Ultimate Load Test of a High-Truss Bridge

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As a result of the construction of the Saylorville Dam and Reservoir on the Des Moines River, six highway bridges crossing the river were scheduled for removal. An old pin-connected, high-truss, single-lane bridge was selected for a testing program that included ultimate load tests. The purpose was to relate design and rating procedures currently used in bridge design to the field behavior of this type of truss bridge. The ultimate load tests consisted of testing one span of the bridge, two I-shaped floor beams, and two panels of the timber deck. The theoretical capacity of each of these components is compared with the results from the field tests. This paper examines the portion of the program related to the ultimate load testing of the trusses. The bridge was rated using the 1970 AASHO maintenance manual. The test span ratings of the trusses averaged about 18 percent of actual capacity and were fairly consistent except for the floor beams, where the lateral support conditions for the compression flange caused considerable variation.

In recent years a considerable number of field tests (1) on bridges have been conducted. However, nearly all of these were conducted at or near design loads. The catastrophic collapses of several old bridges, the approval of load factor design for steel bridges by AASHO (2, 3), and the requirement nationwide for rating highway bridges (4) have all generated considerable interest in testing actual bridges to failure. However, only a limited number of tests (1) have been conducted at substantial overloads or near ultimate capacity. Most of these were performed either on laboratory models or on specially designed bridges, as in the AASHO road tests (5, 6).

The exceptions are a 1960 test of the Glatt Bridge in Switzerland (7), four tests recently completed in Tennessee (8, 9, 10, 11, 12), and a special test that was planned for the summer of 1975 on a bridge in southeast Missouri (13). However, all six of these bridges, as well as the AASHO road test bridges, were beam-and-slab bridges. Therefore, no information was available concerning the behavior of old high-truss bridges typical of those found in Iowa and throughout other parts of the country. Thus, a load test program was designed to provide information on the ultimate load-carrying capability of this type of bridge.

As a result of the construction of the Saylorville Dam and Reservoir on the Des Moines River, six highway bridges crossing that river were scheduled for removal. Five of these were old pin-connected, high-truss, single-lane bridges. However, for these bridge types there are no technical data and no field test data up to ultimate capacity; therefore, the capacity must be determined solely from field examinations. The removal of these five bridges created an excellent opportunity for studying the behavior of truss bridges by testing actual prototypes. The general purpose of the load tests was to relate design and rating procedures currently used in bridge design to the field behavior of this type of truss bridge and to provide data on the behavior of this bridge type in the overload range up to collapse.

The purpose of a study (14) conducted several years ago by Iowa State University was to determine the feasibility of conducting these load tests. The findings of the study included a recommendation to conduct a broad range of programs on several of the truss bridges included in this removal program. Because of the construction schedule, only one of the truss bridges became available for testing and was tested during the summer of 1974. Therefore, a research program to conduct several of these recommended tests was developed and undertaken by Iowa State University. In part, the research program consisted of the following phases:

1. Service load testing of the bridge,
2. Ultimate load testing of several steel floor beams,
3. Ultimate load testing of timber deck sections,
4. Ultimate load testing of trusses in an "as is" condition as well as in a "damaged" condition, and
5. Fatigue testing of tension truss members after the trusses were dismantled.

The original test program (14, 15) consisted of load testing two spans of the bridge to failure. One of the spans was to be tested in its "as is" condition while the other one was tested after a major member had been damaged to simulate the effect of vehicular impact. Since the main thrust (member damage) of the proposed second truss test was accomplished while testing the first truss,

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the ultimate load testing of the second truss was modified. The testing program was changed to include ultimate load tests of the floor beams at panel points 4 and 5.

This paper reports on the portion of the testing program related to the ultimate load testing of the trusses of the bridge. A more detailed analysis of the results is given by Saunders and others (16).

TEST BRIDGE

The high-truss bridge selected for testing was the Hubby Bridge (Figures 1 and 2), located over the Des Moines River in an area that will be included in the Saylorville Reservoir, southern Boone County, about 40.2 km (25 miles) northwest of Des Moines. It was built in 1909 and consisted of four 50.3-m-long (165-ft) modified Parker high-truss simple-spans.

The trusses consisted of tension eye-bars that had both square and rectangular cross sections, built-up laced channels for the end posts and upper chord compression members, and laced channels for the other compression members. Square tension eye-bars ranged in size from 1.9 to 2.9 cm (3/4 to 1 1/8 in) and were used for truss hangers and diagonals. Rectangular tension eye-bars ranged in size from 1.6 by 7.6 cm (5/8 in by 3 in) to 2.1 by 10.2 cm (13/16 in by 4 in) and were used for the truss lower chords and diagonals. The eyes for these two types of eye-bars were formed by bending the end of the bar to form a tear-shaped eye. The end of the bar was forged to form a permanent connection with the rest of the bar. The channels ranged in size from 10.2 to 22.9 cm (4 to 9 in) deep and were used for truss compression members. The deck was built of stringers, crossbeams, and floor planks all made of timber. All of the timber members were 7.6 by 30.5 cm (3 in by 12 in), approximately 5.2 m (17 ft) long, and were made from Douglas fir that had been pressure treated with creosote. A typical panel of deck consisted of 15 stringers, 8 crossbeams, and 16 floor planks. The standard I-section floor beams were 30.5 cm (12 in) deep and weighed 45.5 kg/m (30.6 lb/ft) of length. The floor beams were connected to the truss by means of clip angles and 1.3-cm (1/2-in) bolts.

Based on chemical analysis and physical property tests, it was determined that the tension eye-bars were made of wrought iron and the other members were made of steel. These results are given in Table 1. Tensile tests were conducted on coupons from typical members of both wrought iron and steel to obtain material properties; the stress-strain curves indicated behavior typical of the respective material. The results also indicate that the steel used satisfied the requirements for ASTM A36 steel, even though the steel was manufactured around 1900, and that the wrought iron conformed to ASTM A207 specifications.

TRUSS TEST

To test the span 2 trusses, simulated axle loads were applied at joints L_4 and L_5 in the ratio of 1 to 4; the greater load was applied at L_5. This ratio represented the relationship between the axles on an AASHO H15 truck. Although the load spacing in the truss test was 5.03 m (16.5 ft) since it was limited by floor-beam spacing and panel length, the effect of this difference with the actual 4.27-m (14-ft) specified axle spacing would be minimal because of the large load ratio and would not significantly affect the results.

The loads were applied by using hydraulic jacks connected to weights. Two large reinforced concrete mats that were used to supply the needed dead weights were cast under span 2. A concrete pumping system pumped the concrete from the southwest end of the bridge to the locations where the mats were formed. At the same time, two other mats were cast under span 1 and were used for subsequent floor beam tests. The sizes of the concrete mats varied from 0.46 by 1.83 by 7.62 m (1.5 by 6 by 25 ft) to 0.92 by 3.05 by 7.62 m (3 by 10 by 25 ft) These mats ranged from 15.4 Mg (34 000 lb) to 50.8 Mg (112 000 lb) with soil piled on top of each of the concrete mats to increase its mass.

Rods of 2.5-cm (1-in) diameter were attached to the concrete mats by using concrete inserts and a system of structural tubes (Figure 3). The hydraulic jacks were connected to the rods through a similar system of structural tubes so that the loads could be applied to the truss. Sketches of the loading system are shown in Figure 4.

Truss tests were conducted by applying strain gauges, which were encapsulated and self-temperature compensated for steel, to the truss members (Figure 5). Tension eye-bars were instrumented with one strain gauge on each of the two bar members. Two laced channels or two built-up laced channel members were instrumented with four gauges mounted near the four corners of the member to allow for computing the bending moment in both directions and the axial force for the member. Vertical and horizontal deflection readings were taken at midspan and at the 0.3 and 0.7 points on both sides of the span. For the truss test, 9 deflection indicators were used along with 108 strain gauges on the truss members.

Procedure

The first load was applied in increments of 44.5 kN (10 000 lbf) [35.6 kN (8000 lbf) at L_5 and 8.9 kN (2000 lbf) at L_4]. As loading progressed to higher levels, the increments were reduced to 22.2 kN (5000 lbf) [17.8 kN (4000 lbf) at L_5 and 4.4 kN (1000 lbf) at L_4] until failure was reached.

The truss test proceeded as planned up to a total load of 355.9 kN (80 000 lbf). While the load was proceeding to 400.3 kN (90 000 lbf), yielding took place in one of the hangers at L_5 (downstream side). During the load increment to 489.3 kN (110 000 lbf), there was considerable yielding at L_5. When loading proceeded to 578.3 kN (130 000 lbf) the rust flaked on the hangers at L_5 (upstream side).

At a total load of 591.6 kN (133 000 lbf) [473.3 kN (106 400 lbf) at L_5 and 118.3 kN (26 600 lbf) at L_4], equivalent to AASHO H66.5 truck, one of the hangers at L_5 (upstream side) failed. When the failure occurred, a portion of the load transferred from L_5 to L_4, resulting in a load of 280.2 kN (63 000 lbf) at L_5 and 169.0 kN (38 000 lbf) at L_4 at a loading ratio of 1.66 to 1. At this point, the large truss deflection caused the floor beam to move 10.2 cm (4 in) away from the timber stringers at L_5. This occurred because of the continuity of the floor system and the lack of a positive tie between the timber floor and floor beams. After the hanger failure, the load was reapplied and increased to 556.0 kN (125 000 lbf) [400.3 kN (90 000 lbf) at L_5 and 155.7 kN (35 000 lbf) at L_4]. The load was increased to 498.2 kN (112 000 lbf) at L_5 and 124.6 kN (28 000 lbf) at L_4. At this load, the maximum vertical deflection at midspan was 38.1 cm (15 in), and significant distortion of the bridge was visible (Figure 6).

After readings were taken, the load was removed from L_5 because any further increase in load would cause more distortion of the lower chord at L_5. The load was then applied at L_4 to the capacity of the loading system [349.2 kN (78 500 lbf)] without any additional failures resulting to the truss members.

It was decided that further testing of the trusses would

Figure 1. Hubby Bridge.

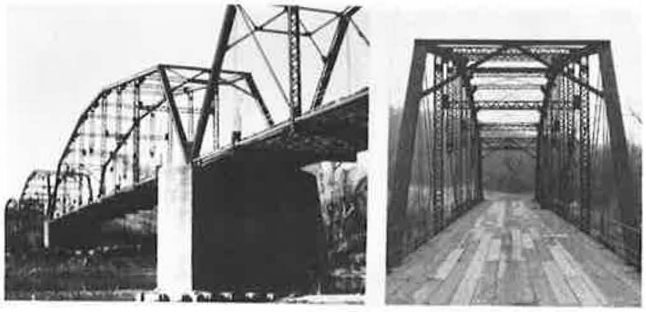
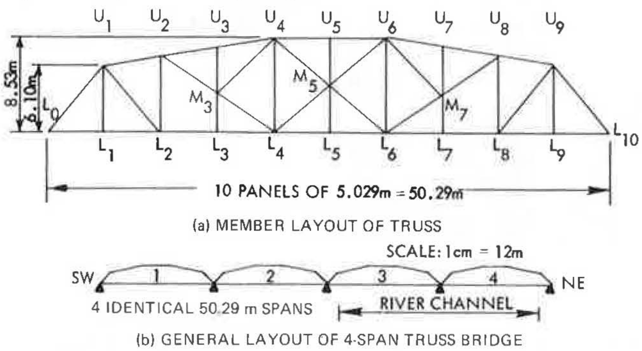


Figure 2. Layout of Hubby Bridge.



Note: 1 m = 3.28 ft.

Figure 3. System used to attach steel rods to concrete mats.

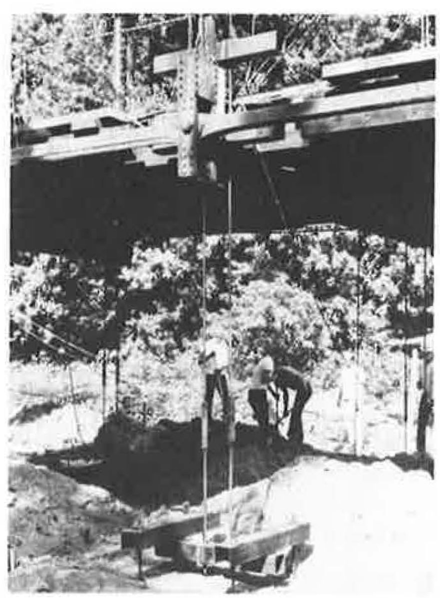


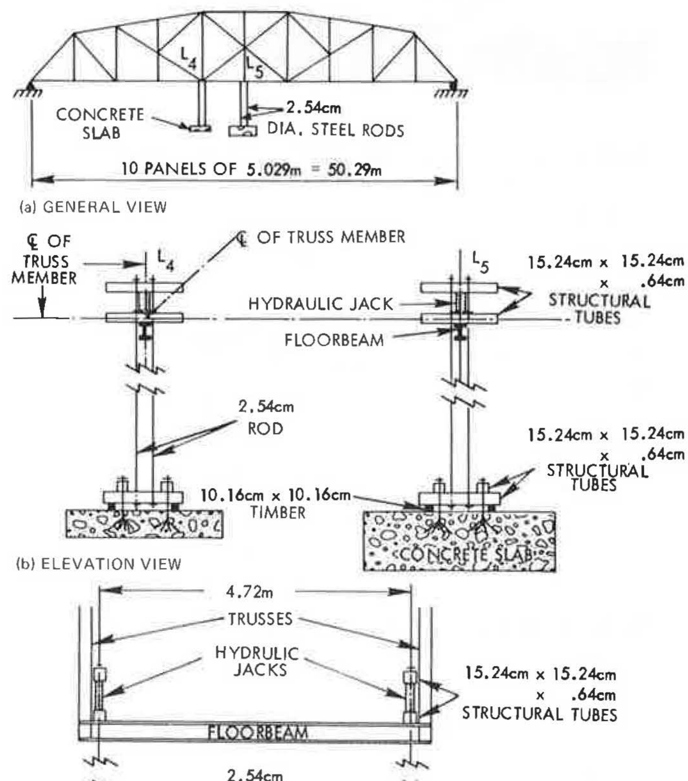
Table 1. Physical properties of wrought iron and steel based on chemical analysis and physical tests.

Properties	Wrought Iron	Steel	Properties	Wrought Iron	Steel
Chemical (%)					
Carbon	<0.03	0.19	Vanadium	<0.01	–
Manganese	<0.05	0.40	Silicon	0.22	<0.05
Phosphorus	0.29	0.012	Cobalt	0.02	–
Sulfur	0.042	0.029			
Nickel	<0.05	<0.05	Physical (MPa)		
Chromium	<0.05	<0.05	σ_y	244.8	289.6
Molybdenum	<0.03	<0.03	σ_{ult}	338.5	404.7
Copper	<0.03	0.03	ϵ	193×10^5	2.13×10^5
Aluminum	0.03	–			

Note: 1 Pa = 0.000 145 lbf/in^2.

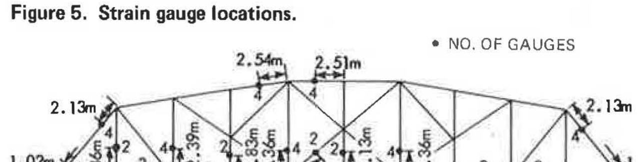
not provide any additional information. Therefore, the objectives of the second truss test, to "damage" one of the key members and reload, were pursued. To simulate the damage, member L_2U_2 was cut with an acetylene torch. This member was selected because it was representative of the laced-channel compression members and because an end post would have required an elaborate loading system to be sufficiently damaged and would have resulted in immediate catastrophic failure. Initially, only one of the two channels of the member was cut. Loading was applied at L_4 and was increased to 311.4 kN (70 000 lbf) without any signs of additional distress. Thus, the other channel of member L_2U_2 was cut, leaving only the web of the channel remaining intact. The load at L_4 was again increased to 311.4 kN (70 000 lbf) without any signs of distress. Again, the load was removed from the bridge and one bar of member L_2U_1 was cut. The truss at L_4 was again loaded to 320.3 kN (72 000 lbf) with no apparent signs of distress. After the load was removed, member L_2U_2 was cut through completely leaving a gap of about 10.2 cm (4 in). Loading at L_4 was

Figure 4. Truss loading system.



Note: 1 cm = 0.393 in; 1 m = 3.28 ft.

Figure 5. Strain gauge locations.



Note: 1 m = 3.28 ft.

Figure 6. Distortion of lower chord at L_5.

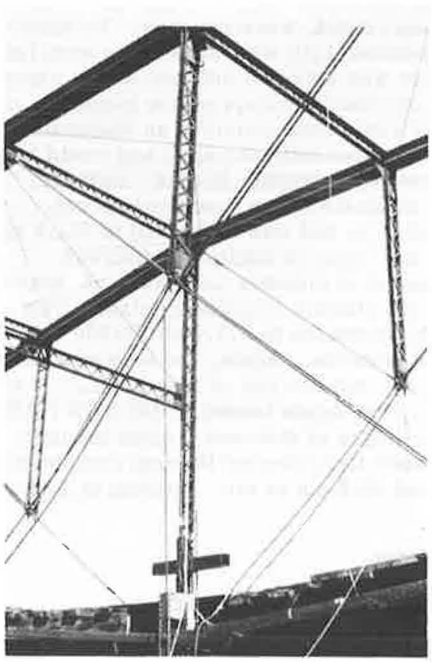


Figure 7. Damaged member after collapse.

Figure 8. Total load-vertical deflection at L_5 for truss test.

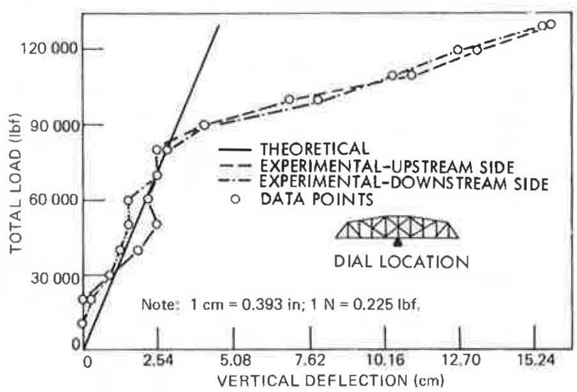


Figure 9. Total load-vertical deflection at L_3 and L_7 for truss test.

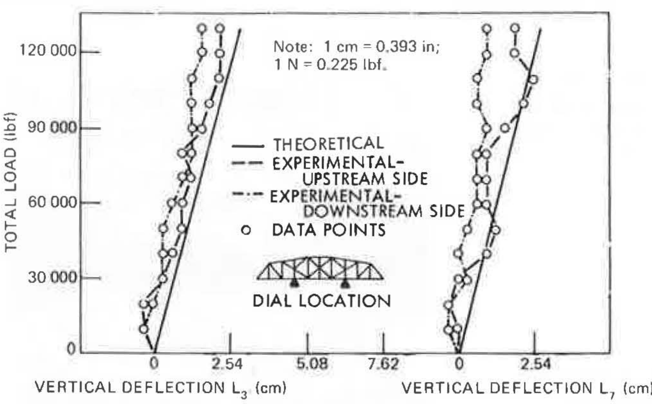


Figure 10. Total load-force in member L_5M_5.

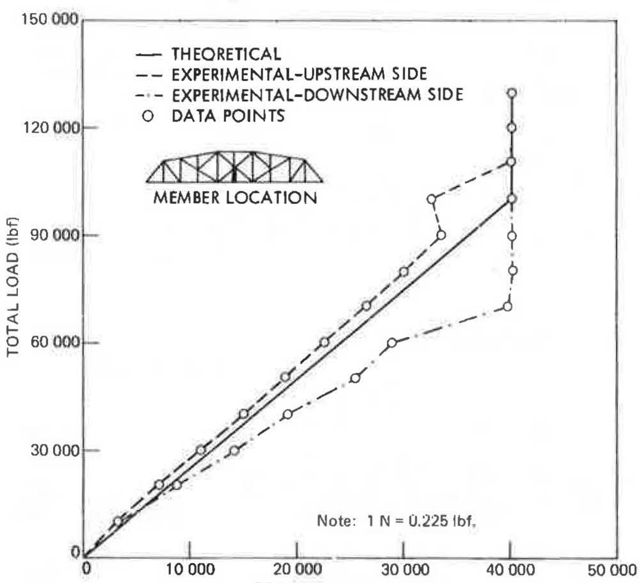
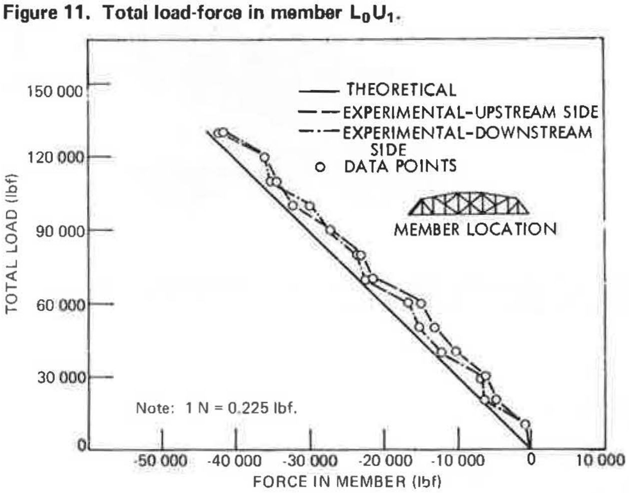


Figure 11. Total load-force in member L_0U_1.



again applied and reached 173.5 kN (39 000 lbf) before the member collapsed upon itself, forming a complete but shorter member at the cut location (Figure 7). The load was then increased to 320.3 kN (72 000 lbf) with no further distress of the truss. The load was removed and all testing terminated. It was postulated that, if a greater gap had been provided in L_2U_2 so that the two sections had not met when initial collapse occurred, the truss might well have collapsed completely.

Results and Analysis

The behavioral indicators for the truss test were the deflection readings taken at midspan and at the 0.3 points and the forces in the truss members that were computed from the strain guage readings taken during the test. Figure 8 shows the theoretical and experimental load-deflection curves for the vertical deflection at midspan, and indicates that yielding began to occur in member L_5M_5 at a total load of approximately 355.8 kN (80 000 lbf). The curve is relatively linear at loads less than 355.8 kN (80 000 lbf), whereas above 355.8 kN (80 000 lbf) the slope of the curve decreases indicating yielding of member L_5M_5. The small nonlinearities at loads below 355.8 (80 000 lbf) indicate the effect that rusting of the members and pins and the distorted shape of some members had on the behavior of the truss. Figure 9 shows the theoretical and experimental load-deflection curves for the vertical deflection at L_3 and L_7 and indicates no yielding or nonlinearity up to the maximum load at which readings were taken. Figure 9 shows that both of the deflection readings taken at the 0.3 points exhibit fairly linear behavior. Although there is some agreement between the two sides of the truss, the small magnitude of the deflections and the apparent effect of the rusted condition of the truss made it difficult to determine whether this agreement was valid. The horizontal deflections of the truss were negligible.

Figure 12. Total load-force in member L_2L_3.

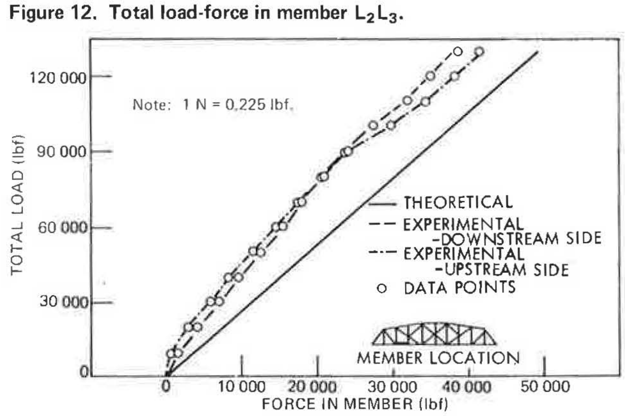


Figure 13. Total load-force in member U_3U_4.

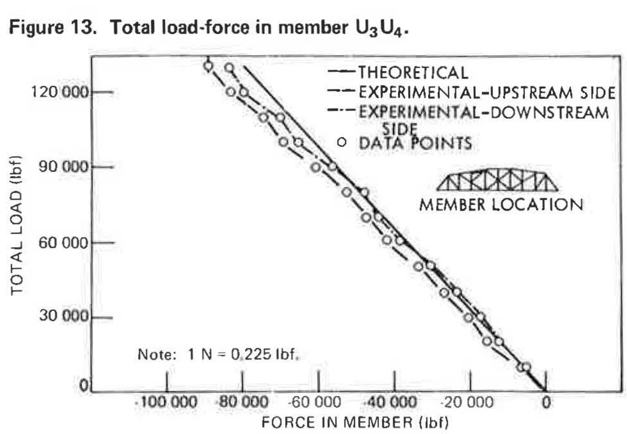


Figure 10 shows the total load-force in truss member L_5M_5, and indicates that this truss member exhibited approximately the same behavior shown in the total load-vertical deflection curve in Figure 8. Representative samples of the curves for other total load-force in truss members are shown in Figures 11, 12, and 13. These curves indicate linear behavior up to the maximum load at which readings were taken.

The theoretical forces shown in Figures 10, 11, 12, and 13 were obtained by analyzing the structure of the truss and by assuming that all of the members were held together by pins at the joints. Most of the experimental forces determined from strain gauge readings agreed quite closely with the theoretical forces determined by analysis. However, in a few cases the experimental data differed in magnitude from the theoretical curves. In these cases, the experimental results were less than the theoretical results. This behavior was due, in part, to the "frozen" condition of the joints that resulted from the rusted members and pins.

Thus, although the actual conditions in the joints were unknown, to consider that the truss was pin-connected provides a realistic method for truss analysis of these old bridges. The tremendous flexibility of the members that allows accommodation of any joint restraint also contributes to this conclusion.

The capacity of the hangers at L_5, calculated by using data from coupon tests, was 489.3 kN (110 000 lbf) [337.2 MPa (48 900 lbf/in^2)]. This was larger than the load that actually caused the fracture of one of these hangers. The actual stress at fracture was 326.8 MPa (47 400 lbf/in^2) thereby indicating that the "lap" near fracture was about 97 percent effective. The current practice in Iowa is to assume that the lap is only 40 percent effective, which is much lower than the actual capacity of the member.

One of the significant portions of this study was the rating of the test span and the comparison of that rating with the actual capacity. Field inspection was used as the basis for the rating calculations and was made by the Maintenance Department of the Iowa State Highway Commission. This information was forwarded to the agencies cooperating in this phase of the study: U.S. Army Corps of Engineers, Iowa State Highway Commission, and Iowa State University. Using these base data, each agency computed bridge rating in accordance with the 1970 AASHO maintenance manual (4).

The operating-level truss ratings (H11.4, H12.7, and H11.9) determined by the three different agencies were quite consistent and were about 18 percent of the test capacity (H66.5). Even though the ratings considered dynamic effects, they still appeared conservative.

SUMMARY AND CONCLUSIONS

The purpose of the ultimate load tests was to relate currently used bridge design and rating procedures to field behavior of this type of truss bridge. The general objective of the test program was to provide data regarding the behavior of this bridge type in the overload range up to collapse. As a result of the ultimate load test performed on this truss bridge, the following conclusions were reached:

1. The experimental forces determined for the truss members agreed closely with those forces determined by analysis for the same members. This indicates that to assume the members were pin-ended was valid for this particular truss.
2. The theoretical capacity of the hangers at L_5

agreed closely with the load capacity that actually caused the fracture of one of these hangers.

3. The current practice of assuming that the lap of an eye-bar must be only 40 percent effective is quite conservative. However, additional tests are required before any recommendation to change this assumption is warranted.

4. The truss ratings averaged about 18 percent of capacity.

At the conclusion of this project, a final summary report will be prepared that will include recommendations for implementation of the findings.

ACKNOWLEDGMENTS

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The opinions, findings, and conclusions expressed are those of the authors and not necessarily those of the Federal Highway Administration, the U.S. Department of Transportation, or the Iowa Department of Transportation.

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