Simplified Strengthening Methods for Halfcap Splices of Timber Bridges

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Abstract

Timber halfcap scarf splices (HCSS) installed in unfavourable (inverted) orientations are prone to splitting at the internal re-entrant corners of the joint, drawing the capacity of the connection into question. Repairing HCSS which show signs of deterioration and distress is a costly and extensive process. The objective of this project is to develop simplified strengthening methods that improve the load carrying capacity of HCSS. Finite element analysis was used to develop and optimise strengthening methods. To verify the performance of strengthening methods, an experimental program was conducted consisting of modified three-point bending tests with fixed-fixed support conditions on twelve 200×75mm 2.4m pine sleeper specimens. Clamps have not been shown to increase the load carrying capacity of the joint during experimental testing. There is potential that clamps improve the ductile failure characteristics of the beam with further investigation required to confirm these observations. The addition of a parallel flanged channel spanning the joint and secured with clamps on the bottom face of the beam has been shown to increase both the load carrying capacity and ductile failure characteristics of the joint. Coach screws have shown no improvement to either metric.

1. Introduction

Western Australian timber bridge structures are comprised of four key structural components. Stringers, corbels, halfcaps and piles (Figure 1) are used to support the bridge deck and accompanying traffic loads. Halfcap beams are used to transfer the load from stringers and corbels into both abutment and pier piles. Historically, local hardwoods such as Jarrah and Wandoo were predominately used in Western Australia to construct timber bridges (Chandler & Adsett, 1998). The maximum available length of sawn timber suitable for halfcap use in Western Australia is 7.5m (Main Roads Western Australia, 1979). It is necessary to join halfcaps in any timber bridge wider than 7.5m, or wherever the available sawn timber is shorter than the required halfcap length. The most common method of joining halfcap beams in Western Australia is the halfcap scarf splice (HCSS).

Two orientations of HCSS exist; upright and inverted (Figure 2). Inverted HCSS are the focus of this project. The geometry, orientation and load transfer behaviour of inverted HCSS make them susceptible to cracking in the region surrounding the internal re-entrant corner of either beam. These cracks form due to the development of tensile stress in the direction perpendicular to the grain.

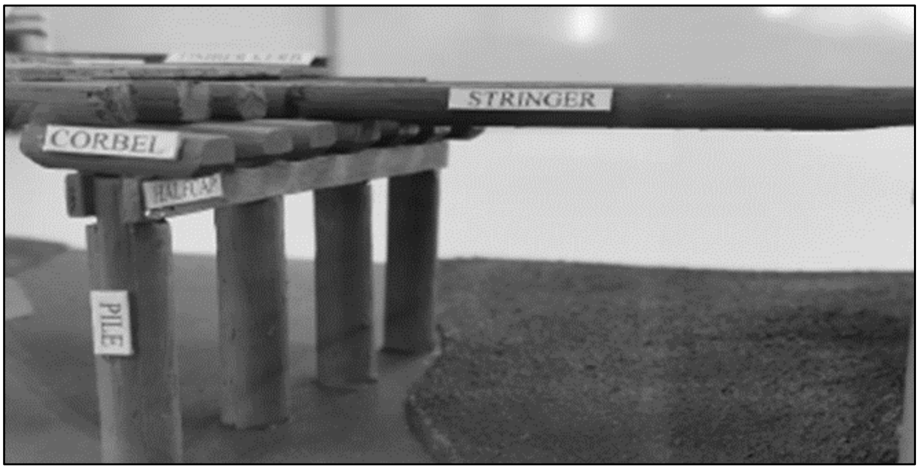

Figure 1 Typical timber bridge structural components (Qi, 2022)

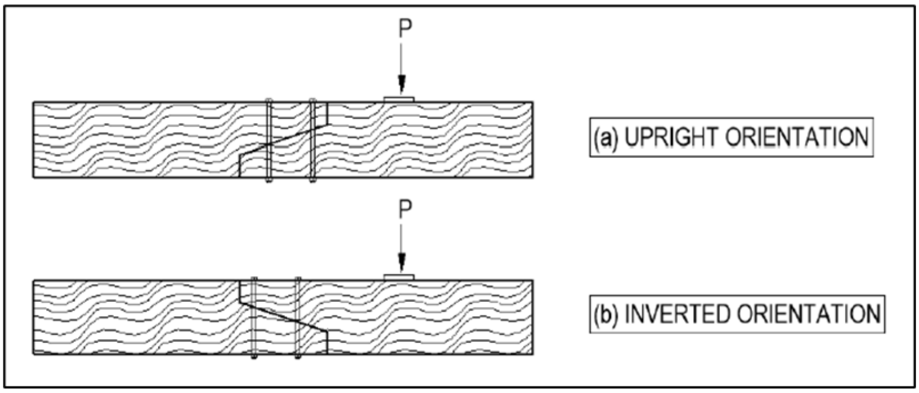

Figure 2 Halfcap scarf splice orientations (Qi, 2022)

Industry design standards and guidelines provide notably low theoretical predictions of the capacity of HCSS, to the extent that permanent loads are considered to exceed the capacity of the joint in some cases (Chang, 2021). Recent research conducted by Chang (2021) found that the joint requires the applied load to be increased beyond the point of crack initiation to continue crack propagation and reach the ultimate failure load of the splice. This finding indicates that if an alternate load path can be provided the structural stiffness and capacity of the beam may be retained.

Currently, intervention and strengthening are only performed on halfcaps and HCSS showing signs of deterioration. Single span strengthening or the complete replacement of halfcap beams with steel parallel flanged channel (PFC) beams are currently performed, however, these methods are labour intensive and costly. Main Roads Western Australia require research to be conducted on simplified methods of strengthening inverted HCSS to make informed decisions on the future of their timber bridge maintenance strategy.

1.1 State of the Art

Patalas et al. (2022) and Karolak (2021) have attempted to use finite element analysis to model scarf joints, which are comparable to HCSS. Unlike steel, timber is an orthotropic material that exhibits unique strength characteristics in three principal directions. Timber is significantly weaker in the radial and tangential directions relative to the direction of the grain. It is common to model timber as transversely isotropic, as the difference between the radial and tangential directions is often small (Baño et al., 2011). This is implemented in Ansys using orthotropic elasticity, which allows for the definition of elastic and shear moduli in addition to Poisson's ratio in all principal directions and shear planes.

Timber typically experiences brittle failure under tensile stress and displays considerable deformation under compression. Plasticity and damage cannot be combined in Ansys without the use of extensive user defined material models. This may not be of concern, as under tension and longitudinal shear timber exhibits nearly linear elastic behaviour up until failure (Šmídová et al., 2022). Progressive damage analysis uses failure theories to predict damage initiation and evolution (Shahbazi, 2016). Continuum damage mechanics introduces strain softening behaviour that is controlled by fracture energy. Hashin damage is the only failure criteria that supports continuum damage mechanics in Ansys (Shahbazi, 2016).

A non-linear fracture mechanics approach known as the cohesive zone model can be used to simulate the behavior of materials exhibiting delamination, debonding, or other forms of cohesive failure (Coureau et al., 2013). Cracking caused by tension perpendicular to the grain generally propagates parallel to the grain (Coureau et al., 2013). As the crack path has already been identified in HCSS, predefined failure surfaces can be simulated in the geometry of the finite element model. The cohesive zone model is less susceptible to convergence issues associated with strain softening of traditional damage models, as it allows for large deformations to occur once delamination of the interface is complete.

1.2 Project Objective

The primary objective of the project is to develop and optimise strengthening methods for HCSS. A finite element model should be created to evaluate the potential of strengthening methods. Strengthening methods must be physically feasible for implementation on in-service timber bridge structures and are required to either provide an increase to the ultimate capacity or improve the ductile failure characteristics of the joint.

2. Methodology

A combination of finite element analysis and scale experimental testing is being used to develop and compare the influence of strengthening methods on ultimate load capacity, load-displacement behaviour and failure modes. This information will then be used to make recommendations on the feasibility of full-scale implementations.

200×75mm 2.4m pine sleepers were chosen to represent halfcap beams as they align closely with the beam depth-width ratio of in-service halfcap beams and are readily available (unlike their Jarrah counterparts). Chang (2021) did attempt to use ex-service Jarrah bridge deck sleepers, however, due to their deterioration reliable results could not be established. Steel Grade 4.8 M8 threaded rod was chosen to maintain the same ratio of bolt tensile strength to beam shear capacity as full-scale scenarios. Elastic and strength properties (Table 1) of pine sleepers are derived, using the approach taken by Patalas et al. (2022), from the modulus of elasticity parallel to the grain and bending strength determined by Chang (2021). The Poisson's ratio and friction coefficients used by Patalas et al. (2022) have been adopted for this analysis. Several other representative material properties have been determined from the work of Gharib et al. (2017).

As HCSS occur in continuous spans of beams, restraint plates are used on the top surface of the beam to limit rotation and create an approximately fixed-fixed scenario using a standard three-point bending apparatus (Figure 3). Load is applied 700mm from the left-hand support of a 1600mm span, with the centre of the scarf splice located 400mm from the left-hand support. This layout attempts to replicated in-service conditions with the centre of the splice close to the theoretical zero bending moment location. During experimental testing a constant displacement of 2.5mm per minute was applied using the UWA 3000kN Instron material testing machine. Load cell data and displacement of the hydraulic piston was recorded at a frequency of 10Hz. Water jet cutting was used to produce accurate scarf splice samples.

Properties	Values		
E_x	7385	MPa	(Chang, 2021)
E_y, E_z	246.17	MPa	
G_{xy}, G_{xz}	461.56	MPa	
G_{yz}	46.16	MPa	
ν_{xy}	0.37		
ν_{yz}	0.47		
ν_{xz}	0.42		
$\mu_{timber\text{-}timber}$	0.5		
$\mu_{timber\text{-}metal}$	0.3		
$\mu_{metal\text{-}metal}$	0.2		
f_b	33.36	MPa	(Chang, 2021)
f_x^t	20.02	MPa	
f_x^c	24.23	MPa	
f_y^t, f_z^t	0.5	MPa	(Gharib et al., 2017)
$G_x^{frc,t}$	60	N mm^{-1}	(Gharib et al., 2017)
$G_y^{frc,t}$,$G_z^{frc,t}$	0.5	N mm^{-1}	(Gharib et al., 2017)

Table 1 Material properties used in the finite element model

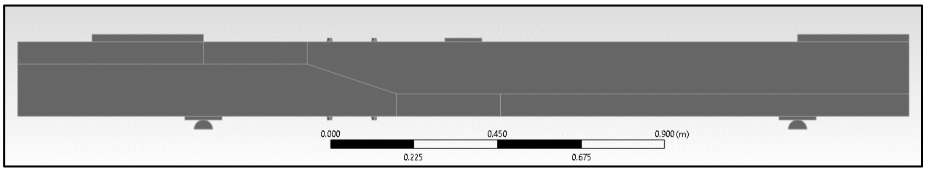

Figure 1 Experimental and FEA test arrangement

3. Results and Discussion

Following a detailed investigation of possible solutions, three methods were selected for experimental testing. Clamps and coach screws were selected as separate methods to provide reinforcement in the vicinity of the re-entrant corner to limit crack expansion. PFC spanning the joint and secured onto the bottom face of the beam with clamps was selected as a method to provide a form of flexural reinforcement. A total of twelve experimental tests have been caried out to date (Table 2). Natural defects such as branch knots, cross graining and the proportion of sap, heart and pith wood have been observed to effect the strength and load-deflection response of test samples. Considerable crushing in the direction perpendicular to the grain has been observed beneath the load and at the supports in the majority of tests. Three specific tests have been identified as outliers (Table 2) due to their significantly reduced ultimate load capacity and abnormal load-deflection response.

Strengthening method	Experimental ultimate load (kN)	Mean ultimate load (kN)	FEA ultimate load (kN)	Relative error
Unstrengthened	37.8 30.1*	37.8	38.1	0.9%
Solid Pine	59.5 58.5	58.8	54.1	–8.0%
30x30 EA Clamp	37.6 34.3*	37.6	43.3	15.1%
100x50 PFC Clamp	38.4	38.4	44.0	14.7%
Short Bottom PFC	50.3	50.3	56.1	11.5%
Long Bottom PFC	50.0	50.0	55.4	10.9%
Coach Screw	36.1	36.1	42.5	17.6%
Coach Screw x 2	37.3	37.3	-	-
Bolt	30.3*	-	-	-

* Outlier - weak sample

Table 2 Summary of experimental and numerical ultimate load results

In the case of an unstrengthened inverted scarf splice, there is good agreement between the numerical and experimental ultimate load capacity. With the exception of a solid pine sleeper, the current numerical model over predicts the ultimate load capacity of a strengthened splice by between 10.9% and 17.6% (Table 2). The numerical model predicts the approximate location and mode of bending failure that was observed during experimental testing. The simulation of re-entrant corner cracking is confined to a predefined delamination plane and assumes a perfectly uniform crack front. This idealised modelling approach may artificially increase the beam's load-bearing capacity since the crack cannot freely propagate through the material and stress concentration features are absent. In addition, the current model does not fully capture the “perpendicular to grain” crushing that was observed, which may also partially account for the differences in capacity. Further investigation should be completed to address these short comings in the model; however, it has been shown to be a useful tool in optimising strengthening solutions.

Clamping of the timber in the area surrounding the scarf splice has shown no significant increase to the ultimate load capacity of the joint. The experimental load-deflection response suggests that there is potential for enhanced post-first-crack residual load carrying capacity, enabling substantial deformation while supporting a significant proportion of the ultimate load. Although additional testing is required for confirmation, this failure behaviour is considered valuable. Timber bridge structures may offer acceptable load redistribution via other members, contingent on the halfcap maintaining post-first-crack load capacity. This contrasts with the project's original focus on ultimate strength, signifying an evolved goal that includes enhancing post-first-crack performance and ductility.

Flexural reinforcement of the splice using 100×50mm PFC has been observed to provide superior capacity and load-deflection response when compared to an unstrengthened sample. Current experimental results indicate a 33% increase relative to unstrengthened capacity, translating to 85% of a solid timber sample, in contrast to an unstrengthened splice's 64% of a solid timber sample. The sole apprehension with this approach is that the weight of a PFC section suitable for reinforcing a full-scale HCSS might pose challenges during installation in unfavourable on-site conditions. This may have potential to be addressed with the use of two separate steel unequal angle sections.

Coach Screws proved ineffective as they withdrew from the timber during testing. Bolts and washers were used to approximate the effect of coach screws if they remained embedded, however, no improvement was observed when compared to an unstrengthened specimen. It must be noted that for this case the timber sample was compromised due to the presence of branch knots. In the numerical model, bonded contact between the coach screw and timber prevented accurate simulation of timber thread stripping observed during testing. Coach screws are not highly supported as they introduce defects to an already vulnerable region of timber and are unsuitable for installation on the top surface of a halfcap beam due to access restrictions.

4. Conclusions and Future Work

The current finite element model appears to overpredict the impact of strengthening methods. Experimental testing has revealed that clamps and flexural reinforcements have potential to enhance the residual load carrying capacity of the joint, however, coach screws have proven ineffective. A parametric finite element investigation is necessary to refine the model's performance. Further scale experimental testing is required to confirm the observations gathered so far. Full scale testing of ex-service HCSS will not occur during the project timeline and is recommended before applying any method to in-service timber bridges structures.

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