DETERMINING LOAD RATING CAPACITY OF SIMPLE LOG STRINGER BRIDGES

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Abstract

This essay uses referenced values of allowable stresses to determine the load rating capacity of a constructed Douglas Fir Log Stringer Bridge. This method compares the applied moments and shear stress from the live vehicle load and the weight of the bridge to the allowable moments and shear stress the bridge is designed to support. The factors which affect the load rating capacity are stringer diameter, depth of gravel decking, span length, stringer species and eccentricity of load. It has been determined that this bridge is capable of safely supporting the Ministry of Forests L-60 On-Highway vehicle loading standard.

1. Introduction

Log stringer bridges used in forest operations are composed of two major components, the substructure and the superstructure. The substructure is the abutment which sits on the ground and supports the superstructure, while the superstructure is the component which spans the distance between the two abutments and directly supports the live load. Log stringers have traditionally been the most commonly used method of superstructure construction for coastal forest resource access roads. This has been the case because good quality building materials have been readily available onsite from harvesting the timber in the road right of way. As the transition from harvesting of old growth to second rotation stands is becoming more frequent, it is common now to use portable steel or permanent concrete/steel composite bridges for stream crossings, as stringers of adequate strength and size are no longer always available from right of way timber (Lyons et al, 2007). However, if it is at all possible to utilize the local materials for bridge construction, building costs, capital investment, and logistical planning can be significantly reduced (Tuomi et al, 1979). For this reason, log stringer bridges remain to be an attractive option for stream crossings in Forest Operations.

In order for a log stringer bridge to be properly constructed and safely support vehicle traffic, it should be built to a design which ensures the bridge is capable of supporting a load greater than will be applied. This essay utilizes a design method which is based on a paper written by (Bradley, 1991) Span Designs for Constructing Temporary Log-Stringer Bridges in Alberta, along with components from the British Columbia Forest Service Bridge Design and Construction Manual (Ministry of Forests, 1999) and (Pronker, 1995) Log Stringer Bridges – Guide for Structural Evaluation, that are more applicable for coastal BC, to come up with a load rating for an existing log bridge. The ministry of forests has various load rating classifications that represent different weights and configurations of vehicle traffic. The vehicle loading diagrams for each rating are found in the MOF Bridge Design Manual and are used in this essay to determine the load rating of the existing bridge. The design criteria from (Bradley, 1991) compare the applied moments and shear stress from the vehicle loading and the weight of the bridge to the allowable moments and shear stress the bridge is determined to be able to withstand. The factors which affect the allowable load capacity in this approach are allowable shear stress, allowable bending stress, stringer diameter, depth of gravel decking, span length, and stringer species.

There are two types of loads which the structure must be able to support, the live load and the dead load. The dead load consists of the weight of the superstructure, which consists of the stringers, gaurd logs, lashing and gravel deck. The live load is the vehicle which the bridge is designed to safely support and an example would be a loaded logging truck, or a lowbed with a yarder on it. The vehicle loading diagrams from the MOF Bridge Manual represent the live load (see appendix 1). The bridge is designed to support the live load at the most critical position; the most likely position to fail. This occurs when the maximum bending moment is applied, or in some cases with shorter spans, when the maximum shear stress is applied (Pronker, 1995).

Log stringer bridges are unique because of the great variability of strength and stiffness in the logs used to construct them. A major challenge for the successful design and construction of log bridges is to spread the load across the bridge deck so that as many stringers as possible bear the load. The variation in size, uniformity, stiffness and strength of the stringers makes it nearly impossible to construct the bridge so that the load is effectively evenly allocated to all stringers. Often the larger and stiffer stringers bear the majority of the load, thus limiting the strength of the bridge to those loaded stringers. Needle beams, cross puncheon, and ties may work well theoretically to evenly distribute the load, but the practical construction of these components is not always successful (Lyons et al, 2007). For this reason it could be detrimental to the safety of the bridge if its design strength depends on one of these components and it is not constructed effectively.

Eccentricity of load occurs when vehicles have not been loaded symmetrically, or are coming out of a corner and leaning to one side (Nagy et al, 1989). This must be accounted for as it can cause a higher load concentration on one side of the bridge than anticipated, and thus exceed the designed capacity. In accordance with the MOF Bridge Manual, this is achieved by applying either a 55% to 45% or a 60% to 40% distribution to each side of the truck (Ministry of Forests, 1999) depending on which load rating classification is used.

The objective of this essay is to use the methodology of (Bradley, 1991) Span Designs for Constructing Temporary Log-Stringer Bridges in Alberta, to determine a safe and accurate load rating for a constructed log-stringer bridge located near Roberts Lake, BC (Fig 1). This process will aid both the author and the reader to form a better understanding of the theory and methodology behind log-stringer bridge design, as well as to help form a better understanding of the limitations and advantages to this method.

Figure 1

2. Methods

The methodology is based on that of (Bradley, 1991), (Pronker, 1995), and (Ministry of Forests, 1999).

2.1 Field Data Collection

Data inputs required for the calculation of the load rating must be measured in the field at the bridge site. The following information was recorded:

- Mid-span diameter of each stringer and guard logs
- Span of bridge
- Depth of gravel
- Number of stringers
- Determined if any gaps were present between stringers

2.2 Determine the Allowable Moment and Shear Stress of One Stringer

This method uses the allowable stress in bending and longitudinal shear to compute the maximum allowable bending resistance and shear resistance for one stringer. The allowable stresses are for Douglas Fir logs and were taken from the Ministry of Forests Log Stringer Bridges Guide for Structural Evaluation. The following methodology was used to compute the bending resistance and shear resistance.

Bending Resistance of One Stringer

(1)

Where:

is the allowable normal stress due to bending

S is the section modulus,

K is the allowable stress modifier

Shear Resistance of One Stringer

(2)

Where:

is the allowable shear stress

K is the allowable stress modifier

2.3 Determine the Applied Load One Stringer Will Support

The percentage of the applied loads one stringer will bear is necessary to compute because this value will be used to calculate the maximum bending moment and shear stress. This percentage is dependent on the depth of gravel, the angle of vertical spread, the average width of the stringers, and the eccentricity of the load. Equations (3) and (4) are used in the calculation to determine the number of stringers which bear the load under each wheel (see figure 2).

(3)

(4)

Where:

N = number of stringers

B = width of the wheel or dual wheel (including gap)

= angle of vertical spread of the gravel

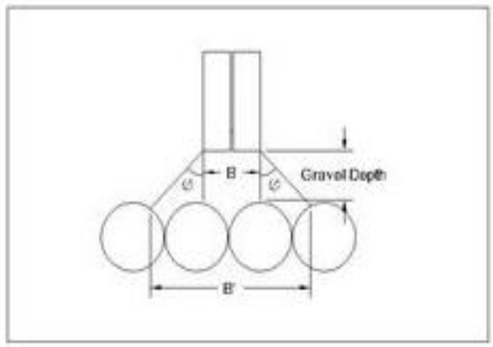

Figure 2

The axle loads applied to the bridge are first multiplied by the coefficient of eccentricity, and secondly divided by the number of stringers which support the load (Equation 5).

$$— \qquad (5)$$

Where:

= resultant axle load per stringer

The resulting axle loads are then used for calculating the applied moments and shear stress per stringer.

2.4 Determine the Maximum Applied Moments and Shear Stress

The critical positions of the vehicle are the positions in which the maximum bending moment and maximum shear stress occur. Generally the maximum shear stress occurs adjacent to one of the abutments and the maximum bending moment occurs somewhere close to the mid-span of the bridge. The max dead and live moments occur at different locations which makes it difficult to calculate the exact value of the total applied moment. A conservative and simple assumption is to add both the moments together; this gives a higher value for the total than the true value. Both the maximum shear stress from the live and dead loads occur at the same location.

Moment Dead Load

Equation (6) is used to calculate the max moment from the dead load and is calculated using values from equations (7), (8), and (9) which account for the dead load from the stringers and the gravel.

= ——— (6)

(7)

(8)

(Pronker, 1995)

= (9)

= (Pronker, 1995)

Shear Dead Load

Equation (10) is used to calculate the shear stress from the dead load.

——— (10)

$L = span = 8.1\text{m}$

Max Moment Live Load

Refer to appendix 1 for the vehicle loading diagrams used in the calculations for the live loads. P1 represents the front axle, P2 and P3 represent the drive axles on the tractor, and P4 and P5 represent the trailer axles.

In Figure 3, P2 and P3 produce the maximum resultant force (Fr). Equations (11), (12), (13), and (14) are used to determine the location of the resultant force, which is required to position the axles for the maximum bending moment.

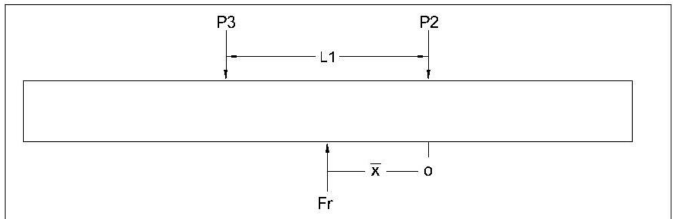

Figure 3

(11)

(12)

(13)

– (14)

The maximum bending moment occurs when P2 is located – distance from the center of the span (see figure 4). Equations (15) and (16) are used to solve for the values of the forces at the supports Ay and By.

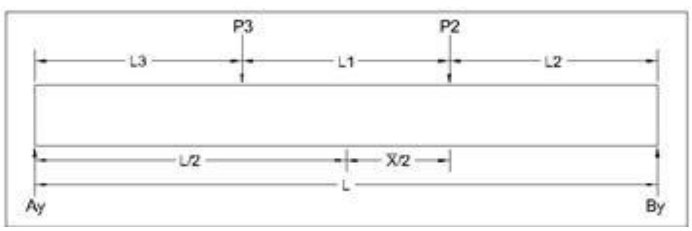

Figure 4

(15)

(16)

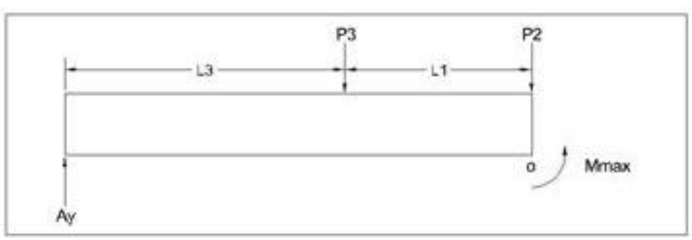

Figure 5

Taking the sum of the moments about (o) in figure 5, the value for the maximum moment is calculated using equations (17) and (18).

(17)

(18)

Max Shear Live Load

Figure 6 shows the axle position which produces the greatest shear stress on the stringer. Equations (19) through (23) are used to solve for this value.

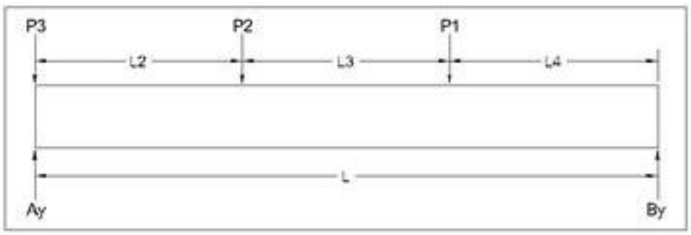

Figure 6

(19)

(20)

– (21)

(22)

– —— (23)

Where *A* is the cross sectional area of the stringer

2.5 Compare Results of Allowable and Applied Stresses

In order for the bridge to meet the loading requirements, the allowable moments and shear stress must be less than the applied moments and shear stress. This concept is shown in table 1.

Table 1

Allowable Stresses	Applied Stresses

3. Results

3.1

The constructed bridge chosen for this essay is a simple log stringer design composed of 13 stringers lashed together with wire rope and gravel decking. Guard logs are present and provide structural support as they are lashed to the stringers, but this is not accounted for in this essay. Data from the bridge site was recorded; refer to tables 2 and 3.

Table 2

Stringer #	Species	Midspan Diameter(cm)
1	Fd	51
2	Fd	54
3	Fd	49
4	Fd	55
5	Fd	51
6	Fd	51
7	Fd	42
8	Fd	42
9	Fd	58
10	Fd	54
11	Fd	49
12	Fd	51
13	Fd	55

Table 3

Average Midspan Diameter (cm)	Span (m)	Lashing Present / Stringers Snug	Depth of Gravel (cm)	Running Surface Width (M)
51	8.1	yes	30	5.8

3.2

Bending Resistance of One Stringer

An allowable normal stress due to bending for Douglas fir of 11000 Kpa was used (Pronker, 1995) and an average radius of 0.25m was used in the computation of the *S* value (see equation (1)). An allowable stress modifier of 1.25 was used to account for the fact that this is a temporary bridge (Bradley, 1991), (Tuomi et al, 1979) Hence, the value is 169.0 kNm.

Shear Resistance of One Stringer

An allowable normal stress due to shear of 5700 Kpa for Douglas Fir was used (Pronker, 1995). This yields a shear resistance value for one stringer ( ) of 7125 Kpa (see equation (2)).

3.3

Using a value of 62 cm for a value of *B*, for through to and 25 cm for a value of *B*, for and a of and equations (3),(4) and (5) the following resultant axle loads were calculated per stringer (See table 4). Refer to Appendix 1 for the vehicle loading diagrams.

Table 4

Axle	L-45	L-60	L-75	L-100	L-150	L-165
	27.2	32	32	43	64	53.9
	32.7	36	46	61.4	92.1	108.9
	32.7	36	46	61.4	92.1	108.9
	21.8	36	46	61.4	92.1	79.5
	21.8	36	46	61.4	92.1	79.5

3.4

Table 5 displays the moment and shear values for the dead load. Table 6 displays the moment and shear values for the live load for each load rating classification (see equations (18) and (23)).

Table 5

Dead Load Moment	Dead Load Shear
33.5 kNm	16.6 Kpa

Table 6

	L - 45	L - 60	L - 75	L - 100	L - 150	L - 165
Max Live Moment (kNm)	112.5	123.8	158.2	222.8	334.3	395.7
Max Live Shear (Kpa)	485	523	640.8	815.6	1222.6	1412.9

3.5

Table 7 compares the total applied stresses to the total allowable stresses. From this table it is apparent that the bridge is capable of supporting L-60.

Table 7

	Allowable Stresses	Applied Stresses					
		L - 45	L - 60	L - 75	L - 100	L - 150	L - 165
Bending Moment (kNm)	169	146	157.3	191.7	256.3	367.8	429.2
Longitudinal Shear (Kpa)	7125	501.6	539.6	657.4	832.2	1239.2	1429.5

4. Discussion

Log stringer bridges are complex structures in terms of analyzing the processes involved in resisting applied loads, so making simple conservative assumptions of the allowable stresses has been the most effective and practical choice for determining designed load rating capacity. However, there are limitations to this method. Log stringers are all variable in size, shape and stiffness to some extent, and have other defects such as knots, rot and spiral grain. These irregularities make it difficult to apply constant values for allowable bending stress and allowable shear stress when in reality these values are extremely variable. To account for this a conservative value is chosen for the allowable stresses. This can cause a significant underestimation of the true capacity of the bridge. While the intent of the design process is to determine a safe load rating for the bridge, if the value is too conservative it may not be possible to construct a bridge from local materials if sufficiently large stringers cannot be found. This is quite often the case when operating in second growth timber and then requires concrete or portable steel bridges to be brought to the site, thus increasing construction costs. The allowable stress calculation method is limited to simple gravel decked bridges. If the bridge were to be lashed with a needle beam or have a timber deck with ties, there would likely be a significant increased value of load rating capacity that would not be accounted for. Some models assume equal allocation to all stringers, which theoretically may be correct, but due to the variability in stiffness and shape of logs it is impractical to have bridges built to these specifications. This could lead to an overestimation of the load rating capacity of the bridge and that may lead to bridge failure (Lyons et al, 2007). There can be much variation in the values to be used for the allowable stresses and . The Canadian highway Bridge Design Code Manual suggest values of 19.5 Mpa and 0.9 Mpa for and respectively. These values are for select structural Douglas Fir which is most likely of greater quality and more uniform than the Log Stringers. For this reason the more conservative values from (Pronker, 1995) were used in this essay.

The allowable stress method from (Bradley, 1991) does not account for any load sharing due to the lashing and as a result the gaurd logs which are lashed to the stringers, also are not accounted for in the calculation of the load rating. (Lyons et al, 2007) developed a finite element model for gravel decked log stringer bridges which determines a value for the load sharing element between different stringers. This model determined that the effect of load sharing due to lashing is very dependent on the location of the lashing and the position of the vehicle loading. While making assumptions for allowable stresses has been a common method for design in the past, there is a shift now to a new method using limit states design procedures (Bennett et al, 2004).

The durability and lifespan of different stringer species should also be taken into account when designing or inspecting log bridges. On the coast, western red cedar, western hemlock and Douglas fir are the most common species used for bridge construction. Western Red Cedar is the most ideal species for long term bridges because it is the most resistant to decay and has a high strength. Cedar cribbing can last up to thirty years, while Douglas Fir lasts eight to ten years (Nagy et al, 1989). Douglas Fir is also a good species to use for stringers as it has a very high strength (Nagy et al, 1989). Western Hemlock has a very short life span, usually a five year period (Nagy et al, 1989) before the logs need to be replaced due to rot and decay, however, hemlock is the most abundant species on the coast and has relatively low market value compared to other species. For this reason it can be the most economical option if it is used in a short term bridge crossing application. Service life is quite variable, and bridges should be inspected every couple years for rot and decay (Nagy et al, 1989).

5. Conclusion

Using the methodology for determining load capacity of log stringer bridges from (Bradley, 1991), it has been determined that the recently constructed log stringer bridge near Roberts Lake, BC meets a maximum of the L – 60 on highway standard from the Ministry of

Forests Bridge Design and Construction Manual. This means the bridge is capable of safely supporting highway logging trucks and low-beds because the anticipated applied bending moment and shear stress from these vehicles are of lesser value than the designed allowable bending moment and shear stress of the structure. Any higher standard such as off-highway L-75, L-100, L-150 or L-165 would apply moments or shear stress greater than the design allows.

6. References

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7. Appendix 1

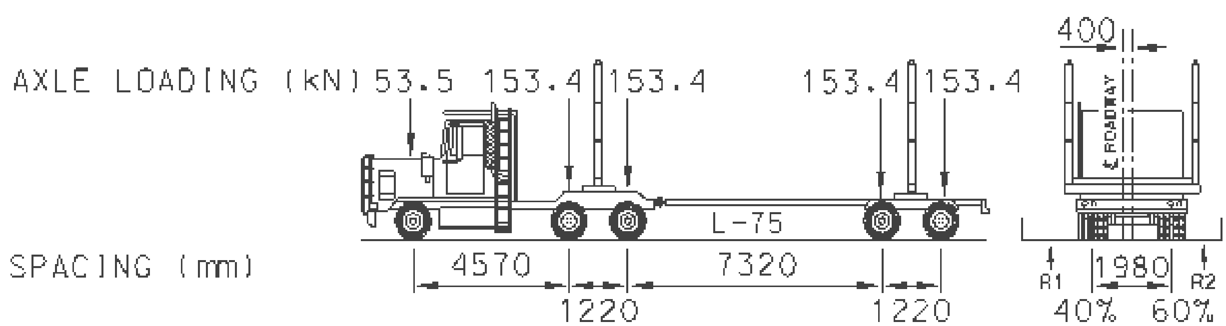
Source for Loading Diagrams and tables (Ministry of Forests, 1999), and (Pronker, 1995). No Diagrams are available for L-45 and L-60.

L-45 (On-Highway)

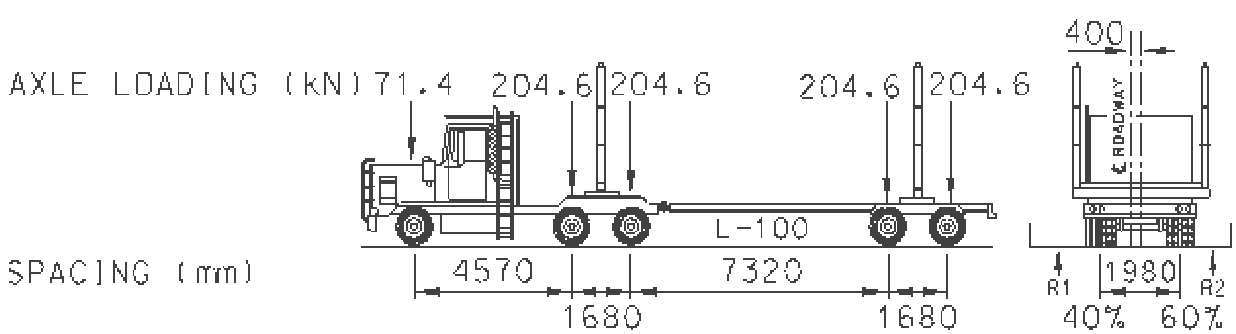
Axle	Force (kN)	Spacing	Distance (mm)
P1	45.4	P1-P2	3660
P2	108.9	P2-P3	1220
P3	108.9	P3-P4	6100
P4	72.6	P4-P5	1220
P5	72.6		

L-75 (Off-Highway)

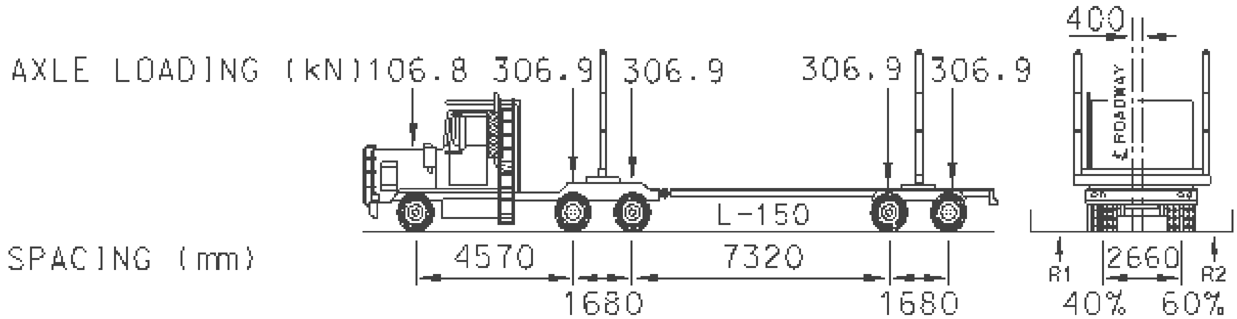
Axle	Force (kN)	Spacing	Distance (mm)
P1	53.4	P1-P2	4270
P2	120.1	P2-P3	1220
P3	120.1	P3-P4	7320
P4	120.1	P4-P5	1220
P5	120.1		



L-100 (Off Highway)



L-150 (Off-Highway)



L-165 (Off-Highway)