FINAL

FAILURE INVESTIGATION

WESTCHESTER NORTH LAGOON BRIDGE

Anchorage, Alaska

August 2014

Prepared for:
Municipality of Anchorage
Project Management and Engineering
P.O. Box 196650
Anchorage, Alaska 99519-6650

Prepared by:
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USKH WO#1419006

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TABLE OF CONTENTS

FIGURES

APPENDICES

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

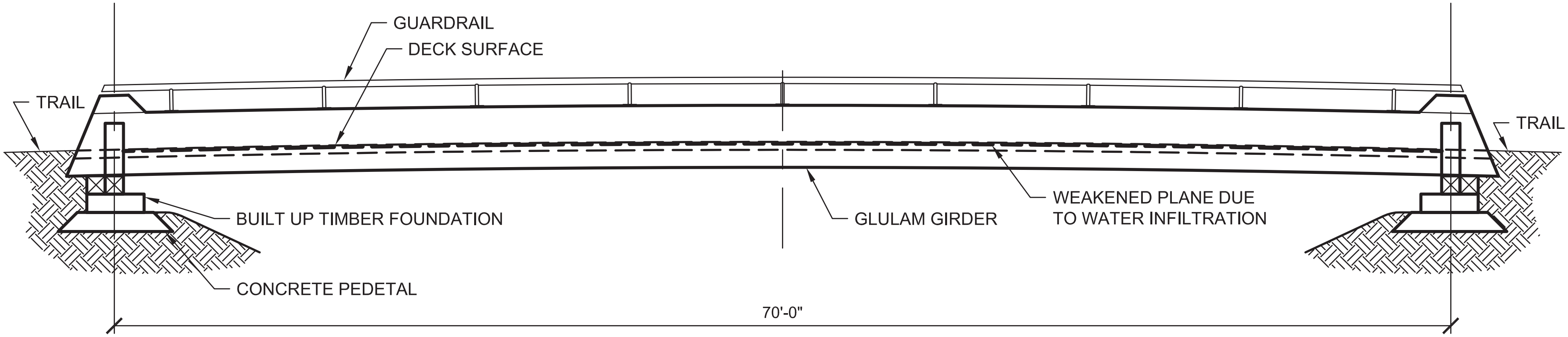
The North Westchester Lagoon Bridge failed on June 16, 2014, as a truck towing a wood chipper crossed the bridge. The MOA Parks and Recreation Department contacted *USKH Inc., now Stantec* (USKH) Structural Engineers and requested an emergency evaluation. As part of the emergency evaluation, MOA requested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

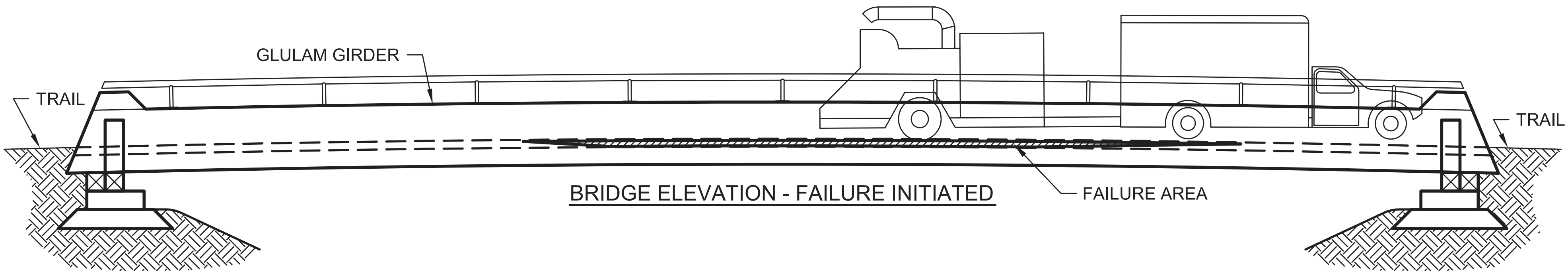
3 DESCRIPTION

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.

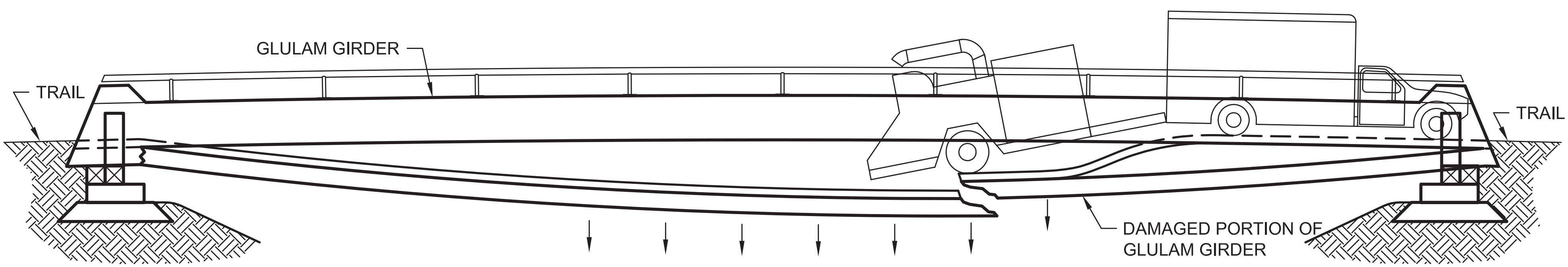
The original construction drawings have a general notes section that lists design criteria and material specifications, see Figure 2. The design codes cited are the 1985 UBC and 1983 AASTHO Specifications. Pertinent design loading indicated is 85 per square foot (psf) uniform live load and an overload 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.



BRIDGE ELEVATION - WEAKENED CONNECTION



BRIDGE ELEVATION - FAILURE INITIATED



BRIDGE ELEVATION - FAILURE

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COASTAL TRAIL BRIDGE ASSESSMENT
NORTH WESTCHESTER LAGOON BRIDGE
Municipality of Anchorage
Alaska

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Checked	RJK
USKH W.O.	1419006

Figure
2

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VICINITY MAP

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NORTH WESTCHESTER LAGOON BRIDGE

Municipality of Anchorage
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Figure
1

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:

	
North Lagoon Bridge failure	Cracked ledger and glulam

	
Splintered glulam	Bolts in failure plane
	
Rotten wood at bolt locations	Bolts in split ledger

5 ANALYSIS

An analysis of the bridge was performed generally based on the American Association of State Highway and Transportation Officials (AASTHO) Load and Resistance Factor Design (LRFD) *Guide Specification for the Design of Pedestrian Bridges*, 2009; AASHTO LRFD *Bridge Design Specifications*, 6th Edition with 2013 Interim Revisions, and the 2005 *National Design Specification for Wood Construction* (NDS).

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 Tornado 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 crossing. 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.

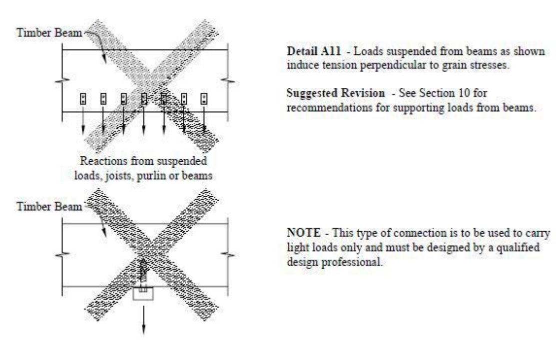
The new handrail and fiberglass deck overlay added in the 2013 trail rehabilitation added approximately 7% weight to the bridge. Modifications made to structures that cause a load increase of less than 10% are usually not considered significant.

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.

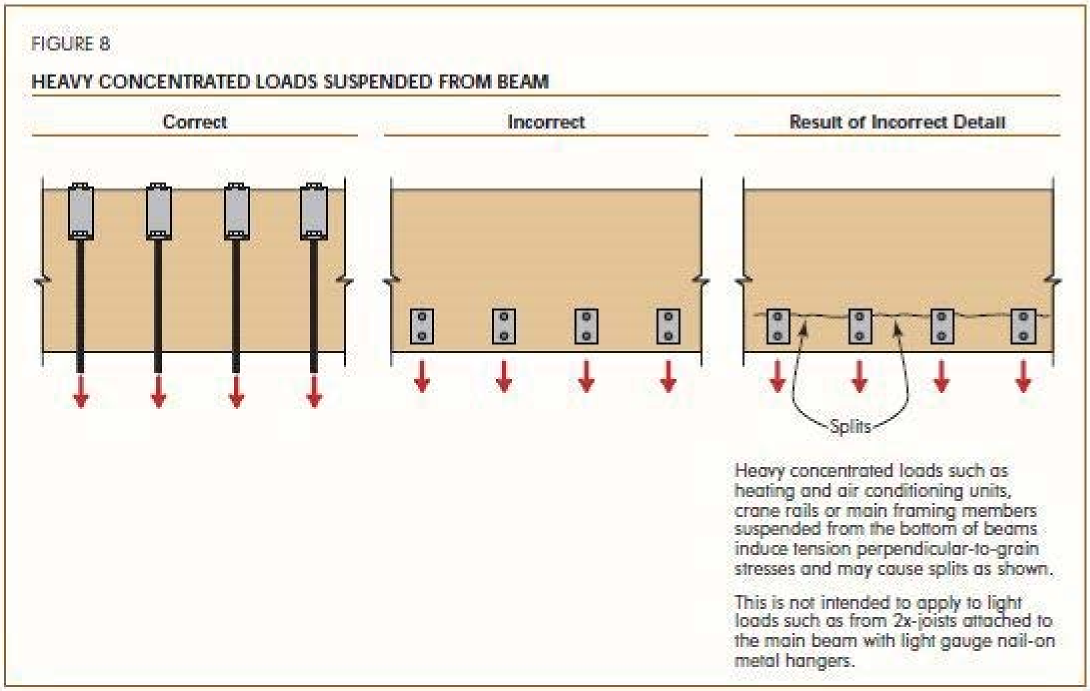
The failure mode, confirmed by analysis and site observations, appears to be cross-grain tension in the glulam beam. The ledger is set inboard of the glulam from 3/4- to 1-inch near the bottom of the glulam beams. The location means the bottom of the beam is loaded inducing cross-grain tension in the 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.



[1]AITC 104-2003 - Typical Construction Details

[1] American Institute of Timber Construction (AITC). AITC 104-2003 Typical Construction Details



[2]Figure 8 Heavy Concentrated Loads Suspended from a Beam

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.

[2] Form No. EWS T300H © 2007 Engineered Wood Systems. www.apawood.org

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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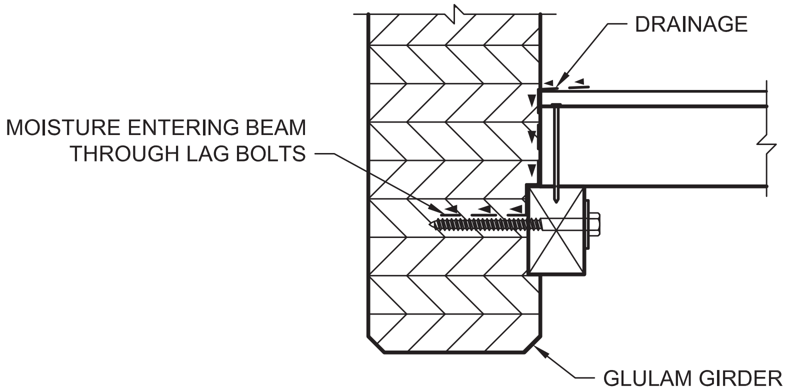
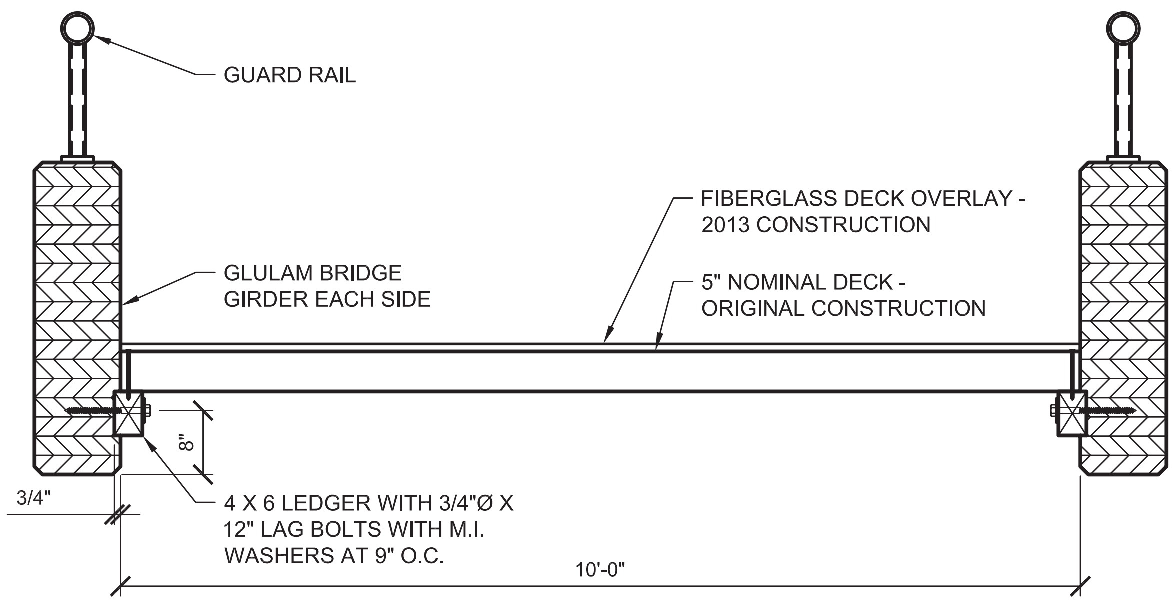
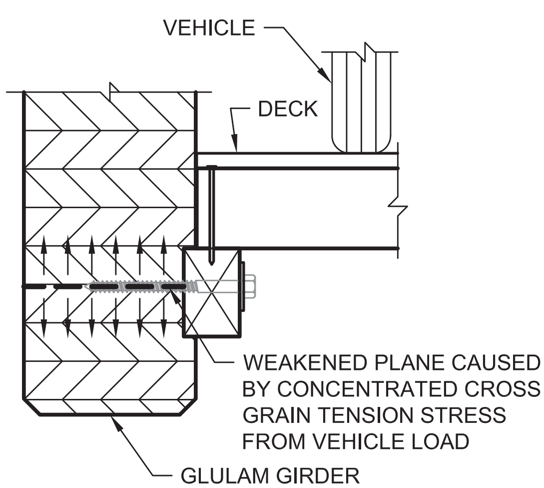


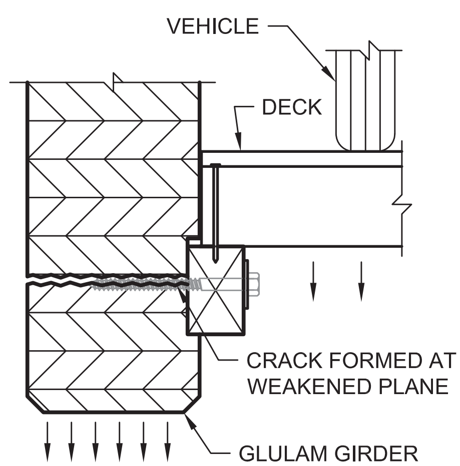
WATER INFILTRATION



BRIDGE CROSS SECTION



WEAKENED CONNECTION



FAILURE INITIATED

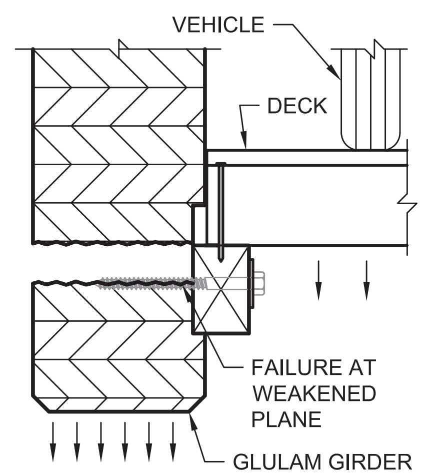


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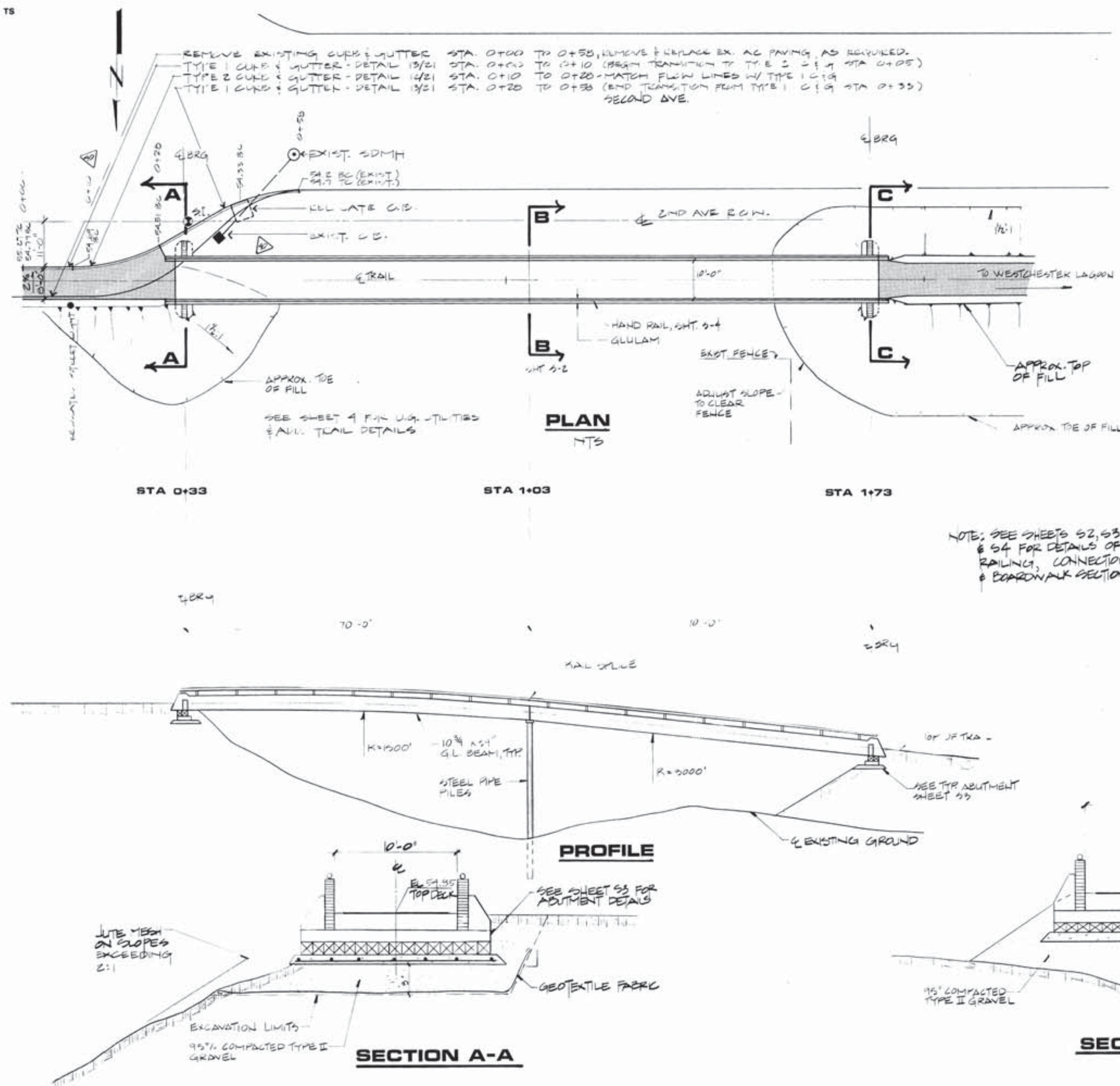
Figure
3

Appendix A – Original Construction Drawings

CT TS



GENERAL NOTES

General: Unless otherwise noted on the Plans, construction shall be in accordance with applicable requirements of MASS.

Design: Plans are in accordance with 1985 UBC and 1983 AASHTO Specifications.

Design Criteria:
1. Live Load = 85 psf Uniform
2. Full Material Dead Loads
3. Snow Load = 82 psf (30-year event)
4. Deflection Limit (L/D) = 300 with 85 psf live load
5. Overload Vehicle = 10,000 lbs. vehicle with 8000 lb. axle w/o impact
6. Seismic Zone 4
7. Footing Pressure = 1.0 ksf (max.)

Foundations: Unless otherwise noted, all backfill shall be compacted gravel with a min. 12-in. below and 18-in. adjacent to wood mudsills. Compaction of backfill materials shall be 95% of maximum dry density ASTM D-1557-78, Method D, unless otherwise specified or directed by Engineer. Driven piles shall be installed in accordance with plan locations and specifications.

Materials:
1. Glue laminated wood shall be premium appearance grade Douglas Fir, Combination 22F-V8 DF/DF, or approved equal fabricated in accordance with the latest AITC specifications and recommendations. Construction must be suitable for wet-use conditions. Members shall be individually wrapped for shipping.
2. Decking shall be 5" nominal depth, potlatch lock-deck, Douglas Fir/Larch or approved equivalent.
3. All other wood shall be Douglas Fir No. 1 or better with a maximum dry moisture content of 19%.
4. All structural steel shall be ASTM A-36 and galvanized after fabrication in accordance with ASTM A-123. Field repair per ASTM A-780.
5. All structural bolts shall be ASTM A-307 and galvanized in accordance with ASTM A-153.
6. Nails and other connectors including washers and malleable iron washers shall be galvanized.
7. Foundation piles shall be per ASTM A-252, Grade 2 or approved equal and shall be galvanized per ASTM A-123.

Treatment and Finish:
1. All wood products shown on the drawings, except the abutment mud sill, shall be pressure treated with penta in light sol. per AWPA C-2 and C-28 to a minimum retention of 0.30 lbs./cu.ft. Resulting finish must be capable of being stained or painted after a set time of 6 months.
2. All wood for abutment mudsill shall be pressure treated with creosote per AWPA C-2 to a minimum net retention of 12 lbs./cu.ft.
3. Treat and paint all steel plates, rails, and other metal components as required by Specifications.

Shop Drawings: Contractor will submit shop drawings for approval prior to fabrication of wood or steel members.

Nailing: Contractor is advised that predrilling of gluelam members may be required prior to nailing.

Filter Fabric: Filter fabric shall be as shown in the specifications.

Concrete: Shall have a minimum compressive strength of 2,500 psi in 28 days.

Reinforcing Steel: Reinforcing steel shall conform to ASTM A-615 Grade 60.

Erosion Control Mat: Jute mesh matting shall be used where fill slopes are 1-1/2:1 or less, or as otherwise directed in the field.

Coarse Aggregate: Materials furnished by the Contractor for use as coarse aggregate backfill shall be an approved material meeting the requirement for AASHTO coarse aggregate Gradation No. 4.

Revisions	
	9/23/87
	10/5/87

COASTAL TRAIL
PHASE 3

COASTAL TRAIL
CONTRACT DOCUMENTS

Burns Clarke Vogan Architects
ARCHITECTURE ■ LANDSCAPE ARCHITECTURE ■ PLANNING
800 WEST FIFTH AVENUE, SUITE 400
ANCHORAGE, ALASKA 99501

Park Identification 605
Plan Set Number 109

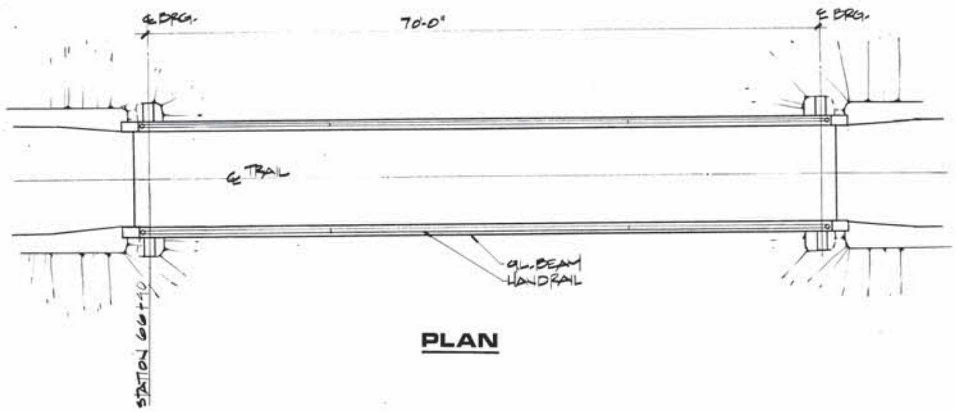
BOARDWALK PLAN & DETAILS
2ND AVENUE & H STREET

Date: 6/1/87 Com. No.: 86.705.50
In Charge : DV
Drawn By :
Checked By :

S1

CT TS

SEE GENERAL NOTES SH-S1



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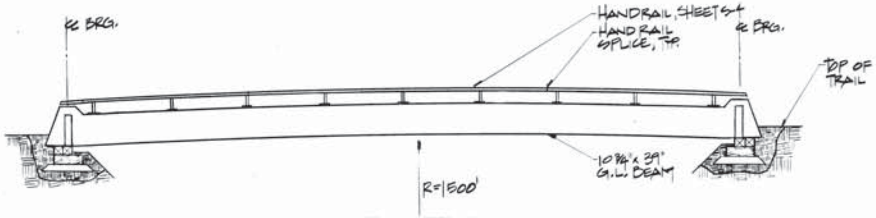

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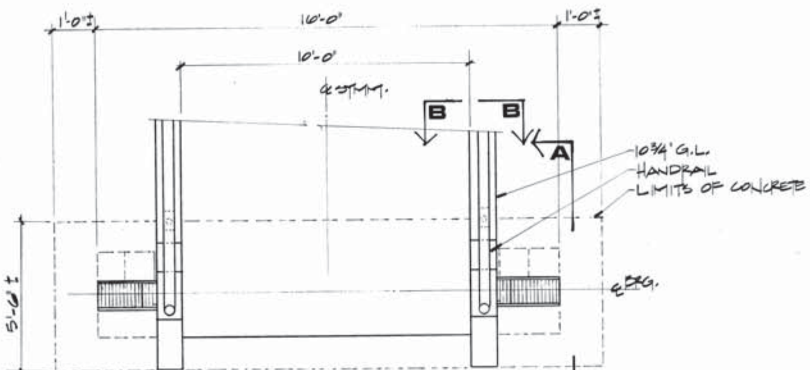
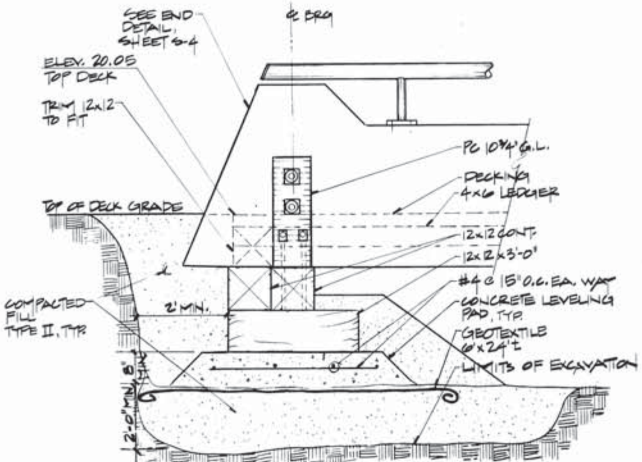
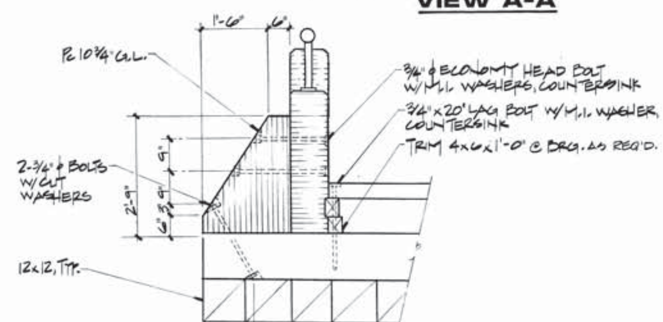


ABUTMENT PLAN

A



VIEW A-A



SECTION B-B

Revisions 9/23/87
10/5/87

COASTAL TRAIL
PHASE 3

COASTAL TRAIL
CONTRACT DOCUMENTS

Burns Clarke Vogan Architects

ARCHITECTURE ■ LANDSCAPE ARCHITECTURE ■ PLANNING
900 WEST FIFTH AVENUE, SUITE 440
ANCHORAGE, ALASKA 99501

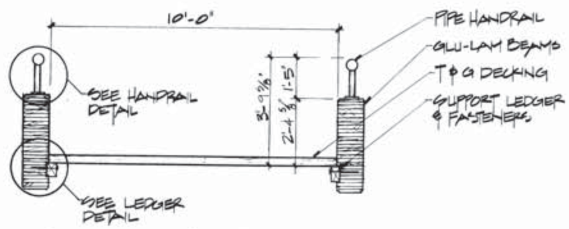
BOARDWALK PLAN & DETAILS
WESTCHESTER LAGOON

Date: 6/1/87 Com. No.: 86.705.50
In Charge : DV
Drawn By :
Checked By :

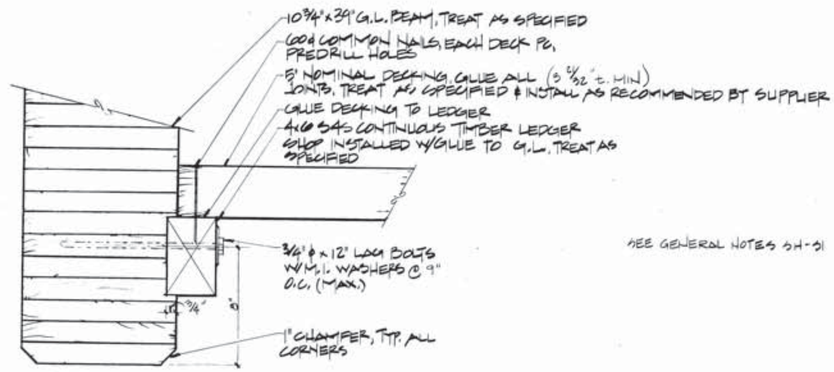
S3

Park Identification 605
Plan Set Number 111

CT TS

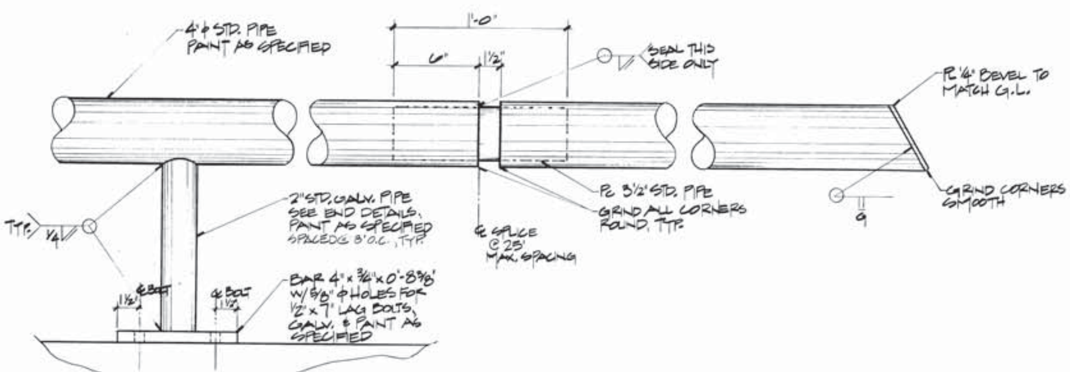


TYPICAL SECTION

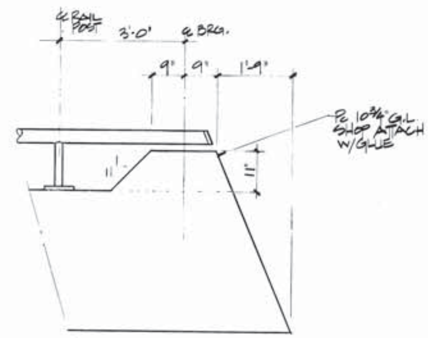


LEDGER DETAIL

SEE GENERAL NOTES SH-31



HAND RAIL DETAILS



END DETAIL

Revisions	9/23/87
	10/5/87

COASTAL TRAIL
PHASE 3

COASTAL TRAIL
CONTRACT DOCUMENTS

Burns Clarke Vogan Architects

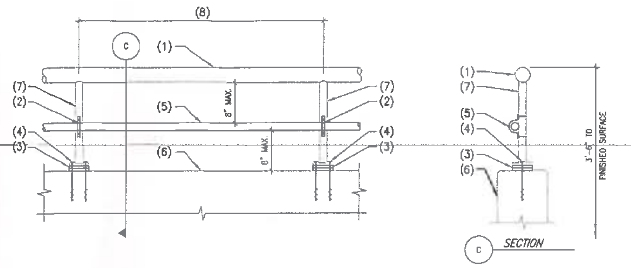
ARCHITECTURE ■ LANDSCAPE ARCHITECTURE ■ PLANNING
800 WEST FIFTH AVENUE, SUITE 440
ANCHORAGE, ALASKA 99501

Park Identification 609
Plan Set Number 112

BOARDWALK DETAILS

Date: 6/1/87 Com. No.: 86.705.50
In Charge : DV
Drawn By :
Checked By :

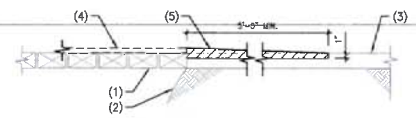
S4



NOTES:

1. EXISTING STEEL PIPE RAILING.
2. NEW GALVANIZED STEEL 2 HOLE PIPE STRAP WITH 1/4" DIA. SELF-DRILLING, SELF-TAPPING SCREWS.
3. NEW 1/2" x 4" x 0'-8 3/8" STEEL SHIMS WITH HOLES FOR LAG BOLTS. GALVANIZE AND FINISH TO MATCH EXISTING RAILINGS. FOR BIDDING PURPOSES, ASSUME (4) SHIMS PER POST. CONTRACTOR TO FIELD VERIFY FOR PROPER RAIL HEIGHT.
4. SHIM RAILING TO ACHIEVE UNIFORM 3'-6" RAILING HEIGHT. REPLACE EXISTING LAG SCREWS WITH NEW GALVANIZED SCREWS, DIAMETER TO MATCH EXISTING AND LENGTH TO ACHIEVE EQUIVALENT EMBEDMENT.
5. NEW 1 1/2" STD. GALVANIZED PIPE. FINISH TO MATCH EXISTING RAILINGS.
6. EXISTING GLULAM BRIDGE BEAM. PROVIDE END CAP AND INTERMEDIATE SLIP CONNECTIONS TO MATCH EXISTING GUARDRAIL.
7. EXISTING STEEL PIPE HANDRAIL SUPPORT POST WITH STEEL BASE PLATE.
8. FIELD VERIFY EXISTING SUPPORT POST SPACING. FOR BIDDING PURPOSES, ASSUME 8'-0" O.C.

05 *EXISTING STEEL HANDRAIL MODIFICATIONS*
SCALE: NOT TO SCALE
213025-SK1-05



NOTES:

1. EXISTING TIMBER DECKING.
2. EXISTING GRADE AS OCCURS.
3. EXISTING ASPHALT PAVEMENT.
4. NEW FINISHED WEARING SURFACE.
5. NEW ASPHALT RAMP. REMOVE AND REPLACE 1" OF EXISTING ASPHALT BELOW RAMP.

04 *NEW ASPHALT RAMP AT NEW FINISHED WEARING SURFACE*
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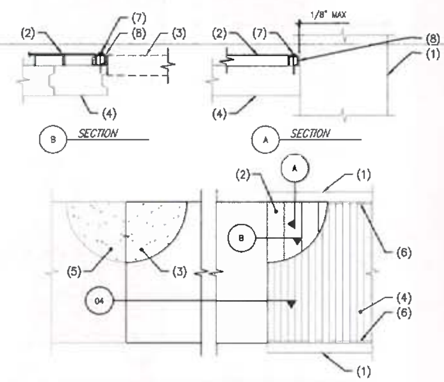
213025-SK1-04



NOTES:

1. EXISTING GLULAM BRIDGE BEAM.
2. NEW FRP DECKING PLANKS.
3. NEW ASPHALT RAMP.
4. EXISTING TIMBER DECKING.
5. EXISTING ASPHALT PAVEMENT.
6. EXISTING TIMBER LEDGER.
7. FASTENER PER SPECIFICATIONS.
8. NEW FRP EDGING PIECE PER SPECIFICATIONS.

EXISTING BRIDGE SURFACE DIMENSIONS FOR ESTIMATING DECKING QUANTITIES

LOCATION	LENGTH	WIDTH	REMARKS
FISH CREEK BRIDGE	102'-0"	10'-0"	---
SOUTH LAGOON BRIDGE EAST SPANS	151'-0"	10'-0"	---
SOUTH LAGOON BRIDGE WEST SPANS	151'-0"	10'-0"	---
SOUTH LAGOON CENTER BOARDWALK	ESTIMATE 800 SQ. FT.		---
NORTH LAGOON BRIDGE	72'-0"	10'-0"	---
2ND AVENUE BRIDGE	142'-0"	10'-0"	REMOVE EXISTING 1/2" TIMBER WEARING SURFACE

03 *NEW FRP DECKING AT EXISTING TIMBER DECKING*
SCALE: NOT TO SCALE
213025-SK1-03

NOTE:
DIMENSIONS AND QUANTITIES SHOWN ON THIS SHEET ARE FOR BIDDING PURPOSES OR AS NOTED OTHERWISE. FIELD VERIFY ALL CONDITIONS.

SCHNEIDER STRUCTURAL ENGINEERS

STATE OF ALASKA 49TH DANIEL M. FOLMAR No. CE11421 03/06/13 REGISTERED PROFESSIONAL ENGINEER
SEAL

Department of Parks and Recreation
Municipality of Anchorage
Healthy Parks Healthy People

TONY KNOWLES COASTAL TRAIL
BRIDGE REHABILITATION
STRUCTURAL DETAILS

SCALE	HOR. N/A VER. N/A	DATE: 03/01/2013	GRID: N/A	SHEET 18 of 18
		STATUS:		

Appendix B – Calculations

Dead Loads

Glulam Beams: $10\frac{3}{4} \times 39$ $419 in^3 = 2.91 ft^3/ft$

$wt = 2.91(0.050\ kcf) = 0.146\ k/ft$

$0.146(2)(70) = 20.4\ k$

Decking: $3\frac{21}{32}$ thick - use 3.75

$\frac{3.75}{12}(0.050) = 0.0156\ ksf$

$0.0156(10)(70) = 10.9\ k$

Ledger: 4x6 $\frac{19.25}{144}(0.050)(70)(2) = 0.9\ k$

Guardrail: w/ connections, 11 lb/ft

$(11)(70)(2) = 1.5\ k$

Total DL = $20.4 + 10.9 + 0.9 + 1.5 = 33.7\ k$
Original

Fiberglass overlay
Strongwell Strongdek wt = 2.58 psf $(2.58)(10)(70) = 1.8\ k$

Additional Guardrail
$(3)(70)(2) = 0.5\ k$

Total Rehab items $= 1.8 + 0.5 = 2.3\ k$

Total DL $= 33.7 + 2.3 = 36.0\ k$ % increase = 6.8%
Current

Uniform wt $= \frac{36\ k}{(10\ ft)(70\ ft)} = 51.4\ psf$ say 52 psf

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.

$$w = 85\left(0.25 + \frac{15}{\sqrt{A_i}}\right) = 85\left(0.25 + \frac{15}{\sqrt{700}}\right) = 69 \text{ psf}$$

Vehicle at Failure:

Chipper:	2005 Morbark Torado 15	7300 LB
Truck:	2006 Ford F550 (Custom)	7099 LB

Chipper has single axle, one tire per side
Truck has Front / Rear axles, double tires per side

Span = 70' width = 10'

Main Beams:

$10\frac{3}{4} \times 39$ GLB 22F-V8 DF/DF

Reference Design Values

$F_{bx0} = 2.2$ ksi

$E_{x0} = 1.7 \times 10^3$ ksi

$G_0 = 0.5$

$F_{v0} = .265$ ksi

$$F_b = F_{bo} C_{KF} C_m (C_P \text{ or } C_V) C_{fu} C_i C_d C_\lambda$$

$$C_{KF} = \frac{2.5}{\phi} = \frac{2.5}{0.85} = 2.94 \text{ (bending)} \qquad = \frac{2.5}{0.75} = 3.33 \text{ (shear)}$$

$$C_m = 0.80$$

$$C_V = \left[\left(\frac{12.0}{39}\right)\left(\frac{5.125}{10.75}\right)\left(\frac{21}{70}\right)\right]^{0.10} \le 1.0 = 0.732$$

$$C_{fu} = 1.0$$

$$C_i = 1.0$$

$$C_d = 1.0$$

$C_\lambda = 0.8$ (consider only Strength I limit state)

$$\Rightarrow F_b = 2.2(2.94)(0.80)(0.732)(1.0)(1.0)(1.0)(0.8) = 3.03 \text{ ksi}$$

$$E = E_0 C_m C_i$$

$$= 1.7 \times 10^3 (0.833)(1.0) = 1.42 \times 10^3 \text{ ksi}$$

$$F_V = 0.265(3.33)(0.875)(1.0)(0.80) = 0.618 \text{ ksi}$$

$M_r = \phi M_N$

$M_n = F_b S C_L$

$$R_b = \sqrt{\frac{70 \times 12 \times 39}{(10.75)^2}} \leq 50 = 16.8$$

$$F_{bE} = \frac{(1.10)(1.42 \times 10^3)}{(16.8)^2} = 5.53$$

$$A = \frac{5.53}{3.03} = 1.83$$

$$C_L = \frac{1 + 1.83}{1.9} - \sqrt{\frac{(1 + 1.83)^2}{3.61} - \frac{1.83}{0.95}} = 0.95$$

C_V is lesser, therefore C_V controls

$$S = \frac{bd^2}{6} = \frac{(10.75)(39)^2}{6} = 2725 \text{ in}^3$$

$$\phi M_N = 0.85\,(3.03 \text{ ksi})(2725 \text{ in}^3)(1.0) = 7020 \text{ in-k}$$

$V_r = \phi V_n$

$$V_n = \frac{F_v b d}{1.5} = \frac{0.618\,(10.75)(39)}{1.5} = 173 \text{ k}$$

$$V_r = 0.75\,(173 \text{ k}) = 130 \text{ k}$$

Glulam beam check – bending

– Uniform live load

$$M_u = \frac{1.25\left[(52\,psf)(5\,ft)\right](70\,ft)^2}{8} + 1.75\,\frac{(85\,psf)(5\,ft)(70\,ft)^2}{8}$$

$$= 655\ ft\text{-}k = 7855\ in\text{-}k$$

$\frac{7855}{7020} = 1.12$ D/C exceeded if consider full 85psf

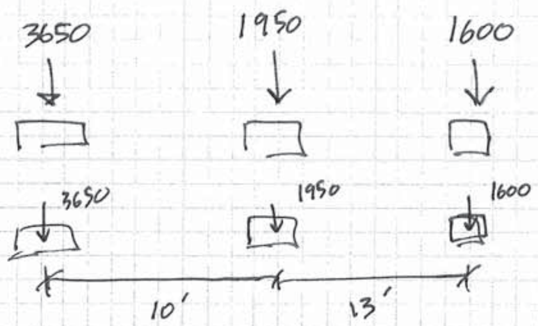
Live load Limit at 1.0

$$7020 - \frac{1.25(52)(5)(70)^2}{8}\left(\frac{12}{1000}\right) = 4631$$

$\left(\frac{1000}{12}\right) 4631 = \frac{1.75\,w(70)^2(5)}{8} \Rightarrow$ $w = 72\,psf$ allowable live load with $D/C = 1.0$
Above the reduced uniform live load

– Consider moment with truck / chipper

Note: The actual truck / chipper combo has greater weight than AASHTO 10000LB design truck



if tires placed on one side, 1ft from edge, 1.2 multiplier for one side

$$V_{u\,max} = 6.3(1.2)(1.75) = 13.2K$$
$$M_{u\,max} = 99.4(1.2)(1.75) = 209\ ft\text{-}k = 2505\ in\text{-}k$$

Add DL $V_u = (52\,psf)\left(\frac{10\,ft}{2}\right)\left(\frac{70\,ft}{2}\right)(1.25) = 11.4K + 13.2K = 24.6K$ $\frac{24.6}{130} = 0.19$ OK

$M_u = 2389 + 2505 = 4894\ in\text{-}k$ $D/C = \frac{4894}{7020} = 0.70$ OK

Main beams ok for vehicle load
but limited to uniform live load of 72psf

Decking

5in wide x 3¾ deep DF Decking

F_{bx0} = 1.45 ksi (sing)
1.650 ksi (rep)

E 1.70 x10^3 ksi

F_{v0} = 0.18 ksi

C_{KF} = 2.94
C_m = 0.85 (decking) 0.97 (shear)
C_F = 1.0 (included in NDS values)
C_{Fu} = 1.0 (included in NDS values)
C_i = 1.0
C_d = 1.0
C_λ = 0.8 (Strength I)

$\Rightarrow F_b = 1.65(2.94)(0.85)(1.0)(1.0)(1.0)(1.0)(0.8) = 3.30$ ksi

$E = E_0 C_m C_i$
$= 1.7\times10^3(0.90)(1.0) = 1.53\times10^3$ ksi

Bending strength: $S = \frac{(5)(3.75)^2}{6} = 11.7\ in^3$

$\phi M_n = 0.85(3.30\ ksi)(11.7\ in^3)(1.0) = 32.8$ in-k per plank

Shear: $\phi = 0.75$

$C_{KF} = \frac{2.5}{0.75} = 3.33$

$F_v = 0.18(3.33)(0.97)(1.0)(0.8) = 0.465$ ksi

$\phi V_n = 0.75\frac{F_v b d}{1.5} = \frac{0.75(0.465)(5)(3.75)}{1.5} = 4.4$k per plank

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– Check LL on decking

Decking wt = 15.6 + 2.6 = 18.2 psf

Pedestrian LL = 85 psf

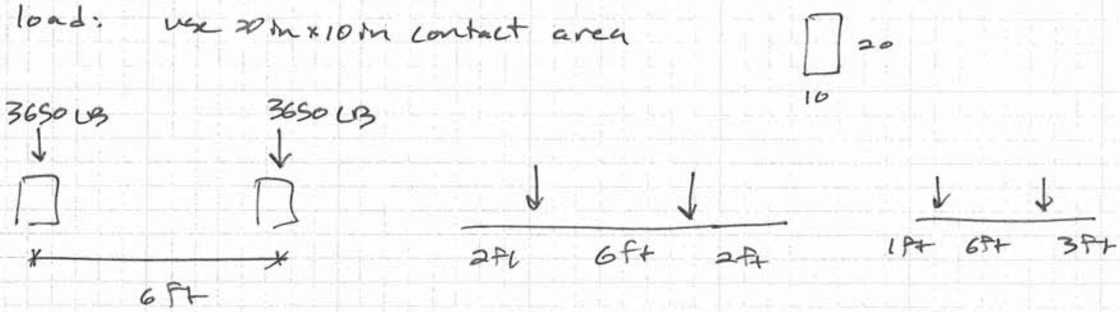
Load Factors

	DC	PL
Str I	1.25	1.75

$\Rightarrow$ load = $18.2(1.25) + 85(1.75) = 171$ psf

$$M_u = \frac{(171)\left(\frac{5}{12}\right)(10)^2}{8} = 891 \text{ ft-lb} = 10.7 \text{ in-k} \quad \underline{ok}$$

Tire load: use 20 in x 10 in contact area



M_u = 153 in-k 184 in-k

V_u = 6.39 k 7.67 k

20 in contact area
would engage at least 6 planks = 30 in

DL only

$17.4(1.25) = 21.8$ psf

$$M_u = \frac{(21.8)(10)^2}{8} = 273 \frac{\text{ft-lb}}{\text{ft}}$$ over 30 in contact area = 681 ft-lb = 8.2 in-k

Total $M_u = 184 + 8 = 192$ in-k

$\phi M_n = 32.8$ (6 planks) = 197 in-k = 0.97 $\underline{ok}$

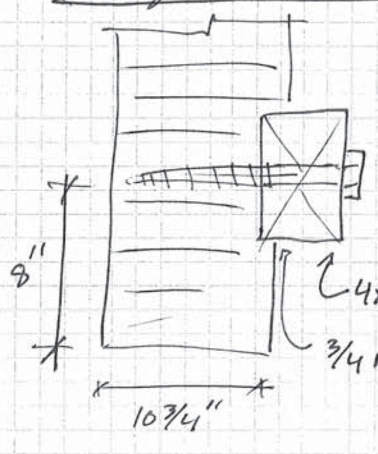
Max V_u : $1.25(17.4)\left(\frac{30}{12}\right)\left(\frac{10}{2}\right) + 7.67\text{k} = 7.94\text{k}$

$\phi V_n = 6(4.4) = 26.4\text{k} \Rightarrow \underline{ok}$

Project Title ______ Page Title ______

W.O. # ______ Date ______ By ______ Checked By ______

Ledger Connection



"Connection design per 2005 NDS"

– If Neglect Notch and bearing on notch → consider only lag in shear.

$$Z' = Z \times C_m \cancel{C_t}\ \cancel{C_g}\ \cancel{C_\Delta}\ \cancel{C_{eg}}\ \cancel{C_{di}}\ \cancel{C_{tn}}\ K_F\ \phi_z\ \lambda$$

$C_m = 0.7$

$K_F = \frac{2.16}{\phi} = \frac{2.16}{0.65} = 3.32$

$\phi_z = 0.65$

$\lambda = 0.8$ (occupancy)

3/4"φ LAG, 3 1/2 side member, 0.50 = G DF $Z_\perp = 520$ lb NDS Table 11J

$Z'_{rp} = 520(0.7)(3.32)(0.65)(0.8) = 628$ LB

equivalent uniform load $= 628\left(\frac{12}{9}\right)\left(\frac{1}{5\text{ft}}\right) = 167$ psf $= w_u$

DL = 17.4 psf (1.2) = 20.8 psf

LL = 85 psf (1.6) = 136 psf

use NDS/ASCE load factors per NDS

157 psf

$D/C = \frac{157}{167} = 0.94$ ok

Actual LL capacity $= \frac{167-21}{1.6} = \underline{91\text{ psf}}$ If only consider lags

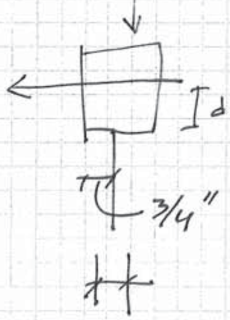
– Consider only lags for tire load

30" length => say 4 lags engaged

(4) (628 lb) = 2512 lb not enough capacity

Ledger connection did not fail, the Glulam failed
so load transfer must have occured by direct bearing
on the 3/4" to 1" notch in the glulam

Consider bearing
– more consistant with observed failure mechanism



$F_{c\perp} = 625\,psi$

$F_{c\perp}' = F_{c\perp} \times C_m \; \cancel{C_t}^{\,1} \; \cancel{C_i}^{\,1} \; C_b \; K_F \; \phi_c \; \lambda$

$C_m = 0.67$

$C_b = \dfrac{l_b + 0.375}{l_b} = \dfrac{0.75 + 0.375}{0.75} = 1.50$

$e = \left(\dfrac{3/4}{2} - \dfrac{3\tfrac{1}{2}}{2}\right) = 1\tfrac{3}{8}''$

$\phi_c = 0.90$

$K_F = \dfrac{1.875}{0.90} = 2.08$

$\lambda = 0.8$

$\Rightarrow F_{c\perp}' = 625(0.67)(1.50)(2.08)(0.90)(0.8) = 941\,psi$

$941\,psi\,(12\,in)(3/4\,in) = 8469\,lb/ft \Rightarrow w_u = 1693\,psf$

Moment resisted by lag

$M = Pe = 1\tfrac{3}{8}P \qquad T = C = \dfrac{M}{d} = \dfrac{1\tfrac{3}{8}P}{\dfrac{3\tfrac{1}{2}}{2}} = 0.786\,P$

3/4 LAG $W = 513\,lb/in$ $\qquad 5\tfrac{1}{2}''$ embed

$W' = W \times C_m \; \cancel{C_t}^{\,1.0} \; \cancel{C_{eg}}^{\,1.0} \; \cancel{C_{tn}}^{\,1.0} \; K_F \; \phi_z \; \lambda$

$C_m = 0.7 \quad K_F = 3.32 \quad \phi_z = 0.65 \quad \lambda = 0.8$

$= 513(5.5)(0.7)(3.32)(0.65)(0.8) = 3410\,lb$

$P_u = \dfrac{3410}{0.786} = 4338\,lb/ft \Rightarrow w_u = 868\,psf$

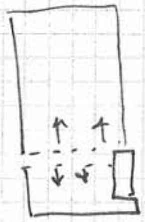
If consider direct bearing, ledger failure probably not a limit state

By loading on bottom section of beam, will induce cross-grain tension stress. → 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 $F_{rt} = 15\,psi$

This is basically same as cross-grain tension



$$F_{rt}' = F_{rt} \times C_m \times C_t \times K_F \times \phi \times \lambda$$

$$= 15(0.7)(1.0)(2.88)(0.75)(0.8)$$

$$= 18.1\,psi$$

consider over 30 in length

$$30\,in\,(10.75 - 0.75)(18.1) = 5440\,LB$$

V_u from Tire loading $= 7670\,LB$ $\frac{D}{C} = 1.41$

41% Exceeded

Alternative: $1/3\, F_v \times C_{vr}$ (wind or earthquake loading or southern pine)

$$F_{rt} = 1/3(265)(0.732) = 65\,psi$$

$$F_{rt}' = 65(0.80)(2.88)(0.75)(0.8) = 89\,psi$$

The lower bound is probably more accurate.

TABLE 1

DESIGN VALUES FOR STRUCTURAL GLUED-LAMINATED SOFTWOOD TIMBER STRESSED PRIMARILY IN BENDING[1,2,3]

Combination Symbol	Species[4] Outer/Core	Balanced/ Unbalanced[5]	Bending About X-X Axis (Loaded Perpendicular to Wide Faces of Laminations): Extreme Fiber in Bending[6]: Tension Zone Stressed in Tension F_{bx}^{+} (psi)	Extreme Fiber in Bending[6]: Compression Zone Stressed in Tension F_{bx}^{-} (psi)	Compression Perpendicular to Grain: Tension Face $F_{c\perp x}$ (psi)	Compression Perpendicular to Grain: Compression Face $F_{c\perp x}$ (psi)	Shear Parallel to Grain (Horizontal)[7] F_{vx} (psi)	Modulus of Elasticity[8] E_x (10^6 psi)
1	2	3	4	5	6	7	8	9
Western Species								
EWS 20F-E/ES1[11]	ES/ES	B	2000	2000	560	560	200	1.8
EWS 20F-E/SPF1[12]	SPF/SPF	B	2000	2000	425	425	215	1.5
EWS 20F-E8M1	ES/ES	B	2000	2000	450	450	200	1.5
EWS 20F-V12	AYC/AYC	U	2000	1400	560	560	265	1.5
EWS 20F-V13	AYC/AYC	B	2000	2000	560	560	265	1.5
EWS 22F-V/POC1	POC/POC	B	2200	2200	560	560	265	1.8
EWS 22F-V/POC2	POC/POC	U	2200	1600	560	560	265	1.8
EWS 24F-E/ES1	ES/ES	U	2400	1700	560	560	200	1.7
EWS 24F-E/ES1M1	ES/ES	B	2400	2400	560	560	200	1.8
EWS 24F-V4	DF/DF	U	2400	1850	650	650	265	1.8
EWS 24F-V4M2[13]	DF/DF	U	2400	1850	650	650	220	1.8
EWS 24F-V8	DF/DF	B	2400	2400	650	650	265	1.8
EWS 24F-V10	DF/HF	B	2400	2400	650	650	215	1.8
EWS 26F-E/DF1[11]	DF/DF	U	2600	1950[14]	650	650	265	2.0
EWS 26F-E/DF1M1[11]	DF/DF	B	2600	2600	650	650	265	2.0
EWS 24F-1.8E Glulam Header[15]	WS,SP/ WS,SP	U	2400	1600	500	500	215	1.8
Southern Pine								
EWS 24F-V3	SP/SP	U	2400	1950	740	740	300	1.8
EWS 24F-V5	SP/SP	B	2400	2400	740	740	300	1.7
EWS 26F-V4	SP/SP	B	2600	2600	740	740	300	1.9
EWS 30F-E2	SP/SP	B	3000	3000	805	805	300	2.1[19]
EWS 30F-E2M2[16]	LVL/SP	B	3000[17]	3000[17]	650[18]	650[18]	300	2.1
EWS 30F-E2M3[16]	LVL/SP	B	3000[17]	3000[17]	650[18]	650[18]	300	2.1
Wet-use factors			0.8	0.8	0.53	0.53	0.875	0.833

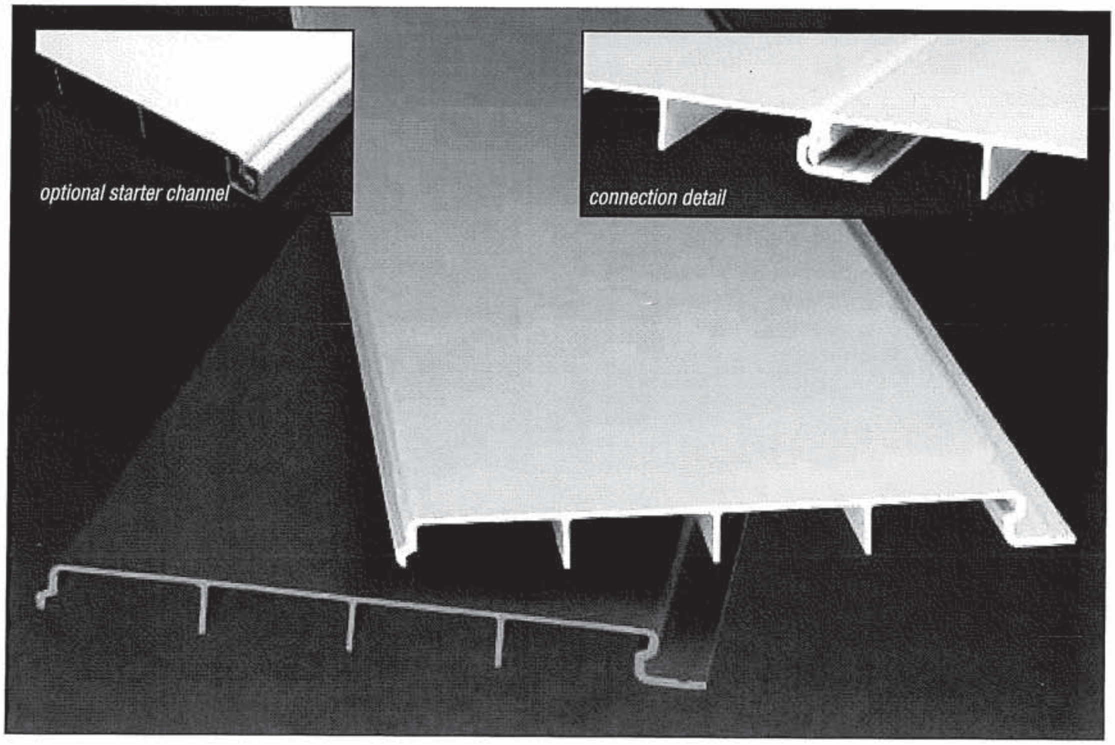
Footnotes on page 8.

Bending About Y-Y Axis (Loaded Parallel to Wide Faces of Laminations)				Axially Loaded			Fasteners		
Extreme Fiber in Bending[9]	Compression Perpendicular to Grain	Shear Parallel to Grain (Horizontal)[7,10]	Modulus of Elasticity[8]	Tension Parallel to Grain	Compression Parallel to Grain	Modulus of Elasticity[8]	Specific Gravity for Dowel-Type Fastener Design		
							Top or Bottom Face	Side Face	
F_{by} (psi)	$F_{c\perp y}$ (psi)	F_{vy} (psi)	E_y (10^6 psi)	F_t (psi)	F_c (psi)	E_{axial} (10^6 psi)	SG		Combination Symbol
10	11	12	13	14	15	16	17	18	
1100	300	175	1.5	1050	1150	1.6	0.41	0.41	EWS 20F-E/ES1[11]
875	425	190	1.4	425	1100	1.4	0.42	0.42	EWS 20F-E/SPF1[12]
1400	315	175	1.4	800	1000	1.4	0.41	0.41	EWS 20F-E8M1
1250	470	230	1.4	900	1500	1.4	0.46	0.46	EWS 20F-V12
1250	470	230	1.4	925	1550	1.5	0.46	0.46	EWS 20F-V13
1500	375	230	1.6	1150	1950	1.6	0.45	0.45	EWS 22F-V/POC1
1500	375	230	1.6	1150	1900	1.6	0.45	0.45	EWS 22F-V/POC2
1100	300	175	1.5	1050	1150	1.6	0.41	0.41	EWS 24F-E/ES1
1100	300	175	1.5	1050	1150	1.6	0.41	0.41	EWS 24F-E/ES1M1
1450	560	230	1.6	1100	1650	1.7	0.50	0.50	EWS 24F-V4
1450	560	230	1.6	1100	1650	1.7	0.50	0.50	EWS 24F-V4M2[13]
1450	560	230	1.6	1100	1650	1.7	0.50	0.50	EWS 24F-V8
1450	375	200	1.5	1100	1550	1.6	0.50	0.43	EWS 24F-V10
1850	560	230	1.8	1400	1800	1.8	0.50	0.50	EWS 26F-E/DF1[11]
1850	560	230	1.8	1400	1800	1.8	0.50	0.50	EWS 26F-E/DF1M1[11]
1300	375	200	1.5	950	1200	1.6	0.42	0.42	EWS 24F-1.8E Glulam Header[15]
1750	650	265	1.6	1150	1650	1.7	0.55	0.55	EWS 24F-V3
1750	650	265	1.5	1150	1650	1.6	0.55	0.55	EWS 24F-V5
2100	650	265	1.8	1200	1600	1.9	0.55	0.55	EWS 26F-V4
1750	650	265	1.7	1350	1750	1.7	0.55	0.55	EWS 30F-E2
1750	650	265	1.7	1350	1750	1.7	0.50	0.50	EWS 30F-E2M2[16]
1750	650	265	1.7	1350	1750	1.7	0.50	0.50	EWS 30F-E2M3[16]
0.8	0.53	0.875	0.833	0.8	0.73	0.833	See NDS	See NDS	

STRONGWELL®

STRONGDEK™

FIBERGLASS ARCHITECTURAL DECKING SYSTEM

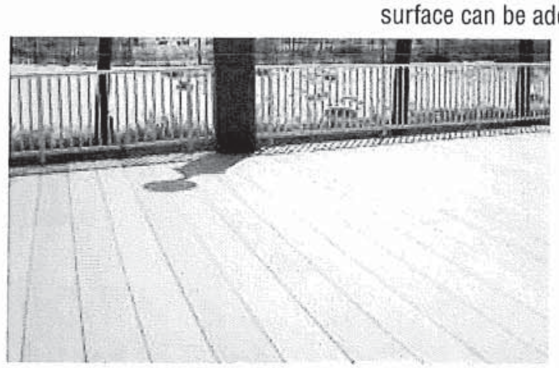


- **Easy to Install**
- **Hidden Fastening System**
- **Rot, Rust and Mildew Resistant**
- **Non-Conductive**
- **Stronger than Wood or Plastic Lumber**
- **Lightweight**

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 Perdido Beach Resort in 2003, and still looks attractive today. The resort's owner, Jim Medlock, said, "The deck has held up very well. During the summer months, it has a function on it just about every Friday and Saturday night!"

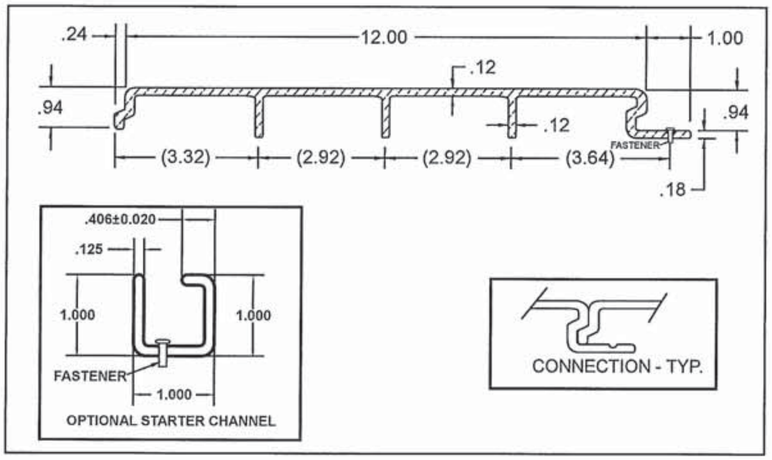
Sizes and Colors

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.

Dimensional Details



STRONGDEK™ Load / Deflection Data

I_{12} = 0.31 in.^{4} Wt = 2.58 lb./lin. ft. (gritted)

SPAN		50 u=2394 c=730	100 u=4788 c=1460	150 u=7182 c=2190	200 u=9576 c=2920	250 u=11970 c=3650	300 u=14364 c=4380	350 u=16758 c=5110	400 u=19152 c=5840	450 u=21546 c=6570	500 u=23940 c=7300	550 u=26334 c=8030	600 u=28728 c=8760	650 u=31122 c=9490
24" 610mm	Δu	0.019	0.026	0.034	0.041	0.048	0.054	0.073	0.080	0.086	0.094	0.100	0.107	0.113
	Δu	0.488	0.671	0.853	1.036	1.219	1.372	1.859	2.042	2.195	2.377	2.530	2.713	2.865
	Δc	0.016	0.022	0.028	0.034	0.04	0.045	0.061	0.067	0.072	0.078	0.083	0.089	0.094
	Δc	0.406	0.559	0.711	0.864	1.016	1.143	1.549	1.702	1.829	1.981	2.108	2.261	2.388
30" 762mm	Δu	0.032	0.041	0.056	0.069	0.081	0.096	0.117	0.131	0.144	0.155	0.165	0.179	
	Δu	0.800	1.029	1.410	1.753	2.057	2.438	2.972	3.315	3.658	3.924	4.191	4.534	
	Δc	0.021	0.027	0.037	0.046	0.054	0.064	0.078	0.087	0.096	0.103	0.11	0.119	
	Δc	0.533	0.686	0.940	1.168	1.372	1.626	1.981	2.210	2.438	2.616	2.794	3.023	
36" 914mm	Δu	0.047	0.065	0.090	0.115	0.140	0.169	0.207	0.227	0.252				
	Δu	1.189	1.646	2.286	2.926	3.566	4.298	5.258	5.761	6.401				
	Δc	0.026	0.036	0.05	0.064	0.078	0.094	0.115	0.126	0.14				
	Δc	0.660	0.914	1.270	1.626	1.981	2.388	2.921	3.200	3.556				
42" 1067mm	Δu	0.067	0.101	0.145	0.191	0.239	0.288	0.340	0.365					
	Δu	1.707	2.560	3.680	4.854	6.081	7.308	8.641	9.281					
	Δc	0.032	0.048	0.069	0.091	0.114	0.137	0.162	0.174					
	Δc	0.813	1.219	1.753	2.311	2.896	3.480	4.115	4.420					
48" 1220mm	Δu	0.096	0.158	0.233	0.310	0.391	0.463							
	Δu	2.438	4.023	5.913	7.864	9.936	11.765							
	Δc	0.04	0.066	0.097	0.129	0.163	0.193							
	Δc	1.016	1.676	2.464	3.277	4.140	4.902							
54" 1372mm	Δu	0.138	0.246	0.370	0.497	0.626								
	Δu	3.498	6.241	9.395	12.619	15.911								
	Δc	0.051	0.091	0.137	0.184	0.232								
	Δc	1.295	2.311	3.480	4.674	5.893								

u = Uniform load in lbs/ft^2 (N/m^2). For example, a 100 lb. uniform load over 3 ft^2 is 300 lbs. of total load.
Δu = Typical deflection under the uniform load in inches (mm)

c = Concentrated load in lbs/ft of width (N/m of width)
Δc = 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.*

STRONGWELL

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