BEHAVIOR OF ALASKAN NATIVE LOG STRINGER BRIDGES

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For many years, native log stringer bridges have served as the primary bridging system for the Alaska forest transportation network. They are constructed from Sitka spruce or western hemlock logs, placed side by side and decked with blast rock, all locally available materials. These bridges have served for ten years or more, and have been economical as well (average cost \$54/sq m, or \$5/sq ft). Inspection and load rating as required by the national bridge inspection standards raised questions about wheel-load distribution and physical strength properties of the bridges. The procedures used in their analysis were based on limited information and the evaluations were, at best, approximate. This applied particularly to the load distribution criteria. A research program was undertaken to obtain the information necessary to provide more realistic rating information. This research program included four phases:

1. Strength evaluations of full size logs,
2. Load distribution tests of four "in-service" bridges (spans 11.6 - 28.0 m; 38-92 ft),
3. Laboratory tests of models of typical bridges, and
4. Analytical evaluation of bridge test data and development of revised load distribution criteria.

The field and laboratory test results supported the analytical findings of better distribution than that of current design. The strength tests provided current data on the ultimate bending strength of Sitka spruce and western hemlock logs. The final result of the study are revised design criteria which show that significantly higher loads can be allowed than are permitted under current criteria.

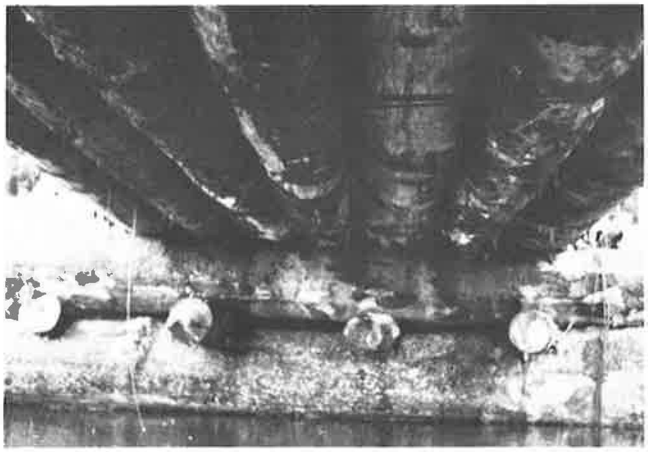
Within the last 20 years, several thousand bridges have been built with native logs in the southeast regions of Alaska (Figure 1). This effort is part of the development of the National Forest Transportation system on the islands of the Alexander Archipelago in the Southeast Alaska Panhandle.

Bridges were needed to span streams, and an abundance of high-quality Sitka spruce logs, up to 1.52 m (5 ft) in diameter, was available locally. Therefore, it proved economically advantageous to build the bridges from these native materials. With trees cut in the proximity of the bridge site, the cost of constructing these bridges was only about \$54.00 per sq m (\$5.00 per sq ft).

Figure 1. Native log stringer bridge in southeast Alaska: Span--28 m (92 ft); Capacity--72.5 Mg (80 tons).

Typically, logs are not sawn into timbers and planks, but are placed butt-to-tip on log crib abutments and tied together with cables (Figure 2). Blast rock is then placed on the logs and bladed to provide a running surface. Brow logs are placed at the sides of the bridge to serve as curbs and guardrails. Although this may seem primitive, the several hundred bridges of this type currently in use function quite adequately for up to ten years or more. Some of the bridges are quite impressive, with clear spans approaching 30 m (100 ft), and carry off-highway logging trucks with gross vehicle weights exceeding 90 Mg (100 tons).

Figure 2. Log crib abutments and bottomside of log stringer superstructure.



Bridge Safety

As a consequence of the "Silver Bridge" disaster in 1967, Congress included a provision in the Federal Aid Highway Act of 1968 which required the Secretary of Transportation to establish national bridge inspection and load rating standards. Those bridges not meeting the loading criteria must be posted for allowable loads accordingly.

Unfortunately, the current knowledge of log stringer bridge analysis and design (1) is limited. Little is known about the bending strength or expected life of large-diameter logs. The recommended allowable design stresses are based on procedures developed for poles and piles. The manner in which wheel loads are distributed between stringers is not well understood.

Inspecting and load-rating existing bridges by current procedures indicated that many log stringer bridges are being seriously overloaded. In a number of instances, the calculated allowable load is much less than the weight of logging trucks which have regularly been using these bridges for several years. It is apparent that new and reliable design information is needed to properly analyze log stringer bridges.

Cooperative Research Program

To obtain the needed information on the performance of log stringer bridges, the U. S. Forest Service-Alaska Region (Region 10) and the U.S. Forest Products Laboratory (FPL) conducted wheel-load distribution tests on existing bridges, and strength evaluations on both used and green Sitka spruce and green western hemlock logs.

The field test data for load distribution analysis were then analyzed by the Engineering Research Institute of Iowa State University (ISU) under a cooperative agreement with the Forest Products Laboratory. Using this data, ISU developed analytical procedures to study a wide range of variables affecting load distribution that cover all types of log stringer bridges. The results were confirmed by scale-model tests conducted in the ISU laboratory.

The final result of the study are recommendations for new allowable log stresses and load distribution criteria that will more accurately predict the behavior of the bridges. This paper summarizes the studies indicated above.

Field Test of Log Stringers (2)

The testing took place at Thorne Bay on Prince of Wales Island. Twenty-five green Sitka spruce logs, 15 green western hemlock logs, and 28 used Sitka spruce logs were tested to destruction in the field test facility. These are the common species for log stringer bridges, with Sitka spruce being used almost exclusively. The logs for the individual stringer tests were representative of the logs used in existing logging operations in the 16-million-acre Tongass National Forest. Log quality was determined by standard design requirements (1). Lengths ranged from 14.3 to 26.2 m (47 to 86 ft), with diameters of up to 1.47 m (4.8 ft) at the butt end.

The test facility (Figure 3) consisted of a hold-down anchor, support cribs, and a loading system. Sixteen rock anchors, capable of resisting 900 kN (200,000 lbf), were installed. Two movable support cribs (adjusted for log length) were built by cross-stacking large logs to a height of about 4.9 m (16 ft) above ground. Loads were applied by cable through two quadruple sheave blocks to increase the force by about 8 to 1. The strongest green log held up a load in excess of 530 kN (120,000 lbf) before it broke. The green western hemlock logs frequently broke explosively, scattering some of the loading apparatus about the site. Green Sitka spruce logs, on the other hand, failed gradually and stayed up on the supports. However, a few of the old, used spruce logs shattered upon failure.

Figure 3. Test set-up (with log in position) for bending tests.

The current procedure for determining allowable bending stress in logs is contained in ASTM D-2899 (3). With this procedure, a statistical point estimate of minimum strength is established which 95 percent of the population should exceed. It is based on the modulus of rupture of small clear wood specimens with appropriate adjustments for growth, shape, variability, etc. The field tests confirmed that this procedure is reasonably accurate.

The log strength properties for green logs are summarized in Table 1. The frequency distributions

Table 1. Log strength properties.

Source	Number of Specimens	Modulus of Rupture Average MPa	Standard Deviation MPa	Coefficient of variation %	Maximum MPa	Minimum MPa
		Western Hemlock				
False Island	4	33.0	4.8	14.4	39.4	28.0
Zarembo Island	4	31.9	4.4	13.9	37.9	27.6
Prince of Wales Island	7	32.1	6.8	21.2	40.2	21.9
All hemlock	15	32.3	5.4	16.7	40.2	21.9
		Sitka Spruce				
False Island	8	31.3	3.7	12.0	39.0	28.1
Zarembo Island	6	29.9	4.1	13.6	35.0	23.6
Prince of Wales Island	11	31.9	7.3	22.8	47.4	20.8
All spruce	25	31.2	5.5	17.6	47.4	20.8

[a] 1 MPa = 145 lbs/in.^{2}

are skewed somewhat to the right and are best represented by log normal distributions which are common for wood. One-sided tolerance limits for the two species at different confidence levels are presented in Figures 4a and 4b. The current design levels are also shown.

The bridge engineer must decide upon the combination of safety and economics that is appropriate for his design. The breaking strengths shown are generally multiplied by a factor of 0.62 to obtain ten-year duration of load stresses. This is quite conservative, since most bridges are under full design load for only a very short time during their service life.

Modulus of rupture values for Sitka spruce logs after 12 years of service were calculated by two methods. The first method was based on the section modulus of the full log diameter, and produced an average strength of 24.0 MPa (3.48 ksi). The second method subtracted peripheral decay from the gross area to estimate the net sound section. The average modulus of rupture of the sound material was 33.2 MPa (4.82 ksi), essentially the same as for fresh logs.

Load Tests of Actual Bridges (Staney Creek) (2)

Four bridges with clear spans ranging from 11.6 to 28.1 m (38 to 92 ft) were selected for the wheel-load distribution tests. The bridges, located on Prince of Wales Island, were loaded with a large off-highway gravel truck with a gross vehicle weight of 39,000 kg (87,000 lb) (Figure 5). Centerline deflections of each stringer and each brow log were recorded for nine different truck positions. The maximum deflection on the 28 m-span (92 ft) bridge was only 33 mm (0.11 ft). There was significant deflection in the brow logs, indicating that they were contributing structurally to the performance of the bridges.

A plan view and cross-section of a typical log bridge are shown in Figures 6 and 7. Typical deflection data for the two positions of loading (concen-

Figure 4a. Short-term breaking strength of western hemlock logs for various tolerance limits and confidence levels.

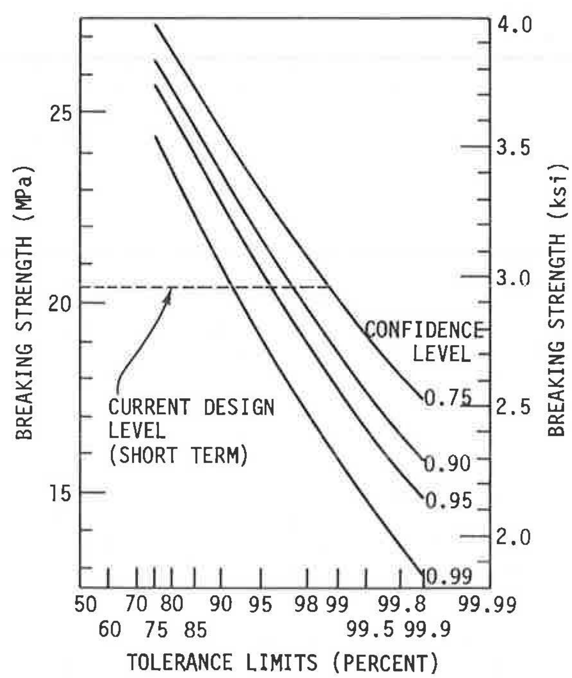
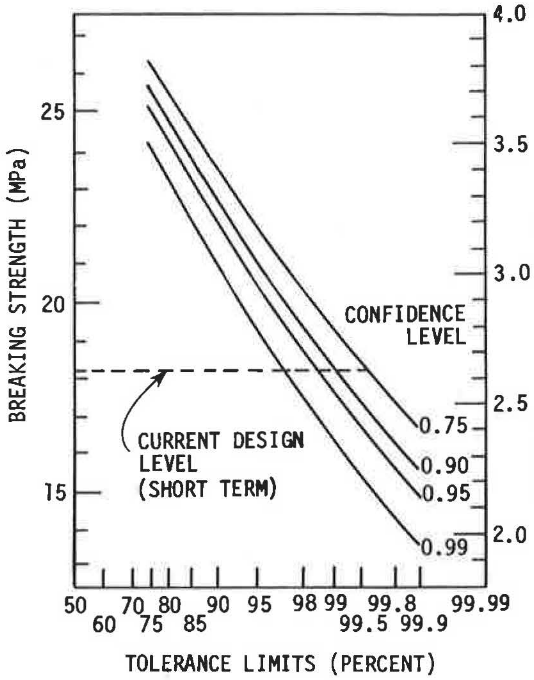


Figure 4b. Short-term breaking strength of Sitka spruce logs for various tolerance limits and confidence levels.



tric and eccentric) are shown in Figure 8 for the truck at midspan. The deflections give an indication of the load distribution, although adjustments are required due to variations in log diameters.

The results of three of the bridges were used to determine the validity of the analytical procedures used to develop the overall behavior of a wide range of log stringer bridges. The fourth bridge failed during testing.

Figure 5. Load test truck (39,000 kg; 87,000 lbs) on bridge.

Analytical Investigation of Bridges

In this portion of the investigation analytical techniques were used to study the load distribution characteristics exhibited by the bridges in the field. The investigation proceded in two parts. First, after a method of theoretical analysis was selected, a means of comparing the load distributions from field tests on the Staney Creek bridges and theoretical load distributions of these same bridges were developed. The second part was the

Figure 6. Plan view of 22 m (72 ft) West Fork Staney Creek Bridge.

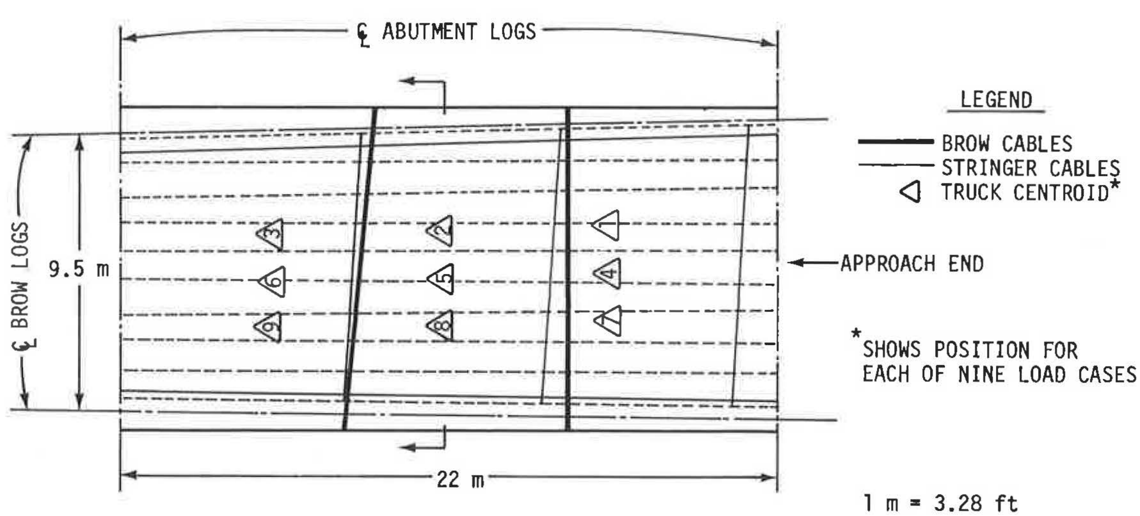
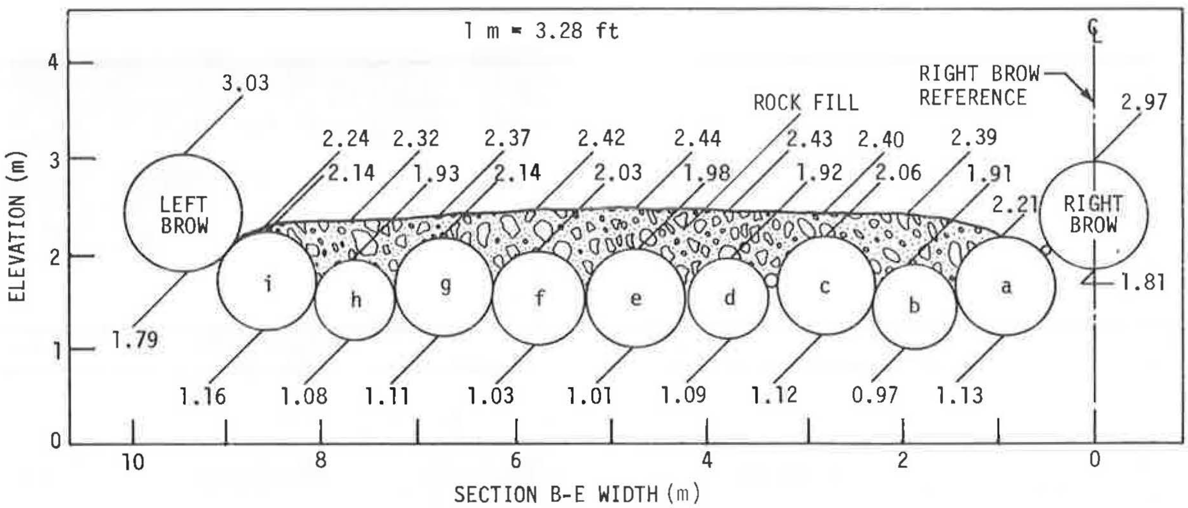


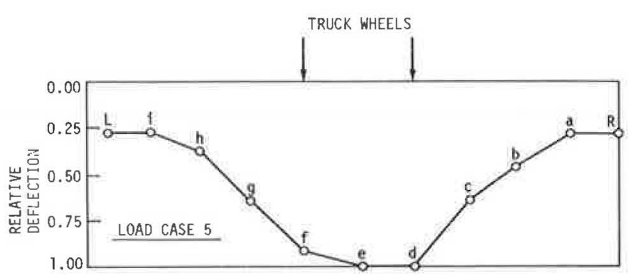
Figure 7. Cross-section of 22 m (72 ft) West Fork Staney Creek Bridge at midspan.



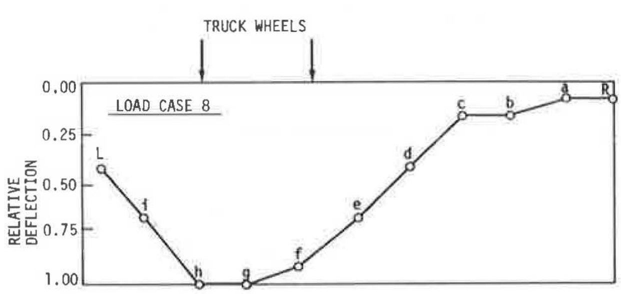
investigation of theoretical load distributions for a range of variable that encompassed the majority of bridges found in Alaska today.

The articulated plate theory (4) was chosen for the analytical investigation, as it is better suited

Figure 8. Deflection diagram for 22 m (72 ft) bridge with test truck at midspan.



(a) Concentric load position



(b) Eccentric load position

to log bridges because load distribution between stringers is primarily accomplished by friction between logs. This type of behavior indicates a load distribution through shear rather than bending; thus, the selection of the articulated plate theory seemed appropriate. The basic assumption in the theory is that only shear is transferred between bridge elements.

The basic comparison between theory and field data is the moment coefficient per foot of log, K_{MPL}. It is the ratio between the moment per foot of bridge width in the log to the average moment per foot of bridge width. To compare theoretical load distribution with the load distribution from field tests, a suitable parametic relationship for use in articulated plate theory was developed. It was shown (4) that the most important cross-section parameter in the determination of transverse wheel load distribution is ϕ, which is approximately

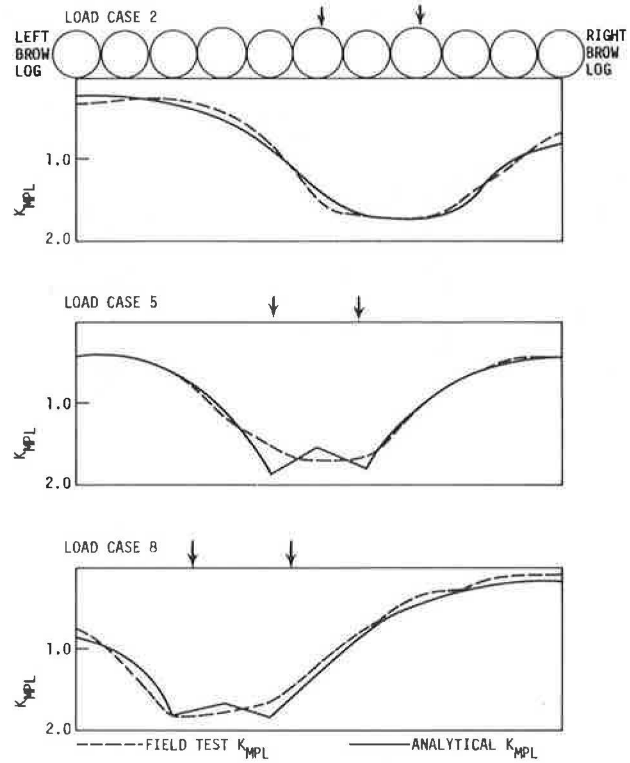
$$\phi = 1.5 \frac{W}{L} \qquad (1)$$

where W = effective bridge width, m (width of bridge plus diameter of brow logs) and L = length of span, m.

The values of K_{MPL} for the stringers in the Staney Creek bridges were computed from deflection data received from the field tests. K_{MPL} values in stringer due to all nine load cases were computed for all three Staney Creek bridges. The midspan load cases were of particular interest because the theoretical analysis only considered bridges with midspan loadings. After the values of K_{MPL} had been computed from the field test deflection data, theoretical values for K_{MPL} in the stringers of the same bridges were calculated using articulated plate theory. Typical load distributions from the Staney Creek field test of the 22 m (72 ft) bridge and the computer analysis using articulated plate theory with ϕ = 1.5 W/L are shown in Figure 9.

Comparison of the load distributions curves for the load cases shows that the theoretical analysis favorably models the actual field test results. Not only do the curves follow the same shape in each load case, but the maximum values of K_{MPL} for the theoretical and field test analysis are nearly equal and occur in about the same locations in both analy-

Figure 9. Comparison of K_{MPL} from theory and from field bridge tests, 22 m (72 ft) Staney Creek Bridge.



sis. This relationship was typical of all cases studied.

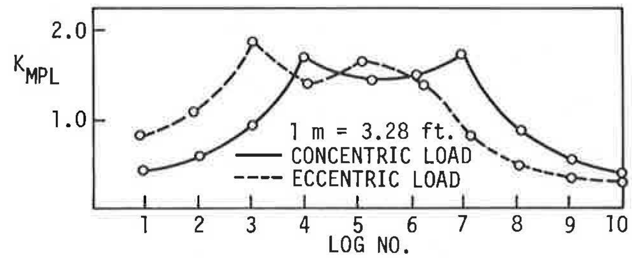
Upon verification of the validity of the theoretical analysis, load distributions were generated for the broad spectrum of bridges. Because the theoretical load distribution in a bridge was based on length and width of the bridge, it was necessary to determine sizes of actual Alaska log bridges. Based upon the results of a field inspection (5), the range of stringer sizes, number of stringers, bridge spans, rock depths, and log configurations were obtained. It was found that the typical bridge had a width of about 4.6 m (15 ft) and a span of 9.1-15.2 m (30-50 ft), and had 8-10 stringers with brow logs.

Using this data, upper and lower bounds were set in width and length of bridges. These bounds set the limits of ϕ (Eq. 1) from 0.25 to 2.50, with a typical value of 0.75.

After determination of the range of sizes of the field bridges in Alaska, theoretical load distributions for bridges within the range were developed. The initial study was limited to bridges without brow logs. The effect of brow logs was determined later.

Computation of these distributions was done using an articulated plate theory computer program. However, instead of using the field truck loading used in generation of theoretical load distribution curves for the Staney Creek bridges, standard loads (unity) were used as the concentrated loads. This standard load computer program (from articulated plate theory) was then used to generate concentric and eccentric load distribution curves for a broad spectrum of bridges. Typical eccentric and concentric load distributions are shown in Figure 10.

Figure 10. Distribution curves for typical log stringer bridges.



Experimental Investigation of Model Bridges

The experimental investigation of bridges was needed to supplement test data provided by the Forest Service (2) so that confirmation of general design criteria could be obtained. Comparison of the results from the field test data and the computer analysis proved favorable, but experimental work was needed before generalizations on the nature of load distribution in stringers, including the effect of brow logs, could be made. The experimental work consisted of tests of laboratory model bridges.

The model bridges were constructed of 7.6 m (25 ft) utility poles with diameters ranging 20.3-26.7 cm (8-10.5 in.), as shown in Figure 11. The bridges were either at a scale of 1:5 or 1:4, depending on the log diameter in comparison to the span length being modeled. Over 25 model bridges were tested. The significant parameters were the width of the bridge and addition of brow and/or stabilizer (transverse) logs (Figure 12).

The objectives of the experimental investigation were twofold. The first was to gain additional load distribution data to supplement the information provided by the field bridge tests. These additional data were needed to verify the assumptions made in the theoretical load distribution study.

The second objective was to isolate the effect of brow logs and stabilizer logs on load distribution. By the systematic addition and removal of brow logs and stabilizer logs to test bridges of a given length and width combination, their effect on load distribution was determined.

A comparison of the lab test, field test, and theoretical load distribution is shown in Figure 13. The lab test distribution is taken from calculations based on the diameter and deflection of the stringers (Figure 12) in the model bridge. The theoretical distribution is found by using the standard load program with width and number of logs of the model bridge as input. The field test distribution is taken from articulated plate analysis of the Staney Creek bridge. All three curves are approximately the same shape, and the peak percentages of load carried by a stringer in each bridge are close. The peak theoretical and field percentages of total load in a stringer are 18.8 percent and 17.5 percent, respectively, which is a difference of 7 percent. The experimental lab test peak percentage of load in a stringer is slightly higher at 21.5 percent. Comparing all three distributions, the peak percentage of total load in a bridge stringer is about 20 percent.

After comparing the distribution curves (includ-

Figure 11. Laboratory model test bridge ready for test.

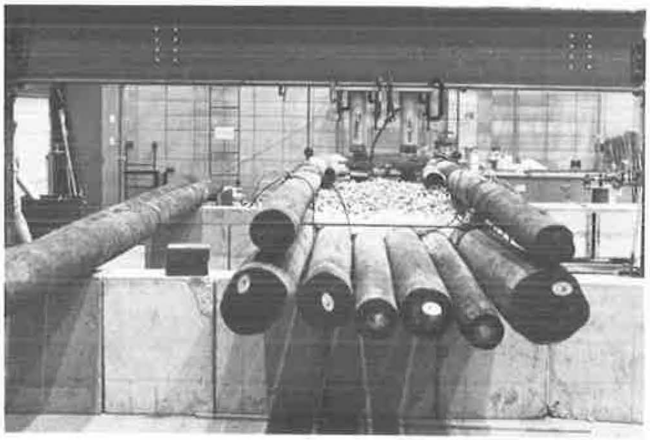


Figure 12. View showing stabilizer log and deflection dials.

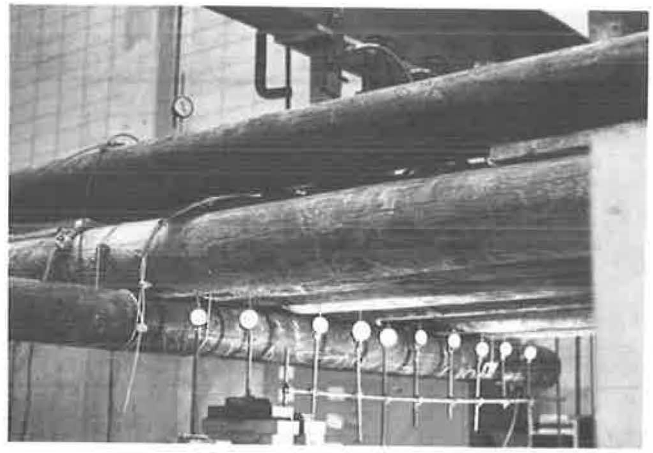
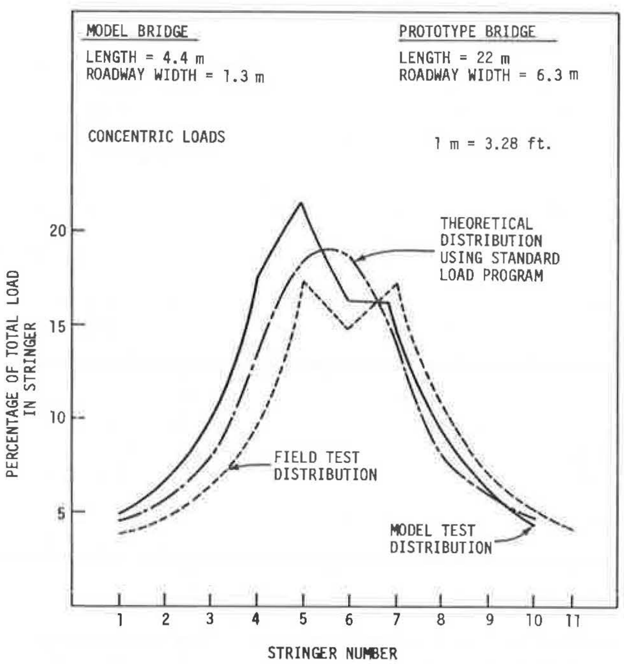


Figure 13. Comparison of theoretical, field test and model test load distribution for 22 m (72 ft) Staney Creek Bridge.



ing Figure 13) for the model bridges, it was seen that the first objective in the experimental investigation had been accomplished, i.e., that the load distributions obtained in the lab tests had consistently matched load distributions from field testing and the theoretical study. Thus, the theoretical results previously outlined could not only be considered valid for bridges similar to those field tested, but also for the entire range of Alaskan bridges as typified by the model bridges. The model bridges also showed the effect of the brow logs and stabilizer logs on load distribution. Reductions in the maximum percentage of load in the critical stringer decreased from about 5 to 15 percent with the addition of brows. Reductions of the percentage of load carried by the critical stringer in the model bridges due to the addition of a stabilizer log ranged from 6 to 27 percent.

Brow logs do much more for a bridge than merely act as a guardrail. It was shown that the brow logs are an essential structural entity of a bridge, since they can decrease the amount of load carried by the critical stringer by an average of 10 percent. Stabilizer logs can also play a role in load distribution. By placing a midspan stabilizer on a bridge, the stiffness of the bridge is increased, which can decrease the amount of load taken by the critical stringer by about ten percent. However, for the stabilizer to distribute the load across the width of the bridge, it is important that the stabilizer logs make contact with all stringers. In reality, this is difficult, thus making their practical use very questionable.

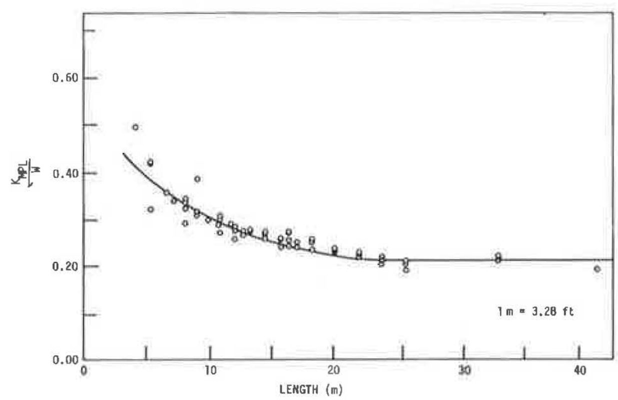
Formulation of Load Distribution Criteria

After completion of the analytical and experimental investigation of load distribution, the final phase of the ISU study was to incorporate the findings of the two investigations into a new rating criteria. The analytical investigation had shown that the behavior of the field bridges in Alaska (2) could be theoretically explained using a modified form of articulated plate theory. Once the load distribution in the Staney Creek bridges had been verified, a series of theoretical load distribution curves were produced to predict the behavior of a range of bridges commonly in use. The experimental investigation provided additional laboratory tests to supplement the actual field tests done in Alaska, and helped justify the assumptions made in the theoretical investigation.

The final step of the analytical investigation was to produce a set of theoretical load distribution curves for the range of bridges used in Alaska. The maximum values of K_{MPL} for the range of bridge sizes were compiled for both concentric and eccentric load cases. For cases with realistic configurations, these values are plotted in Figure 14. The plot relates the distribution factor, K_{MPL}/W, to the bridge span. It should be noted that this plot is for bridges without brow logs.

A least squares fit for the data relating the distribution factor with span is also shown in Figure 14. This data is being analyzed and will be developed into tabular form so that the engineer can readily determine the final design or rating factor for any bridge to be studied. This final factor will consider the effects of variables such as workmanship and variations in log properties.

Figure 14. $\frac{K_{MPL}}{W}$ vs. L for bridges without brow logs.



The maximum vehicle moment per log (M_L) can then be computed from

$$M_L = \frac{K_{MPL}}{W} M_v d \qquad (2)$$

where M_v is the maximum applied vehicle moment, d is the diameter of log (m) being rated.

The effect of brow logs has been shown to reduce the maximum moment coefficient as much as 10 percent. This method results in an average stringer moment of about 22 percent of the total truck moment, as compared to 30 percent in the current design and rating criteria (1). Since the dead load in most bridges is relatively low, the use of the revised criteria could result in an increase in rated capacities of about 30 percent.

Summary

This paper presents a summary of the results of a three-part study of Alaskan log stringer bridges. The investigations included the load testing of actual logs to determine ultimate bending stress, field load testing of four log stringer bridges to obtain experimental distribution of live load, and a comprehensive analytical and laboratory model test study of load distribution with the development of a proposed revision to the current load distribution criteria (1).

The results of the project are more accurate ultimate bending stresses for log stringers and a more realistic load distribution criteria for rating of bridges. These results will be used to develop revisions to the current design and rating procedures (1) for Alaskan native log stringer bridges and will be considered by the U.S. Forest Service.

Acknowledgments

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