FINAL

STRUCTURAL ASSESSMENT

COASTAL TRAIL BRIDGES ASSESSMENT

Anchorage, Alaska

August 2014

Prepared for:
Municipality of Anchorage
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USKH WO#1419006

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ACRONYMS

AASHTO .... American Association of State Highway and Transportation Officials
DMV .... Alaska Department of Motor Vehicles
Glulam .... glued laminated timber
lb(s)...................... pound(s)
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 Coastal Trail Bridge Assessments were directed as a result of the Westchester North Lagoon Bridge failure. That bridge failed under vehicle load. 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.

Since the other Coastal Trail bridges are of similar construction, there are similar issues and concerns. The main ledger lag bolted connection is a conduit for moisture into the main glulam and the connection also induces cross grain tension stress along this plane. Core samples taken from the main girders of the Coastal Trail bridges confirm the glulam beams are saturated to varying degrees. The saturation is inducing decay inside the glulam and weakening a plane similar to the plane of weakness evident in North Lagoon Bridge. Of the bridges examined, the 2nd Avenue Bridge has the most advanced decay with the core sample being totally saturated and crumbling when withdrawn. The main girders also have significant cracks along the connection plane in various locations.

It may be possible to temporary reinforce the wood ledger and glulam beams on some bridges to allow trail grooming and other vehicles to pass over those bridges for a short term (1-2 year) timeframe. This is not a substitute for permanent bridge replacements.

Some summary points regarding the bridges site observations, core samples, and analysis:

- The bridge designs did not consider the cross-grain tension induced by the connection.
- Normal pedestrian loading will probably not initiate failure until the wood decay is more advanced.
- The lag bolts are allowing moisture to infiltrate the interior of the glulam beams causing decay and weakness to occur in the connection zone. The beams are preservative treated but the treatments penetrate wood only to a depth of 2 inches. The lag bolts create a path for water to penetrate into the untreated core of the beam.
- The bridges with the smaller ledger not inset have a significantly reduced capacity and no capacity for vehicle loading.
- The South Lagoon Boardwalk has a reduced capacity due to the long joist spans.
- The ledger detail induces cross-grain tension in the main supporting member, which is not recommended in wood design.
- Vehicles must be kept off the bridges until they are replaced or repaired. A permanent bollard must be installed in front of every bridge access point to prevent vehicles from driving onto the bridges.
- The bridges, particularly 2nd Avenue Bridge, should be regularly monitored for signs of failure along the connection zone.
- A short term reinforcement (1-2 year design life) could be installed for an estimated rough order of magnitude cost of about $200,000 - $300,000 per bridge.

A recommended priority of bridge replacements is:

1. 2nd Ave
2. Fish Creek
3. South Lagoon Boardwalk
4. Sisson

2 INTRODUCTION

As a result of the North Westchester Lagoon Bridge failure on June 16, 2014, the Municipality of Anchorage (MOA) Parks and Recreation Department contacted *USKH Inc., now Stantec* (USKH) Structural Engineers and requested a structural assessment of similar bridges along the Coastal Trail. These bridges include: 2nd Avenue, South Lagoon Boardwalk, Fish Creek, and Sisson.

The 2nd Avenue Bridge is located along 2nd Avenue near H Street, see Figure 1. The bridge runs adjacent to 2nd Avenue near an electrical substation. The Sisson Loop Bridge is near the west end of the Ted Stevens Anchorage International Airport runways over a canyon, see Figure 2. The Fish Creek Bridge spans Fish Creek and is located southwest from Westchester Lagoon, see Figure 3. The South Lagoon Boardwalk consists of two bridge spans and a center platform area. This boardwalk is located along the south edge of the Lagoon, see Figure 4. Original contract documents were available for this investigation and were reviewed (Appendix A). Site visits were conducted on June 17, 18, and 19, 2014.

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VICINITY MAP

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COASTAL TRAIL BRIDGE ASSESSMENT
2ND AVENUE BRIDGE

Municipality of Anchorage
Alaska

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Figure
1

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SISSON BRIDGE

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Figure
2

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COASTAL TRAIL BRIDGE ASSESSMENT
FISH CREEK BRIDGE

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Figure 3

VICINITY MAP

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COASTAL TRAIL BRIDGE ASSESSMENT
SOUTH LAGOON BOARDWALK

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Figure
4

3 DESCRIPTIONS

3.1 2nd Avenue Bridge

The 2nd Avenue Bridge is approximately 140 feet long with two equal 70-foot spans, and is framed with glued laminated timber (glulam) beams and wood decking. The glulams are spaced 10 feet apart with the decking 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 on either end composed of timbers resting on a concrete leveling pad. A driven steel pile and steel beam foundation supports the bridge where the two spans meet at the center of the bridge. It is our understanding that the bridge has not had any significant rehabilitation work or modifications, except in 2013 when a 1/2-inch wear surface was removed and a fiberglass overlay was secured to the decking and the guardrails were modified by adding another pipe railing. For details of this bridge section, see Figure 5 in Section 5.

The original construction drawings were completed in 1987 as part of Phase 3 of the Coastal Trail construction, and have a general notes section that lists design criteria and material specifications, see Appendix A. The design codes cited are the 1985 Uniform Building Code (UBC) and 1983 American Association of State Highway and Transportation Officials (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 is Douglas Fir No. 1. All lumber was specified to be pressure treated.

3.2 Sisson Loop Bridge

The Sisson Loop Bridge is approximately 140 feet long with two equal 70-foot spans, and is framed with glued laminated timber (glulam) beams and wood decking. The glulams are spaced 10 feet apart with the decking 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 on either end composed of timbers resting on a concrete leveling pad. A driven steel pile and steel beam foundation supports where the two spans meet at the center of the bridge. It is our understanding that the bridge has not has any significant rehabilitation work or modifications, except in 2013 when a 1/2-inch wear surface was removed and the existing decking was removed. New 4x6 decking (laid long direction vertical) was installed with a fiberglass overlay secured to the decking. Additional work included guardrail modifications by adding another pipe railing. For details of this bridge section, see Figure 5 in Section 5.

The original construction drawings were completed in 1987 as part of Phase 4 of the Coastal Trail construction, and have a general notes section that lists design criteria and material specifications, see Appendix A. The design codes cited are the 1985 UBC and 1983 AASHTO Specifications. Pertinent design loading indicated is 85 psf uniform live load and an overload vehicle with 10,000lb weight and 8,000 lb axle load. The glulams specified are 22F-V8 DF/DF and other lumber is Douglas Fir No. 1. All lumber was specified to be pressure treated.

3.3 Fish Creek Bridge

The Fish Creek Bridge is approximately 100 feet long in two 50-foot spans. The glulams are spaced 10 feet apart with the decking between supported by wood ledgers. The ledgers are flush mounted to the glulam, and secured with lag bolts. The structure foundation is a shallow foundation system on either end composed of timbers resting on a concrete leveling pad. A driven steel pile and steel beam foundation supports where the two spans meet at the center of the bridge. It is our understanding that the bridge has not has any significant rehabilitation work or modifications except in 2013 when a fiberglass overlay was secured to the decking and the guardrails were modified by adding another pipe railing. The original construction drawings were completed in 1986 as part of the Phase 1 Coastal Trail construction, see Appendix A. For details of this bridge section, see Figure 6 in Section 5.

3.4 South Lagoon Boardwalk

The South Lagoon Boardwalk consists of two long 150-foot "bridge" sections intersecting a center platform area. The "bridge" sections consists of elements similar to the Fish Creek Bridge – main glulams spaced 10 feet apart with decking between supported by wood ledgers. The ledgers are flush mounted to the glulam (not inset) and secured with lag bolts. The spans are 50 feet and are steel pile/beam supported over the wetland areas and shallow foundation supported where the structure intersects the paved trail. The center platform section is supported by dimensional lumber framing bearing on glulam beams and steel piles. Wood decking overlays the dimensional lumber framing. It is our understanding that the bridge has not has any significant rehabilitation work or modifications. A bid package was completed in 2013 showing an overlay of fiberglass decking and guardrails modifications but this work was not completed. The original construction drawings were completed in 1986 as part of the Phase 1Coastal Trail construction, see Appendix A. For details of this boardwalk section, see Figure 6 in Section 5.

4 SITE OBSERVATIONS

Site visits were conducted June 17, 18, and 19, 2014. Based on the findings of the *DRAFT Failure Investigation Westchester North Lagoon Bridge* (USKH now Stantec, 2014), core samples were drilled and removed to examine the ledger connection area for moisture and decay. 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 on two of the bridges.

The general appearance of the bridge structures was the same for all bridges. This is expected since they were constructed at relatively the same time. The outside of the glulam beams were weathered and grey. There was some checking and cracking in the sides of the beams but larger checking and splitting in the top of the beams in line with the lag bolts that secure handrail mounts. Ledgers generally have areas of water staining and moisture on the surface. The top of the wood decking could not be observed on three bridges because it was covered by the fiberglass deck overlay. Specific prominent bridge observations are noted below, photos for each bridge follow the narratives.

4.1 2nd Avenue Bridge

The 2nd Avenue Bridge had a more recently added safety fence located on the north side. It is unclear when this fence was installed. The weight of this fencing is not significant. The general configuration and members match the original construction drawings except 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. Some of the wood decking extended below the other decking. It is assumed this decking was replaced sometime in the past with thicker material that had to be notched to fit flush on the top side. There was surface dampness at the ledger and the bottom of the glulam. Some significant cracks were noted at the ledger location in some areas. Checking and cracking was noted near the top at the guardrail mounting locations and on the sides. A core sample was taken from the exterior side of the southwest glulam girder near midspan. The sample was very wet near the outside edge and saturated inside; half the sample fell apart as it was extracted from the girder.

2nd Avenue Bridge

2nd Avenue Bridge – Cracks near bearing point

2nd Avenue Bridge – Checks in the plane of the lag bolts

2nd Avenue Bridge – Delamination near top of Glulam girder

	
2nd Avenue Bridge – Disintegrated core sample	2nd Avenue Bridge – Core location

4.2 Sisson Loop Bridge

The Sisson Loop Bridge was generally dryer than the other bridges but there were areas of dampness along the bottom of the deck, glulam, and at the ledger. The general configuration and members match the original construction drawings except 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 girders have less cracking and checking than the other bridges. The ledger on the southwest side was wet with some cracking. Two cores were cut and removed, one up from the bottom and the other from the outside face. The samples were taken approximately 15 feet from the south abutment on the west side. Both cores were slightly damp with most of the moisture located near the ledger connection.

	
Sisson Loop Bridge	Sisson Loop Bridge – Checking at top of glulam girder
	
Sisson Loop Bridge – Ledger connection	Sisson Loop Bridge – Bottom core hole

Sisson Loop Bridge – Exterior core hole

Sisson Loop Bridge – Ledger core hole

Sisson Loop Bridge – Core samples

4.3 Fish Creek Bridge

The Fish Creek Bridge has some checking at the top of the glulam girders. The deck, ledgers, and bottom of the glulams were somewhat damp. A core sample was taken approximately 10 feet from the support. The core was solid but damp, with wetness increasing on the end nearest to the ledger.

Fish Creek Bridge – Glulam girder

Fish Creek Bridge – Bolted girder connection

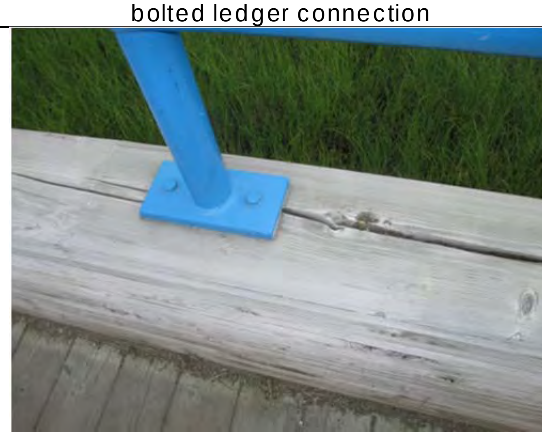
Fish Creek Bridge – Checking at top of glulam

Fish Creek Bridge – Boring hole at glulam girder

Fish Creek Bridge – Core sample

4.4 South Lagoon Boardwalk

The South Lagoon Boardwalk did not have a fiberglass decking overlay and the decking could be observed. The decking was weathered and warped but there did not appear to be "soft" areas indicative of rot. Galvanized steel angles were bolted near the midspan of the decking. The "bridge" areas had ledgers and undersides of beams that were damp. A core sample was taken approximately 8 feet from the east abutment, south beam. The sample was slightly damp on the inside. Another sample was taken through the ledger. The inside of the ledger was slightly damp. The center section of the boardwalk is framed with dimensional lumber joists bearing on glulam beams. The joists span over interior glulam beams and attach to the "exterior" beams with ledgers and top flange joist hangers. The hangers are galvanized and little corrosion was seen. The interior joists had clips tying the joist to the glulam and these appeared in serviceable condition. The nails were rusty so it is not clear what coating they had originally. Since the joists are in direct bearing at midspan or in hangers at the ledger, and the decking helps to secure the top flange hangers, the rusty nails are probably not a concern. The joists did not have signs of decay or rot but were moisture stained and damp in locations.

	
South Lagoon Boardwalk – Glulam girders	South Lagoon Boardwalk – Moisture at deck and bolted ledger connection
	
South Lagoon Boardwalk – Moisture at plaza framing	South Lagoon Boardwalk – Checking at top of glulam girder

	
South Lagoon Boardwalk – Boring hole at ledger	South Lagoon Boardwalk – Boring hole at glulam exterior

South Lagoon Boardwalk – Core sample

5 ANALYSIS

An analysis of the bridges were performed generally based on the AASHTO 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). Full calculations are included in Appendix B.

Only Limit State Strength I was checked to determine dead and live load capacity under a failure condition similar to the recently collapsed Westchester North Lagoon Bridge. Deflection and other serviceability limit states, seismic, wind, and foundations were not checked as they are not applicable to that 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 some bridges, the design live load for the bridge girders could be reduced down to 69 psf under previous editions of the specification. The deck and connections would still need to meet full live load design regardless of area. The design documents reference 85 psf for all bridges so it was assumed the bridge was designed for the reduced load.

In addition to the uniform loads specified by AASHTO, the capacities of the four bridges that are the subject of this report were also checked using the wheel loads from the truck and chipper combination that caused the Westchester North Lagoon Bridge to fail. MOA provided information regarding the truck and the chipper. 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. These Coastal Trail Bridges were checked against both this vehicle and also the vehicles listed on the construction documents, either a 10,000 lb vehicle or a 7,000 lb vehicle.

The new handrail and fiberglass deck overlay added in the 2013 trail rehabilitation added approximately 7 percent to the weight of each bridge, except the South Lagoon Boardwalk where it was not installed. Modifications made to structures that cause a load increase of less than 10 percent are usually not considered significant. The Sisson Loop Bridge had an additional weight increase due to the complete wood deck replacement. The additional weight of the increased thickness decking, guardrail, and fiberglass deck is approximately 23 percent.

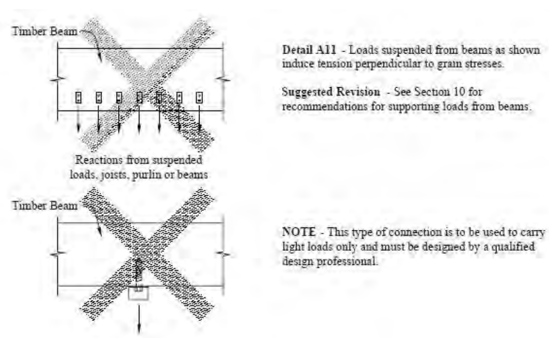
Wood member allowable stresses were taken from the current AASHTO or NDS provisions based on the grades of wood noted in the general notes on the original construction documents.

5.1 2nd Avenue Bridge

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, and 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.

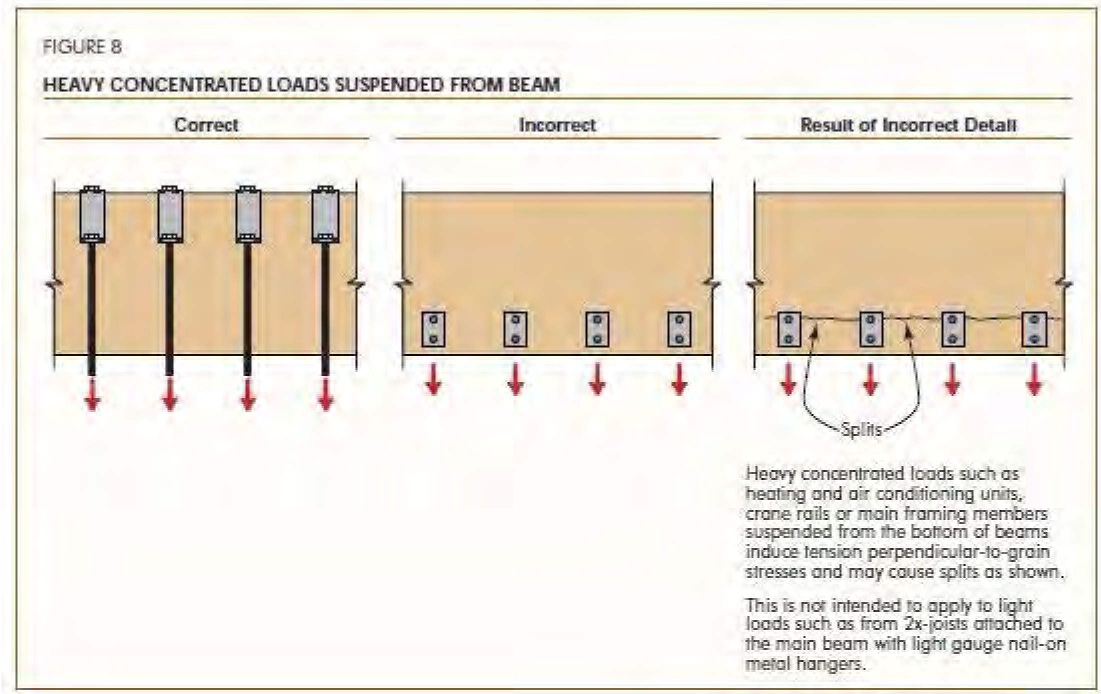
The likely limit state, based on failure analysis of the Westchester North Lagoon Bridge, appears to be cross-grain tension in the glulam beam induced by the ledger connection. The ledger is set inboard of the glulam from 3/4- to 1-inch near the bottom of the glulam beams. The ledger, located near the bottom of the bridge girder, transfers load to only the bottom third of the girder. The load from the deck is then transferred to the stronger upper two thirds of the girder by tension that develops across the grain, i.e. the bottom third has to pull down on the upper two thirds of the beam. 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. In an unrelated part of the code, 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 truck/chipper combination exceeds the allowable stress by about 40 percent. The code has a typical average factor of safety of 2, so assuming new condition with no decay, this connection would probably hold, but it is not recommended.

Reference the figures below from American Institute of Timber Construction (AITC) 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

5.2 Sisson Loop Bridge

The Sisson Loop Bridge is similar to the 2nd Avenue Bridge except the original wood decking had been replaced with a greater strength decking and then overlaid with fiberglass decking. The decking was not the load limiting factor so this change neither increases the bridge capacity nor does it help the cross-grain tension problem at the ledger. The additional weight does reduce the allowable uniform live load of the main bridge members down to 67 psf. This is within an acceptable margin of error considering a reduced live load allowed by older codes.

Like the 2nd Avenue Bridge, the limit state is the ledger connection considering cross-grain tension of the glulam girder at the connection interface. The significant moisture and decay present at the interface is alarming and will reduce the connection capacity significantly.

5.3 Fish Creek Bridge

The Fish Creek Bridge has a similar configuration to the other bridges but section sizes are different due to the reduced spans. The ledger connection is smaller, and is not inset to the glulam, which greatly limits the ledger strength. The decking called out is smaller than the other bridges, which also reduces decking capacity. The main glulam beams are adequate for the full uniform load or a vehicle similar to the truck/chipper combination. The decking is adequate for the listed design vehicle (7,000 lb street sweeper) but appears deficient for the truck/chipper combination. The limit state for the deck is the ledger connection. The connection is flush mounted with no notch like the 2nd Avenue or Sisson Loop

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

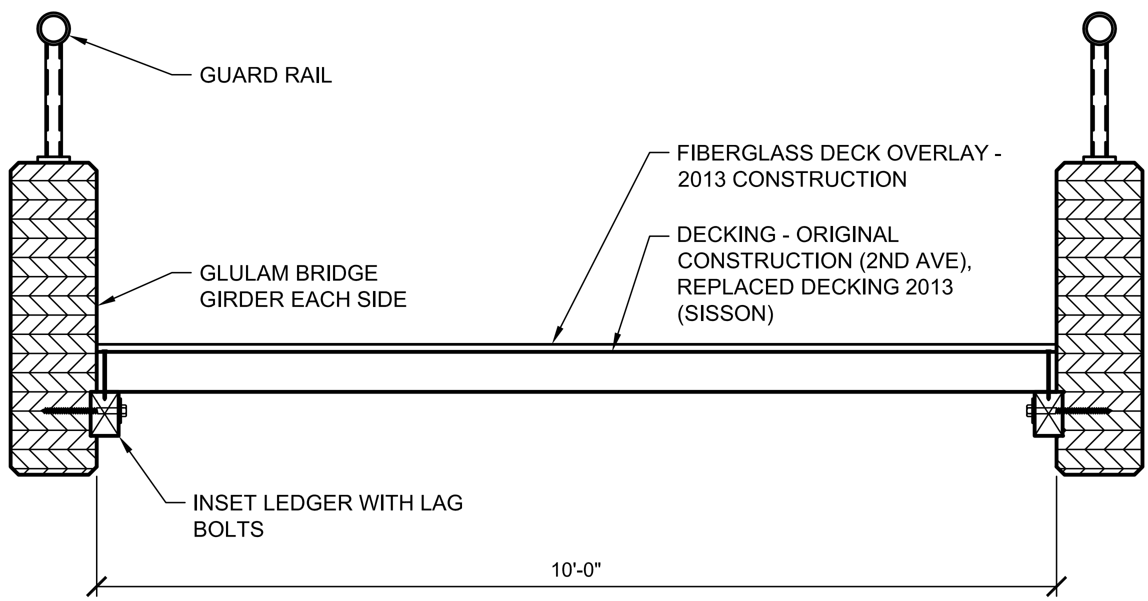
bridges so the entire load must be transferred by shear through the lag bolts. The ledger is only a 4x4, which has an actual dimension of 3-1/2 inches. This lag bolt is centered on the ledger, which puts it close to both top and bottom edges. This results in a significant capacity reduction of the connection. This connection limits live load capacity to 51 psf and the ledger is deficient for both vehicle loadings. This connection also results in cross grain tension on the bottom side of the beam, which is not recommended.

5.4 South Lagoon Boardwalk

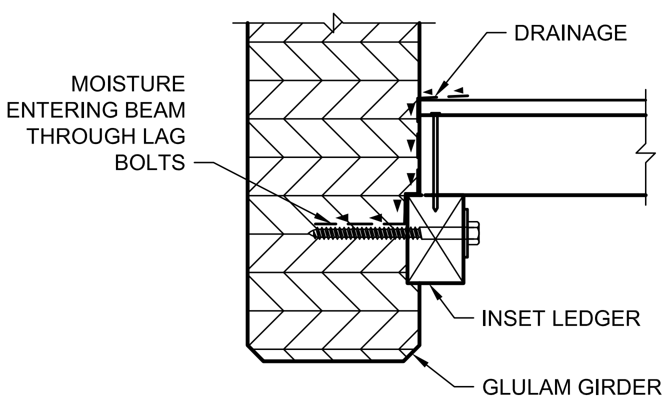
The "bridge" spans of the boardwalk are of similar configuration and construction to the Fish Creek Bridge. The main beams are adequate for full uniform or vehicle loading, while the decking is limited to a smaller vehicle. The ledger connection limits the overall capacity to 51 psf, and it is deficient with regard to concentrated vehicle loads. The center "platform" area is framed with dimensional lumber joists bearing on glulams. All the glulams supporting the center platform are adequate for full uniform or vehicle loading. Where the joists span is typically less than 10 feet the area is good for full live load. A section of platform extending from the Harwood Circle entrance to the "bridge" intersection has a longer joist span (up to 18 feet). This area has a reduced live load of 35 psf. Analysis shows the dimensional lumber cannot support vehicle loading in any location in the platform.

5.5 Possible Temporary Reinforcement

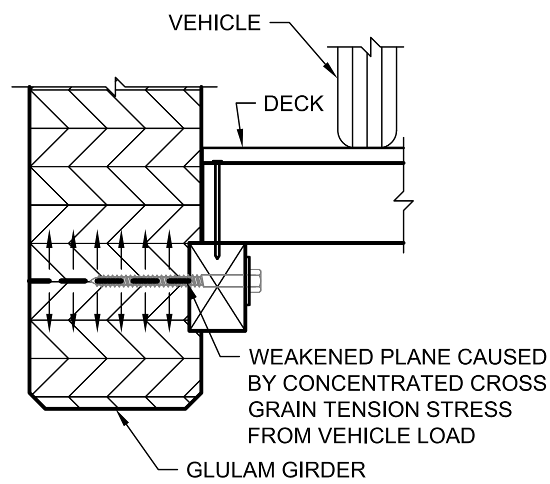
It may be possible to temporary reinforce the wood ledger and glulam beams on some bridges to allow trail grooming and other vehicles to pass over those bridges for a short term (1-2 year) timeframe. This would not be a permanent repair but short term reinforcement. The reinforcement would need to directly support the ledger and transfer load around the weakened plane up to competent wood. The reinforcement would involve steel brackets and bolting similar as depicted in Figure 7. A rough order of magnitude cost to install steel brackets on one bridge is about $200,000 - $300,000.



BRIDGE CROSS SECTION



WATER INFILTRATION



WEAKENED CONNECTION

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Figure
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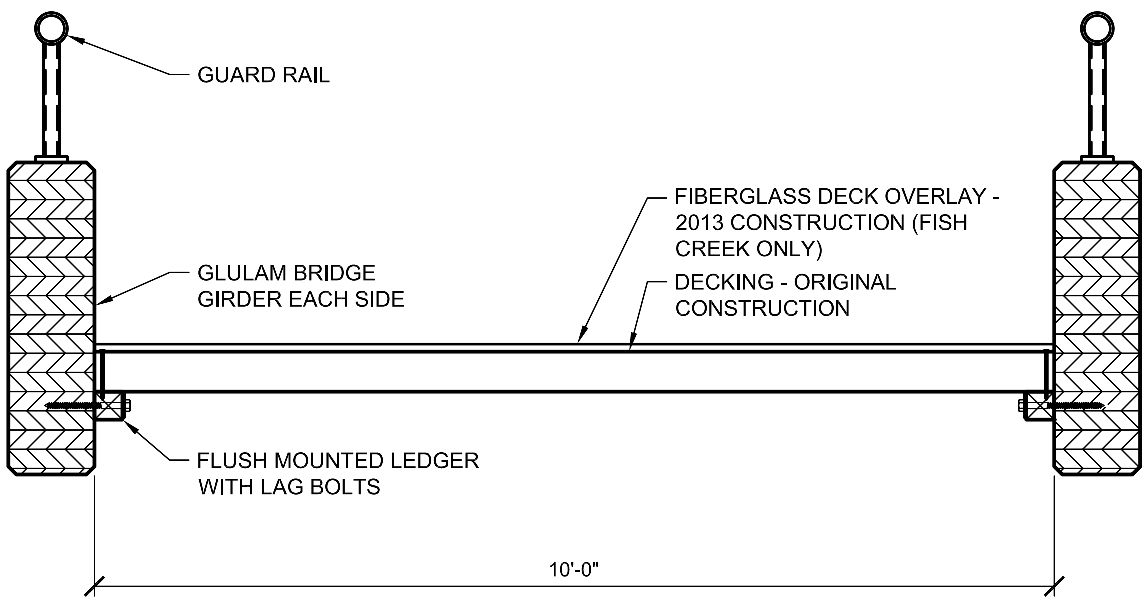

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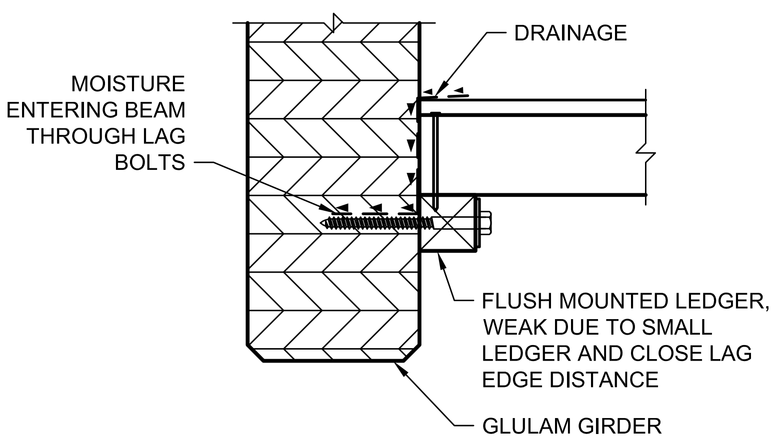

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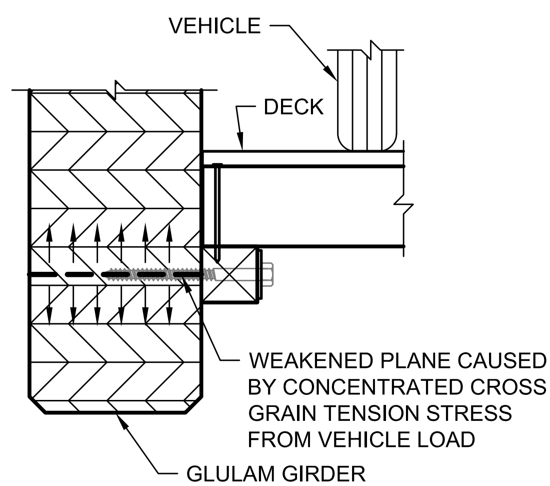
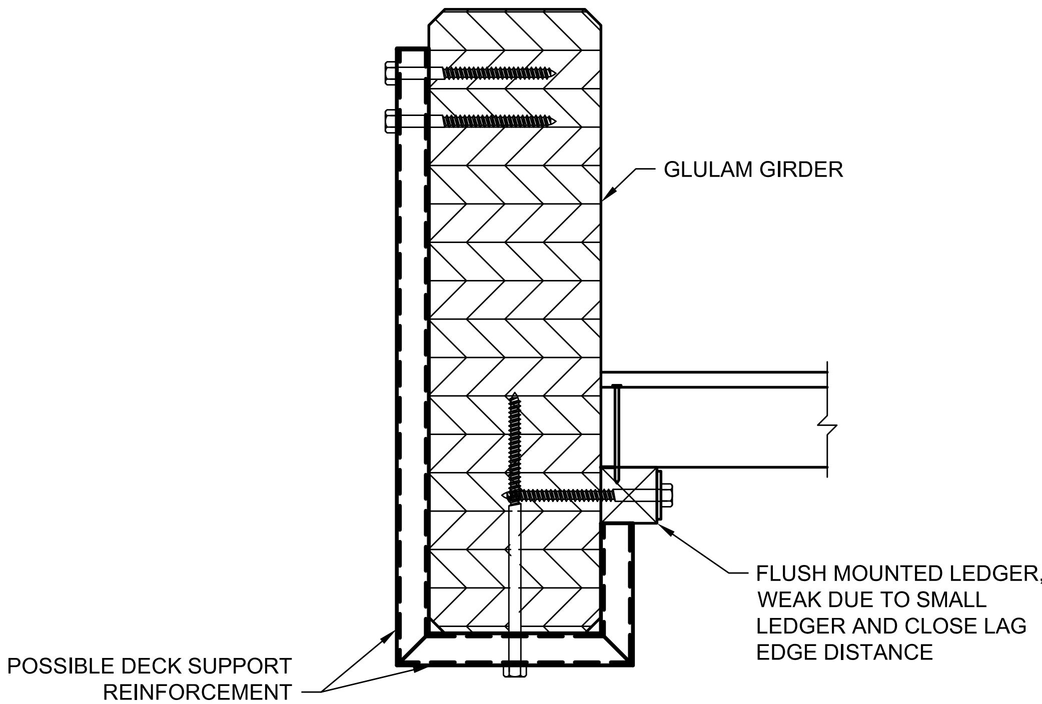
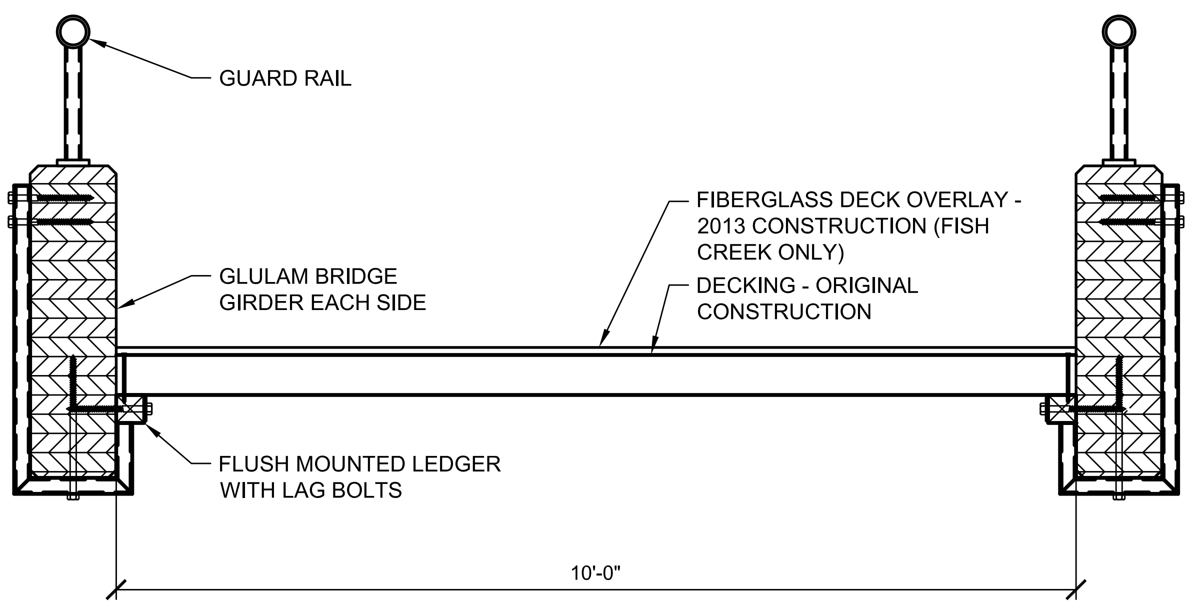


WEAKENED CONNECTION



SHORT-TERM GIRDER REINFORCEMENT



BRIDGE CROSS SECTION

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Figure
7

6 CONCLUSIONS

The Coastal Trail Bridge Assessments were directed as a result of the Westchester North Lagoon Bridge failure. That 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 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 the beam decayed over time. The beams are preservative treated but the treatment only treats the wood on the perimeter and the interior remains vulnerable to decay as moisture is introduced into the center of the beam. 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.

Since the other Coastal Trail bridges are of similar construction, there are similar issues and concerns. The main ledger lag bolted connection is a conduit for moisture into the main glulam and the connection also induces cross grain tension stress along this plane. Core samples taken from the main girders of the Coastal Trail bridges confirm the glulam beams are saturated to varying degrees. The saturation is inducing decay inside the glulam and weakening a plane similar to the plane of weakness evident in North Lagoon Bridge. Of the bridges examined, the 2nd Avenue Bridge has the most advanced decay with the core sample being totally saturated and crumbling when withdrawn. The main girders also have significant cracks along the connection plane in various locations.

It may be possible to temporary reinforce the wood ledger and glulam beams on some bridges to allow trail grooming and other vehicles to pass over those bridges for a short term (1-2 year) timeframe. This is not a substitute for permanent bridge replacements.

Some summary points regarding the bridges site observations, core samples, and analysis:

- The bridge designs did not consider the cross-grain tension induced by the connection.
- Normal pedestrian loading will probably not initiate failure until the wood decay is more advanced.
- The lag bolts are allowing moisture to infiltrate the interior of the glulam beams causing decay and weakness to occur in the connection zone. The beams are preservative treated but the treatments penetrate wood only to a depth of 2 inches. The lag bolts create a path for water to penetrate into the untreated core of the beam.
- The bridges with the smaller ledger not inset have a significantly reduced capacity and no capacity for vehicle loading.
- The South Lagoon Boardwalk has a reduced capacity due to the long joist spans.
- The ledger detail induces cross-grain tension in the main supporting member, which is not recommended in wood design.
- Vehicles must be kept off the bridges until they are replaced or repaired. A permanent bollard must be installed in front of every bridge access point to prevent vehicles from driving onto the bridges.
- The bridges, particularly 2nd Avenue Bridge, should be regularly monitored for signs of failure along the connection zone.
- A short term reinforcement (1-2 year design life) could be installed for an estimated rough order of magnitude cost of about $200,000 - $300,000 per bridge.

A recommended priority of bridge replacements is:

1. 2nd Ave
2. Fish Creek
3. South Lagoon Boardwalk
4. Sisson

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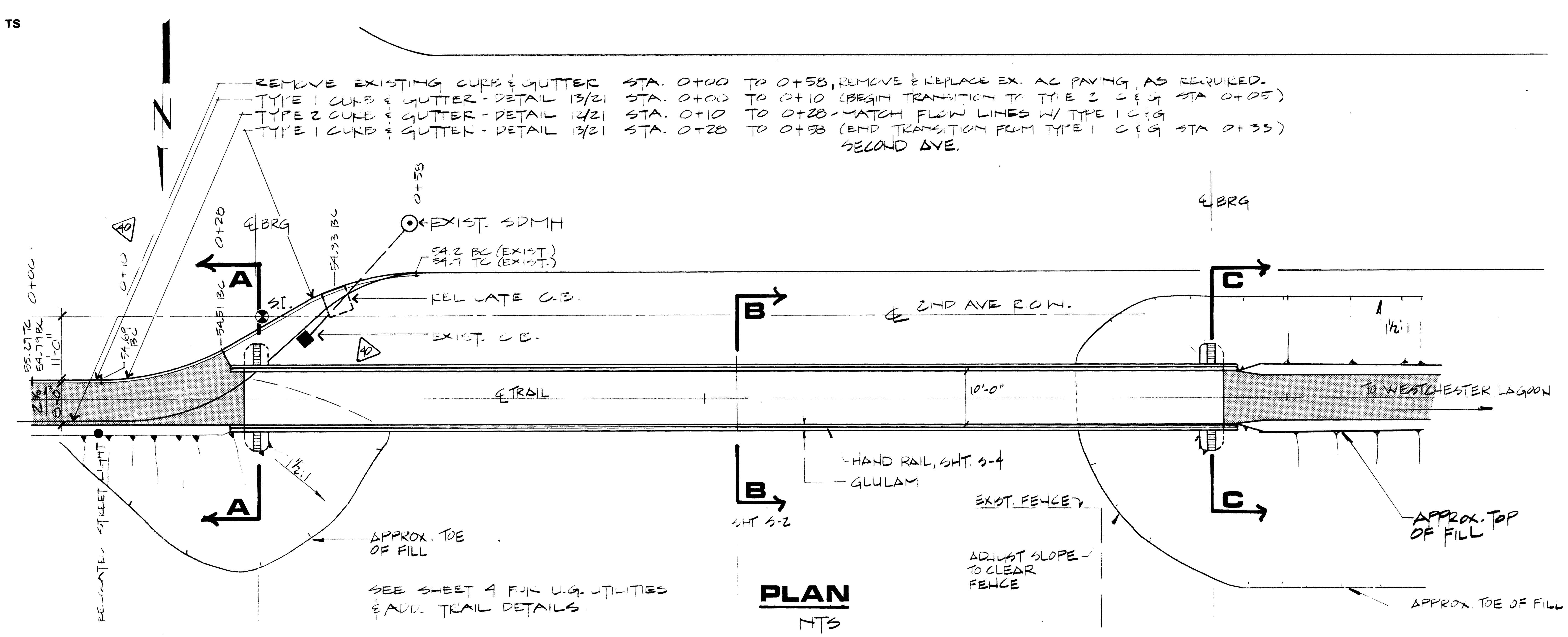
Appendix A – Original Construction Drawings

CT TS

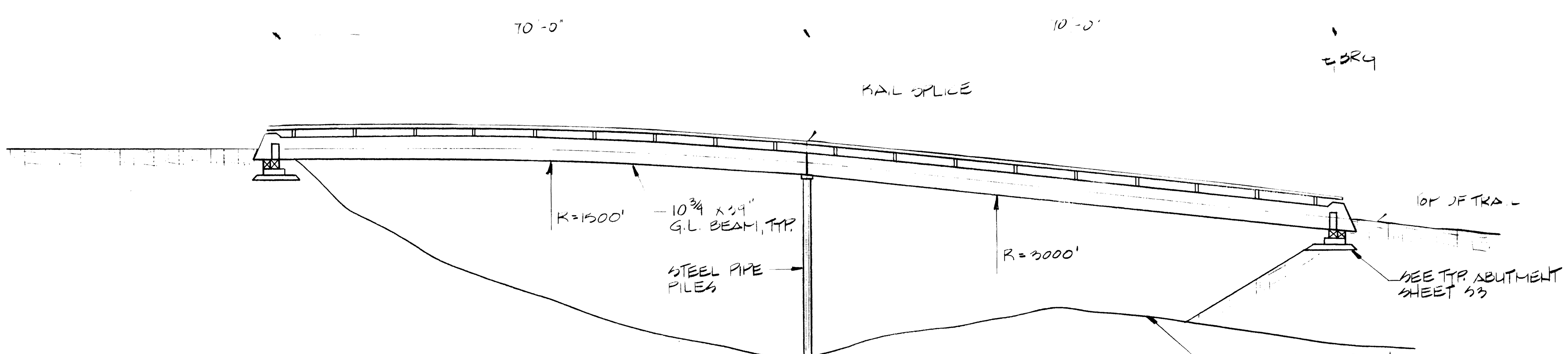


PLAN

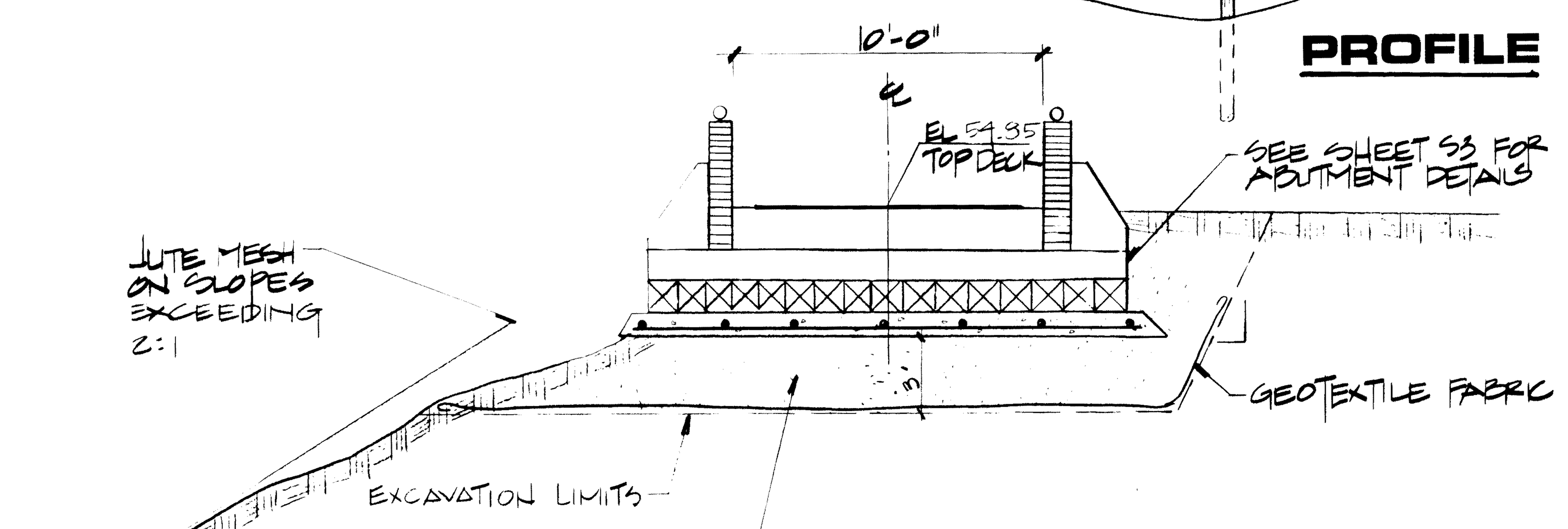
NTS

STA 0+33 STA 1+03 STA 1+73

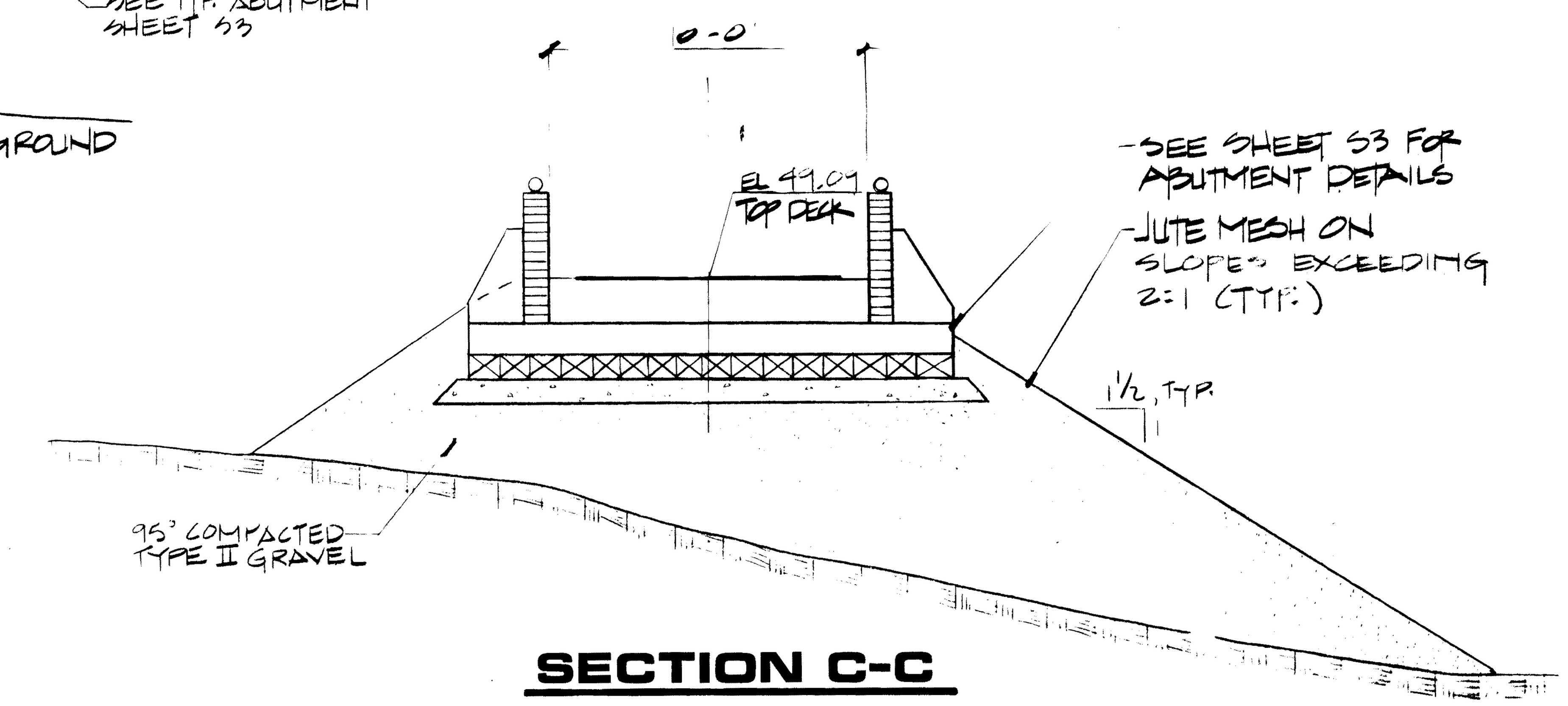
NOTE: SEE SHEETS S2, S3 & S4 FOR DETAILS OF RAILING, CONNECTIONS & BOARDWALK SECTION



PROFILE



SECTION A-A



SECTION C-C

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-VB 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
900 WEST FIFTH AVENUE, SUITE 440
ANCHORAGE, ALASKA 99501

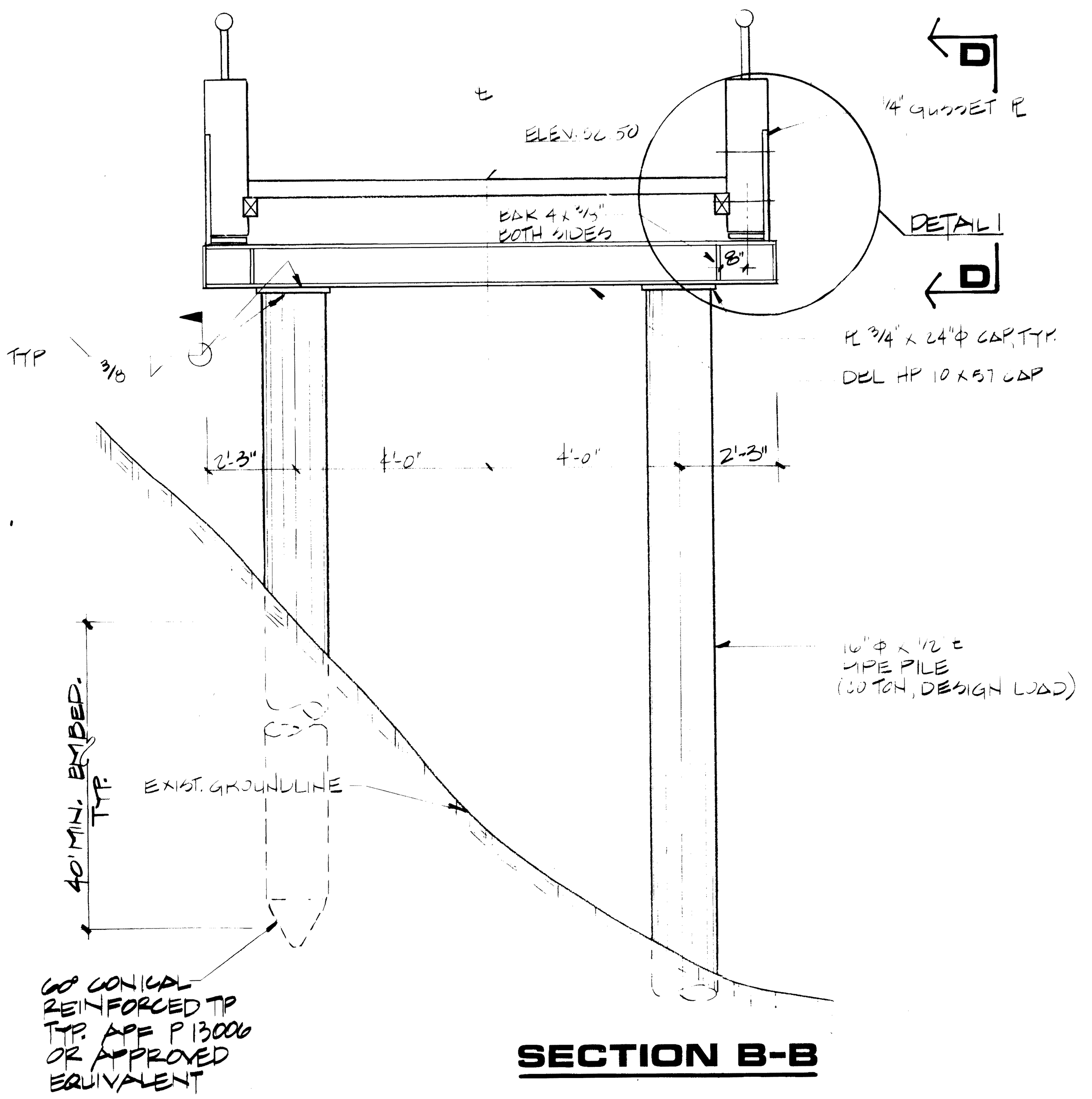
Park Identification 605
Plan Set Number 109

BOARDWALK PLAN & DETAILS
2ND AVENUE & H STREET

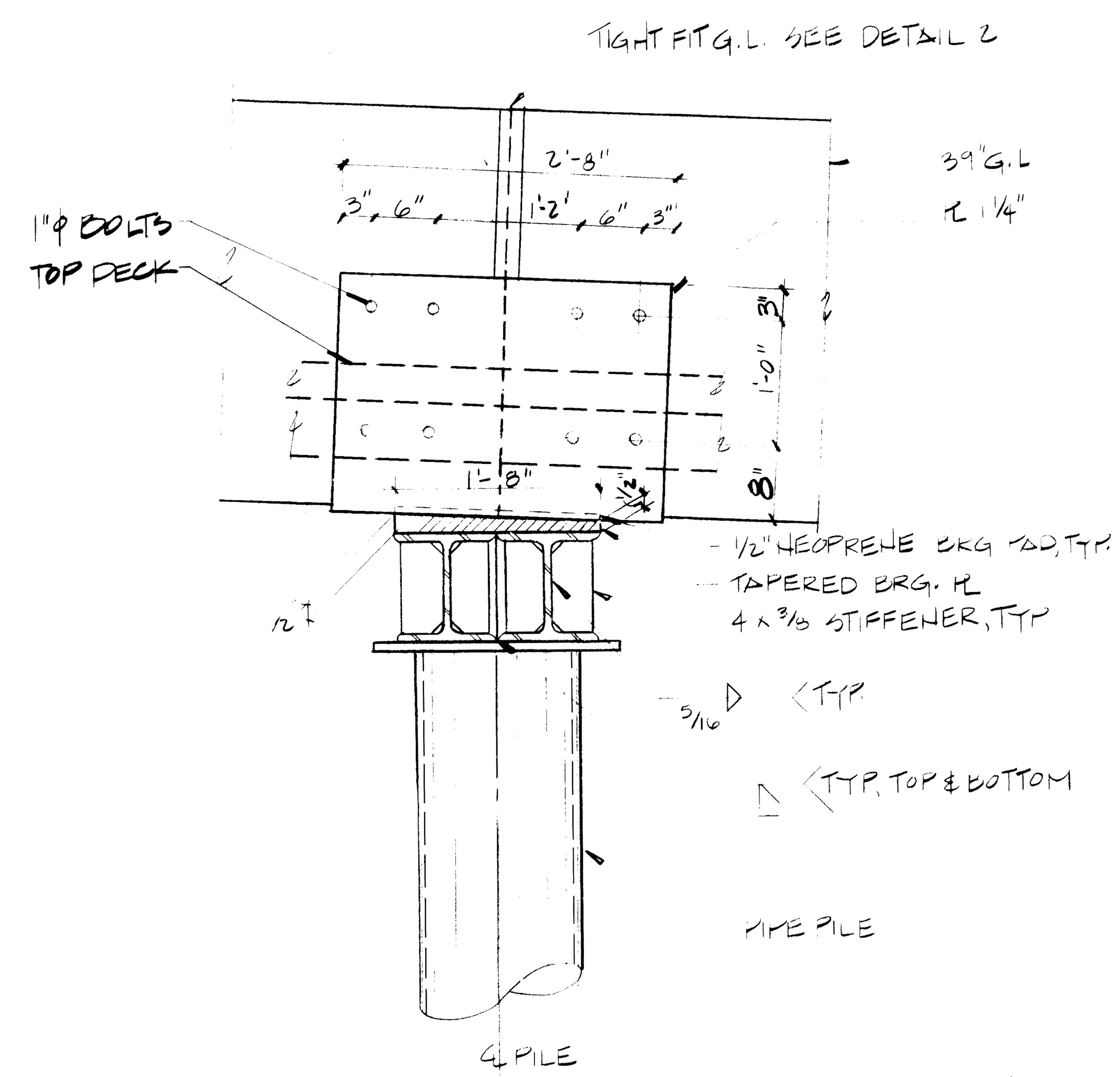
Date: 6/1/87 Com. No.: 86.705.50
In Charge: DV
Drawn By:
Checked By:

S1

CT TS



SECTION B-B



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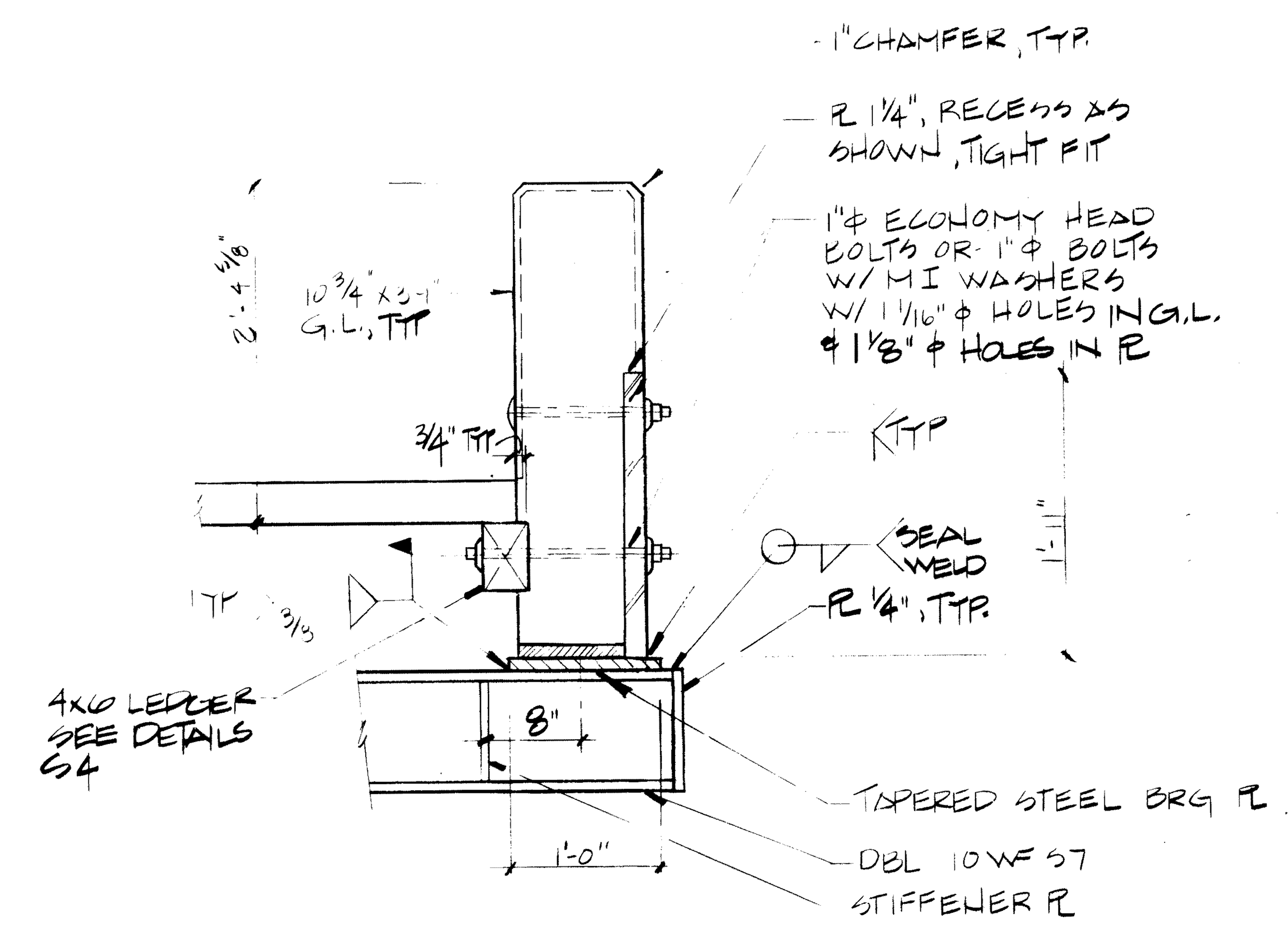
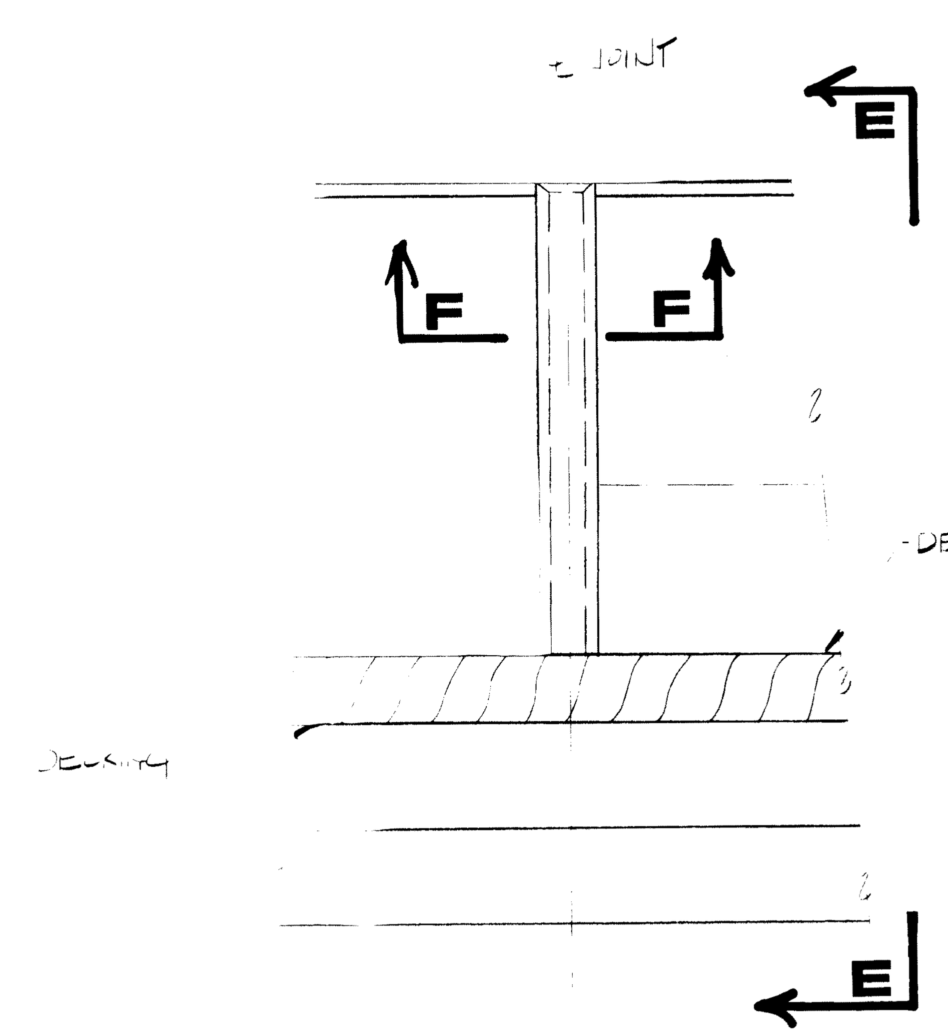
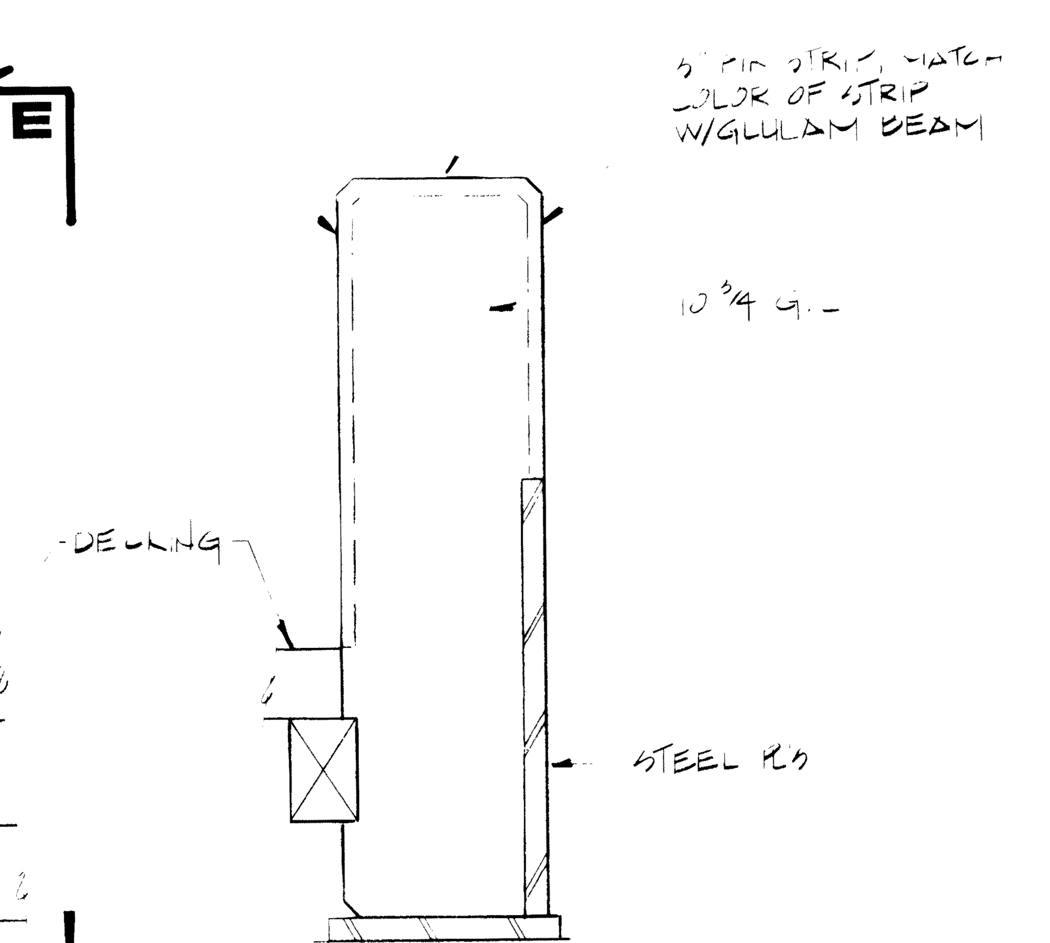


DETAIL 1





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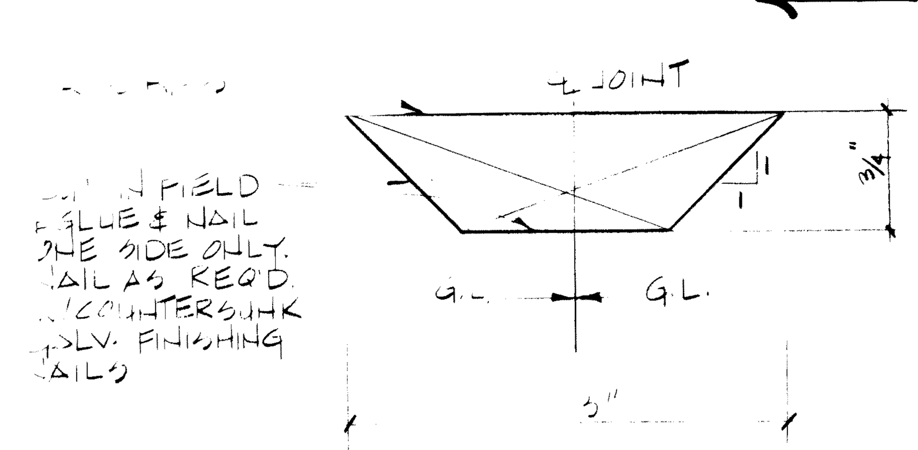


SECTION F-F

DETAIL 2

Revisions 9/23/87 10/5/87

COASTAL TRAIL
PHASE 3

COASTAL TRAIL
CONTRACT DOCUMENTS

Clarke
Vogan

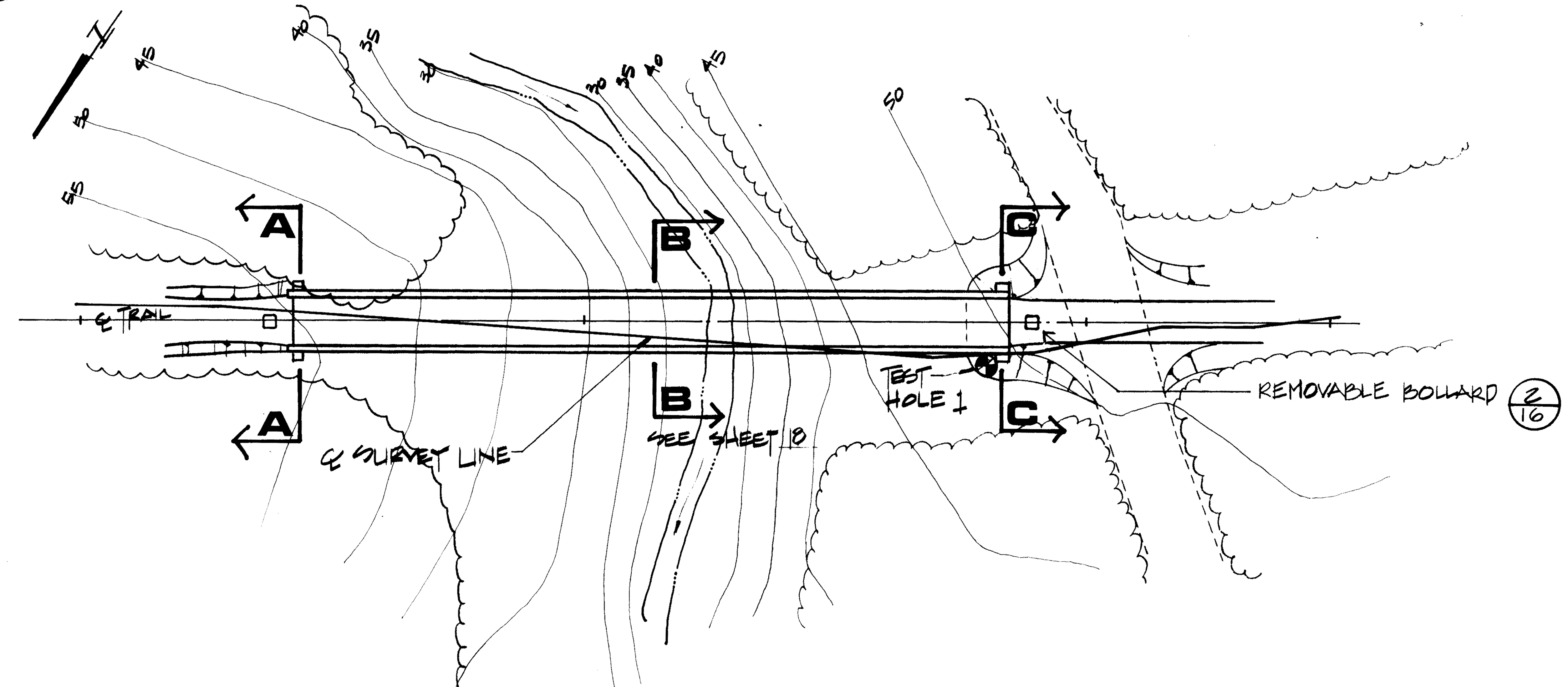
LANDSCAPE ARCHITECTURE ■ PLANNING
WEST FIFTH AVENUE. SUITE 440
NCHORAGE, ALASKA 99501

Identification 605
Set Number 110

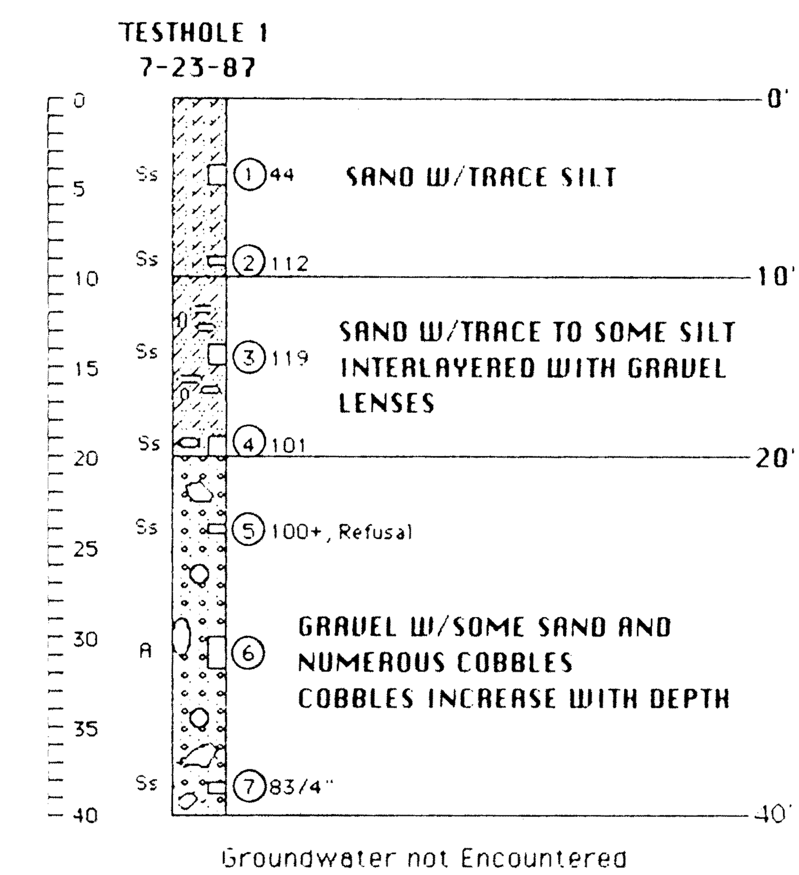
BOARDWALK DETAILS
2ND AVENUE & H STREET

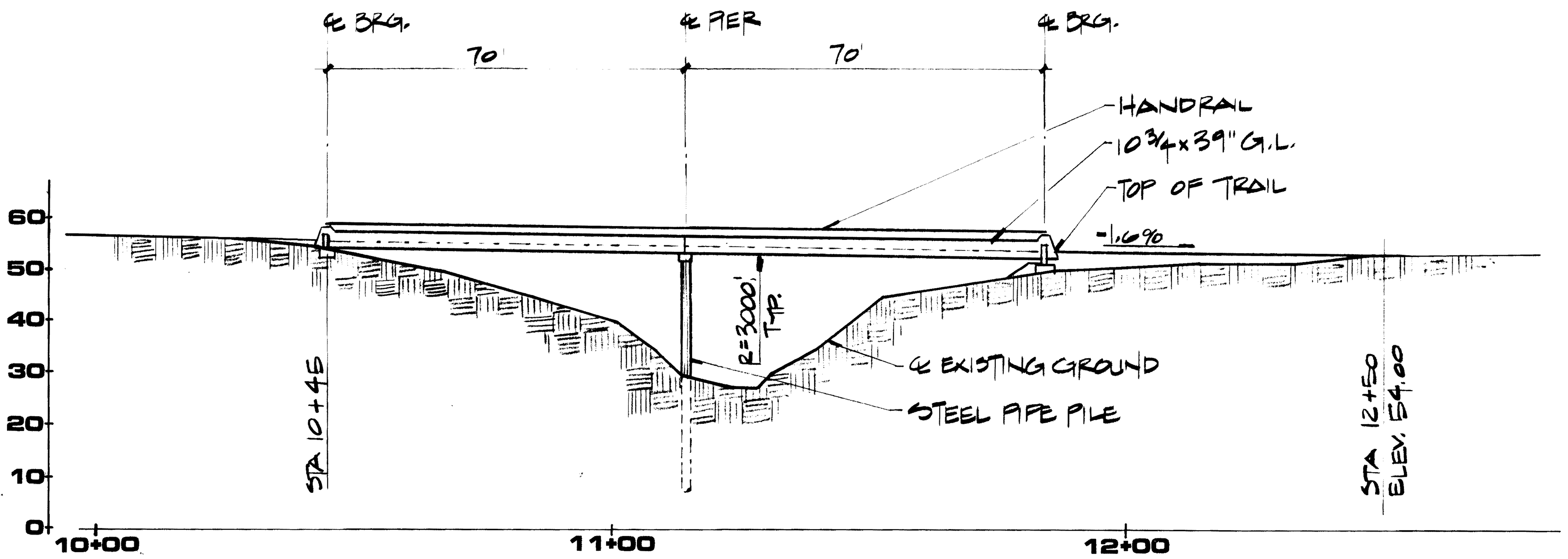
Date: 6/1/87 Com. No.: 86.705.50
Charge : DV
Drawn By :
Checked By :

S2



PLAN





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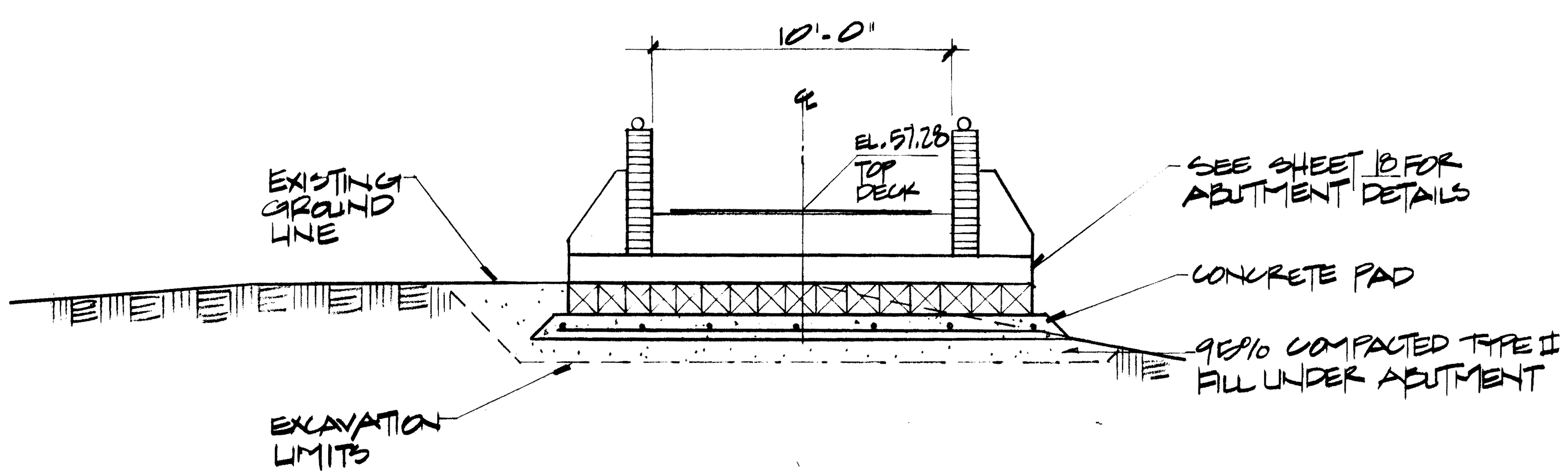
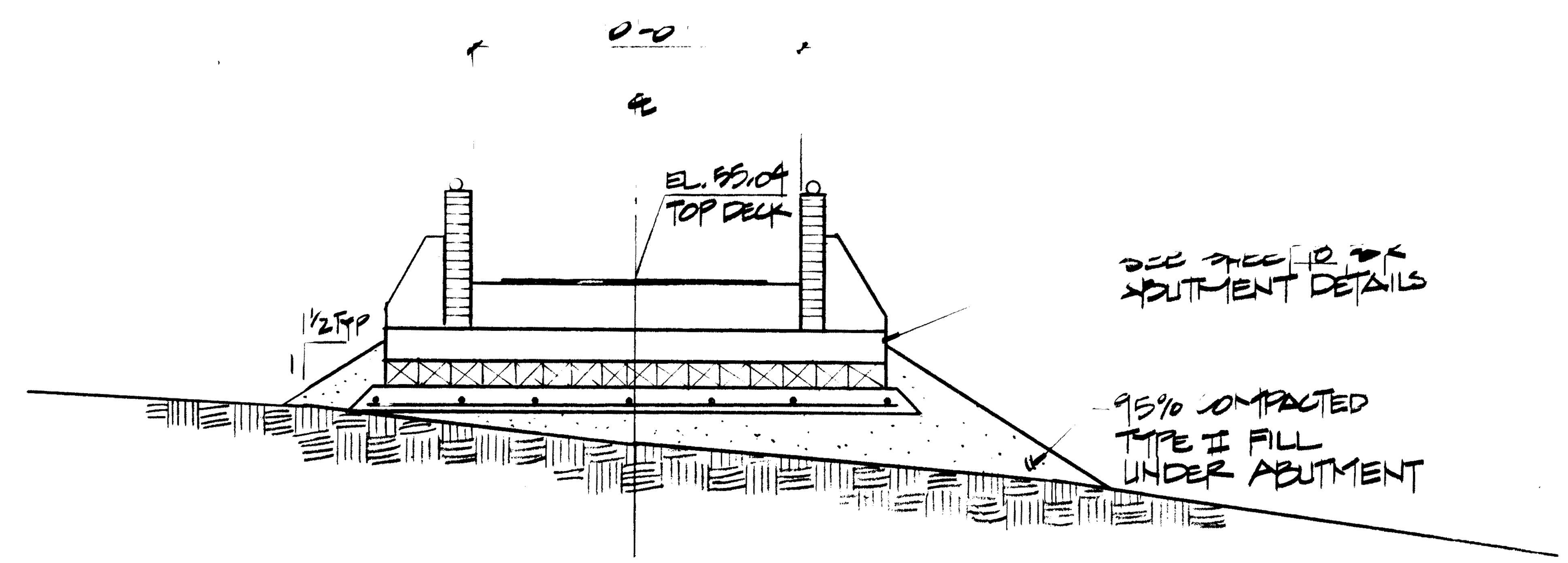


SECTION A-A



SECTION C-C

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. (HOT DIP)
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.

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.

Park Identification 605
Plan Set Number 143

Burns Clarke Vogan Architects
ARCHITECTURE ■ LANDSCAPE ARCHITECTURE ■ PLANNING
900 WEST FIFTH AVENUE, SUITE 440
ANCHORAGE, ALASKA 99501

Date: 7/24/87 Com. No.: 86-705.60
In Charge :
Drawn By :
Checked By :

MUNICIPALITY OF ANCHORAGE
CAPITAL PROJECTS OFFICE

COASTAL TRAIL
PHASE 4 BRIDGE PLANS

Drawing Number
Sheet Number

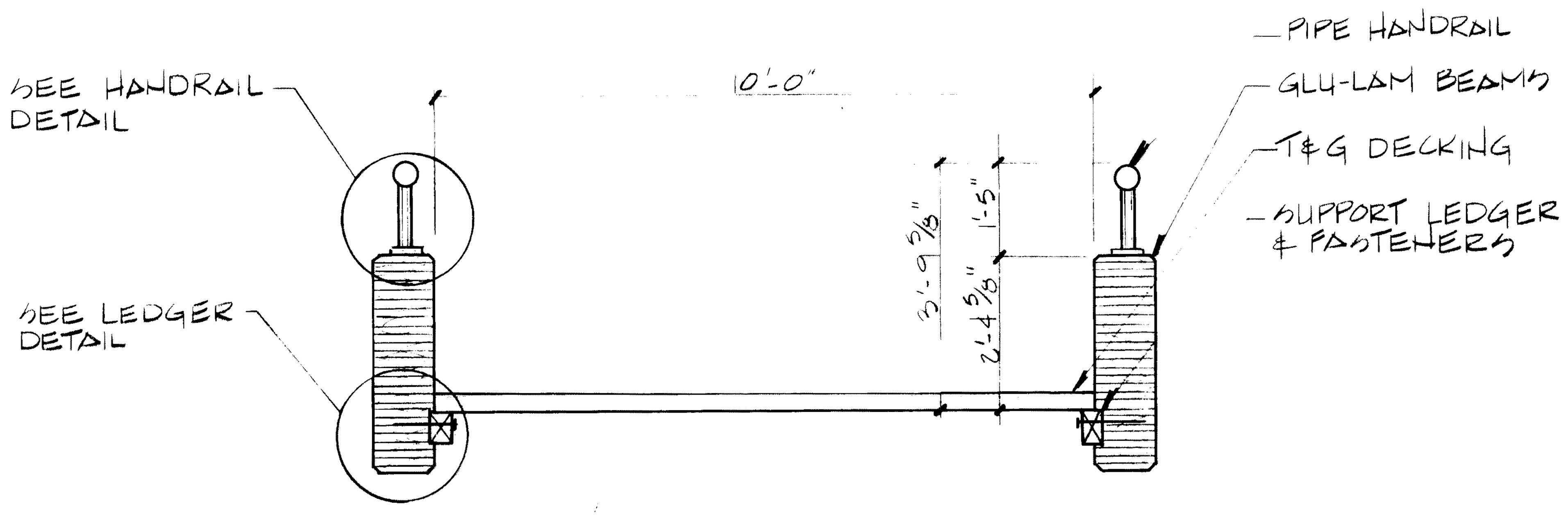
Burns Clarke Vogan Architects
ARCHITECTURE ■ LANDSCAPE ARCHITECTURE ■ PLANNING
900 WEST FIFTH AVENUE, SUITE 440
ANCHORAGE, ALASKA 99501

Date: 7/24/87 Com. No.: 86-705.60
In Charge :
Drawn By :
Checked By :

MUNICIPALITY OF ANCHORAGE
CAPITAL PROJECTS OFFICE

COASTAL TRAIL
PHASE 4 BRIDGE DETAILS

Drawing Number
Sheet Number
18 of 19



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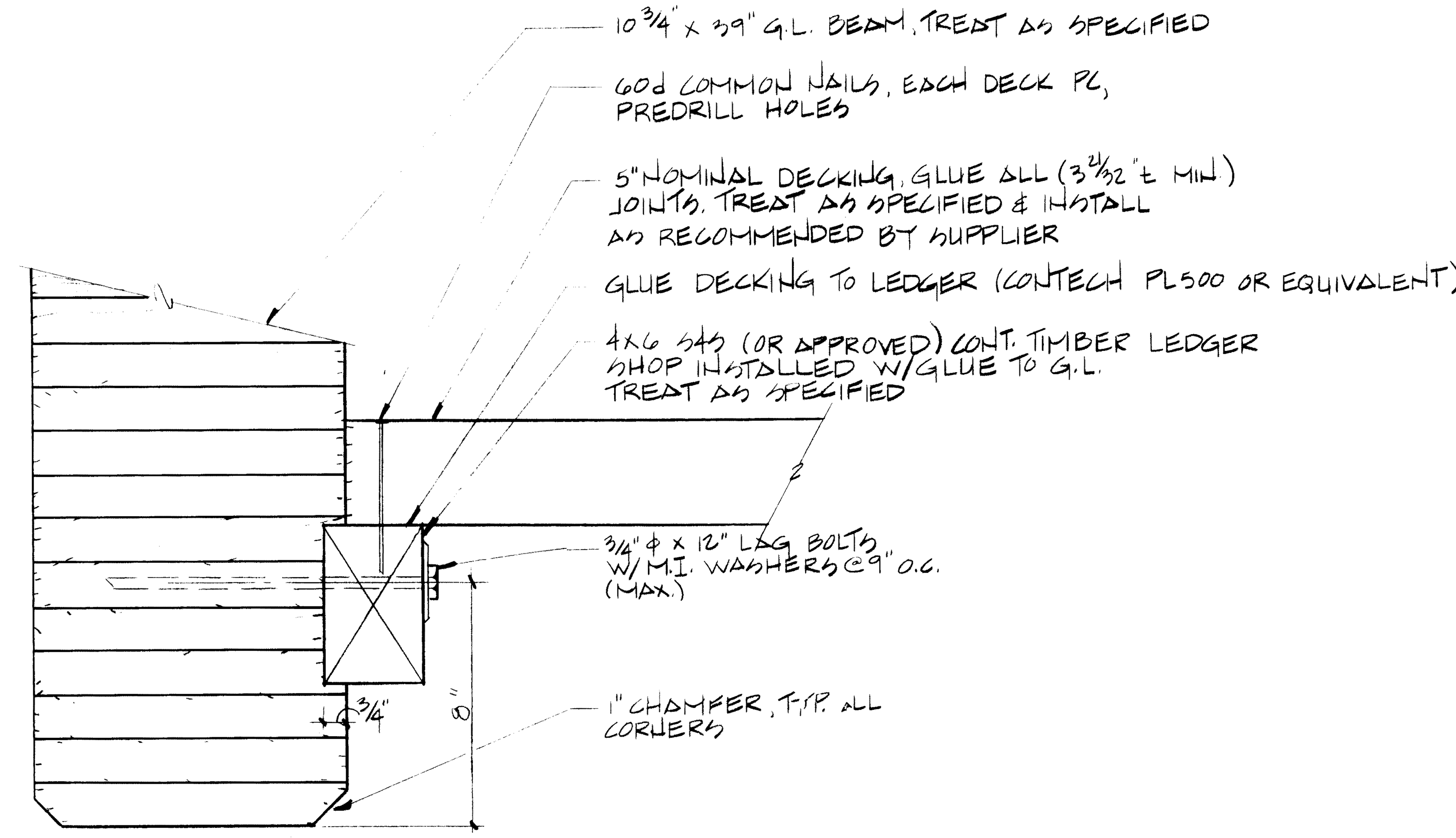

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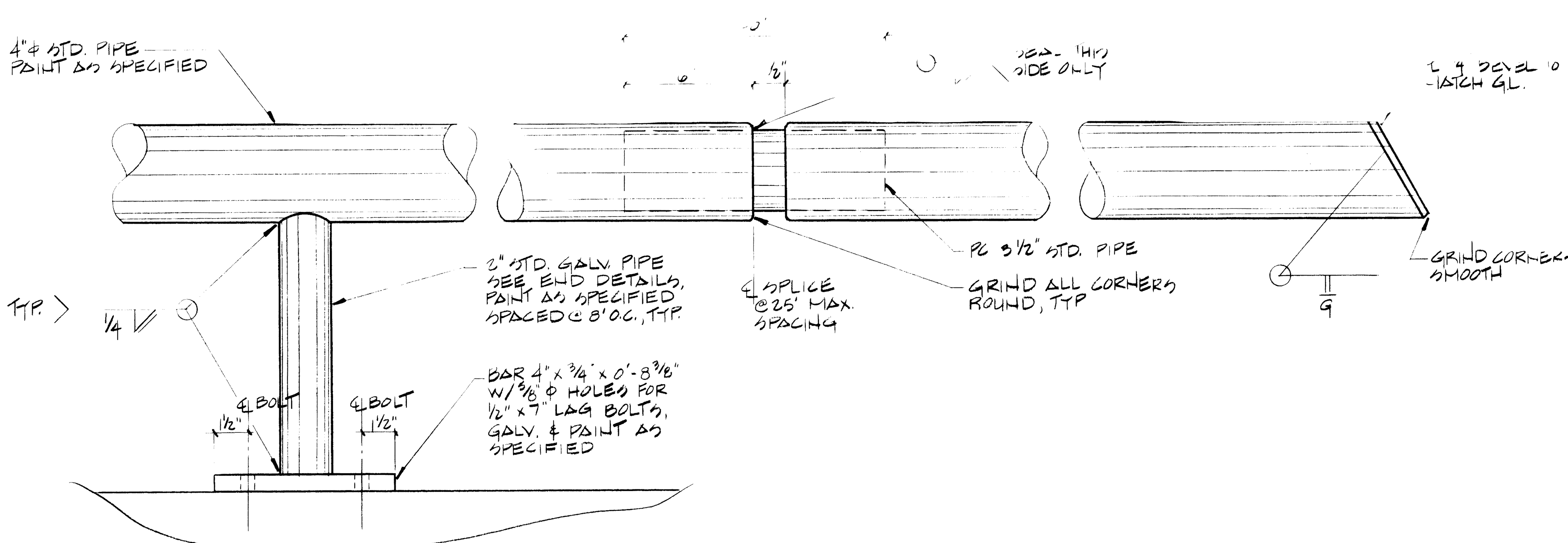
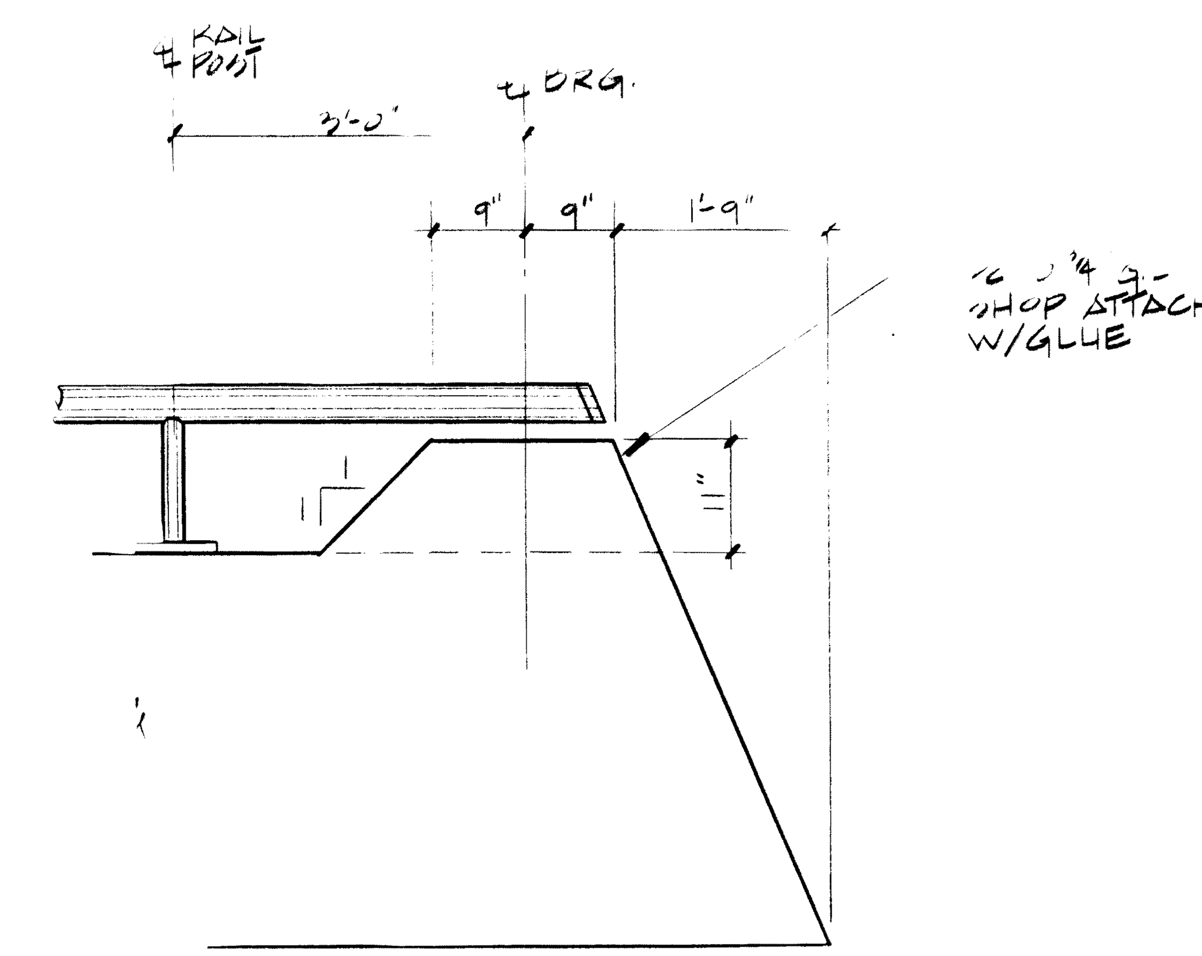


HAND RAIL DETAILS



END DETAIL

Park Identification 605
Plan Set Number 145

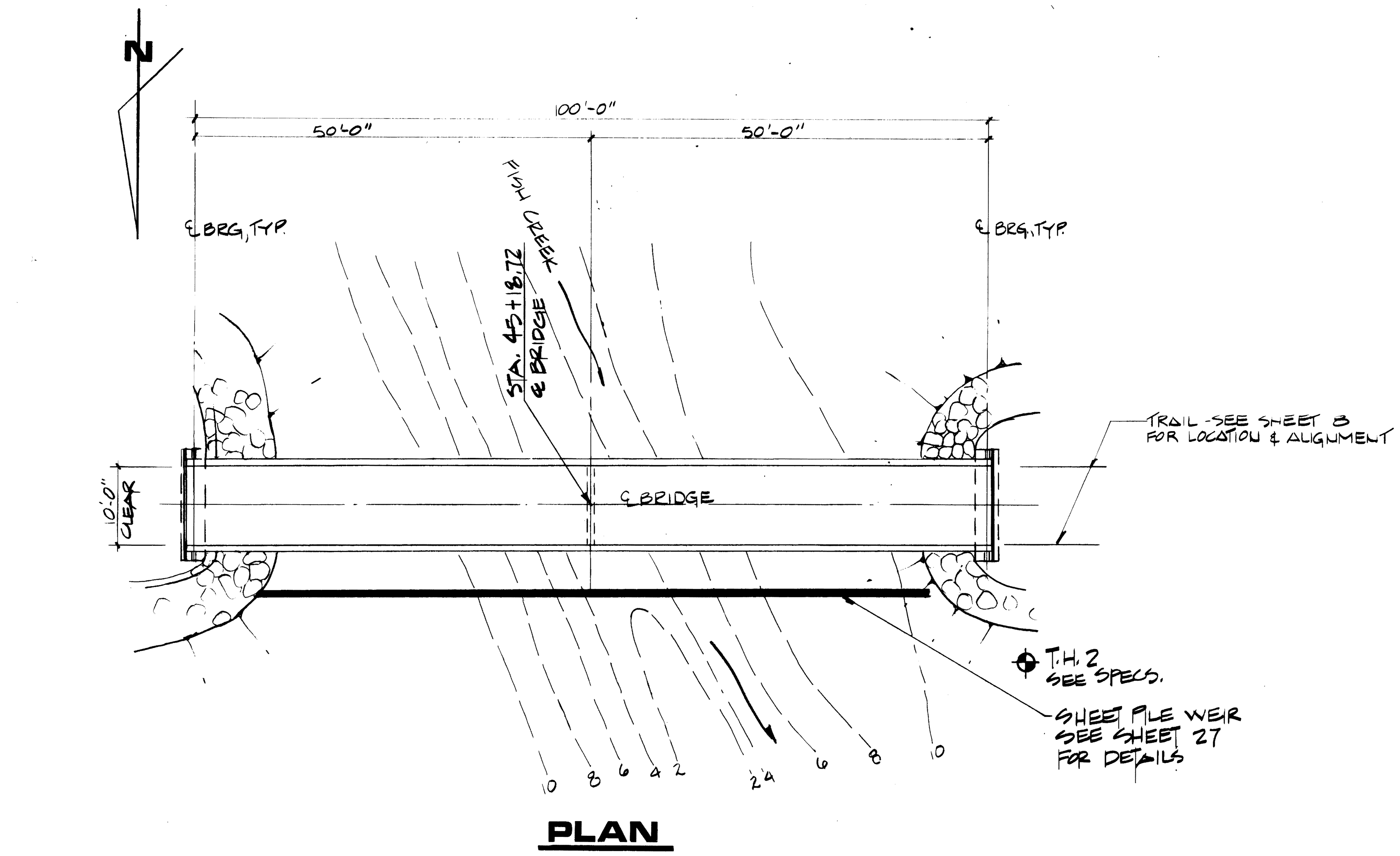
Burns Clarke Vogan Architects
ARCHITECTURE ■ LANDSCAPE ARCHITECTURE ■ PLANNING
900 WEST FIFTH AVENUE, SUITE 440
ANCHORAGE, ALASKA 99501

Date: 7/24/87 Com. No.: 86-705.60
In Charge :
Drawn By :
Checked By :

MUNICIPALITY OF ANCHORAGE
CAPITAL PROJECTS OFFICE

COASTAL TRAIL
PHASE 4 BRIDGE DETAILS

Drawing Number
Sheet Number
19 of 19



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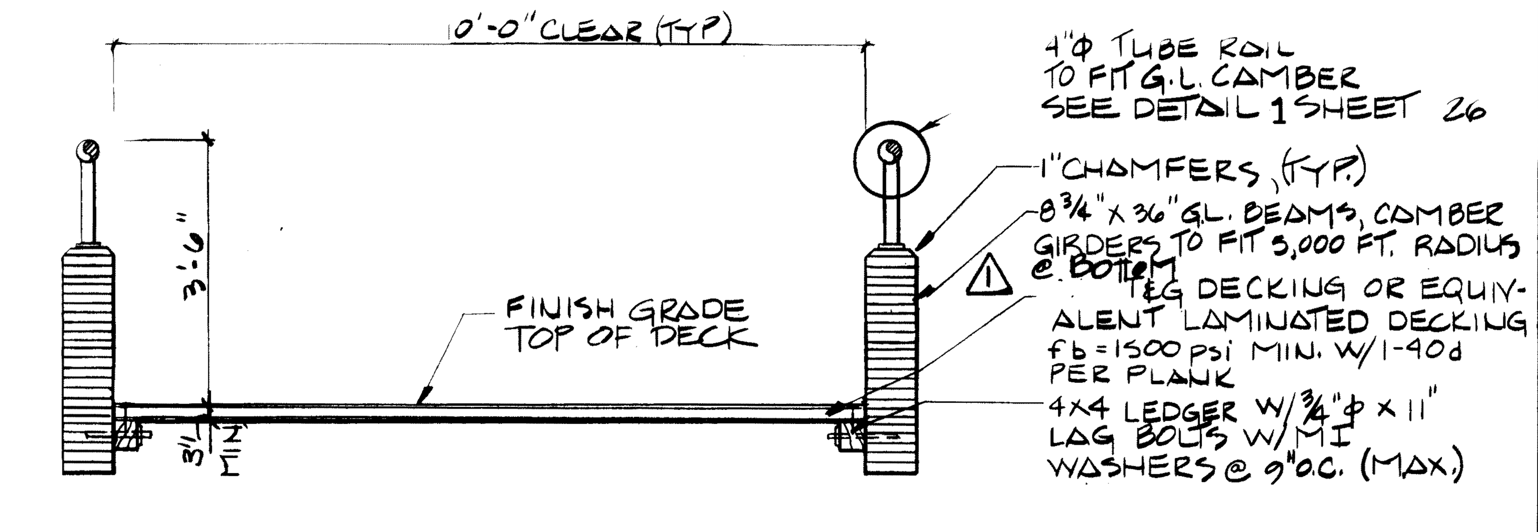

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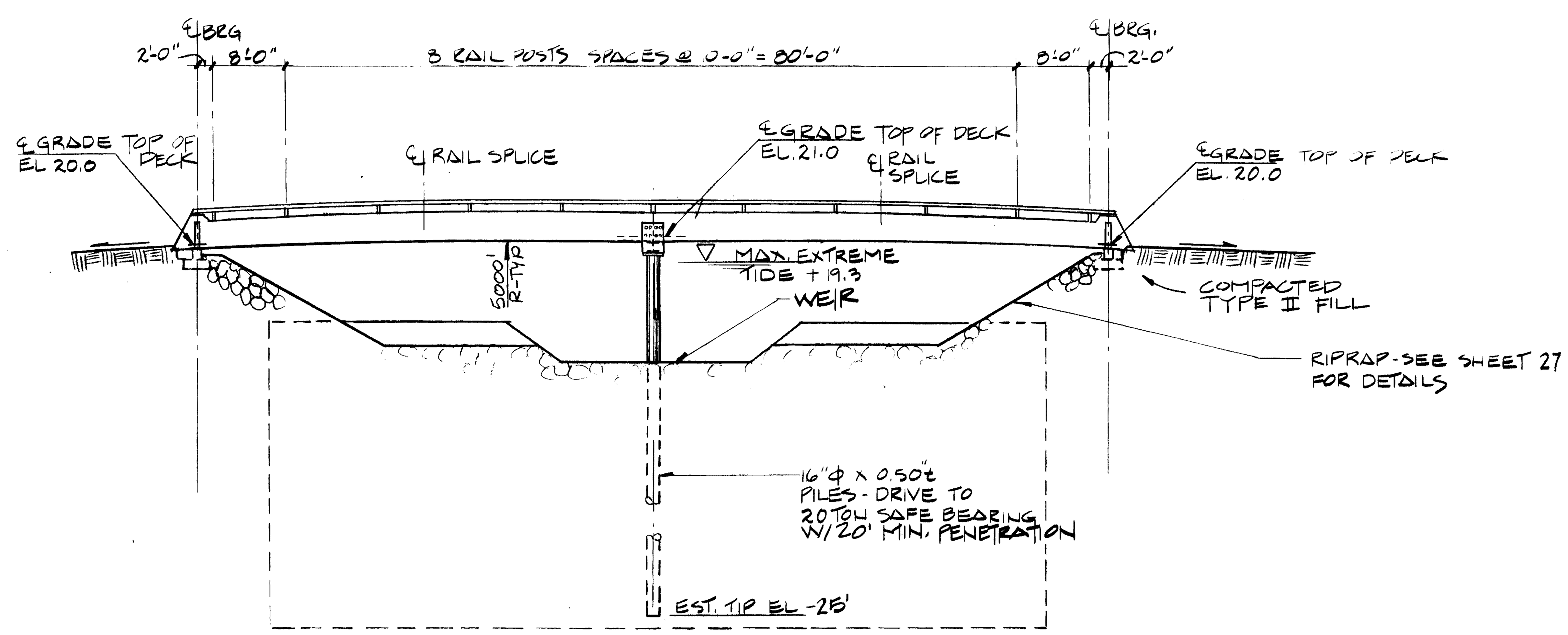


ELEVATION

NOTE: PROFILE AND CONTOUR DATA PROVIDED BY ALASKA DEVELOPMENT CONSULTANTS (ADC), 1983. THE CONTRACTOR MUST VERIFY ACTUAL GROUND CONDITIONS AT TIME OF CONSTRUCTION.

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 1982 UBC and 1977 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
5. Maintenance Vehicle = (STREET SWEEPER 7000 G.V.W.)
6. Seismic Zone 4
7. Footing Pressure = 1.0 ksf (max.)
8. Ice Load = 2,000 lb./pile at midpoint

Foundations: Backfill with compacted gravel min. 12-in. below and 18-in. adjacent to wood mudsills. Compaction of backfill materials shall be 90% 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 appearance grade Douglas Fir, combination 2,200F 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. All other wood shall be Douglas Fir No. 1 or better.
3. All structural steel shall be ASTM A-36 and galvanized after fabrication in accordance with ASTM A-123. Field repair per ASTM A-780.
4. All structural bolts shall be ASTM A-307 and galvanized in accordance with ASTM-153.
5. Nails and other connectors including washers and malleable iron washers shall be galvanized.
6. Foundation piles shall be per ASTM A-252 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., after treatment dry to a moisture content of 19%.
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.

NEOPRENE PAD: THE NEOPRENE SHALL BE OF HARDNESS OF 60±5 PER ASTM D2240

CALL BEFORE YOU DIG!

ANCHORAGE WATER & WASTEWATER	786-5557
ANCHORAGE TELEPHONE UTILITY	564-1555
CHUGACH ELECTRIC ASSOC.	562-2278
ENSTAR NATURAL GAS COMPANY	264-3740
MATANUSKA ELECTRIC ASSOC.	694-2161
MATANUSKA TELEPHONE ASSOC.	694-3134
MULTI VISIONS LTD.	562-3133
MUNICIPAL LIGHT & POWER	276-7671
STATE STORM DRAINS	337-9481
STATE STREET LIGHTS	333-5548
TRAFFIC SIGNAL CABLE	786-8355

ANCHORAGE AREA UTILITY ASSOC.

Park Identification 605
Plan Set Number 46

Revisions

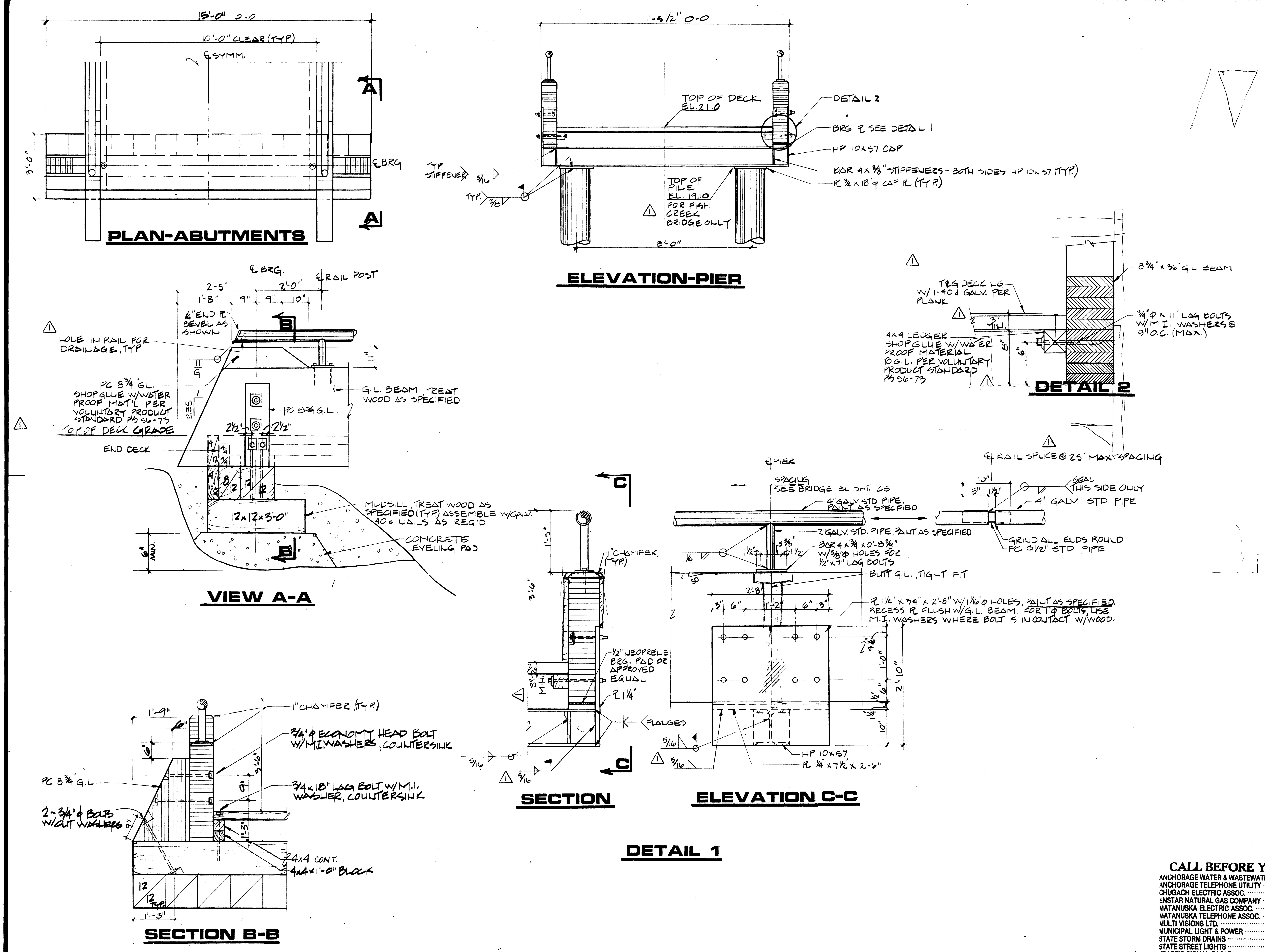
FISH CREEK BRIDGE

STATE OF ALASKA 49TH

PAVA Design, Inc.
900 West Fifth Avenue
Suite 740
Anchorage, Alaska 99501
(907) 279-5533

PAVA

Drawn: H.T.S.
Date: 1-20-86
Job No.: 85-698
Sheet: 25



CALL BEFORE YOU DIG!

ANCHORAGE WATER & WASTEWATER	786-5557
ANCHORAGE TELEPHONE UTILITY	564-1555
CHUGACH ELECTRIC ASSOC.	562-2278
ENSTAR NATURAL GAS COMPANY	264-3740
MATANUSKA ELECTRIC ASSOC.	694-2161
MATANUSKA TELEPHONE ASSOC.	694-3134
MULTI VISIONS LTD.	562-3133
MUNICIPAL LIGHT & POWER	279-7671
STATE STORM DRAINS	337-9481
STATE STREET LIGHTS	333-5548
TRAFFIC SIGNAL CABLE	786-8355

ANCHORAGE AREA UTILITY ASSOC.

Park Identification 005
Plan Set Number 47

Revisions

FISH CREEK BRIDGE DETAILS

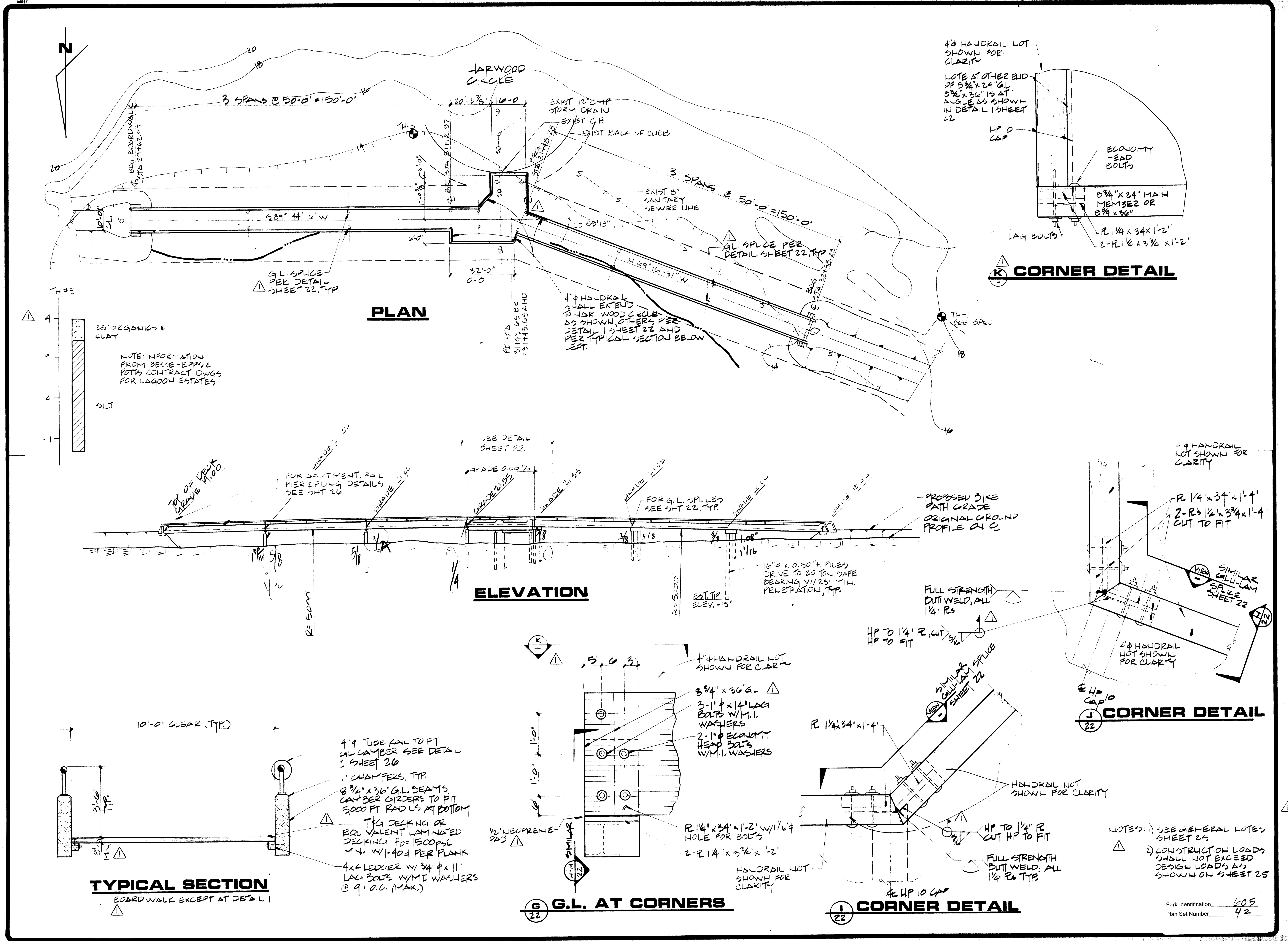
COASTAL TRAIL CONTRACT DOCUMENTS

STATE OF ALASKA 49TH Roy A. Pavia No. 1100-E REGISTERED ENGINEER

PAVA

PAVA Design, Inc.
900 West Fifth Avenue
Suite 740
Anchorage, Alaska 99501
(907) 272-5533

Drawn H.T.S.
Date 1-20-86
Job No. 85-596
Sheet 26
of 84 Sheets



Park Identification 605
Plan Set Number 42

Revisions

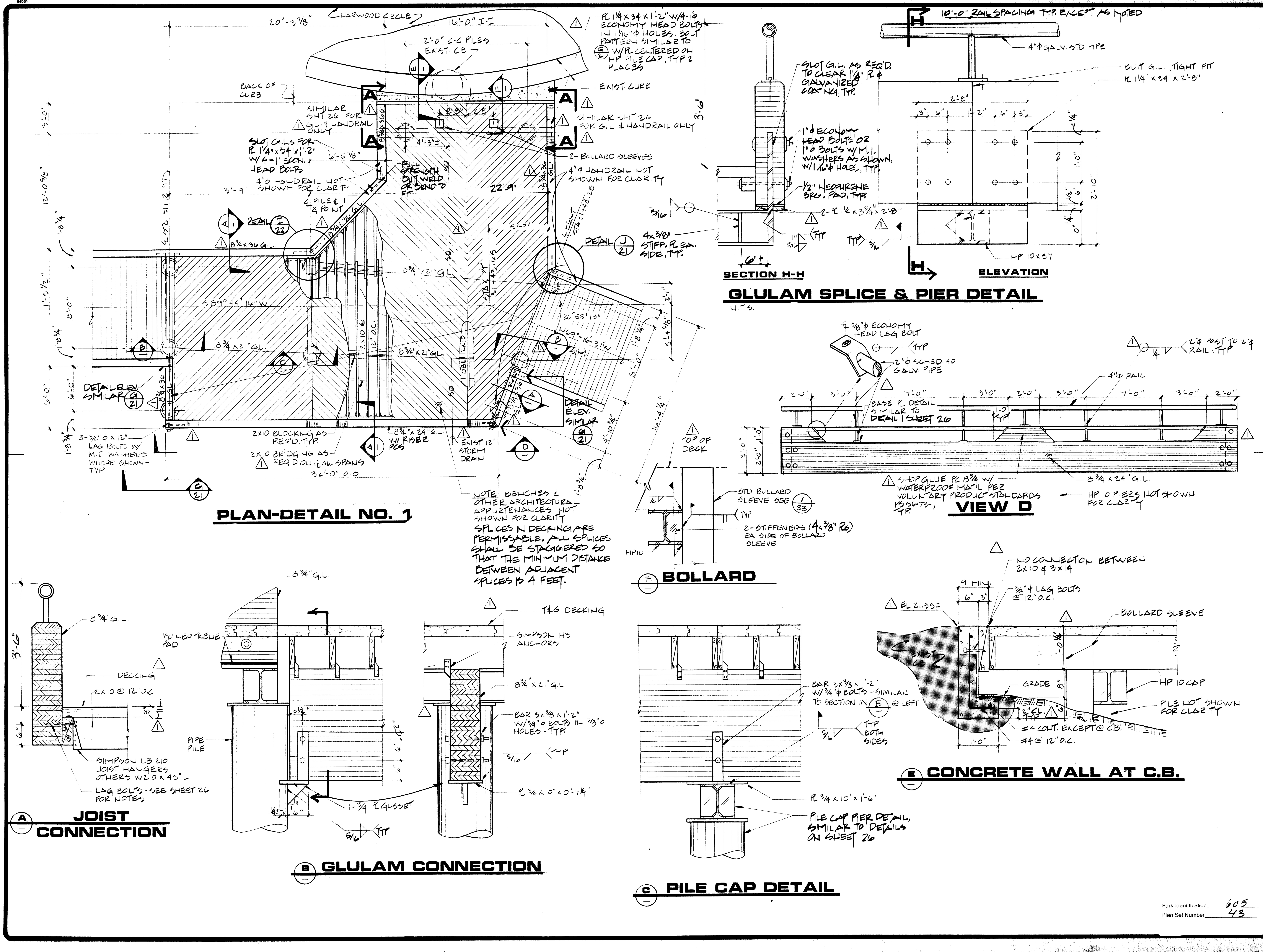
BOARDWALK PLAN & ELEVATION

COASTAL TRAIL

PAVA Design, Inc.
900 West Fifth Avenue
Suite 740
Anchorage, Alaska 99501
(907) 272-5533

PAVA

Drawn	N.T.S.
Date	1-20-86
Job No.	85-5??
Sheet	21

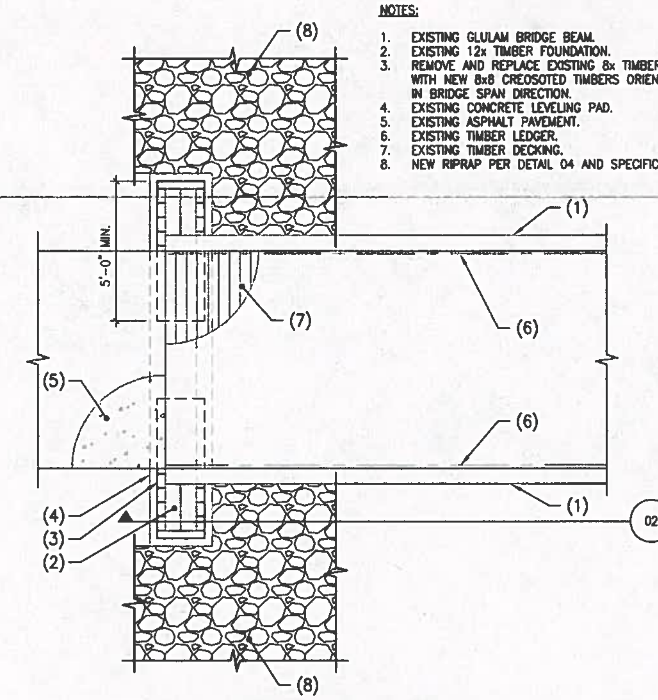


BOARDWALK DETAILS

PAVA Design, Inc.
900 West Fifth Avenue
Suite 740
Anchorage, Alaska 99501
(907) 272-5533

Drawn	N.T.S.
Date	1-20-86
Job No.	85-586
Sheet	22 of 34 Sheets

Park Identification 605
Plan Set Number 43

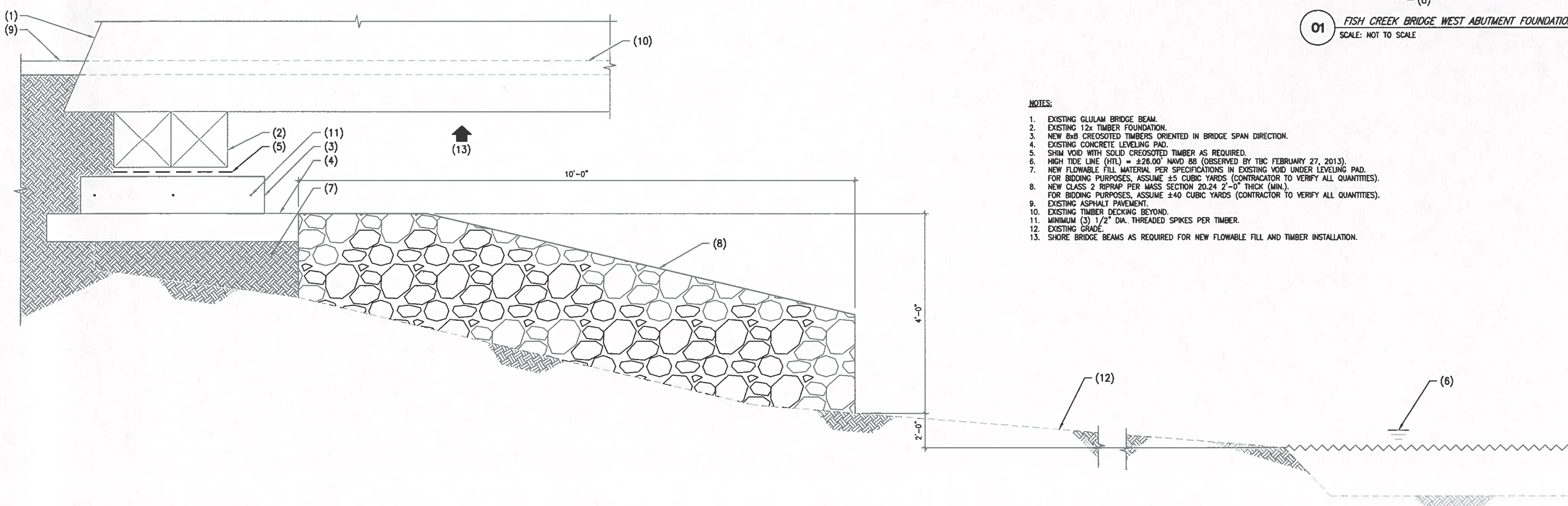


NOTES:

1. EXISTING GLULAM BRIDGE BEAM.
2. EXISTING 12x TIMBER FOUNDATION.
3. REMOVE AND REPLACE EXISTING 8x TIMBERS WITH NEW 8x8 CREOSOTED TIMBERS ORIENTED IN BRIDGE SPAN DIRECTION.
4. EXISTING CONCRETE LEVELING PAD.
5. EXISTING ASPHALT PAVEMENT.
6. EXISTING TIMBER LEDGER.
7. EXISTING TIMBER DECKING.
8. NEW RIPRAP PER DETAIL 04 AND SPECIFICATIONS.

01 *FISH CREEK BRIDGE WEST ABUTMENT FOUNDATION REPAIR*
SCALE: NOT TO SCALE
213025-SK1-01

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



NOTES:

1. EXISTING GLULAM BRIDGE BEAM.
2. EXISTING 12x TIMBER FOUNDATION.
3. NEW 8x8 CREOSOTED TIMBERS ORIENTED IN BRIDGE SPAN DIRECTION.
4. EXISTING CONCRETE LEVELING PAD.
5. SHIM VOID WITH SOLID CREOSOTED TIMBER AS REQUIRED.
6. HIGH TIDE LINE (HTL) = ±26.00' NAVD 88 (OBSERVED BY TBC FEBRUARY 27, 2013).
7. NEW FLOWABLE FILL MATERIAL PER SPECIFICATIONS IN EXISTING VOID UNDER LEVELING PAD. FOR BIDDING PURPOSES, ASSUME ±5 CUBIC YARDS (CONTRACATOR TO VERIFY ALL QUANTITIES).
8. NEW CLASS 2 RIPRAP PER MASS SECTION 20.24 2'-0" THICK (MIN.). FOR BIDDING PURPOSES, ASSUME ±40 CUBIC YARDS (CONTRACTOR TO VERIFY ALL QUANTITIES).
9. EXISTING ASPHALT PAVEMENT.
10. EXISTING TIMBER DECKING BEYOND.
11. MINIMUM (3) 1/2" DIA. THREADED SPIKES PER TIMBER.
12. EXISTING GRADE.
13. SHORE BRIDGE BEAMS AS REQUIRED FOR NEW FLOWABLE FILL AND TIMBER INSTALLATION.

02 *FISH CREEK BRIDGE WEST ABUTMENT FOUNDATION REPAIR*
SCALE: NOT TO SCALE
213025-SK1-02

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

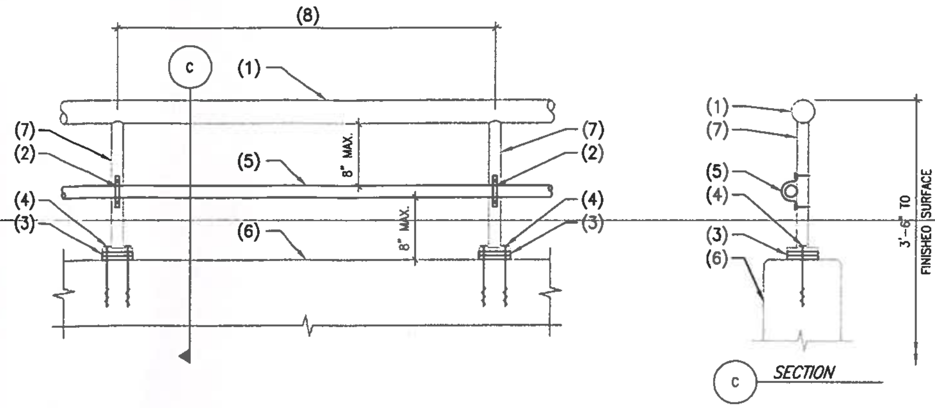
MUNICIPALITY OF ANCHORAGE

TONY KNOWLES COASTAL TRAIL
BRIDGE REHABILITATION
STRUCTURAL DETAILS

SCALE HOR. N/A VER. N/A	DATE: 03/01/2013	GRID: N/A	SHEET 17 of 18
	STATUS:		

NOTES:

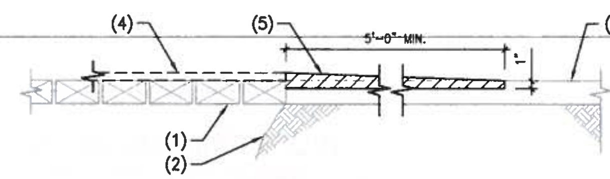
1. EXISTING STEEL PIPE RAILING.
2. NEW GALVANIZED STEEL 2 HOLE PIPE STRAP WITH 1/4" DIA. SELF-DRILLING, SELF-TAPPING SCREWS.
3. NEW 1/2" x 4" x 0'-8 3/8" STEEL SHIMS WITH HOLES FOR LAG BOLTS. GALVANIZE AND FINISH TO MATCH EXISTING RAILINGS. FOR BIDDING PURPOSES, ASSUME (4) SHIMS PER POST. CONTRACTOR 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:

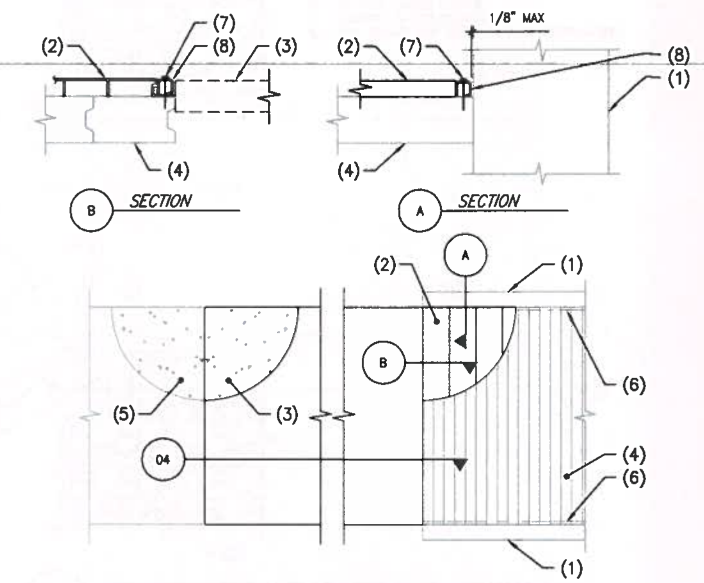
1. EXISTING TIMBER DECKING.
2. EXISTING GRADE AS OCCURS.
3. EXISTING ASPHALT PAVEMENT.
4. NEW FINISHED WEARING SURFACE.
5. NEW ASPHALT RAMP. REMOVE AND REPLACE 1" OF EXISTING ASPHALT BELOW RAMP.



04 NEW ASPHALT RAMP AT NEW FINISHED WEARING SURFACE
SCALE: NOT TO SCALE
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:
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SCHNEIDER STRUCTURAL ENGINEERS

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

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Municipality of Anchorage

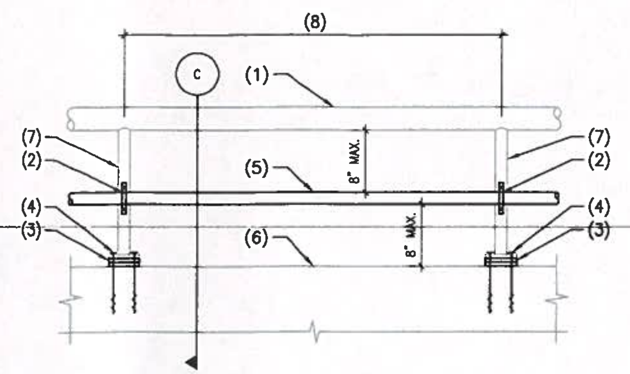
Healthy Parks Healthy People

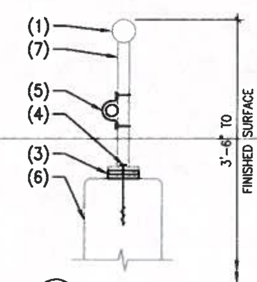
TONY KNOWLES COASTAL TRAIL

BRIDGE REHABILITATION

STRUCTURAL DETAILS

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





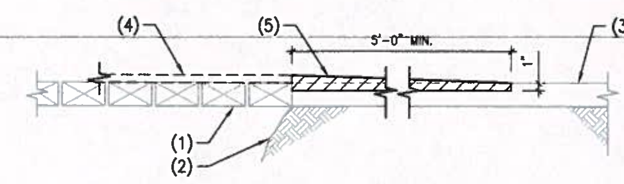
NOTES:

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'-D" O.C.

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

NOTES:

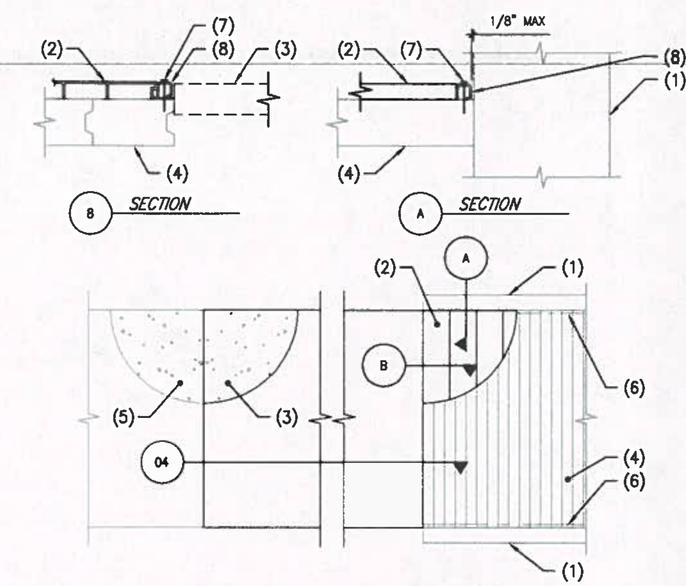
1. EXISTING TIMBER DECKING.
2. EXISTING GRADE AS OCCURS.
3. EXISTING ASPHALT PAVEMENT.
4. NEW FINISHED WEARING SURFACE.
5. NEW ASPHALT RAMP. REMOVE AND REPLACE 1" OF EXISTING ASPHALT BELOW RAMP.



04 *NEW ASPHALT RAMP AT NEW FINISHED WEARING SURFACE*
SCALE: NOT TO SCALE
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:
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SCHNEIDER STRUCTURAL ENGINEERS

STATE OF ALASKA 49TH
DANIEL M. FOLMAR
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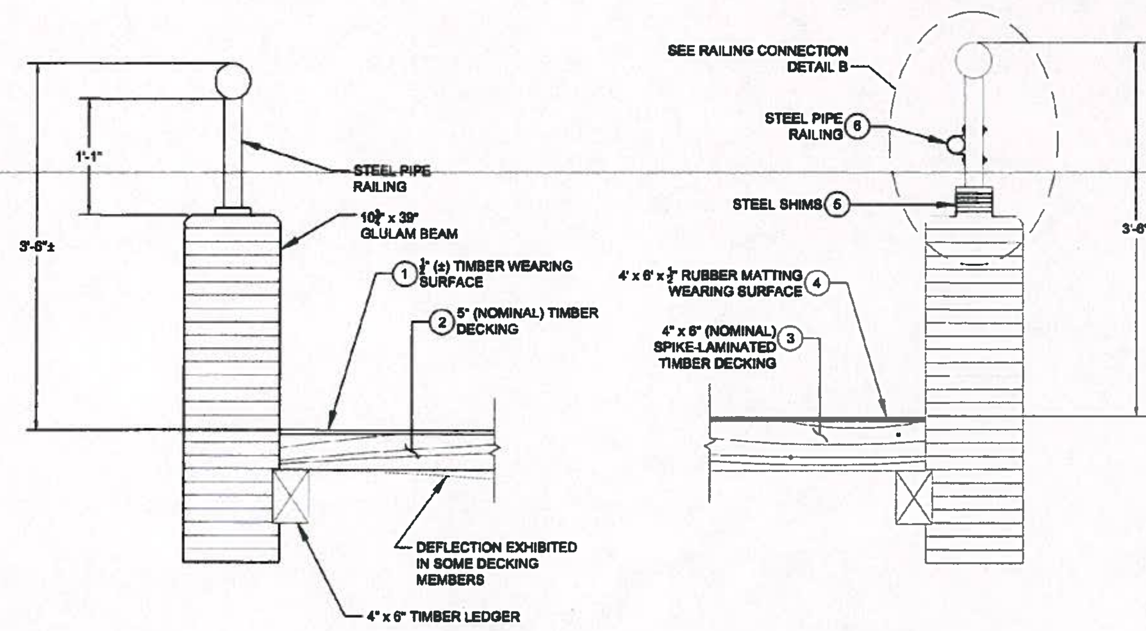
Healthy Parks Healthy People

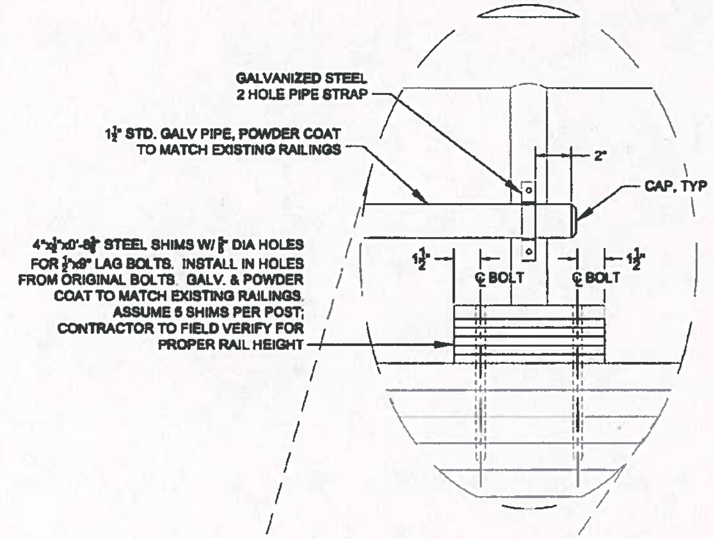
TONY KNOWLES COASTAL TRAIL
BRIDGE REHABILITATION
STRUCTURAL DETAILS

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



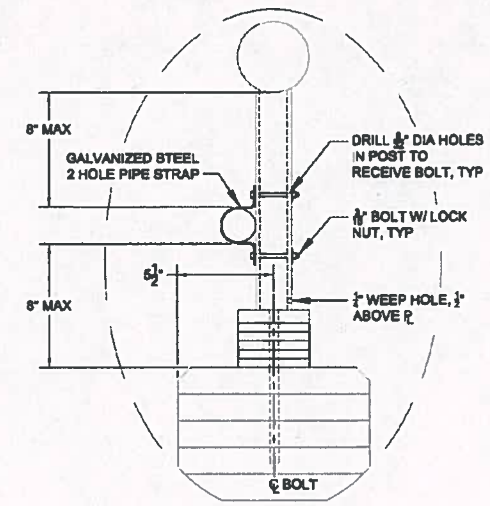
BRIDGE TYPICAL SECTIONS

NOT TO SCALE



RAILING CONNECTION DETAIL A

NOT TO SCALE

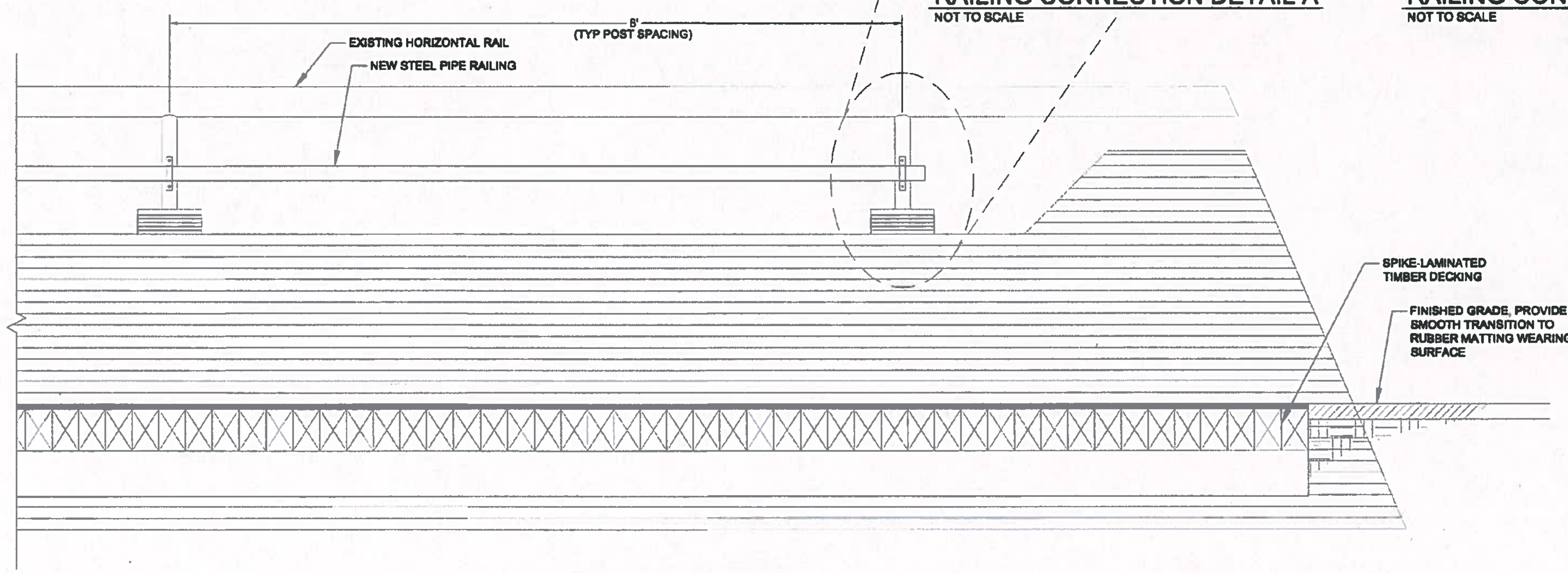


RAILING CONNECTION DETAIL B

NOT TO SCALE

REPAIR ITEMS:

1. REMOVE AND DISPOSE OF EXISTING TIMBER WEARING SURFACE.
2. REMOVE AND DISPOSE OF EXISTING TIMBER DECKING.
3. INSTALL NEW SPIKE-LAMINATED TIMBER DECKING UPON EXISTING 4"x6" LEDGERS.
4. INSTALL RUBBER MATTING WEARING SURFACE.
5. SHIM RAILING TO ACHIEVE UNIFORM 3'-6" RAILING HEIGHT; REPLACE 7" LAG SCREWS WITH 9" LAG SCREWS.
6. INSTALL NEW STEEL PIPE RAILING.



ELEVATION VIEW

NOT TO SCALE

HDR Alaska, Inc.

CONSULTANT

STATE OF ALASKA 49TH

8-6-2012

Department of Parks and Recreation

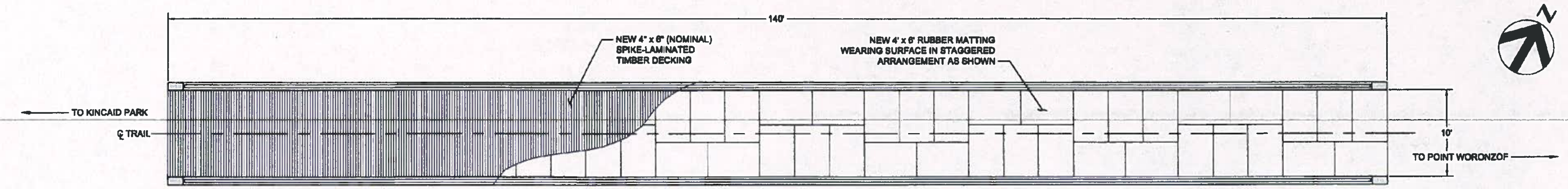
Municipality of Anchorage

Healthy Parks Healthy People

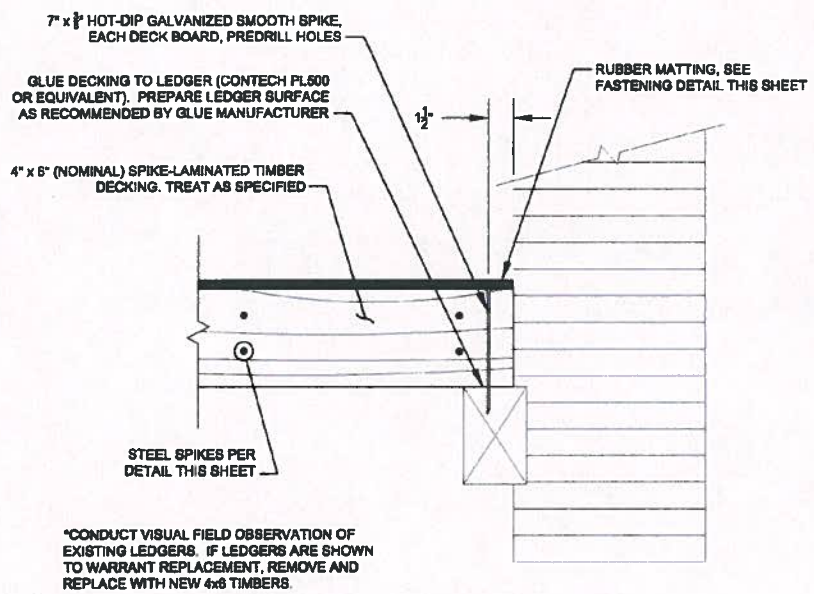
COASTAL TRAIL BRIDGE ASSESSMENT

SISSON LOOP BRIDGE REPAIRS
EXHIBIT 1

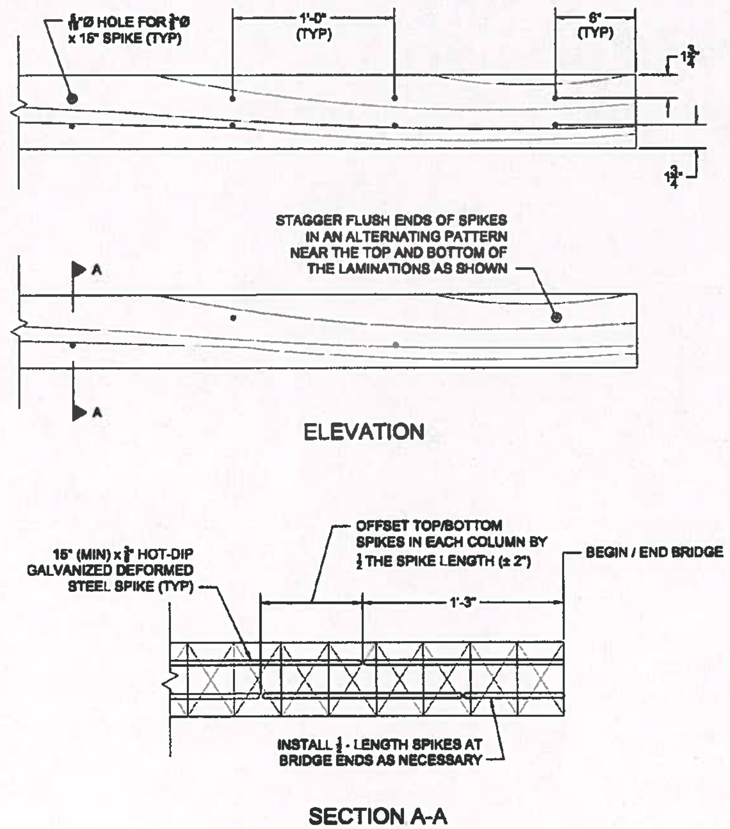
SCALE HOR. VER. | DATE AUG 2012 | STATUS | SHEET 8 of 10



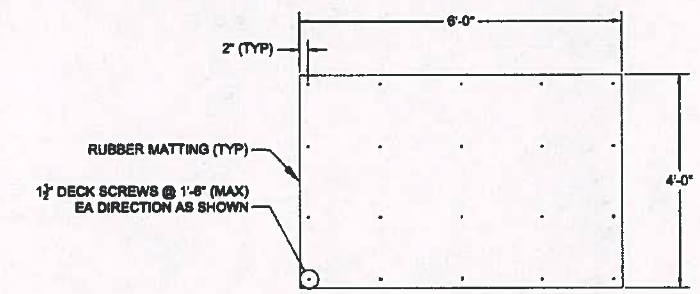
BRIDGE PLAN VIEW
NOT TO SCALE



CONNECTION DETAILS
NOT TO SCALE



SPIKE-LAMINATED DETAILS
NOT TO SCALE



RUBBER MATTING FASTENING DETAIL
NOT TO SCALE

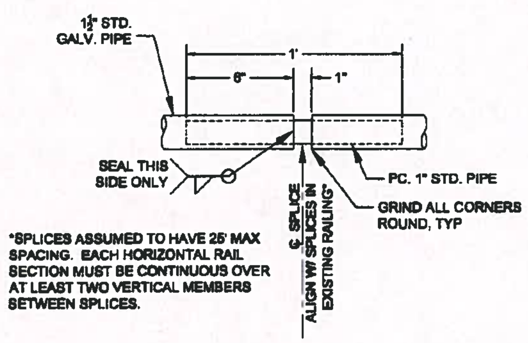
NOTES:

1. DECKING SHALL BE DOUGLAS FIR-LARCH NO. 1 OR BETTER WITH A MAXIMUM DRY MOISTURE CONTENT OF 19%, OR APPROVED EQUIVALENT.

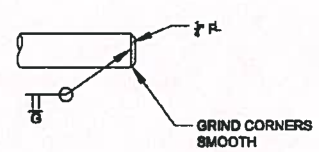
2. DECKING SHALL BE PRESSURE TREATED WITH PENTA IN LIGHT SOL. PER AWPA C-2 AND C-28 TO A MINIMUM RETENTION OF 0.30 LBS/CU.FT. RESULTING FINISH MUST BE CAPABLE OF BEING STAINED OR PAINTED AFTER A SET TIME OF 6 MONTHS.

3. ALL MISCELLANEOUS STEEL AND HARDWARE SHALL BE HOT-DIP GALVANIZED IN ACCORDANCE WITH AASHTO M-111 OR M-232 (ASTM A-123 OR A-153). POWDER COAT STEEL PIPE RAILING AND SHIMS TO MATCH EXISTING COASTAL TRAIL RAILINGS.

4. RUBBER MATTING SHALL BE 4 FT x 6 FT x ½ IN RUBBER HORSE STALL MAT.



RAIL SPLICE DETAIL
NOT TO SCALE



RAIL CAP DETAIL
NOT TO SCALE

HDR Alaska, Inc.
CONSULTANT

Department of Parks and Recreation

Municipality of Anchorage

Healthy Parks Healthy People

8-6-2012

COASTAL TRAIL BRIDGE ASSESSMENT

SISSON LOOP BRIDGE REPAIRS
EXHIBIT 2

SCALE HOR. VER. | DATE AUG 2012 | STATUS | SHEET 9 of 10

Appendix B – Calculations

Dead Loads

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

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

$0.146(2)(70) = 20.4 \text{ k}$

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

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

$0.0156(10)(70) = 10.9 \text{ k}$

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

Guardrail : w/ connections, 11 lb/ft

$(11)(70)(2) = 1.5 \text{ K}$

Total Original DL = $20.4 + 10.9 + 0.9 + 1.5 = 33.7 \text{ K}$ (one span)

Fiberglass overlay

Strongwell Strongdek wt = 2.58 psf $(2.58)(10)(70) = 1.8 \text{ k}$

Additional Guardrail

$(3)(70)(2) = 0.5 \text{ K}$

Total Rehab Items $= 1.8 + 0.5 = 2.3 \text{ K}$

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

Uniform wt $= \frac{36 \text{ K}}{(10 \text{ ft})(70 \text{ ft})} = 51.4 \text{ psf}$ 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. (one span)

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

Vehicle at Failure For North Lagoon Bridge

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

$F_{bxo} = 2.2$ ksi

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

$G_o = 0.5$

$F_{vo} = .265$ 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)

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

$$E = E_o C_m C_i$$

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

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

$M_r = \phi M_N$

$M_n = F_b S C_L$

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

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

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

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

Cv is lesser, therefore Cv controls

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

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

$V_r = \phi V_n$

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

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

Glulam beam check - bending

- uniform live load

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

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

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

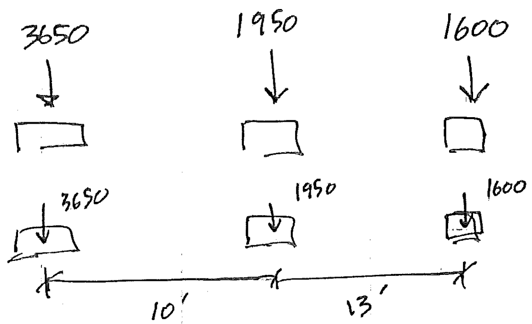
Live load Limit at 1.0

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

$\left(\frac{1000}{12}\right)4631 = \frac{1.75\,w(70)^2(5)}{8} \Rightarrow$ $w = 72psf$ 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, 1 ft from edge, 1.2 multiplier for one side

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

Add DL $V_u = (52psf)\left(\frac{10ft}{2}\right)\left(\frac{70ft}{2}\right)(1.25) = 11.4K + 13.2K = 24.6K$ $\frac{24.6}{130} = 0.19$ ok
$M_u = 2389 + 2505 = 4894 \text{ in-k}$ $D/C = \frac{4894}{7020} = 0.70$ ok

$\boxed{\text{Main beams ok for vehicle load but limited to uniform live load of 72psf}}$

Decking

5in wide x 3¾ deep

DF Decking

$F_{bxo} = 1.45$ ksi (sing)
1.650 ksi (rep)

E 1.70 $\times 10^3$ ksi

$F_{vo} = 0.118$ 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)

$C_r = 1.0$

$C_d = 1.0$

$C_\lambda = 0.8$ (Strength I)

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

$E = E_o C_M C_i$

$= 1.7 \times 10^3 (0.90)(1.0) = 1.53 \times 10^3$ ksi

Bending strength:

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

$\phi M_n = 0.85 (3.30 \text{ ksi})(11.7 \text{ in}^3)(1.0) = 32.8$ 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

Project Title
Page Title
W.O. #
Date
By
Checked By

– Check LL on decking

Decking wt = 15.6 + 2.6 = 18.2 psf

Pedestrian LL = 85 psf

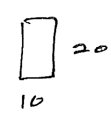
Load Factors

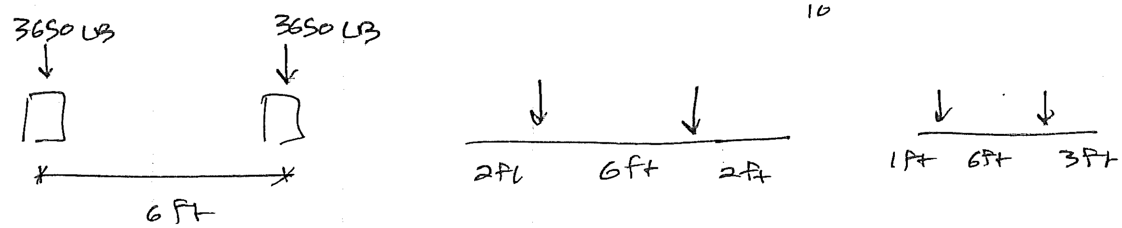
	DC	PL
Str I	1.25	1.75

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

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

Tire load: use 20 in × 10 in contact area.





	2 ft – 6 ft – 2 ft	1 ft – 6 ft – 3 ft
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

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

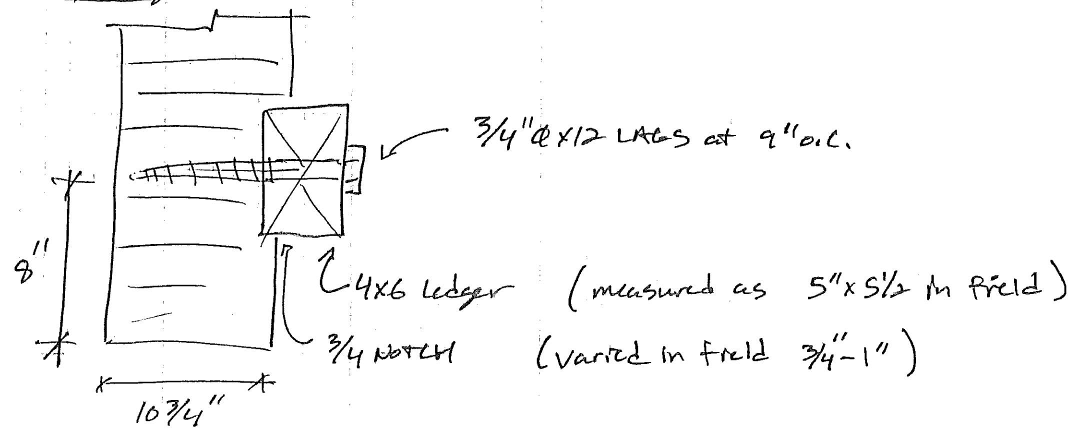
Total $M_u = 184 + 8 \Rightarrow 192$ in-k

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

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

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

Ledger Connection



"Connection design per 2005 NDS"

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

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

$C_m = 0.7$

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

$\phi_z = 0.65$

$\lambda = 0.8$ (occupancy)

3/4"φ LAG, 3½ side member, 0.50 = G DF $Z_\perp = 520$ lb NDS Table 11J

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

equivalent uniform load $= 628\left(\frac{12}{9}\right)\left(\frac{1}{5ft}\right) = 167$ 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} = $ 91 psf If only consider lags

— Consider only lags for tire load

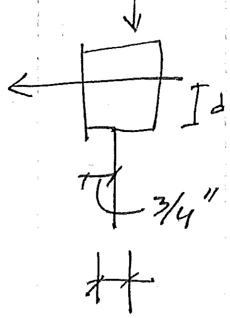
30" length => say 4 lags engaged

(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

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



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

$C_m = 0.67$; $C_t = 1$; $C_i = 1$

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

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

$\phi_c = 0.90$

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

$\lambda = 0.8$

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

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

Moment resisted by lag

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

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

$W' = W \times C_m \; \cancel{C_t} \; \cancel{C_{eg}} \; \cancel{C_{tn}} \; K_F \; \phi_z \; \lambda = 513(5.5)(0.7)(3.32)(0.65)(0.8) = 3410\,lb$

$C_m = 0.7$; $C_t = 1.0$; $C_{eg} = 1.0$; $C_{tn} = 1.0$; $K_F = 3.32$; $\phi_z = 0.65$; $\lambda = 0.8$

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

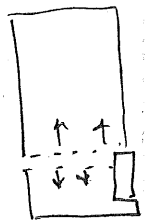
If consider direct bearing, ledger failure probably not a limit state

By loading on bottom section of beam, will induce cross-grain tension stress. → $\boxed{\text{Actual failure mechanism observed}}$ → At North Lagoon

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

$30\text{in}\,(10.75 - 0.75)(18.1) = 5440$ 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]

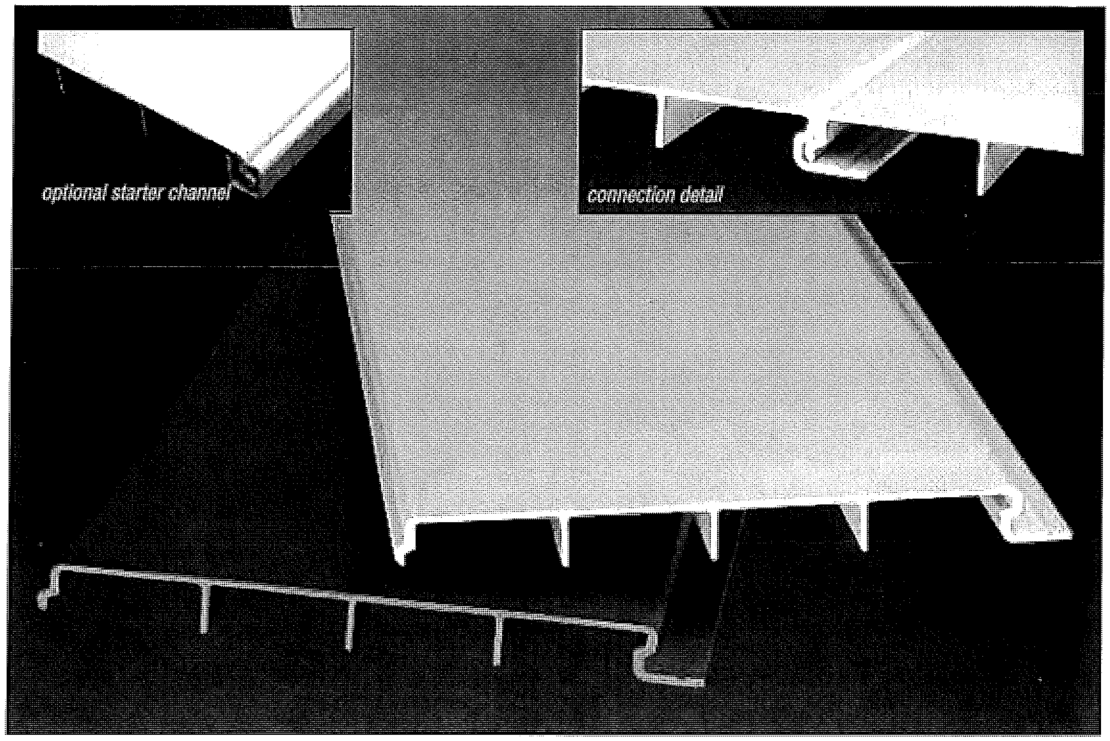
			Bending About X-X Axis (Loaded Perpendicular to Wide Faces of Laminations)					
			Extreme Fiber in Bending[6]		Compression Perpendicular to Grain		Shear Parallel to Grain (Horizontal)[7]	Modulus of Elasticity[8]
			Tension Zone Stressed in Tension	Compression Zone Stressed in Tension	Tension Face	Compression Face		
Combination Symbol	Species[4] Outer/Core	Balanced/ Unbalanced[5]	F_{bx}^{+} (psi)	F_{bx}^{-} (psi)	$F_{c\perp x}$ (psi)		F_{vx} (psi)	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	

STRONGDEK™

FIBERGLASS ARCHITECTURAL DECKING SYSTEM

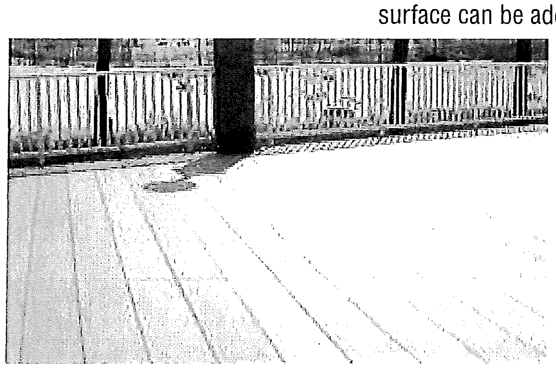


- **Easy to Install**
- **Hidden Fastening System**
- **Rot, Rust and Mildew Resistant**
- **Non-Conductive**
- **Stronger than Wood or Plastic Lumber**
- **Lightweight**

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!"

STRONGWELL®

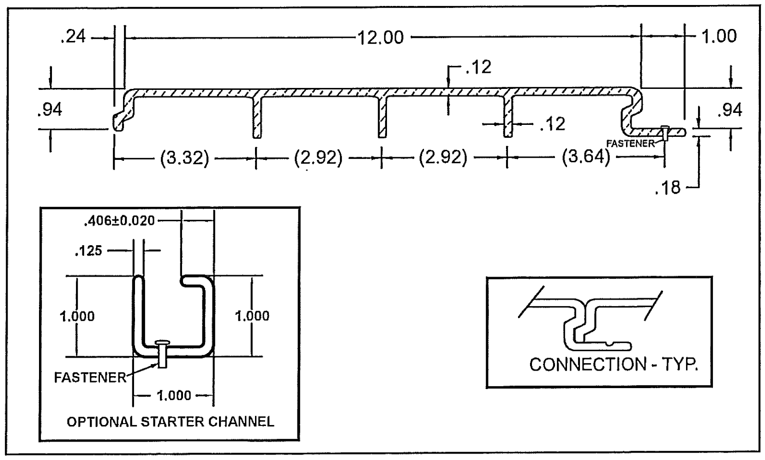
Sizes and Colors

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

ISO-9001:2008 Quality Certified and ISO-14001:2004 Environmentally Certified Manufacturing Plants

BRISTOL LOCATION
400 Commonwealth Ave., P.O. Box 580, Bristol, VA 24203-0580 USA
(276)645-8000 FAX (276)645-8132

CHATFIELD LOCATION
1610 Highway 52 South, Chatfield, MN 55923-9799 USA
(507)867-3479 FAX (507)867-4031

www.strongwell.com

ST1013

Dead Loads

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

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

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

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 $\quad \frac{19.25}{144}(0.050)(70)(2) = 0.9\,k$

Guardrail: w/ connections, 11 lb/ft

$(11)(70)(2) = 1.5\,K$

Total Original DL = $20.4 + 10.9 + 0.9 + 1.5 = 33.7\,K$ (one span)

New 4x6 Decking: $\frac{5.5}{12}(0.050)(10)(70) = 16.0\,K$ $\quad 16.0 - 10.9 = 5.1\,K$ Additional

Fiberglass overlay

Strongwell Strongdek $\quad wt = 2.58\,psf$ $\quad (2.58)(10)(70) = 1.8\,k$

Additional Guardrail

$(3)(70)(2) = 0.5\,K$

Total Rehab items = $5.1 + 1.8 + 0.5 = 7.4\,K$

Total Current DL = $33.7 + 7.4 = 41.1\,K$ $\qquad$ % increase = 23%

Uniform wt = $\frac{41.1\,K}{(10\,ft)(70\,ft)} = 58.7\,psf$ say 59 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. (one span)

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

Vehicle at Failure For North Lagoon Bridge.

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

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

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

$C_m = 0.80$

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

$C_{fu} = 1.0$

$C_i = 1.0$

$C_d = 1.0$

$C_\lambda = 0.8$ (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

$$M_u = \frac{1.25\left[(59\,psf)(5ft)\right](70ft)^2}{8} + 1.75\frac{(85psf)(5ft)(70ft)^2}{8}$$

$$= 681\ ft\text{-}k = 8177\ in\text{-}k$$

$\frac{8177}{7020} = 1.16$ D/C exceeded if consider full 85psf

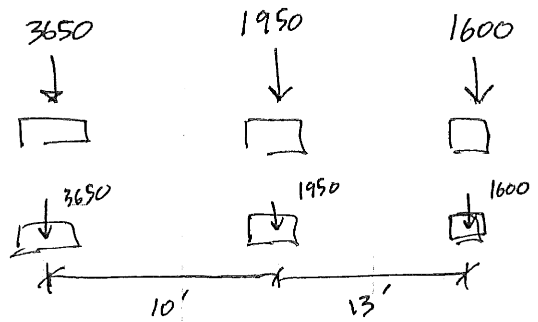
Live load Limit at 1.0

$$7020 - \frac{1.25(59)(5)(70)^2}{8}\left(\frac{12}{1000}\right) = 4310\ in\text{-}k$$

$\left(\frac{1000}{12}\right) 4310 = \frac{1.75\,w(70)^2(5)}{8} \Rightarrow$ $w = 67$ psf allowable live load with D/C = 1.0
within 3% of 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 $V_u = (59psf)\left(\frac{10ft}{2}\right)\left(\frac{70ft}{2}\right)(1.25) = 12.9K + 13.2K = 26.1K$ $\frac{26.1}{130} = 0.20$ ok

$M_u = 2710 + 2505 = 5215\ in\text{-}k$ $D/C = \frac{5215}{7020} = 0.74$ ok

$\boxed{\text{Main beams ok for vehicle load but limited to uniform live load of 67 psf}}$

USKH

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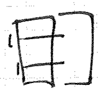
Sisson Decking Rehab



4x6 on edge spiked together

$F_{bxo} = 1.45$ ksi (sing)
1.65 ksi (rep)

$F_{vo} = 0.18$ 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) ← Decking not on flat but close enough.
$C_i = 1.0$
$C_d = 1.0$
$C_\lambda = 0.8$ (Strength I)

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

Bending strength: $S = \frac{(3.5)(5.5)^2}{6} = 17.6\ in^3$

$\phi M_n = 0.85(3.30)(17.6)(1.0) = 49.5$ in-k per plank

Shear: $\phi = 0.75$, $C_{KF} = 3.33$

$F_v = 0.14(3.33)(0.97)(1.0) = 0.465$ ksi

$\phi V_n = 0.75\frac{(0.465)(3.5)(5.5)}{1.5} = 6.0$ k per plank

Project Title ______ Page Title ______
W.O. # ______ Date ______ By ______ Checked By ______

– Check LL on decking

Decking wt = 22.9 + 2.6 = 25.5 psf

Pedestrian LL = 85 psf

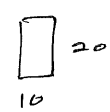
Load Factors

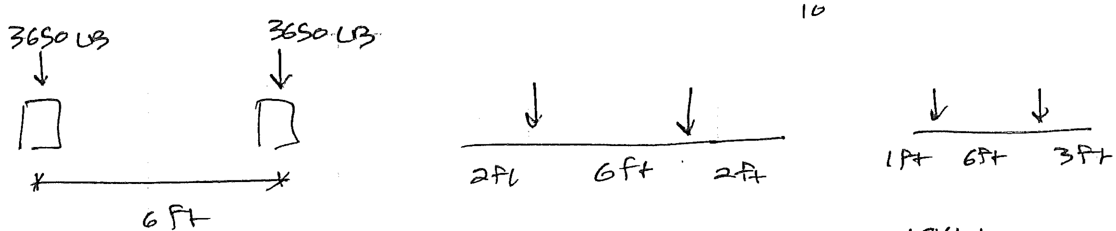
	DC	PL
Str I	1.25	1.75

$\Rightarrow$ load = $25.5(1.25) + 85(1.75) = 181$ psf

$M_u = \dfrac{(181)(\frac{3.5}{12})(10)^2}{8} = 660$ ft-lb $= 7.9$ in-k $\underline{\text{ok}}$ (one plank)

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 8 planks = 28 in

DL only

$22.9(1.25) = 28.6$ psf

$M_u = \dfrac{(28.6)(10)^2}{8} = 358\ \dfrac{\text{ft-lb}}{\text{ft}}$ over 28 in contact area = 835 ft-lb = 10.0 in-k

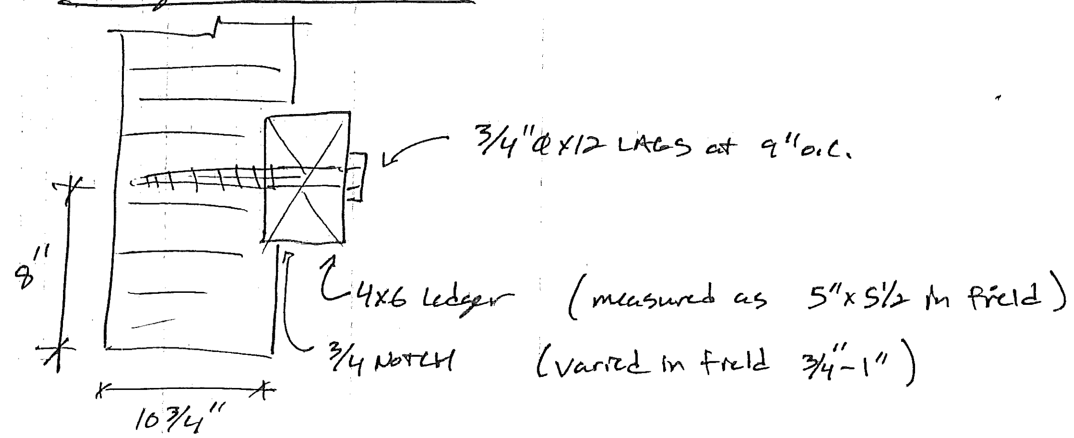
Total $M_u = 184 + 10.0 > 194$ in-k

$\phi M_n = 49.5\,(8 \text{ planks}) = 396$ in-k $= 0.46$ $\underline{\text{ok}}$

Max V_u: $1.25(22.9)\left(\frac{30}{12}\right)\left(\frac{10}{2}\right) + 7.67\text{k} = 8.03$ k

$\phi V_n = 8\,(6.0) = 48.0$ k $\Rightarrow$ $\underline{\text{ok}}$

Ledger Connection



"Connection design per 2005 NDS"

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

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

$C_M = 0.7$

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

$\phi_z = 0.65$

$\lambda = 0.8$ (occupancy)

3/4"Ø LAG, 3½ side member, 0.50 = G DF $Z_\perp = 520\text{ lb}$ NDS Table 11J

$Z'_n = 520(0.7)(3.32)(0.65)(0.8) = 628\text{ LB}$

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

DL $= 22.9\text{ psf}\ (1.2) = 27.5\text{ psf}$

LL $= 85\text{ psf}\ (1.6) = 136\text{ psf}$

use NDS/ASCE load factors per NDS

163 psf

$D/C = \dfrac{163}{167} = 0.98$ ok

Actual LL capacity $= \dfrac{167-28}{1.6} = \underline{87\text{ psf}}$ $\underline{\underline{\text{If only consider lags}}}$

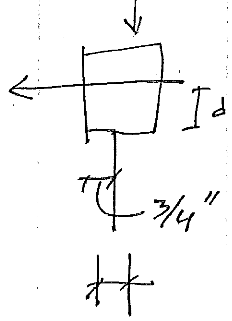
– Consider only lags for tire load

30" length => say 4 lags engaged

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

In North Lagoon Bridge,
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
use similar assumption for Sisson Loop

consider bearing
— more consistant with observed failure mechanism



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

$$F_{c\perp}' = F_{c\perp} \times \underset{0.67}{C_m}\ \overset{1}{\cancel{C_t}}\ \overset{1}{\cancel{C_i}}\ C_b\ K_F\ \phi_c\ \lambda$$

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

$$c = \left(\frac{3/4}{2} - \frac{3\tfrac{1}{2}}{2} = 1\tfrac{3}{8}''\right.$$

$\phi_c = 0.90$

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

$\lambda = 0.8$

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

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

Moment resisted by lag

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

3/4 LAG W = 513 lb/in   5½" embed

$$W' = W \times \underset{0.7}{C_m}\ \overset{1.0}{\cancel{C_t}}\ \overset{1.0}{\cancel{C_{eg}}}\ \overset{1.0}{\cancel{C_{tn}}}\ \underset{3.32}{K_F}\ \underset{0.65}{\phi_z}\ \underset{0.8}{\lambda} = 513(5.5)(0.7)(3.32)(0.65)(0.8) = 3410\,lb$$

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

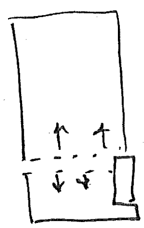
If consider direct bearing, ledger failure probably not a limit state

By loading on bottom section of beam, will induce cross-grain tension stress. → $\boxed{\text{Actual failure mechanism observed}}$ → At north Lagoon

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.

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

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

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]

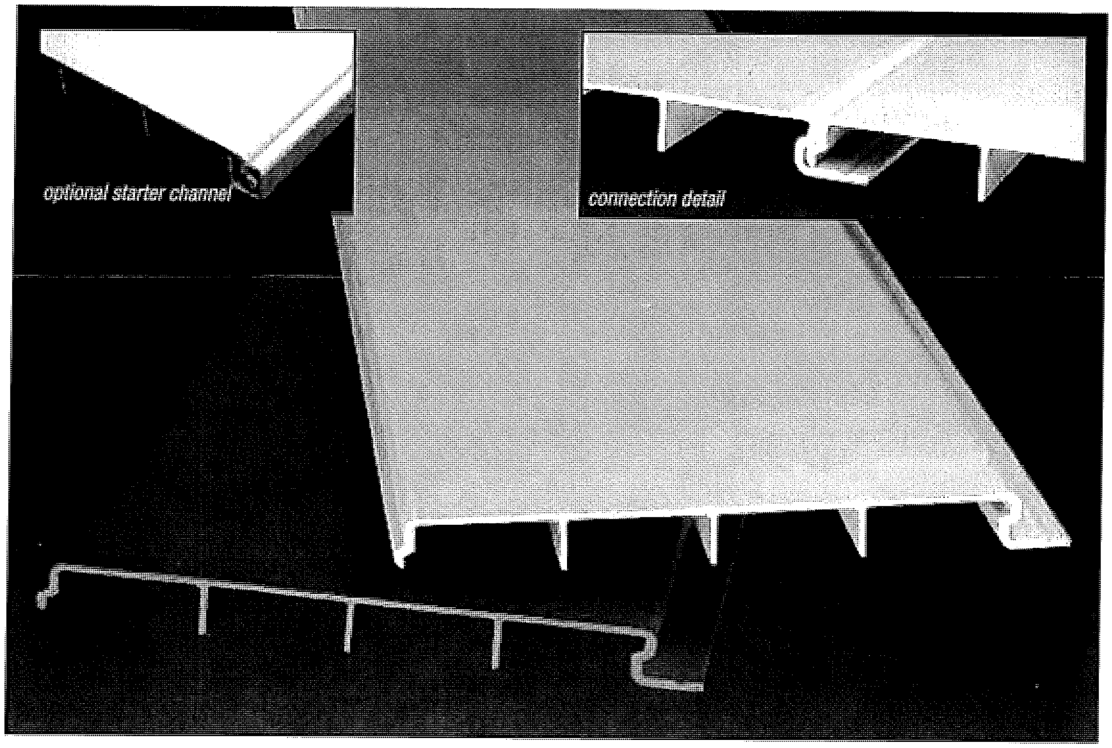
			Bending About X-X Axis (Loaded Perpendicular to Wide Faces of Laminations)					
			Extreme Fiber in Bending[6]		Compression Perpendicular to Grain		Shear Parallel to Grain (Horizontal)[7]	Modulus of Elasticity[8]
			Tension Zone Stressed in Tension	Compression Zone Stressed in Tension	Tension Face	Compression Face		
Combination Symbol	Species[4] Outer/Core	Balanced/ Unbalanced[5]	F_{bx}^{+} (psi)	F_{bx}^{-} (psi)	$F_{c\perp x}$ (psi)		F_{vx} (psi)	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	

STRONGDEK™

FIBERGLASS ARCHITECTURAL DECKING SYSTEM

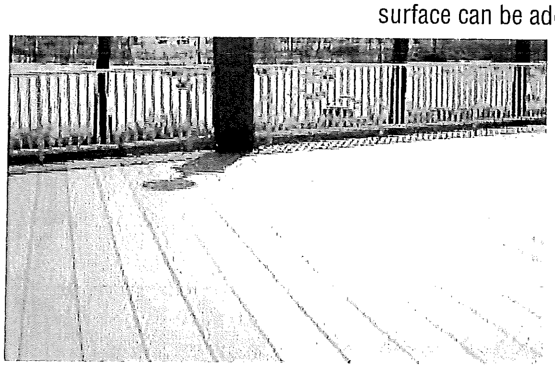


- **Easy to Install**
- **Hidden Fastening System**
- **Rot, Rust and Mildew Resistant**
- **Non-Conductive**
- **Stronger than Wood or Plastic Lumber**
- **Lightweight**

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!"

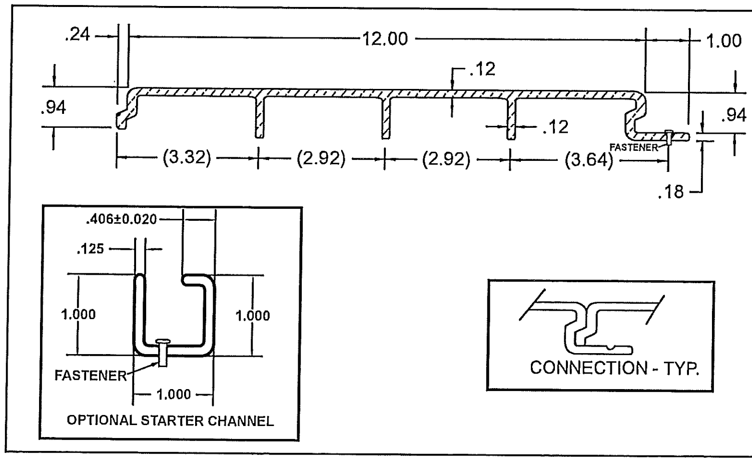
Sizes and Colors

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

ISO-9001:2008 Quality Certified and ISO-14001:2004 Environmentally Certified Manufacturing Plants

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Dead Loads

Glulam Beams: $8\frac{3}{4} \times 36$ $315 \text{ in}^3 = 2.19 \text{ ft}^3/\text{ft}$

$wt = 2.19(0.050 \text{ pcf}) = 0.109 \text{ k/ft}$

$0.109(2)(50) = 10.9 \text{ K}$

Decking: 3" min

$\frac{3}{12}(0.050) = 0.0125 \text{ ksf}$

$0.0125(10)(50) = 6.3 \text{ K}$

Ledger: 4x4 $\frac{12.25}{144}(0.050)(50)(2) =$

Strongback: $4.1(50) = 0.2 \text{ K}$

Guardrail: w/ connections, 11 lb/ft

$(11)(50)(2) = 1.1 \text{ K}$

Total DL (per span) $= 10.9 + 6.3 + 0.2 + 1.1 = 18.5 \text{ K}$

Fiberglass overlay
strongwell strongdek $wt = 2.58 \text{ psf}$ $2.58(10)(50) = 1.3 \text{ K}$

Additional Guardrail

$(3)(50)(2) = 0.3 \text{ K}$

total Rehab Items $= 1.3 + 0.3 = 1.6 \text{ K}$

Total DL current $= 18.5 + 1.6 = 20.1 \text{ K}$ % increase = 8.7%

only Fish Creek but doesn't make significant difference in result for South Lagoon "Bridges"

uniform $wt = \frac{20.1}{(10)(50)} = 40.2$ say 41 psf

– South Lagoon Boardwalk "Bridge" Sections
Project Title – Fish Creek Bridge Analysis Page Title

W.O. # Date By Checked By

Live Loads

Uniform = 85 psf

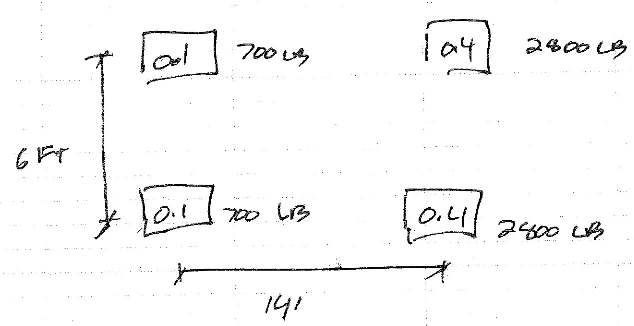
Vehicle: "Street Sweeper 7000 GVW"

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.

consider one span, $A = (10)(50) = 500\text{sf}$

$$w = 85\left(0.25 + \frac{15}{\sqrt{A}}\right) = 85\left(0.25 + \frac{15}{\sqrt{500}}\right) = 78\text{psf}$$

Vehicle



Span = 50' width = 10'

Main Beams:

$8\frac{3}{4} \times 36$ GLB 22F-V8 DF/DF

Reference Design Values

$F_{bxo} = 2.2\,ksi$

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

$G_o = 0.5$

$F_{vo} = 0.265\,ksi$

$$F_b = F_{bo} C_{KF}\, C_m\, (C_F \text{ or } C_v)\, C_{fu}\, C_i\, C_d\, C_\chi$$

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

$C_m = 0.8$

$$C_v = \left[\left(\frac{12}{36}\right)\left(\frac{5.125}{8.75}\right)\left(\frac{21}{50}\right)\right]^{0.10} \leq 1.0 \quad = 0.779$$

$C_{fu} = 1.0$

$C_i = 1.0$

$C_d = 1.0$

$C_\chi = 0.8$ (consider only strength I limit state)

$$\Rightarrow F_b = 2.2(2.94)(0.80)(0.779)(1.0)(1.0)(1.0)(0.8) = 3.22\,ksi$$

$$E = E_o\, C_m\, C_i$$

$$= 1.7 \times 10^3 (0.833)(1.0) = 1.42 \times 10^3\,ksi$$

$$F_v = 0.265(3.33)(0.875)(1.0)(0.80) = 0.618\,ksi$$

$M_r = \phi M_n$

$M_n = F_b S C_L$

$R_b = \sqrt{\frac{50 \times 12 \times 36}{(8.75)^2}} \leq 50 = 16.8$

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

$A = \frac{5.53}{3.22} = 1.72$

$C_L = \frac{1+1.72}{1.9} - \sqrt{\frac{(1+1.72)^2}{3.61} - \frac{1.72}{0.95}} = 0.94$

C_V is lesser, therefore C_V controls

$S = \frac{bd^2}{6} = \frac{(8.75)(36)^2}{6} = 1890 \text{ in}^3$

$\phi M_n = 0.85(3.22 \text{ ksi})(1890 \text{ in}^3)(1.0) = 5170 \text{ in-k}$

$V_r = \phi V_n$

$V_n = \frac{F_v b d}{1.5} = \frac{0.618(8.75)(36)}{1.5} = 130 k$

$V_r = 0.75(130k) = 97 k$

Glulam beam check - bending

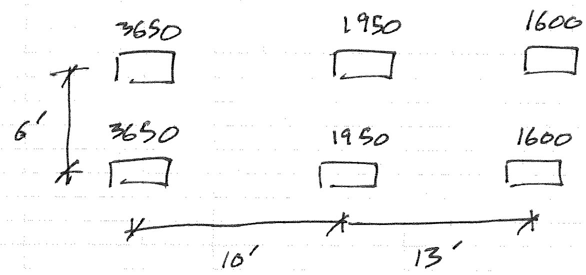
- uniform Live load

$$M_u = 1.25\frac{[(41psf)(5ft)](50ft)^2}{8} + 1.75\frac{(85psf)(5ft)(50ft)^2}{8}$$

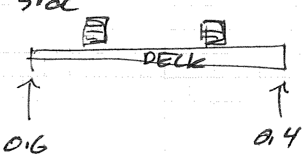
$$= 313\ ft\text{-}k = 3750\ in\text{-}k$$

$$\frac{3750}{5170} = 0.73\ D/C$$ ok with no LL reduction

- consider moment with truck/chipper



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



$$V_{u\ max\ LL} = 5.9(1.2)(1.75) = 12.4\ k$$

$$M_{u\ max\ LL} = 64.0(1.2)(1.75) = 134\ ft\text{-}k = 1610\ in\text{-}k$$

Add DL $V_u = (41psf)\left(\frac{10ft}{2}\right)\left(\frac{50ft}{2}\right)(1.25) = 6.4\ k$

$6.4\ k + 12.4\ k = 18.8\ k$ $\quad \frac{D}{C} = \frac{18.8}{89} = 0.21$ ok

$M_u = 961 + 1610 = 2571\ in\text{-}k$ $\quad \frac{D}{C} = \frac{2571}{5170} = 0.50$ ok

$\boxed{\text{Main beams ok for vehicle load and full uniform load}}$

Decking

7¼ wide x 3" deep DF decking

F_{bvo} = 1.45 ksi (sing)
1.65 ksi (rep)

E 1.70x10³ ksi

F_{vo} = 0.18 ksi

$C_{KF} = 2.94$
$C_m = 0.85$ (sawn lumber)
$C_F = 1.0$ (included in NDS values)
$C_{Fu} = 1.0$ (included in NDS values)
$C_i = 1.0$
$C_d = 1.0$
$C_\lambda = 0.8$ (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: $S = \frac{(7¼)(3)^2}{6} = 10.9 \text{ in}^3$

$\phi M_n = 0.85(3.30 \text{ ksi})(10.9 \text{ in}^3)(1.0) = 30.6$ 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

$\phi V_n = 0.75 \frac{F_v b d}{1.5} = \frac{0.75(0.465)(7¼)(3)}{1.5} = 5.1$ k per plank

– Check LL on decking

Decking wt = 12.5 + 2.6 = 15.1 psf

Pedestrian LL = 85 psf

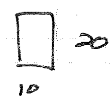
Load Factors

Str I 1.25 DL 1.75 LL

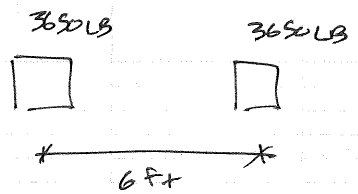
$\Rightarrow$ load = 15.1 (1.25) + 85 (1.75) = 168 psf

$$mu = \frac{(168)\left(\frac{7.25}{12}\right)(10)^2}{8} = 1269 \text{ ft-lb} = 15.2 \text{ in-k} \quad \underline{ok}$$

Tire load: use 20 in x 10 in contact area



chipper Load



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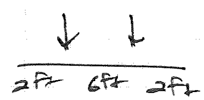


$mu = 153$ in-k

$Vu = 6.39$ K

offset one side

$mu = 184$ in-k

$Vu = 7.67$ K

20 in of contact area would engage at least 5 planks = 36.25 in

DL only: 15.1 (1.25) = 18.9 psf

$$mu = \frac{(18.9)(10)^2}{8} = 236 \frac{\text{ft-lb}}{\text{ft}} \quad \text{over } 36.25 \text{ in} = \begin{matrix} 713 \text{ ft-lb} \\ 8.6 \text{ in-k} \end{matrix}$$

Total $mu = 184 + 9 = 193$ in-k $\qquad \frac{D}{C} = 1.26$ probably marginal for chipper, maybe ok

$\phi mn = 30.6 (5) = 153$ in-k

$$Vu = (18.9)\left(\frac{36}{12}\right)\left(\frac{10}{2}\right) + 7.67 = 7.95 \text{K}$$

$\phi Vn = 5 (5.1 \text{K}) = 25.5 \text{K} \gg \underline{ok}$

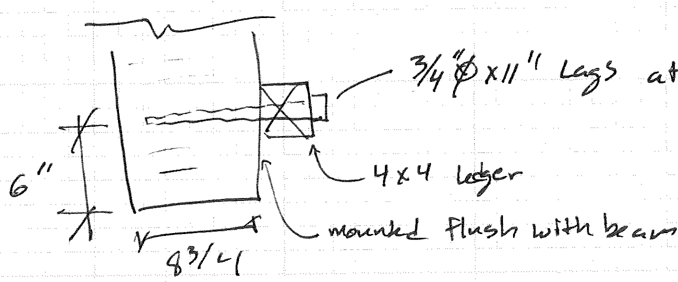
– if scale back to 7000 LB sweeper wt

max wheel load = 40% (7000) = 2800 LB $\quad \frac{2800}{3650} = 0.767$

$0.767 (1.26) = 0.97$ <u>ok</u>

$\boxed{\therefore \text{ Planks ok for design vehicle, marginal for chipper}}$

Ledger Connection



"Connection design per 2005 NDS"

consider only lag in shear:

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

$C_M = 0.7$

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

$\phi_z = 0.65$

$\lambda = 0.8$ (occupancy)

C_Δ : min edge dist loaded edge $= 4D = 4(3/4) = 3\text{in}$

Actual $= \frac{4x}{2} = \frac{3.5}{2} = 1.75\text{in}$

$\Rightarrow C_\Delta = \frac{1.75}{3} = 0.59$

3/4"Ø LAG, 3½" side member, 0.50 = G DF $\quad Z_\perp = 520\text{ LB}$ NDS Table 11J

$Z' = 520(0.7)(0.59)(3.32)(0.65)(0.8) = 371\text{ lb}$

equivalent uniform load $= 371\left(\frac{12}{9}\right)\left(\frac{1}{5\text{ft}}\right) = 99\text{psf} = w_n$

DL $= 15.1\text{psf}\,(1.2) = 18.1$

LL $= 85\text{psf}\,(1.6) = 136$

$\overline{154\text{ psf}}$

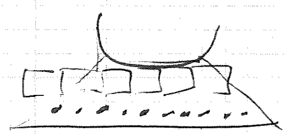
$\frac{D}{C} = \frac{154}{99} = 1.56$ NG

Actual LL capacity : $\frac{99-18}{1.6} = \underline{51\text{psf}}$

- consider tire load

2 in per lag = 371 lb

if consider 5 planks => 36" wide plus additional load distribution on ledger



$\frac{36}{9} + 2 = 6$ lags

$6(371) = 2226$ lb

$\frac{D}{C} = \frac{7670}{2226} = 3.45$ NC

$V_u = 7670$ lb chipper

$V_u = 5880$ lb design vehicle $\frac{5880}{2226} = 2.64$ NC

Ledger connection lacks strength for concentrated vehicle loading
Ledger connection only good for 51 psf LL

TABLE 1

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

			Bending About X-X Axis (Loaded Perpendicular to Wide Faces of Laminations)					
			Extreme Fiber in Bending[6]		Compression Perpendicular to Grain		Shear Parallel to Grain (Horizontal)[7]	Modulus of Elasticity[8]
			Tension Zone Stressed in Tension	Compression Zone Stressed in Tension	Tension Face	Compression Face		
Combination Symbol	Species[4] Outer/Core	Balanced/ Unbalanced[5]	F_{bx}^{+} (psi)	F_{bx}^{-} (psi)	$F_{c\perp x}$ (psi)		F_{vx} (psi)	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	

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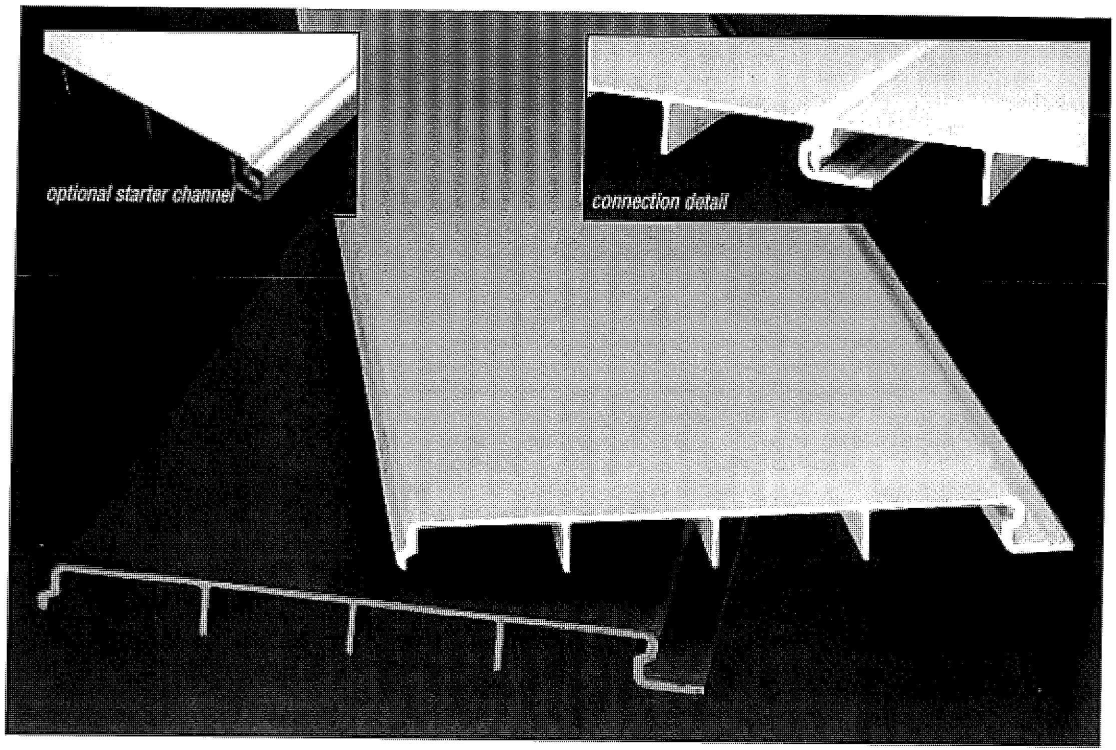
STRONGDEK™

FIBERGLASS ARCHITECTURAL DECKING SYSTEM

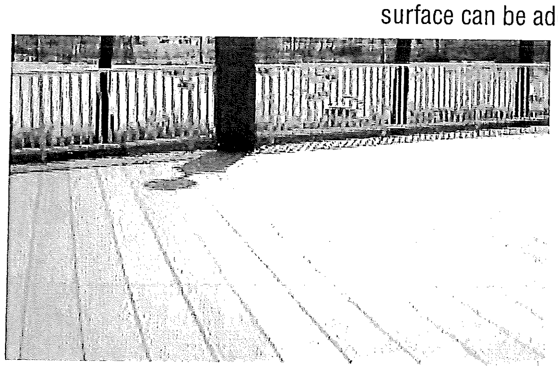


- **Easy to Install**
- **Hidden Fastening System**
- **Rot, Rust and Mildew Resistant**
- **Non-Conductive**
- **Stronger than Wood or Plastic Lumber**
- **Lightweight**

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!"

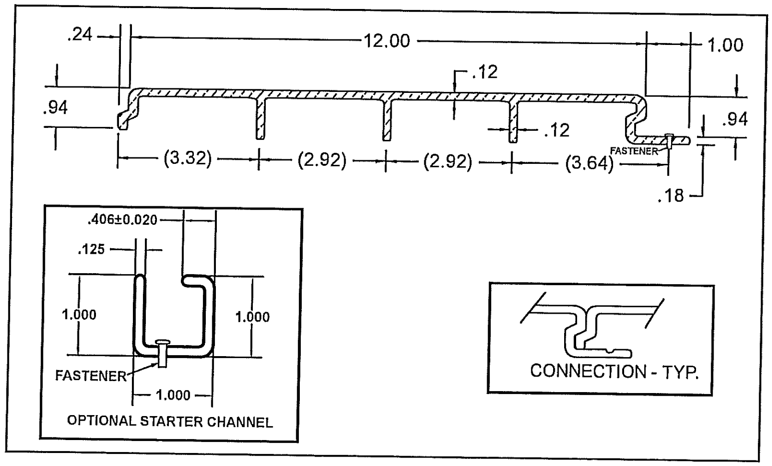
Sizes and Colors

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/ft² (N/m²). For example, a 100 lb. uniform load over 3 ft² 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

ISO-9001:2008 Quality Certified and ISO-14001:2004 Environmentally Certified Manufacturing Plants

BRISTOL LOCATION
400 Commonwealth Ave., P.O. Box 580, Bristol, VA 24203-0580 USA
(276)645-8000 FAX (276)645-8132

CHATFIELD LOCATION
1610 Highway 52 South, Chatfield, MN 55923-9799 USA
(507)867-3479 FAX (507)867-4031

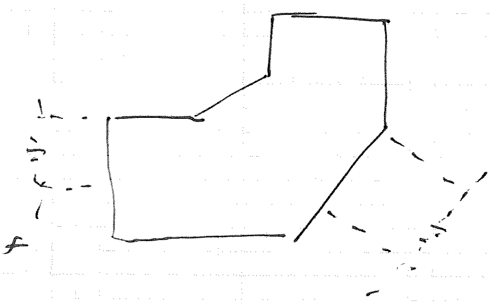
www.strongwell.com

ST1013

Center boardwalk platform

Dead Loads

Deck structure: 2x10 @ 12" o.c. 4.8
3" Decking 12.5
misc 0.7
18.0 psf



Beams:	8 3/4 x 24	32 LF x 73	=	2.4 K
	8 3/4 x 21	58 LF x 64	=	3.7 K
	8 3/4 x 36	56 LF x 109	=	6.1 K
	Ledgers	88 LF x 6.3	=	0.6 K
	Handrail	86 LF x 11	=	1.0 K
				13.8 K

Deck area ≈ 870 sf

⇒ Total DL = 870(18) + 13.8 = 29.5K
equivalent uniform load = 34 psf

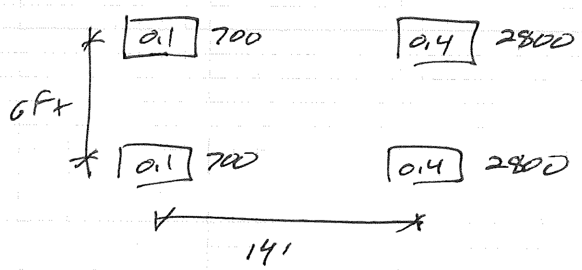
Live Loads

uniform = 85 psf

vehicle: "Street Sweeper 7000 GVW"

Influence area of members appear small
therefore no LL reduction taken

Vehicle



Main GLB's

use $F_b = 3.22$ ksi for 36" GLB

C_v is increased for other sizes

use $C_v = 0.85$

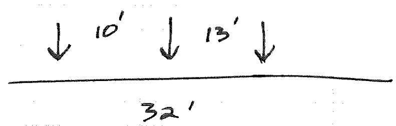
$\Rightarrow F_b = 3.51$ for other sections

size	S	ϕM_n	ϕV_n
8 3/4 x 21	643	1920 in-K	52K
8 3/4 x 24	840	2510 in-k	59K
8 3/4 x 36	1890	5170 in-k	89K

By inspection the 8 3/4x36 GLBs spans very short, therefore strength not a problem.

- 8 3/4 x 24:

consider tires near edge like fish creek.



$V_{u_{LL}} \, max = 5.2(1.2)(1.75) = 10.9K$

$M_{u_{LL}} \, max = 35.5(1.2)(1.75) = 74.6 \text{ ft-k} = 895 \text{ in-k}$

DL : $V_{u_{DL}} = 34 psf \left(\frac{9.33 ft}{2}\right)\left(\frac{32 ft}{2}\right)(1.25) = 3.2k$

$10.9 + 3.2 = 14.1k \quad \frac{D}{C} = \frac{14.1}{59} = 0.24$ ok

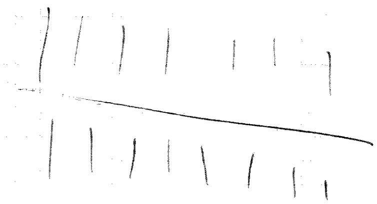
$M_{u_{DL}} = \frac{34\left(\frac{9.33}{2}\right)(32)^2}{8}(1.25) = 25378 \text{ ft-lb} = 305 \text{ in-k}$

$895 + 309 = 1200 \text{ in-k} \quad \frac{D}{C} = \frac{1200}{2510} = 0.48$ ok

uniform load: $M_u = \frac{85\left(\frac{9.33}{2}\right)(32)^2}{8}(1.75) + 305 = 914 \text{ in-k} \Rightarrow$ ok

- 8¾x21 max center span = 22 ft

Trib width = 13 ft



uniform load:

$Mu_{DL} = \frac{34(13)(22)^2}{8}(1.25) = 33426\text{ ft-lb} = 401\text{ in-k}$

$Vu_{DL} = \frac{34(13)(22)}{2}(1.25) = 6.1\text{k}$

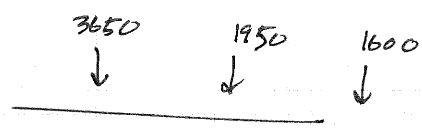
$Mu_{LL} = \frac{85(13)(22)^2}{8}(1.75) = 1404\text{ in-k}$

$Mu_{TL} = 1805\text{ in-k}$ $\quad \frac{D}{C} = \frac{1805}{1920} = 0.94$ ok

$Vu_{LL} = \frac{85(13)(22)}{2}(1.75) = 21.3\text{k}$

$Vu_{TL} = 6.1 + 21.3 = 27.4$ $\quad \frac{D}{C} = \frac{27.4}{52} = 0.53$ ok

vehicle load:



one moving tire load
chipper

$M = 21.79\text{ ft-k}$
$V = 4.5\text{k}$

if consider full wt on one beam - conservative

$Mn = 2(21.79)(1.75)(12) = 915\text{ in-k}$

$Mu_{TL} = 401 + 915 = 1320\text{ in-k}$ $\quad \frac{D}{C} = \frac{1320}{1920} = 0.69$ ok

shear not a problem

$\boxed{\therefore \text{ Main GLBs ok under uniform and vehicle loading}}$

-Joist Capacity

DF No. 1

$F_{bo} = 1.0$ ksi
$F_{vo} = 0.18$ ksi
$E_0 = 1700$ ksi

$F_b = F_{bo} C_{KF} C_m (C_F \text{ or } C_v) C_{Fu} C_i C_d C_x$

$C_{KF} = 2.94$ bending 3.33 shear
$C_m = 0.85$ (sawn lumber)
$C_F = 1.5$ (sawn lumber)
$C_{fu} = 1.0$
$C_I = 1.0$
$C_d = 1.0$
$C_X = 0.8$

$F_b = 1.00(2.94)(0.85)(1.5)(1.0)(1.0)(1.0)(0.8) = 3.00$ ksi

$E = E_0 C_m C_i$
$= 1700(0.90)(1.0) = 1.53\times10^3$ ksi

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

$S = \dfrac{(1.5)(9.25)^2}{8} = 16.0 \text{ in}^3$

$\phi M_n = 0.85(3.00 \text{ ksi})(16.0 \text{ in}^3)(1.0) = 40.8$ in-k

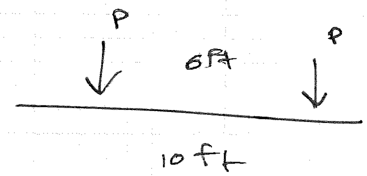
$\phi v_n = 0.75 \dfrac{(0.465)(1.5)(9.25)}{1.5} = 3.2$K

– Joist Loading

Vehicles:

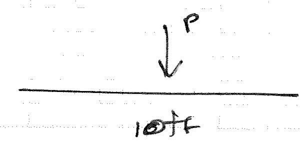
2x10 @ 12'' o.c.

Joist perp to travel direction



$M = 2P$ (FT-K)
$V = P$ (K)

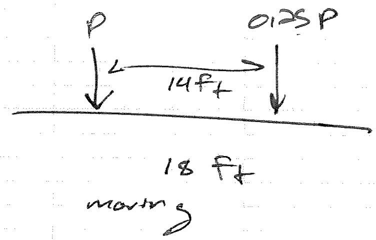
$\frac{M_u}{\lambda} = \frac{1.25(18.0)(18ft)^2}{8} = 911$ ft-lb $= 10.9$ in-k



$M = 2.5P$ (FT-K)
$V = 0.5P$ (K)

$\frac{M_u}{\lambda} = \frac{1.25(18.0)(10ft)^2}{8} = 281$ ft-lb $= 3.4$ in-k

Joist para to travel direction



$M_{max} = 4.5P$ (FT-K)
$V_{max} = 1.10P$ (K)

– uniform load:

18ft span $M_u = \frac{1.25(18.0)(18ft)^2}{8} + \frac{1.75(85)(18)^2}{8} = 6936$ ft-lb $= 83.2$ in-k NC

$\frac{D}{C} = \frac{83.2}{40.8} = 2.04$

10ft span $M_u = \left(\frac{10}{18}\right)^2 83.2 = 25.7$ in-k OK

Load limit

$40.8 - \frac{1.25(18)(18)^2}{8}\left(\frac{12}{1000}\right) = 29.9$ in-k

$\Rightarrow 35$ psf LL

$V_u = \frac{1.25(18.0)(18)}{2} + \frac{1.75(85)(18)}{2} = 1.5K$ OK For either span

– Concentrated load:

Decking distributed load 20 in + 2 planks

20in + 2(7) = 34in load distributed over 3 joist

18ft span area sweeper: $P = 0.4(7000) = 2800$ $M_{max\,LL} = 1.75(4.5)(2800)\frac{12}{1000} = 265$ in-k

over 3 joist $= \frac{265}{3} = 88$ in-k $88.0 + 10.9 = 98.9$ in-k $\frac{98.9}{40.8} = 2.42$ NC

Chipper will be $\frac{3650}{2800}$ more $= 1.31$ $\frac{D}{C} = 3.30$ NC

10ft span area

$M_{max\,LL} = 1.75(2.5)(2800)\frac{12}{1000} = 147$ in-k

over 3 joists $= \frac{147}{3} = 49$ in-k $\quad 49.0 + 3.4 = 52.4$ in-k

$\frac{D}{C} = \frac{52.4}{40.8} = 1.28$ NC probably ok though.

For chipper $\frac{D}{C} = 1.68$ NC

Joist hanger: Simpson LB210 top flange hanger

allowable load = 915 lb

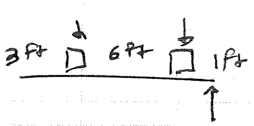
Limit state Design = 1420 lb

3 joists $\Rightarrow \phi R_n = 3(1420) = 4260$ lb

$V_{u\,DL} = (18\,psf)(3\,ft)\left(\frac{10\,ft}{2}\right)(1.25) = 0.34$ k

$V_{u\,LL} = 1.75(1.2)(2800) = 6.1$ k

load near end



$0.34 + 6.1 = 6.4$ k $\quad \frac{D}{C} = \frac{6.4}{4.3} = 1.49$ NC

probably ok though since Simpson hanger value very conservative