# An Experimental Evaluation of Autostress Design

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ABSTRACT

Autostress design has been proposed as an economical and rational method for designing steel bridges. It is an extension of existing AASHTO load factor design, and it utilizes three load level limit states. Limited plastic redistribution of load is permitted during overload and maximum load limit states, because of the ductility of steel. However, the deflections are controlled to assure continued serviceability. This concept is somewhat new for bridge design, and extensive research has been performed to substantiate the method. However, it was believed that a full-scale field test was needed to assure that the autostress method performs as expected under practical conditions. This paper describes such a test program. In this paper, the bridge, instrumentation, and test program are described, and the test results are discussed and analyzed. Theoretical predictions are made and compared to the test results, and long-term observations of the bridge behavior are noted. The study shows that the autostress method performs well under service load and overload. Plastic deformations occurred but they were controlled. Therefore, permanent deflection and cracking of the concrete deck were minimal, and the design method should result in satisfactory performance of the bridge for many years.

Autostress design (ASD) has been proposed (1) as an economical and rational method for designing steel bridges. It is an extension of existing AASHTO (2) load factor design (LFD) and it utilizes three load level limit states. Under service load, the ASD method has identical design provisions to the LFD method including factors such as fatigue and deflection control. However, steel bridges are ductile and may have considerable reserve strength, because of plastic redistribution of loads, and so ASD permits limited yielding during overloads that may occur a few times in the life of the structure. The deflections caused by yielding, however, are controlled to assure continued serviceability. The maximum load occurs only once or twice in the life of a bridge, and so the ASD method permits plastic analysis methods for this load condition.

The concept of deliberate yielding is widely understood in some types of structural design, but is new for bridge design. Therefore, extensive research (3-8) has been performed to substantiate the method. Testing (6,8) of full-scale components has been performed to verify the load capacity and rotational ductility of bridge girders. A scale model bridge has been tested (4,7) to verify the general concept, and linear and nonlinear analyses have been performed. Thus, the ASD method has been well documented, but a full-scale field test was needed to assure that the ASD method performs as predicted under practical field conditions.

The Whitechuck River Bridge was this test structure (9,10). The bridge was designed by the ASD method and was constructed in 1982. A nondestructive load test, which simulated overload and induced yielding, was performed immediately after construction. Strains, deflections, temperatures, and concrete deck cracking were measured and observed during the load test, and a series of long-term observations were taken for the 2-year period after the load test.

The results of this test program are presented in this paper. The bridge is described and the instrumentation used during the study is noted. The load test is discussed and the test results are analyzed. Theoretical predictions are made and compared to the test results. Finally, a series of long-term observations are described and compared to the earlier results. This study shows that a bridge designed by using the ASD method may perform well under service load and overload. Plastic deformations occurred but they were controlled. Therefore, permanent deflection and cracking of concrete bridge decking were minimal and so deterioration of the bridge and bridge deck should not be excessive for this method.

## THE TEST BRIDGE

The Whitechuck River is located in the Glacier Peak area of Mount Baker and Snoqualmie National Forest near Darrington, Washington. The river is crossed by an unpaved road that is heavily used by logging trucks and recreational traffic. There are minimal constraints on the speed and weight of the logging vehicles. On occasion, the road is used by heavy timber yarding equipment that may overload the structure. A timber truss bridge crossed this river before 1982, but the U.S. Forest Service determined that it was not up to the required standards. As a result, the new bridge was designed by the ASD method through the joint efforts of the U.S. Steel Research Laboratory and the Denver Office of the U.S. Department of Transportation's Office of the Western Bridge Design. The resulting structure is a single-lane, 3-span bridge with two continuous, composite girders and an orthogonal crossing as shown in Figure 1. The wide flanges are made of ASTM A588 (AASHTO M222) steel and are relatively shallow because of the economy achieved with the ASD Method.

The new bridge was designed for AASHTO HS-20 service loads. The overload limit state used a 1.0 load factor applied to the axle loads shown in Figure 2 plus impact, and the maximum load limit state used a load factor of 1.3 applied to the overload. ASD permits controlled yielding during overload and plastic limit analysis behavior during maximum load.

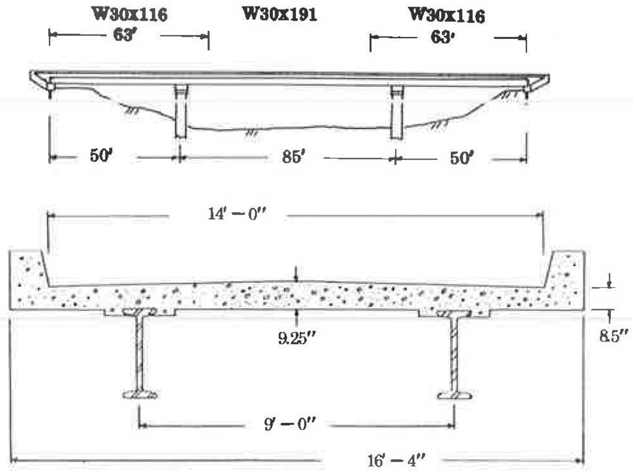


FIGURE 1 Geometry of the Whitechuck River Bridge.

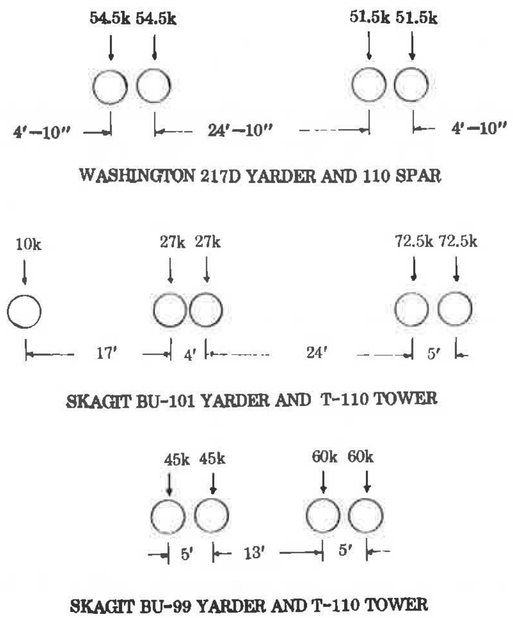


FIGURE 2 Design vehicle axle loads for overload limit state.

This yielding causes residual stress and strain and permanent deflections. The designers estimated a permanent deflection of 0.69 in. for the design overload, and the bridge girders were cambered for this deflection.

INSTRUMENTATION

Because a load test was performed to simulate overload and induce limited yielding before the bridge was opened to normal traffic, all instrumentation was installed during the construction process. Stainless steel bolts were machined, painted, and bolted to the web of the steel beam to provide accurate, visible targets as shown in Figure 3. Twenty targets were attached to each beam at 9-10 ft intervals, and deflections of the targets were measured to approximately 0.01 in. accuracy with theodolites placed at one of four different stations. The theodolites typically had a sight distance of less than

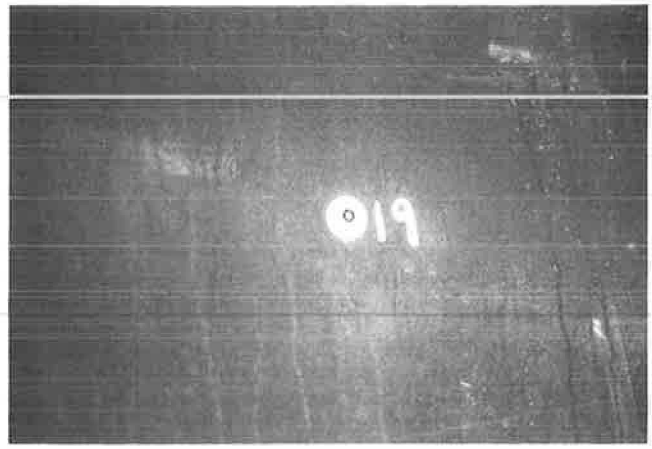


FIGURE 3 Typical theodolite target used for deflection measurements.

200 ft, so a 0.01 in.-deflection approximately coincided with 1-sec of angle. The theodolite supports were steel pipes filled with concrete and anchored into a concrete base.

One hundred and twenty strain gauges were attached to the bridge to measure strain level, locate neutral axis, determine initiation of yielding, estimate the effective width of the composite slab, and evaluate bending moments. Ninety-six of the gauges were attached to the steel, wide flanges in groups of six at eight locations of each girder as shown in Figure 4. This arrangement permitted redundancy of measurement and provided an estimate of

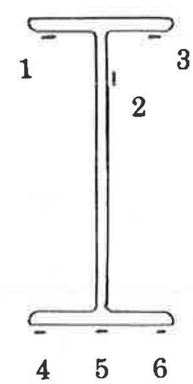


FIGURE 4 Typical strain gage configuration on the steel beams.

the distribution of strain over the beam depth and flange width. Two of the six groups were in the zone of yielding and were installed to estimate the initiation of yielding and locate the neutral axis. The other six locations were in areas that remained elastic, and they were also used to estimate bending moments. Four strain gauges were attached to longitudinal reinforcing bars over the bridge piers and steel girders. They showed the tensile strains in reinforcement and these strains were correlated to tensile cracking in regions of negative bending of the bridge deck. Twenty additional gauges were embedded in the bridge deck to measure concrete strains and evaluate the effective width. The strains were measured with a Hewlett Packard HP85 computer and HP 3054A data acquisition system.

## LOAD TEST

Four thermocouples were used to measure the temperature of the steel girders and extensive material property tests were performed. Test cylinders were taken from each concrete mixer load for the deck, cured at the bridge site, and tested within 2 days of the load test. The average strength and elastic modules of the in-situ concrete were much larger than design values with results of 6.5 ksi and 4,600 ksi, respectively. Properties of the steel wide flanges and reinforcing bar were also measured and are given in Tables 1 and 2. Wheel loads of the test vehicles were measured with portable scales that were calibrated before the test. Finally, the as-built geometry was measured before testing and tension cracks in the concrete deck were monitored before, during, and after the test.

TABLE 1 Properties of Steel Reinforcing Bar

| | Bar Size | |
|---|---|---|
| Properties | No. 5 | No. 6 |
| Yield stress (ksi) | 84 | 73 |
| Modulus of Elasticity Based on Nominal Area (ksi) | 35,000 | 36,000 |

TABLE 2 Properties of Steel Shape

| | Heat Number | |
|---|---|---|
| Properties | X49411 | K49721 |
| Yield Stress (mill report) (ksi) | 56.4 to 62.9 | 57.3 |
| Tensile Strength (mill report) (ksi) | 73.8 to 78.3 | 75.7 |
| Structural Shape | W30 x 116<br>W30 x 191 | W30 x 191 |
| Static Yield Stress (ksi) | 50.1 | 50.5 |
| Elastic Modulus (ksi) | 29,000 | 29,000 |

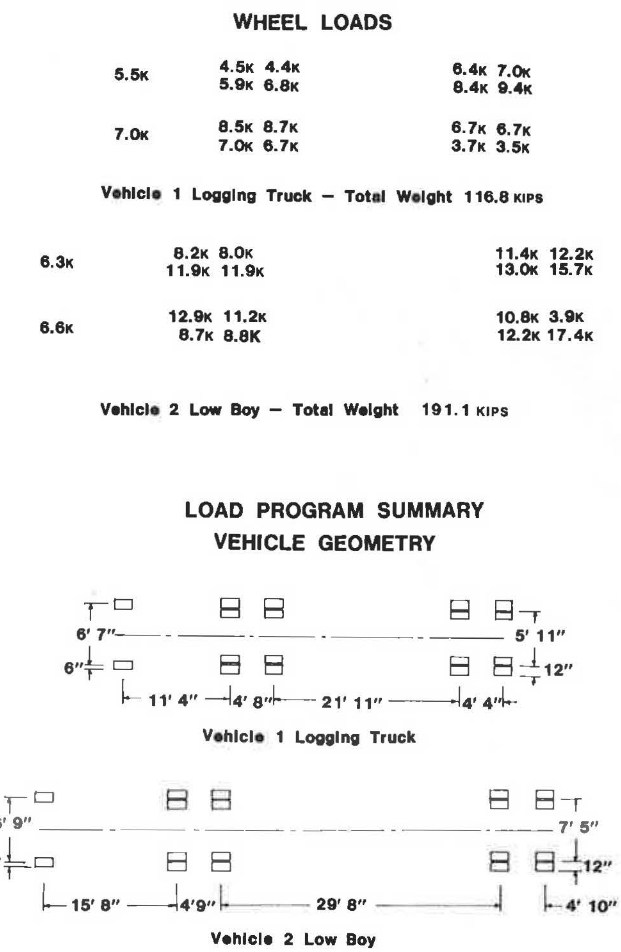

FIGURE 5 Wheel weights and axle geometry for two test vehicles.

The temperature during the test, which was performed in November 1982, was reasonably constant (approximately 30°F) during the test, and so thermal strain had little effect on the results. Two load vehicles were used with wheel loads and geometry shown in Figure 5. The original intent had been to use a single vehicle with the axle loads and spacing of the Skagit BU-99 unit shown in Figure 2, but at the time, no such vehicle was available. Discussions with representatives of logging firms in the area suggest that the axle loads of Figure 2 do not represent a real vehicle, since yarding equipment is usually custom built to customer specifications around a general model. Thus, the two vehicles of Figure 5 were a compromise selection. Both vehicles exceeded the normal HS-20 service load condition, and the combination of the two vehicles simulated the overload limit state.

Eighteen major load points, LP1 to LP18, and eleven minor load points (LP1A, LP2A, etc.) were used in the load test. LP1 through LP3 simulated service load conditions and no yielding was expected, since a single vehicle was used. Although a single vehicle was used in LP4 and LP5, limited yielding was expected because the loads were applied at critical locations (near the curb and with the heavier, longer vehicle straddling the piers). LP6, 7, 8, and 9 all used both vehicles back-to-back and near the curb as shown in the photo of Figure 6. Significant yielding was expected during LP6 and LP7, but little or no yielding was expected during


FIGURE 6 Photograph of two test vehicles applied simultaneously back-to-back during LP6.

LP8 and LP9, because of the Automoment (1,11,12) formed during earlier yielding (i.e., the structure experiences shakedown).

The trucks were moved into position for each of the load points, and strains and deflections were measured after a 15-min delay. The delay permitted completion of all yielding before measurements were made. The concrete deck over the bridge pier was closely examined for tension cracks at each load point. Concrete cracking was smaller and less widespread than expected. A single tension crack formed over approximately the center of each pier during the test (see Figure 7). The cracks started initiation during LP2 and were formed or were visible on the deck surface through LP9. The cracks were not visible on the deck surface after the loads were removed.

**FIGURE 7 Photograph of deck crack marked and observed during LP6.**

## TEST RESULTS

The results of these tests clearly indicate that yielding occurred during the test program. Figure 8 is a plot of the measured deflections of the bridge girders with no load applied at various times during the test. It can be seen that permanent deflections started to develop early in the test and generally continued to increase through LP7A, but remained essentially constant after LP7A. This suggests that yielding started at LP1 and continued through LP7, when shakedown occurred. The maximum permanent deflections were 0.16 and 0.11 in. for the north and south girders, respectively. This permanent deflection is considerably smaller than the 0.6 in. predicted by nonlinear analysis for the test vehicles.

The measured strains also indicated that yielding started early in the test but stopped after LP7. Figure 9 shows the strain in the reinforcing bar in the bridge deck over the steel girders and interior piers with no loads on the bridge. Tensile residual strain develops at these locations when plastic deformation occurs, and Automoments form. These residual strains first developed at the west pier, because LP1 and LP2 used loads on the west and center spans only. After LP7, the residual strains did not change and the structure remained elastic.

The strain gauges on the steel girders were used to locate the neutral axis and measure the curvature. Figure 10 shows the measured permanent curvature for LP2A, LP4A, and LP8A. Curvature generally increases during the early load points and remains essentially constant after LP8A. Curvature at the interior piers is caused by yielding of the steel. The smaller curvatures noted in the spans is elastic curvature caused by positive residual bending moments [i.e., the Automoments (1,12)]. These curvatures are small and of opposite sign to that noted at the piers. The experimental data of Figure 10 are connected with straight lines for simplicity, but the curvature distribution between measured points would be quite different for the actual structure.

The previous data indicate that yielding occurred due to overload of the structure. However, the yielding started sooner than was expected and the permanent deflection and concrete cracking were smaller than was anticipated. Figure 11 helps to illustrate why these unexpected results were noted. This figure shows the reported camber of the steel girders in the fabrication shop, the camber with computed, elastic, dead load deflections removed, and the absolute deflection of the bridge girders at the start of the test. (Note that the deflections shown in Figure 8 and all later figures are relative to this absolute value.) The figure shows that the actual dead load deflection was larger than expected. The girders were cambered by the flame-cambering process, and this process, when combined with hot-

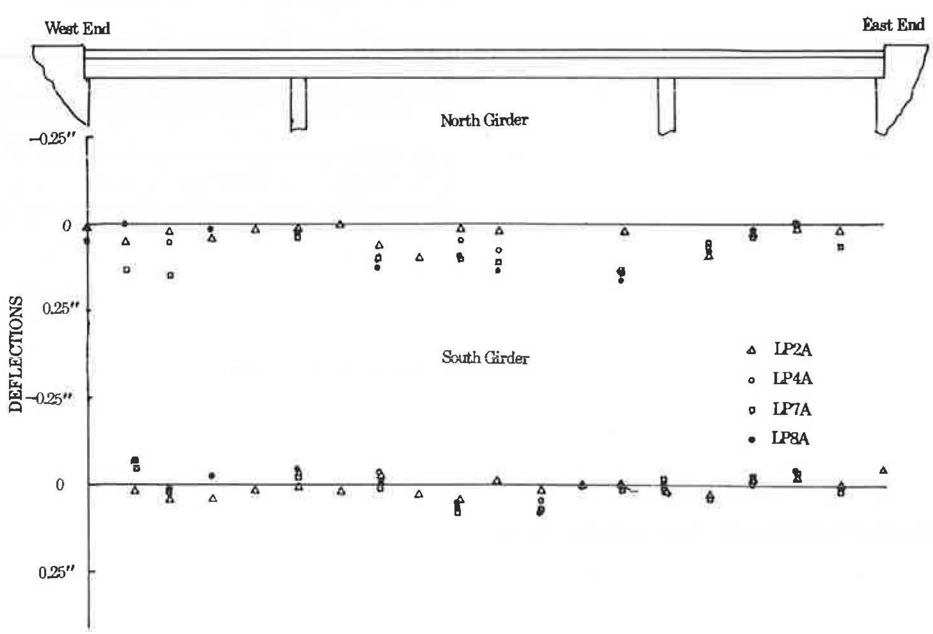


**FIGURE 8 Measured deflections of bridge girders with no loads on the bridge.**

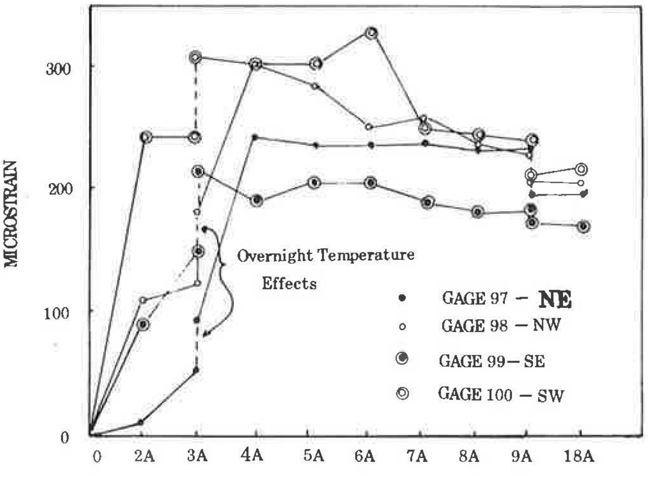


**FIGURE 9 Measured strains in the reinforcing bar in the bridge deck over the steel girders and interior pier with no load on the bridge.**

rolling, introduces residual stresses approaching the yield stress (13). The residual stresses caused the steel to yield during placement of the dead load, and permanent deflection had occurred before any live load was applied. These plastic dead load deflections were larger at the east end because the deck concrete was placed from east to west. During the load test, yielding continued through LP1 to LP7, but the permanent deflections were much smaller than expected during these later loads, because much of the permanent deflection had already occurred. Fabrication residual stresses affect the order and time of yielding but they have no impact on the ultimate strength of the bridge girders (12,13).

COMPARISON OF THEORETICAL PREDICTIONS WITH TEST RESULTS

A series of linear, elastic, theoretical predictions were performed (9,10) using the average measured material properties and bridge geometry, and the results were compared with the experimental observations. Figures 12 and 13 are typical comparisons for the deflections and bending moments, respectively. These comparisons showed that the concrete curb and steel guardrail significantly contributed to the bridge stiffness, and therefore had to be considered in the elastic analysis and prediction of bending moments. Further, the uncracked stiffness of the concrete deck was used even in the region of negative bending. The uncracked stiffness provided good comparison between theoretical predictions and experimental observations and agreed well with the observation that little deck cracking occurred during the load test. The actual in situ concrete had an average strength that was 85 percent larger than the design value. This stiff, strong concrete probably contributed to the minimal cracking that was observed. This stiff material also necessitated the use of uncracked concrete stiffness and contributed to the small permanent deflections, since these permanent deflections are elastic deflections due to positive residual moments (1), such as Automoments.

The distribution of load between girders was also studied. One simple method, which is commonly used in design, distributes the loads to the individual girders by simple transverse equilibrium. This method implicitly assumes that the torsional stiffness of the bridge is warping stiffness, and it resulted in much larger moment and deflection predictions for the heavily loaded girder when the bridge was eccentrically loaded as shown in Figures 12 and 13. A more refined analysis including Saint Venant and warping torsion stiffness (14,15) was then used and much better comparison with experimental data resulted as shown in Figures 12 and 13. The experi-

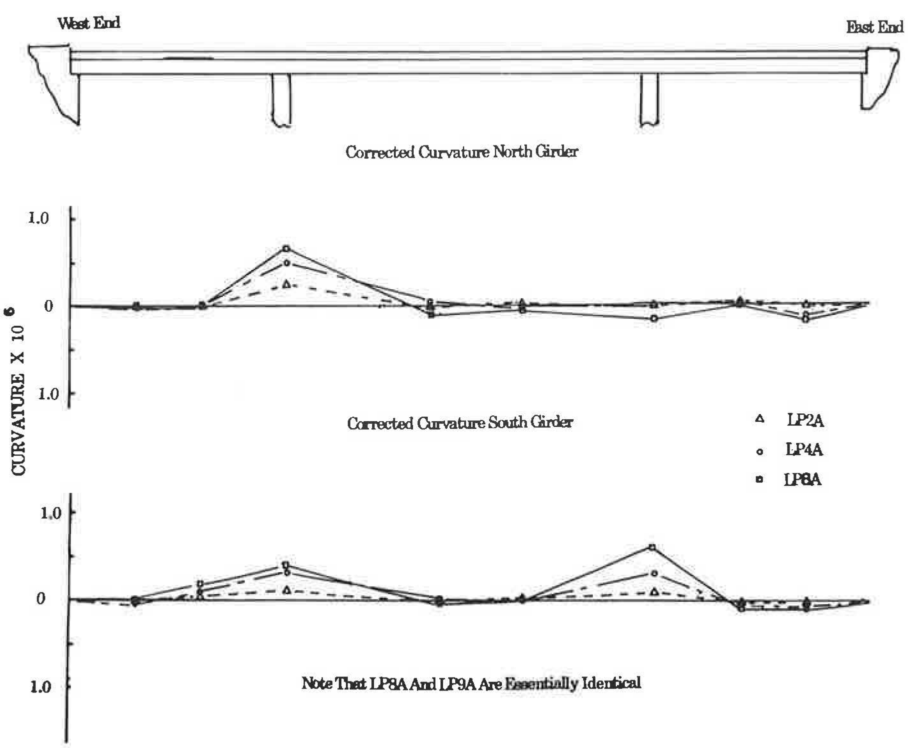


**FIGURE 10 Measured curvature of the bridge girders with no load on the bridge.**

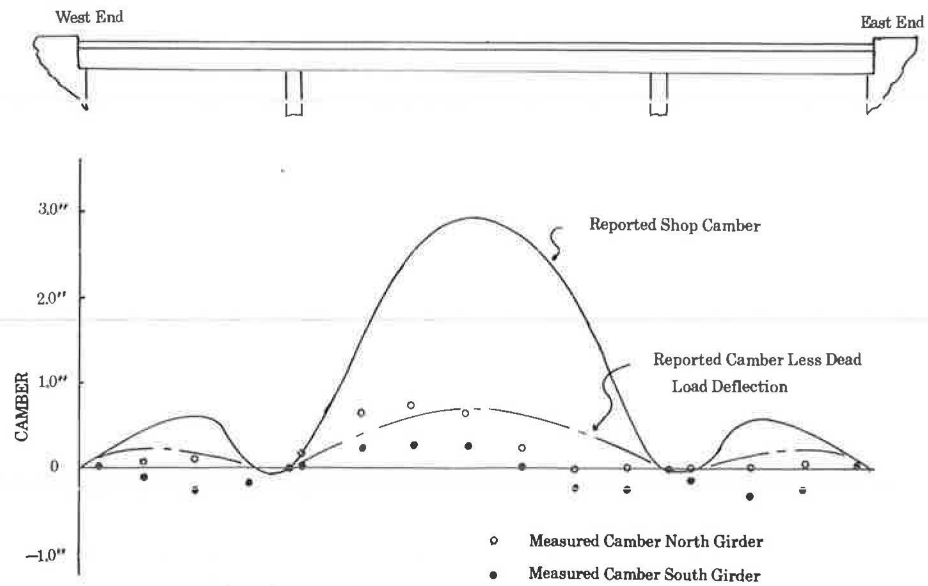


**FIGURE 11** Reported camber for the bridge girders at the fabricator shop and measured camber prior to testing.

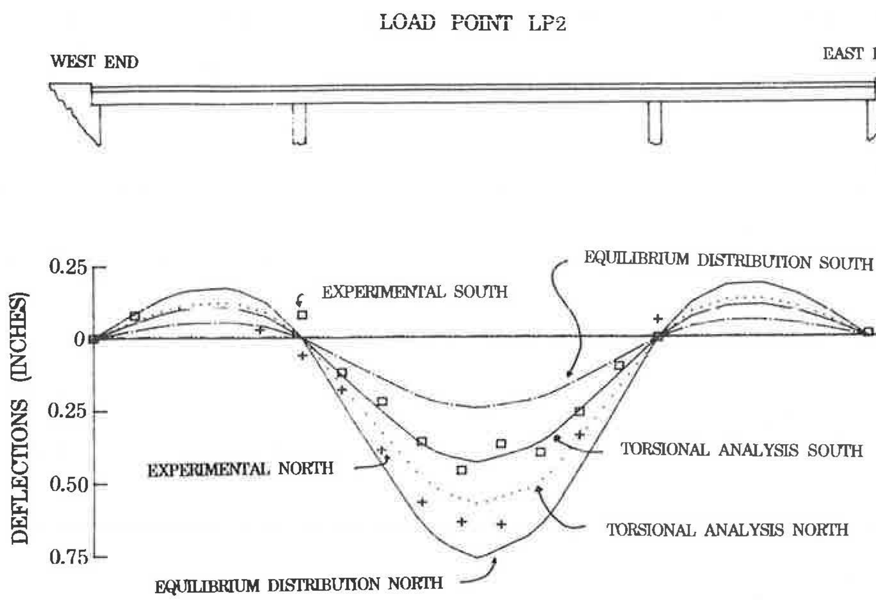


FIGURE 12 Measured and computed deflections for load point LP2.

mental data shown in Figures 12 and 13 include the permanent deflections and Automoments caused by yielding, while the theoretical predictions do not. Therefore, the experimental data should overestimate the theoretical deflection and underestimate the positive and bending moment by approximately 8 percent. This comparison indicates that the Saint Venant torsion stiffness caused by the thick concrete deck dominated the solution, and the more refined analysis clearly provided more reliable results. Saint Venant torsion helped to distribute the load between the girders and reduced the maximum strains and deflections in the beams.

LONG-TERM OBSERVATIONS

A series of long-term observations were made at approximately 6-month intervals for the 2 years following the load test. These were to verify shakedown and assure that no deterioration in the structural performance had occurred. Deflections were measured in the early morning while the bridge was in thermal equilibrium (i.e., the temperature was essentially constant throughout the bridge). This negated thermal deflections, and the resulting changes in bridge deflections over the 2-year period can be seen in Figure 14. The changes were consistently less than 0.02 in., which represents the statistical reliability of the measuring technique (9). Few outlying data points can be seen but those that can be seen have long sighting distances and potentially larger errors.

Cracking of the concrete bridge deck was also carefully monitored. At the end of the load test, a single tension crack existed in the bridge deck over the interior piers. The crack was closed and not visible on the deck surface unless live loads were

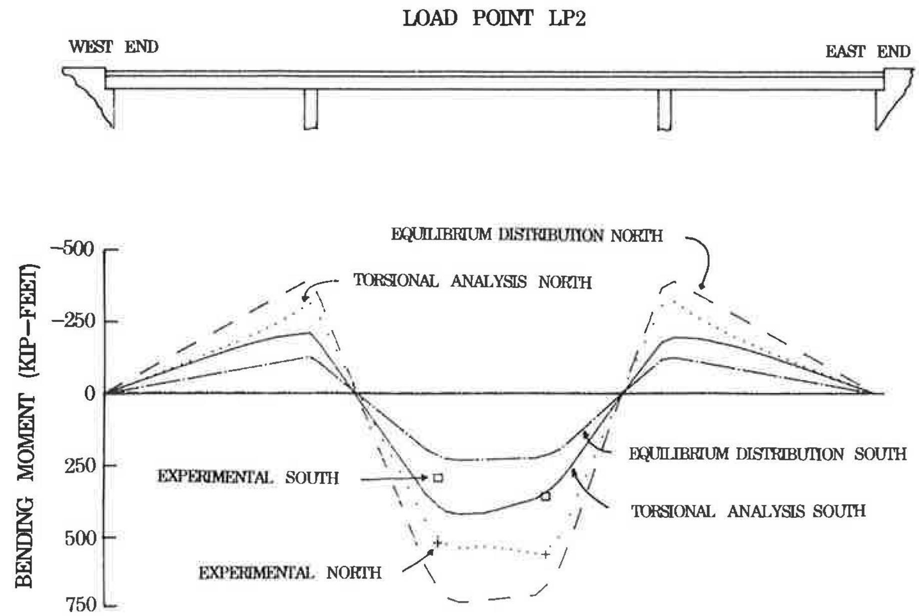


**FIGURE 13** Bending moments estimated from experimental data and computed by analysis for load point LP2.

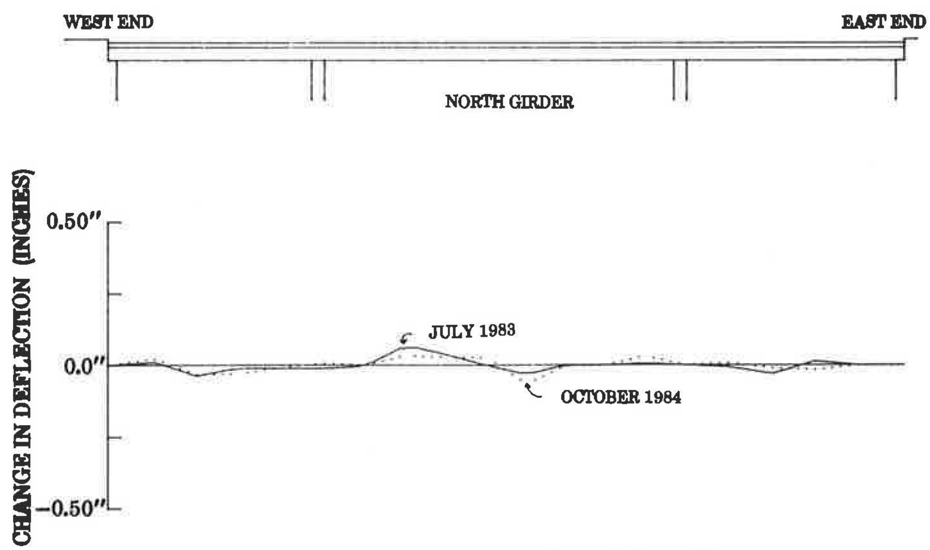


**FIGURE 14** Measured change in permanent deflection observed during the two years following the load test.

applied, but it could be seen on the underside of the deck, which was smooth. The size of these cracks did not grow during the long-term observation period, and they may have become smaller. However, the number of cracks increased slightly during this period. In October 1984, two small cracks could be seen over the west pier and three over the east pier. They are approximately 12 in. apart, and visible on the smooth underside of the deck only. In view of these observations, it is believed that no additional yielding of the bridge girders occurred during the 2 years following the load test, and the bridge deck shows no tendency toward deterioration.

## SUMMARY AND CONCLUSIONS

The ASD method is a new design method that permits limited, controlled yielding during the overload and maximum load limit states. The Whitechuck River Bridge was designed by this method, constructed, load tested, and observed. The bridge performed well during this period as only controlled yielding of the steel occurred during overload and the permanent deflection and tension cracking of the concrete deck were less severe than expected. After the yielding occurred, the structure remained elastic for all future loadings. Long-term measurements substantiate these load test observations. The deck surface was nearly straight at the end of the test, and the bridge should have a normal service life before it.

The following conclusions are made:

1. The thick concrete slab helped to distribute the loads between the bridge girders. This reduced the maximum strains and deflections in the bridge girders well below simple design predictions. Torsional analysis methods that include warping and

Saint Venant torsion provide a reasonable estimate of this reduction.

2. The permanent deflections and tension cracking of the concrete deck appear to be major concerns with the ASD method since they represent possible sources of serviceability and deterioration problems within the bridge. This test program appears to suggest that these concerns may be overstated. The ASD method has the unique advantage that limited yielding is anticipated. This yielding is accounted for with additional camber. Virtually all cambering methods cause large residual stresses, and these residual stresses induce early yielding when live or dead loads are applied. However, they do not affect the ultimate strength of the structure. Therefore, much of the permanent deflection may be completed before the concrete is hardened and live loads are applied. This is consistent with the Whitechuck Bridge observations and other earlier (13) research results.

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