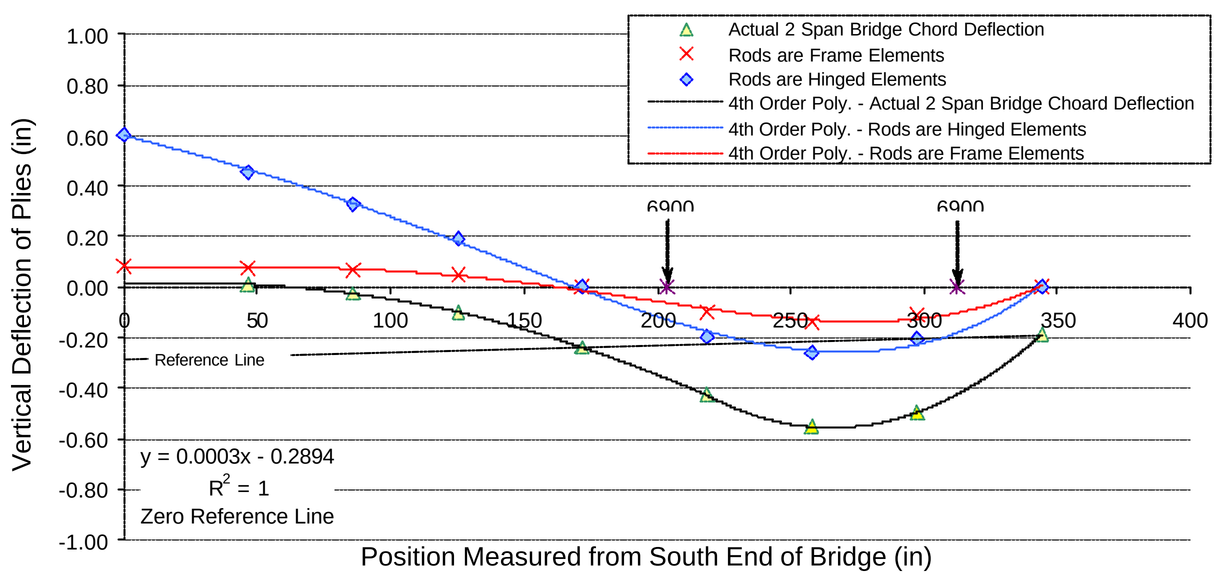


Figure 1.64: Chord 2/4 - 2-Point Load, Deflection Profile for Hinged and Framed Models

Figures 1.65 and 1.66 illustrate the measured deflection profiles for Chord 1/4 for the one-point load and two-point load cases, respectively. The end support locations were based on the clear distance between the interior faces of the actual physical supports, i.e. 158". This was done because in the actual test the ends of the plies rotated and were bearing on the inside corners of the caps. The "square" symbols along the horizontal axis represent these corner locations. The reference line for the measured deflection profiles indicate downward support motion of about .10" to .15" occurred in the tests. The analytical results for the hinged tie and framed tie models also are shown, and essentially are coincident to each other, which is due to the single-span configuration, i.e. with the timber chord being statically determinate. Thus the ideal simply-supported beam model was not done, as it also would be essentially coincident with the other models. If the reference line for each measured deflection profile is shifted to the horizontal, it is seen that the results are above those predicted by the model. The adjusted measured values at mid-span are about 70 percent to 80 percent of the predicted values. Hence the model is too flexible.

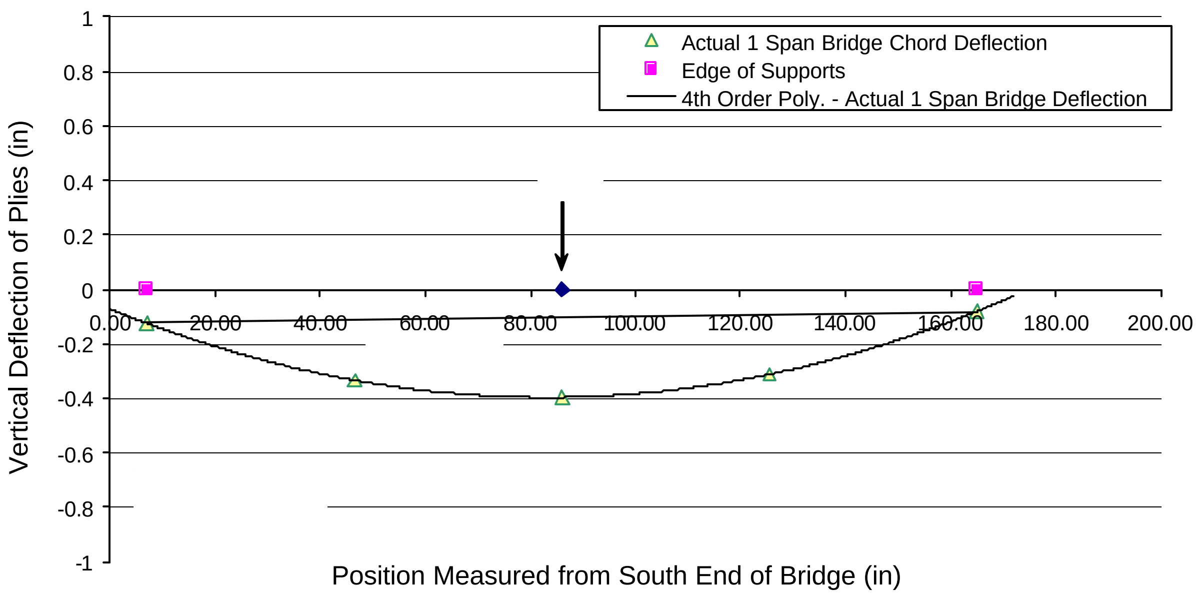


Figure 1.65: Chord 1/4 - 1-Point Load, Measured Deflection Profile

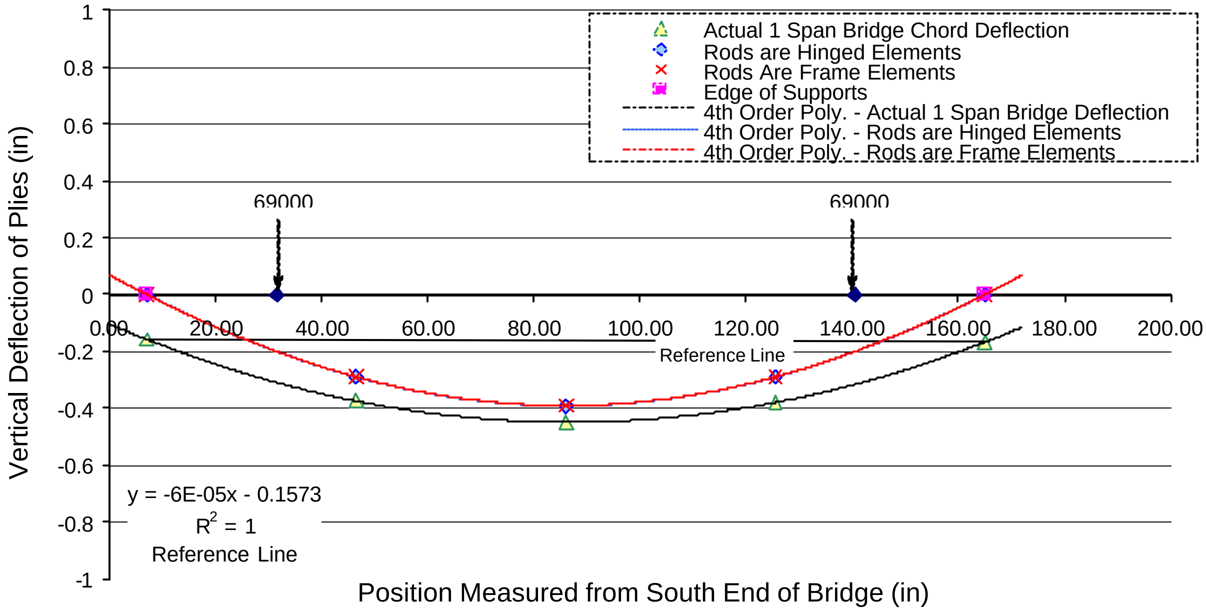


Figure 1.66: Chord 1/4 - 2-Point Load, Deflection Profile for Hinged and Framed Models

EFFECT OF FIELD CONDITIONS

As noted earlier, some of the bridge chord specimens were subjected to intentional modifications of physical conditions of bearing support, removal of ties, and centering and not centering the steel rail over plies. The outcomes of these investigations are described below.

Differential Bearing

Chord 2/4 was retested after several adjustments of the bearing conditions. Figure 1.67 illustrates the three cases examined in this study and the reference number for the plies. Initially, all plies were shimmed with 3/4" thick plywood. In each of the three alterations, plywood was removed at one location. In Case 1, the plywood was removed from the end support of ply 1. In Case 2, plywood was removed from the interior support of ply 1. In Case 3, plywood was removed from the interior support of ply 2. These were individual events, not cumulative. The loading was a single point load in the right span. In each case ply tie rods were in place at mid-span.

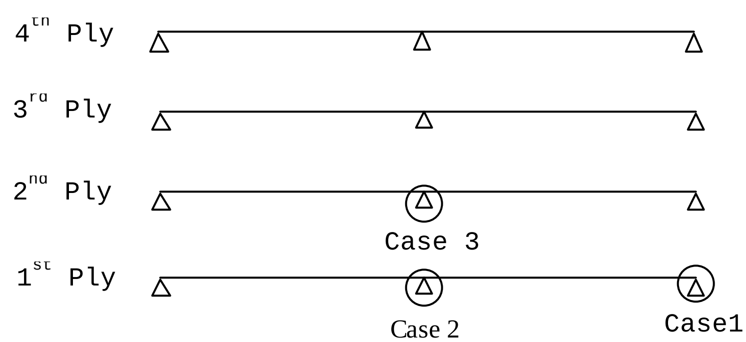


Figure 1.67: Locations of Unsupported Plies – 2-Span Bridge Chord

Figure 1.68 illustrates the mid-span ply deflections of the loaded span for the condition of all plies supported vs. each of the three cases. In Case 1, as expected, ply 1 deflected the most. The adjacent plies 2, 3, and 4 each had less deflection each as one moves away from ply 1. Except for ply 4, which is farthest away from ply 1, the ply deflections increased relative to the base case of all plies supported by plywood. Thus, the greater flexibility of ply 1 contributed to higher overall deflection, which diminished as one moves farther way laterally. Note that in the base case, exterior plies deflected more than the interior plies, even though one might expect an exterior ply to carry less share of the load than the interior plies. However, this latter point is not necessarily true, as reported by Gutkowski et al. (1999).

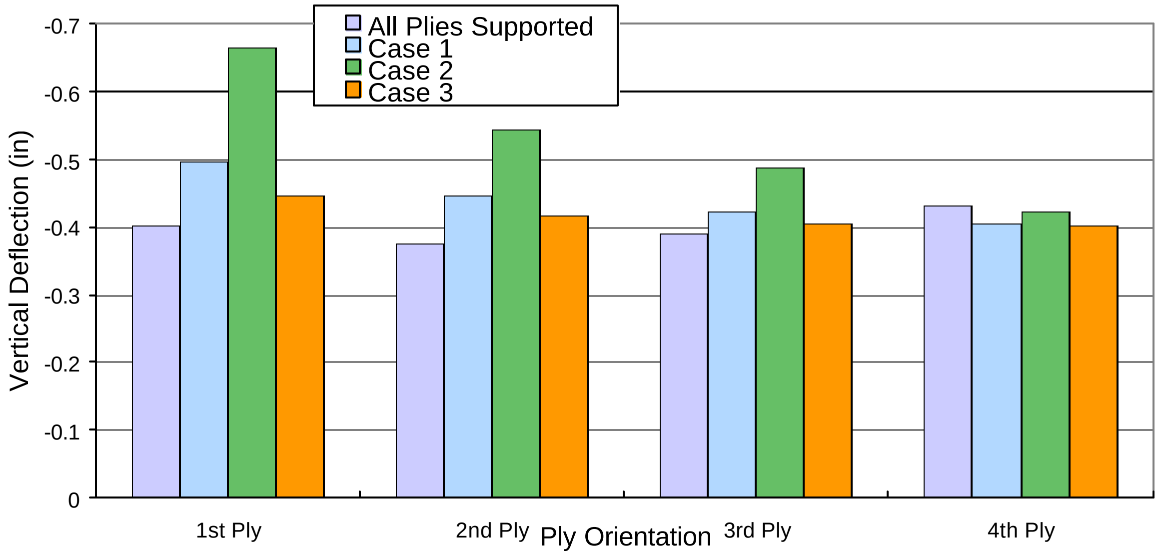


Figure 1.68: Mid Span Deflections of Loaded Span, 1-Point Load Tests

In Case 2, the same ply deflection pattern is evident as in Case 1 except all deflections are higher. In the ideal case of a two-span continuous beam with constant EI, the interior support would carry 69 percent more reaction than the exterior support, so the observed deflections appear reasonable.

Case 3 also exhibits the same trend of ply deflections as in Cases 1 and Case 2, but with less deflection than for either of them. However, except for ply 4, the deflections increased relative to the base condition. The base case suggests that the exterior plies carried moderately more load share than the interior plies, as one factor to consider. It is also surmised that possibly the ply tie rods were tight enough to have adjacent plies be forced to move somewhat together, i.e. plies 1 and 3 may have helped support the relaxed ply 2.

Centering vs. Not Centering the Steel Rail

Chord 1/4 originally was tested with the steel rail centered over the four plies. It was then tested in the same load location after adding a ply to the outside of the first ply (ply 1). Then the steel rail was centered over the five plies and the specimen was retested, again. The single-point load was used in each case. For the third case the specimen was shifted laterally to center it under the fixed load point. Thus, its support points were altered.

Figure 1.69 summarizes the results. For the original configuration, ply deflections were moderately different, with the interior plies deflecting the most. Ply 4 deflected the least.

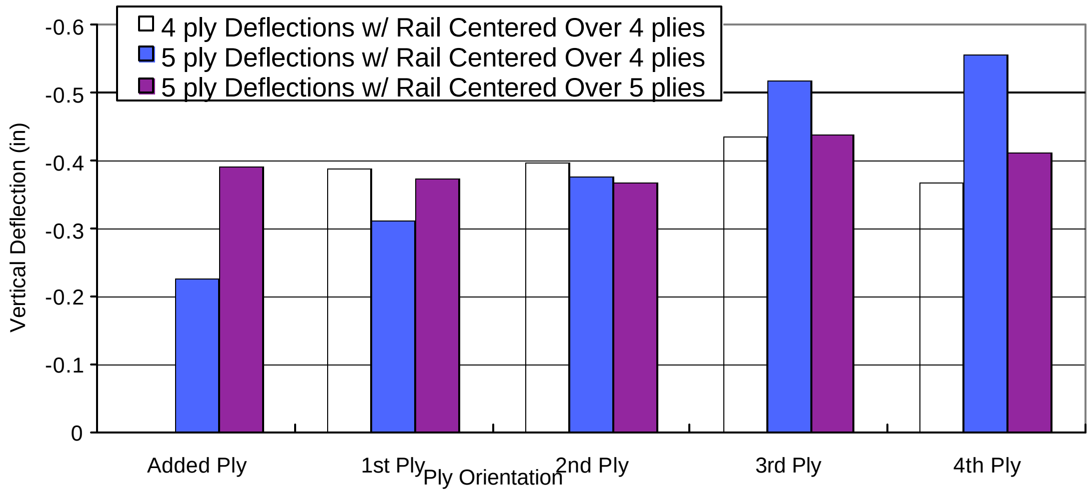


Figure 1.69: 1-Span Bridge Chord, Effects of Rail Placement

With a ply added next to ply 1 the new ply picked up a share of the load, but one much lower than the rest, perhaps because it was farthest away from the load and also may have not have been a tight fitting ply. The load shares of the nearest plies, ply 1 and ply 2 decreased, which is rational with the added ply relieving them of some load. Inexplicably, though, plies 3 and 4 had a significant increase in deflection. This is not plausible, unless upon reload some free motion occurred in the plies due to vertical slippage of the ends. Thus, measurement error may have occurred.

With the steel rail shifted to be centered on the five plies, the added ply and ply 1 experienced an increase in deflection, relative to the eccentric rail case. Again, it is noted that the specimen itself had to be shifted. Ply 2 (now the middle ply) had modest decrease in deflection (this is not rational, unless slippage occurred as the load was placed on steel rail above this ply). Plies 3 and 4 had a decrease in deflection, which is rational for having reoriented the load toward the added ply and ply 1 and away from ply 3 and ply 4. In absolute terms, the ply deflections are reasonably balanced, but the exterior plies deflected more than the interior plies 1 and 2. Ply 3 had the highest deflection.

Ply Tie Rod Configuration

Chord 1/4 was altered to produce several tie rod configurations. The chord was tested with: 1) one ply tie rod placed at mid-span, 2) no tie rods in place (i.e removing the mid-span ply tie rod), and 3) with ply tie rods installed at the quarter points and at mid-span. The single-point loading was applied three times and the average results used for assessment.

Figure 1.70 is a plot of the results. The ply deflections essentially are the same for both cases of ply tie rods present. Thus, ply tie rods added at the quarter point locations had no effect. With no ply tie rods present, deflections of all three plies increased moderately, up to a 6 percent increase.

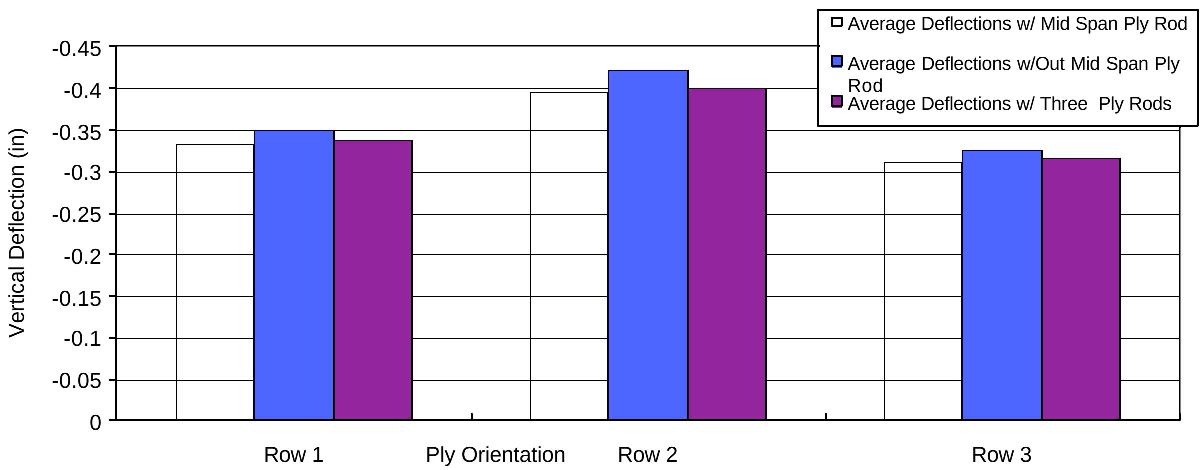


Figure 1.70: 1-Span Bridge Chord, Effects of Ply Tie Rods

CONCLUSIONS

The main conclusions from phase 1 of the research project are:

- The unintended occurrence of support uplift compromised intended comparisons with actual field performance. However, the data obtained from the laboratory tests allowed progress to be made in structural modeling.
- The simple beam models establish bounds on the response of the specimens. However, they neglect support motion (due to gaps and support deformation) that occurred at interior supports. and the uplift of the end supports. Thus, the specimens were not modeled successfully by them.

- Modified beam models were developed to simulate the actual test specimen behavior in the presence of end support uplift. The addition of the steel rail and consideration showed that if the interior support motion was removed the interior loaded span could be modeled closely, but not the end spans.
- The presence of ply tie rods had only a modest influence on deflection response.
- Addition of an exterior ply to the single span chord relieved the adjacent plies of some load share and, thus, was beneficial.
- Centering of the steel rail in Chord 1/5 resulted in the expected shift in load shares, leading to more balanced distribution of the load among the plies.

RECOMMENDATIONS

Based on observations made in this report, the next phase should include consideration of the following recommendations:

- It is desirable to reconstruct one or more of the specimens and adjust end supports to prevent uplift. Effort should be taken to eliminate support motion due to gaps before the retest. The steel rail should be attached to replicate field practice. If disassembly becomes an issue, and only one retest is possible, the three-span specimen is the priority as it can be compared with field test of Bridge No. 101.
- The modified beam models should be adapted to the revised specimens and examined for suitability for modeling them.
- If downward support motion due to gaps is not prevented, the model should be extended to include it.
- Each support area for ply reaction forces and ply end displacements so direct data is available.
- More rigorous analytical models should be considered, such as a 3-D space structure including the steel rail and wood cross-ties.

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