FINAL
FAILURE INVESTIGATION
WESTCHESTER NORTH LAGOON BRIDGE
Anchorage, Alaska

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TABLE OF CONTENTS

ACRONYMS.................................................................................................................................................. II
1 EXECUTIVE SUMMARY....................................................................................................................... 1
2 INTRODUCTION.................................................................................................................................... 1
3 DESCRIPTION ....................................................................................................................................... 1
4 SITE OBSERVATIONS ............................................................................................................................ 5
5 ANALYSIS .............................................................................................................................................. 6
6 CONCLUSIONS .................................................................................................................................... 8

FIGURES
Figure 1 – Vicinity and Location Map ...................................................................................................... 3
Figure 2 – Bridge Elevation ......................................................................................................................... 4
Figure 3 – Bridge Cross Section ................................................................................................................ 10

APPENDICES
Appendix A – Original Construction Drawings
Appendix B – Calculations
ACRONYMS

AASHTO ............... American Association of State Highway and Transportation Officials
DMV ..................... Alaska Department of Motor Vehicles
Glulam.................. glued laminated timber
LRFD..................... Load and Resistance Factor Design
MOA....................... Municipality of Anchorage
NDS....................... National Design Specification for Wood Construction
PM&E..................... Project Management and Engineering
Psf ...................... pounds per square foot
UBC....................... Uniform Building Code
USKH..................... USKH Inc. – Now Stantec
1 EXECUTIVE SUMMARY

The North Lagoon Bridge failed under vehicle load when a truck and chipper crossed the bridge. The failure was caused by a connection detail that allowed a constant influx of moisture into the main supporting glulam beam. The moisture kept the interior of the beam in a saturated condition and produced decay over time. The highly concentrated force from the chipper tire applied a significant shear load across a relatively small area of glulam beam. A local failure developed at one point in the decayed area, then like a zipper, the failure plane spread outward down the beam both sides from the initial point of failure. This bottom side of the beam, ledger, and decking tumbled into the wetland.

Some summary points regarding the analysis site observation and analysis of cause of failure:

- While the new fiberglass decking added load to the bridge, it was not the cause of failure.
- Normal pedestrian loading could not have initiated this type of failure until the wood decay was significantly more advanced.
- The lag bolts allowed moisture to infiltrate the interior of the glulam beam causing decay and weakness to occur in the connection zone. The beams were preservative treated but the treatments penetrate wood only to a depth of 2 inches. The lag bolts created a path for water to penetrate into the untreated core of the beam.
- Had that connection zone not been decayed, the truck and chipper would not have caused failure.
- The ledger detail induced cross-grain tension in the main supporting member, which is not recommended in wood design.

2 INTRODUCTION

The North Westchester Lagoon Bridge failed on June 16, 2014, as a truck towing a wood chipper crossed the bridge. The MOA Parks and Recreation Department contacted USKH Inc., now Stantec (USKH) Structural Engineers and requested an emergency evaluation. As part of the emergency evaluation, MOA requested an analysis and a determination of the cause of failure.

The North Lagoon Bridge is located on the northwest end of Westchester Lagoon, see Figure 1. The bridge crosses over a wetland area between the lagoon and the railroad. Original contract documents were available for this investigation and reviewed (Appendix A). Site visits were conducted on June 16 and 17, 2014.

3 DESCRIPTION

The North Lagoon Bridge spans approximately 70 feet and is framed with glued laminated timber (glulam) beams and wood decking. The glulams are spaced 10 feet apart with the decking spanning in between supported by wood ledgers. The ledgers are inset into the glulam, and secured with lag bolts. The structure foundation is a shallow foundation system composed of timbers resting on a concrete leveling pad. A small utility conduit was attached to the bottom of the decking running along near the east glulam beam. It is our understanding that the bridge has not has any significant rehabilitation work or modifications except in 2013 when an overlay of fiberglass decking was secured to the decking and the guardrails were modified by adding another pipe railing. The original construction drawings were completed in 1987 as part of Phase 3 of the Coastal Trail construction.
The original construction drawings have a general notes section that lists design criteria and material specifications, see Figure 2. The design codes cited are the 1985 UBC and 1983 AASHTO Specifications. Pertinent design loading indicated is 85 per square foot (psf) uniform live load and an overload vehicle with 10,000-pound (lb) weight and 8,000 lb axle load. The glulams specified are 22F-V8 DF/DF and other lumber Douglas Fir No. 1. All lumber was specified to be pressure treated.
4 SITE OBSERVATIONS

Site visits were conducted June 16 and June 17, 2014. The truck and chipper had been removed prior to the site visits and information regarding the equipment was supplied by the Municipality of Anchorage (MOA). The dimensions of the main bridge glulam beams, ledger, decking, and lag bolts were measured and compared to the construction document drawings. The glulam and lag bolts match the specific dimensions but the ledger connecting the decking to the glulam was slightly wider than called out. The called out ledger was a 4x6 which has actual dimensions of 3-1/2 by 5-1/2 inches. The measured depth matched the 5-1/2 inches but the measured width varies from 5 to 5-1/4 inches. The ledger inset dimension appeared to vary from 3/4- to 1-inch. This variation would not significantly change the capacity of this connection.

The east side glulam beam failed when the bottom part of the beam split off from the main beam and came to rest in the wetlands. Most of the deck ledger was split. A portion remained connected to the glulam while other portions remained connected to the deck. The upper portion of the west glulam was still intact spanning the wetland. The east glulam appeared undamaged. The east ledger was rotated with the twisted surface of the decking and had a crack in line with the bolts at the points of maximum twisting.

The surfaces of the glulam at the exposed failure plane were closely examined. The surfaces appeared moist and brown; stained from infiltrated moisture. The lag bolts were corroded, indicative of having been exposed to moisture. The darkest staining of the failure plane was near the point where the deck collapsed beneath the wood chipper, as shown in the newspaper photos.

The outside both glulam beams were weathered and grey. There was some minor checking and cracking in the sides of the remaining beam. There was larger checking and splitting in the top of the beams in line with the lag bolts that secure handrail mounts. The east ledger has areas of water staining and moisture on the surface. The top of the wood decking could not observed because it was covered by the fiberglass deck overlay.

Photos taken during the site visits follow:
5 ANALYSIS


Only Limit State Strength I was checked to determine dead and live load capacity under the failure condition. Deflection and other serviceability limit states, seismic, wind, and foundations were not checked as they are not applicable to the failure. The current pedestrian bridge specification recommends 90 psf loading, non-reducible. Older versions of the pedestrian specifications recommend 85 psf, which was reducible when the area of deck supported exceeds a minimum threshold value. Given the area of the bridge deck for this bridge, the design live load could be reduced down to 69 psf under previous editions of the specification. The design documents reference 85 psf so it was assumed the bridge was designed for the reduced load.

MOA provided information regarding the truck and the chipper crossing the bridge as it failed. The truck was a 2006 Ford F-550 with an Alaska Department of Motor Vehicles (DMV) registration weight of 7,099 lbs. The chipper was a 2005 Morbark Tomado 15 with a listed shipping weight of 7,300 lbs. The truck has single tires in front and dual tires in the rear. The truck was empty of tree chippings at the time of
The chipper has a single axle with single tires on each side. The truck appeared custom with a cargo section on the rear so an estimation of front-rear distribution of load and wheelbase of the truck was made by reviewing manufacturer specifications regarding the type of truck. The chipper was assumed to apply little tongue weight to the truck, so the weight of the chipper was fully applied to the single axle.

The new handrail and fiberglass deck overlay added in the 2013 trail rehabilitation added approximately 7% weight to the bridge. Modifications made to structures that cause a load increase of less than 10% are usually not considered significant.

Wood member allowable stresses were taken from the current AASHTO or NDS provisions based on the grades of wood noted in the general notes on the original construction documents.

The analysis shows the capacity of the main glulam beams is limited by an overall uniform live load of approximately 72 psf. The main beams have adequate capacity for the design vehicle and actual truck plus chipper vehicle loading. The decking is limited by the point loading from vehicles. It is loaded to its maximum capacity by the vehicle that crossed the bridge. The ledger connection appears to have adequate capacity for both uniform load and concentrated vehicle load assuming a bearing limit state on the notched glulam. Full calculations are included in Appendix B.

The failure mode, confirmed by analysis and site observations, appears to be cross-grain tension in the glulam beam. The ledger is set inboard of the glulam from 3/4- to 1-inch near the bottom of the glulam beams. The location means the bottom of the beam is loaded inducing cross-grain tension in the glulam. The NDS does not recognize this type of loading since wood is extremely weak in cross-grain tension, so designing connections that rely on cross-grain tension is not a recommended practice. The NDS gives recommendations for radial tension in curved glulam beams, which is basically the same as cross-grain tension. Using the allowable radial tension stress limitations given in NDS, and an assumption for length influenced by the tire loading, gives a Demand-Capacity Ratio of about 1.4. This means under the assumptions listed, the stress caused by the tire exceeds the allowable stress by about 40%. The code has a typical average factor of safety of 2, so in practice the vehicle loading did not cause failure until the plane was weakened when moisture infiltration and decay reduced the strength.

Reference the figures below from American Institute of Timber Construction and American Panel Association addressing this type of connection.

![Diagram](image)

1 AITC 104-2003 - Typical Construction Details

1 American Institute of Timber Construction (AITC). AITC 104-2003 Typical Construction Details
A review of the as-built drawings and site investigation shows how this plane of weakness developed. Water drains off the deck onto the ledger below and eventually migrates down to the lag bolts drilled into the glulams. While in the glulams, the moisture collects in the wood and although some moisture will dissipate through the wood and back to the surface, most will stay. This results in this location of the glulam remaining in a saturated condition over time causing decay. The black/brown stains and the moistness of the beams at this area confirm this decay zone. The ledger detail relies on cross-grain tension developed in the bottom of the beam to transfer the load. This load transfer detail is not recommended as wood is weakest in this loading condition. A calculation using some assumptions showed that this was the weak link of the design. The connection held until strength reduction from wood decay reduced this capacity gradually over time.

6 CONCLUSIONS

The North Lagoon Bridge failed under vehicle load. The failure mechanism was cross-grain tension along the plane of the lag bolts attaching the deck support ledger to the main glulam beams. Although this was not a recommended detail, it did not fail until years of water infiltration and decay weakened a plane defined by a line of lag bolts through the ledger.
Water draining off the deck onto the ledger below and eventually migrated down the lag bolts drilled into the glulams. The constant influx of moisture into the glulam beam kept the plane in a saturated condition and decayed over time. The ledger detail relied on cross-grain tension developed in the bottom of the beam to transfer the load. The highly concentrated force from the chipper tire applied a significant shear load across a relatively small area of glulam beam. A local failure developed at one point, then like a zipper, the failure plane spread outward down the beam both sides from the initial point of failure. This bottom side of the beam, ledger, and decking tumbled into the wetland.

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- Had that connection zone not been decayed, the truck and chipper would not have caused failure.
- The ledger detail induced cross-grain tension in the main supporting member, which is not recommended in wood design.
WEAKENED CONNECTION

FAILURE INITIATED

FAILURE

GLULAM GIRDER

WEAKENED PLANE CAUSED BY CONCENTRATED CROSS GRAIN TENSION STRESS FROM VEHICLE LOAD

GLULAM GIRDER

CRACK FORMED AT WEAKENED PLANE

GLULAM GIRDER

FAILURE AT WEAKENED PLANE

GLULAM GIRDER

4 X 6 LEDGER WITH 3/4"Ø X 12" LAG BOLTS WITH M.I. WASHERS AT 9" O.C.

DRAINAGE

MOISTURE ENTERING BEAM THROUGH LAG BOLTS

WATER INFILTRATION

GUARD RAIL

FIBERGLASS DECK OVERLAY - 2013 CONSTRUCTION

5" NOMINAL DECK - ORIGINAL CONSTRUCTION

BRIDGE CROSS SECTION
Appendix A – Original Construction Drawings
**TYPICAL SECTION**

**LEDGER DETAIL**

**HAND RAIL DETAILS**

**END DETAIL**
Appendix B - Calculations
Dead Loads

Glulam Beams: \(10\frac{3}{4} \times 39\) \(\quad 419 \text{ in}^3 = 2.91 \text{ ft}^3/\text{ft}\)

\[wt = 2.91 \left(0.050 \text{ kcf}\right) = 0.146 \text{ k/ft}\]

\[0.146 \times 70 = 20.4 \text{ k}\]

Decking: \(3\frac{1}{2}\) thick - use \(3.75\)

\[
\frac{3.75}{12} \left(0.050\right) = 0.0156 \text{ ksf}
\]

\[0.0156 \times 10 \times 70 = 10.9 \text{ k}\]

Ledger: \(4 \times 6\)

\[
\frac{19.25}{144} \left(0.050\right) \times 2 = 0.19 \text{ k}
\]

Guardrail: w/ connection, 11 lb/ft

\[(11) \times 70 \times 2 = 11.5 \text{ k}\]

Total DL = \(20.4 + 10.9 + 0.9 + 11.5 = 33.7 \text{ k}\)

Fiberglass Overlay

strongwell Strongdek \( wt = 258 \text{ psf}\)

\[
(2.58) \times (10) \times 70 = 1.8 \text{ k}
\]

Additional Guardrail

\[(3) \times 70 \times 2 = 0.15 \text{ k}\]

Total Rehab Items = \(1.8 + 0.15 = 2.3 \text{ k}\)

Total DL = \(33.7 + 2.3 = 36.0 \text{ k}\)

\%

\% increase = 6.8%
Live Loads

Uniform = 85 psf

Vehicle: 10000 lb with 8000 lb axle w/o impact

Under older Guide Specifications for Design of Pedestrian Bridges, the main supporting members may have the loading reduced if the deck influence area exceeds 400 square feet.

\[ w = 85 \left( 0.25 + \frac{15}{1000} \right) = 85 \left( 0.25 + \frac{15}{1700} \right) = 69 \text{ psf} \]

Vehicle at Failure:

Chipper: 2005 Morbark Terra Do 15 7300 LB
Truck: 2006 Ford F550 (Custom) 7894 LB

Chipper has single axle, one tire per side
Truck has front / rear axles, double tires per side
Span = 70'   width = 10'

Main Beams:
10 1/4" x 39 G-LB

Reference Design Values
F_{by} = 21.2 \text{ ksi}
E_{x0} = 11.7 \times 10^3 \text{ ksi}
G_{0} = 0.15
F_{y0} = 0.265 \text{ ksi}

F_0 = F_{by} C_{kp} C_m (C_f \text{ or } C_v) C_{fu} C_i C_d \phi

C_{kp} = \frac{2.15}{\phi} = \frac{2.15}{0.95} = 2.24 \text{ (bending)}
C_m = 0.80
C_v = \left[ \left( \frac{12.0}{39} \right) \left( \frac{5.125}{10.75} \right) \left( \frac{21}{70} \right) \right]^{0.10} \leq 1.0 = 0.732
C_{fu} = 1.0
C_i = 1.0
C_d = 1.0
C_x = 0.80 \text{ (consider only strength in limit state)}

\Rightarrow F_0 = 21.2 (2.24)(0.80)(0.732)(1.0)(1.0)(1.0)(0.8) = 3.03 \text{ ksi}

E = E_0 C_m C_i
= 11.7 \times 10^3 (0.80)(1.0) = 1.42 \times 10^3 \text{ ksi}

F_v = 0.265 (3.33)(0.975)(1.0)(0.80) = 0.618 \text{ ksi}
\[ M_r = \phi M_n \]
\[ M_n = F_{PVE} SCL \]
\[ R_b = \sqrt{\frac{70 \times 12 \times 39}{(10.75)^2}} \leq 50 = 16.18 \]
\[ F_{PVE} = \frac{(1.10)(1.42 \times 10^3)}{(16.18)^2} = 5.53 \]
\[ A = \frac{5.53}{3.03} = 1.83 \]
\[ C_L = 1 + 1.83' = \sqrt{\frac{(1 + 1.83)^2}{3.61} - \frac{1.83}{0.95}} = 0.95 \]

CV is lesser, therefore CV Controls

\[ S = \frac{bd^2}{6} = \frac{(10.75)(39)^2}{6} = 2725 \text{ in}^3 \]
\[ \phi M_n = 0.85 \left(3.03 \text{ ksi}\right) (2725 \text{ in}^3) (10) = 7020 \text{ in-k} \]

\[ V_r = \phi V_n \]
\[ V_n = \frac{F_{vb} d}{1.15} = \frac{0.648 (10.75)(39)}{1.15} = 173 \text{ k} \]
\[ V_r = 0.75 (173 \text{ k}) = 130 \text{ k} \]
Glulam beam check - bending

- Uniform live load

\[ M_u = \frac{1.25 \left[ \frac{52 \text{ ft}^2}{5 \text{ ft}} \right] (70 \text{ ft})^2}{8} + 1.75 \left( \frac{8.5 \text{ psf}}{5 \text{ ft}} \right) (5 \text{ ft}) (70 \text{ ft})^2 \]

\[ = 6555 \text{ ft-k} = 7855 \text{ in-k} \]

\[ \frac{7855}{7020} = 1.12 \text{ % exceeded if consider full 8.5 psf} \]

Live load limit at 1.10

\[ 7020 = 1.25 \left[ \frac{52 \text{ ft}^2}{5 \text{ ft}} \right] (70 \text{ ft})^2 \left( \frac{12}{1000} \right) \]

\[ \left( \frac{1000}{12} \right) 4631 = 1.75 \left( \frac{52 \text{ ft}^2}{5 \text{ ft}} \right) \Rightarrow w = 72 \text{ psf} \]

allowable live load with \( P/L = 1.10 \)

Above the reduced uniform live load

- Consider moment with truck / chipper

\[ \begin{aligned}
3650 & \quad 1950 & \quad 1600 \\
\downarrow & \quad \downarrow & \quad \downarrow \\
\hline & \quad & \\
3650 & \quad 1950 & \quad 1600 \\
\downarrow & \quad \downarrow & \quad \downarrow \\
10' & \quad 11' & \quad 13'
\end{aligned} \]

\[ V_u \max = 6.3 (1.12) (1.75) = 13.2 \text{ k} \]

\[ M_{u \max} = 99.4 (1.12) (1.75) = 209 \text{ ft-k} = 2505 \text{ in-k} \]

Add OL \[ V_u = \left( \frac{52 \text{ ft}^2}{5 \text{ ft}} \right) \left( \frac{10 \text{ ft}}{2} \right) (425) = 11.4 \text{ k} + 13.2 \text{ k} = 24.6 \text{ k} \]

\[ \frac{24.6 \text{ k}}{130} = 0.19 \text{ OK} \]

\[ M_u = 2399 + 2505 = 4834 \text{ in-k} \]

\[ \frac{4834}{7020} = 0.70 \text{ OK} \]

Main beams ok for vehicle load 

but limited to uniform live load of 72 psf
Decking

5" wide x 3 3/4" deep

DF Decking

F_{D_{Ct}} = 1.45 \text{ ksi} \ (5/2)

1.65 \text{ ksi} \ (\text{rep})

E = 1.7 \times 10^3 \text{ ksi}

F_{vo} = 0.112 \text{ ksi}

C_{KF} = 2.94

C_m = 0.45 \ (\text{decking}) \ 0.97 \ (\text{shear})

C_p = 1.10 \ (\text{included in NOS values})

C_{pm} = 1.10 \ (\text{included in NOS values})

C_f = 1.10

C_d = 1.0

C_x = 0.05 \ (\text{strength})

F_b = 1.65 \times (2.94)(0.45)(1.10)(1.10)(1.10)(0.8) = 3.30 \text{ ksi}

E = E_0 C_{mf} G

= 1.7 \times 10^3 \times (0.97)(1.10) = 1.53 \times 10^3 \text{ ksi}

Bending strength:

S = \left( \frac{5}{6} \right) \left( \frac{3.75}{11.7} \right)^2 = 11.7 \text{ in}^3

\phi_{bw} = 0.85 \left( 3.30 \text{ ksi} \right) \left( \frac{6}{11.7} \right) \left( \frac{10}{1} \right) = 32.8 \text{ in} - \text{k per plank}

Shear:

\phi = 0.175

C_{KF} = \frac{25}{0.175} = 143

F_v = 0.18 \left( 3.33 \right) \left( 0.97 \right) \left( 1.10 \right) \left( 0.8 \right) = 0.4165 \text{ ksi}

\phi_{Vw} = 0.175 \ F_{v bd} = 0.175 \left( 0.465 \right) \left( 5 \right) \left( 3.75 \right) = 4.4 \text{ k per plank}

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Project Title

Page Title

W.O. # Date By Checked By

Page 6 of
- Check LC on decking

  Decking wt = 18.6 + 2.6 = 21.2 psf
  Pedestrian LL = 85 psf

  Load Factors

  DC  PL
  S+R  1.25  1.75

  ⇒ load = 19.2 (1.25) + 85 (1.75) = 171 psf

  
  \[ m_u = \frac{(171)(5^2)}{8} = 891.17 \text{ ft-lb} = 101.7 \text{ m-k} \quad \text{OK} \]

  Tire load: use 20 m x 10 m contact area

  \[ 20 \times 10 \]

  \[ 3650 \text{ lbs} \]

  \[ 6 \text{ ft} \]

  \[ m_u = 153 \text{ m-k} \]

  \[ V_u = 6.39 \text{ k} \]

  \[ 7.67 \text{ k} \]

  PL only

  \[ 17.1(1.25) = 21.4 \text{ psf} \]

  \[ m_u = \frac{(21.4)(10)^2}{8} = 273 \text{ ft-lb} \quad \text{over 30 m contact area} \]

  \[ = 681 \text{ ft-lb} \]

  \[ = 81.2 \text{ m-k} \]

  Total \[ m_u = 184 + 8 = 192 \text{ m-k} \]

  \[ d/mn = 32.8 \text{ (6 planks)} = 19.7 \text{ m-k} = 0.97 \quad \text{OK} \]

  \[ m_u V_u = 17.1(10)(5^2)(10) = 7.67 \text{ k} = 7.94 \text{ k} \]

  \[ d/V_u = 6 \times 17.1(10) = 264.1 \text{ k} \quad \text{OK} \]
"Connection design per 2005 NDS"
- If neglect notch and bearing on notch, consider only leg in shear.

\[ Z' = Z \times C_m \times \phi \times f_d \times f_s \times f_k \times f_y \times \phi_e \times \gamma \]

\[ C_m = 0.7 \]
\[ k_f = \frac{2.16}{0.65} = 3.32 \]
\[ d_2 = 0.165 \]
\[ \chi = 0.8 \ (occupancy) \]

\[ Z_i = 520 \times (0.17) \times (3.32) \times (0.65) \times (0.8) = 628 \text{ lb} \]

Equivalent uniform load = \( 628 \times \left(\frac{12}{9}\right) \times \left(\frac{1}{5.4}\right) = 167 \text{ psf} = w_u \)

\( dL = 17.4 \text{ psf} \times (1.2) = 20.8 \text{ psf} \)

\( uL = 85 \text{ psf} \times (1.6) = 136 \text{ psf} \)

Actual LL capacity = \( \frac{167 - 21}{16} = 9.1 \text{ psf} \)

If only consider legs.
Consider only lags for the load

30” length ⇒ say 4 lags engaged

\[(4)(628\text{ lb}) = 2512\text{ lb} \text{ not enough capacity}\]

Ledger connection did not fail, the glulam failed
So load transfer must have occurred by direct bearing
on the 3/4” to 1” notch in the glulam
Consider bearing

- more consistent with observed failure mechanism

\[ F_{c1} = 6.25 \text{ psi} \]

\[ F_{c1} = F_{c1} \times C_m \left( \frac{C_b}{C_f} \right) K_p \phi_c \lambda \]

\[ 6.25 = \frac{C_b}{C_f} \]

\[ C_b = \frac{2b + 0.375}{0.75} \]

\[ = 0.75 + 0.375 \]

\[ = 1.125 \]

\[ \phi_c = 0.90 \]

\[ K_p = \frac{1.875}{0.90} = 2.08 \]

\[ \lambda = 0.8 \]

\[ \Rightarrow F_{c1} = 6.25 \left( 0.167 \right) \left( 1.50 \right) \left( 2.08 \right) \left( 0.90 \right) \left( 0.8 \right) = 941 \text{ psi} \]

941 psi \( (12 \text{ in}) \left( \frac{3}{4} \text{ in} \right) \) = 8469 1/2 ft-

\( m \) = resist mom.

\[ m = 8c = 13/4 \ P \]

\[ T = c = \frac{m}{d} = \frac{13/4}{3/2} = 0.786 \ P \]

3/4 4 x 6 w = 513 1/2 in

5 1/2" embed

\[ W' = W \times C_m \left( \frac{C_b}{C_f} \right) K_p \phi_c \lambda \]

\[ = 513 \left( 5.5 \right) \left( 0.7 \right) \left( 3.52 \right) \left( 0.65 \right) \left( 0.8 \right) \]

\[ = 34101 \text{ lb} \]

\[ P = \frac{3410}{0.786} = 4378 \text{ 1/2 ft} \]

\( \Rightarrow w_n = 868 \text{ psi} \)

If consider direct bearing, ledger failure probably not a limit state.

Project Title

Page Title

W.O. # Date By Checked By

Page 10 of
By loading on bottom section of beam will induce cross-grain tension stress. The actual failure mechanism observed consider area of stress. This is not considered by AASHTO or NDS.

NDS does have guidance for radial tension stress at curved members. \( F_{t} = 15 \text{ psi} \).

This is basically same as cross-grain tension:

\[
F_{t} = F_{t} \times C_{m} \times C_{l} \times k_{p} \times \delta \times \lambda
\]

\[
= 15 \times (0.7) \times (1.0) \times (2.88) \times (0.75) \times (0.18)
\]

\[
= 13.1 \text{ psi}
\]

Consider over 30in length

\[
30 \text{in} \times (10.75 - 0.175)(18.1) = 5440 \text{ LB}
\]

\[
\frac{V_{u}}{V_{l}} \text{ from tire loading} = 7670 \text{ LB} \quad \frac{D}{c} = 1.14, \quad 41\% \text{ Exceeded}
\]

Alternative: \( \frac{1}{3} F_{x} \times C_{u} \times r \) (wind or earthquake loading or southern pine)

\[
F_{x} = \frac{1}{3}(265)(0.75) = 65 \text{ psi}
\]

\[
F_{t} = 65 \times (0.75)(2.88)(0.75)(0.18) = 89 \text{ psi}
\]

The lower bound is probably more accurate.
### TABLE 1
DESIGN VALUES FOR STRUCTURAL GLUED-LAMINATED SOFTWOOD TIMBER STRESSED PRIMARILY IN BENDING<sup>(1,2,3)</sup>

(Loaded Perpendicular to Wide Faces of Laminations)

<table>
<thead>
<tr>
<th>Combination Symbol</th>
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<th>Extreme Fiber in Bending&lt;sup&gt;(6)&lt;/sup&gt;</th>
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Wet-use factors:

|            |            |            | 0.8        | 0.8        | 0.53       | 0.53       | 0.875    | 0.833    |            |            |

Footnotes on page 8.
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<th>Extreme Fiber in Bending[^9]</th>
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[^1]: © 2008 APBA - The Engineered Wood Association
**STRONGDEK™**
FIBERGLASS ARCHITECTURAL DECKING SYSTEM

- Easy to Install
- Hidden Fastening System
- Rot, Rust and Mildew Resistant
- Non-Conductive
- Stronger than Wood or Plastic Lumber
- Lightweight

STRONGDEK™ fiberglass decking is an attractive, low-maintenance architectural decking system that offers an alternative to traditional decking materials. The panels will not rot, rust, chip or mildew, which make them ideal for high-moisture environments, including saltwater.

STRONGDEK™ panels are designed to connect to form a continuous solid surface utilizing an innovative interlocking design. The deck sections are easily installed with screw-like fasteners that are not visible, creating a smooth, attractive surface.

STRONGDEK™ panels have intermediate ribs on each panel that help provide extra stiffness and strength, allowing the deck to perform ideally in areas with pedestrian traffic. An optional grit surface can be added to provide a non-skid surface.

Typical applications of STRONGDEK™:
- Hotel Recreational Areas
- Homes and Condominiums
- Buildings in Coastal Areas
- Marinas and Docks

STRONGDEK™ decking was installed at the Piedido Beach Resort in 2003, and still looks attractive today. The resort's owner, Jim Medlock, said, "The dock has held up very well. During the summer months, it has a function on it just about every Friday and Saturday night!"
Sizes and Colors

STRONGDEK™ is 12" wide and standard 24' long panels are available in stock. Panels can also be produced in any length that is practical. Standard colors are light gray or beige. Panels can be produced with an optional grit surface.

Available Accessories

A STRONGDEK™ starter channel can be used to provide a finished look to lengthwise ends, while equal leg angles can be used for end closures and/or cantilever supports.

STRONGDEK™ Load / Deflection Data

\[ I_x = 0.31 \text{ in.}^4 \quad W_t = 2.58 \text{ lb./lin. ft.} \text{ (gritted)} \]

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| 30"  | 0.533 | 0.686 | 0.940 | 1.168 | 1.372 | 1.626 | 1.981 | 2.210 | 2.438 | 2.616 | 2.794 | 3.023 |]

\[ u = \text{Uniform load in lbs/ft}^2 (\text{lb/ft}^2) \text{. For example, a 100 lb. uniform load over 3 ft}^2 \text{ is 300 lbs. of total load.} \]

\[ w = \text{Typical deflection under the uniform load in inches (mm).} \]

\[ c = \text{Concentrated load in lbs/ft of width (lb/in of width).} \]

\[ a = \text{Typical deflection under concentrated load in inches (mm).} \]

**NOTE:** STRONGDEK™ panels were attached to beams with tek screws and tested in a multi-panel configuration. This data was used to create the STRONGDEK™ load table above for a single panel.


Bristol Location

400 Commonwealth Ave., P.O. Box 580, Bristol, VA 24203-0580 USA
(276)545-9000 FAX (276)545-8132
www.strongwell.com

Chatfield Location

1610 Highway 52 South, Chatfield, MN 55923-9799 USA
(507)667-9479 FAX (507)867-4031

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