SACRAMENTO PLACERVILLE TRANSPORTATION CORRIDOR JOINT POWERS AUTHORITY

BRIDGE INSPECTIONS

Prepared By

DOKKEN ENGINEERING

March 2024

Project Overview

The Sacramento Placerville Transportation Corridor Joint Powers Authority (SPTC-JPA) is a public entity formed in 1991. There are four Member Agencies of the SPTC-JPA: County of El Dorado, City of Folsom, County of Sacramento, and Sacramento Regional Transit. The SPTC-JPA purchased 53 miles of the Placerville Branch right of way from the Southern Pacific Transportation Company's Placerville Branch railroad in 1996 to provide reciprocal use agreements for transportation and transportation preservation uses as desired by the SPTC-JPA's member agencies. The SPTC-JPA is an ongoing agency with the purpose of preserving the corridor for transportation uses and overseeing property management.

This report provides a summary of the condition of all culverts along approximately 33 miles of the corridor from the "Wye" property, at the intersection of Folsom Boulevard and Bidwell Street in the City of Folsom, to Missouri Flat Road in El Dorado County. An aerial view of the corridor is shown in Figures 1a through 1d.

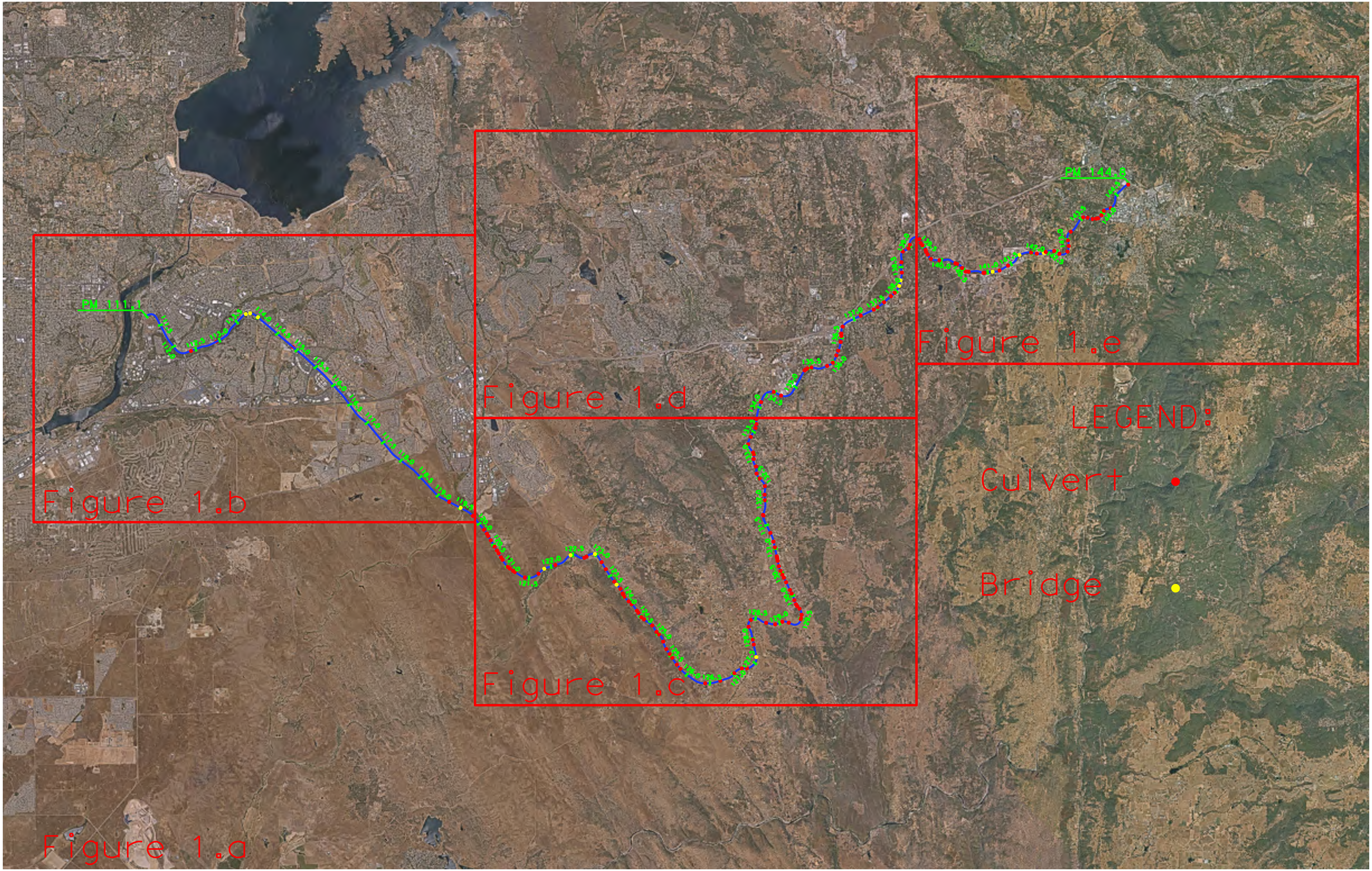
Dokken Engineering staff traversed the entire 33-mile corridor, inspecting all culverts. An inspection log was developed for each culvert, and photographs were taken to document the culvert's condition. A total of 14 bridges were identified. A brief description of each bridge, including location, type, dimensions, condition, and serviceability is provided. Table 1 provides summary information for all bridges.

Observation Summary

Dokken located a total of 14 bridges along the stretch. These types of bridges include a concrete girder bridge, 3 steel girder bridges, and timber girder bridges. The following report will show a calculated bridge rating for each different type of bridge. Each bridge was field measured to calculate the capacity of the bridge using its section properties, calculating beam allowable stresses, and unit weights comparing it to E80 train live loads. Moment and shear capacities were calculated to determine the rating capacity of the bridge. A description and summary of each measured bridge is provided below. The AREMA manual and other historical references were used as a reference to determine the structural capacity of each bridge.

SPTC JPA Bridge Inspection - Structures and Culverts

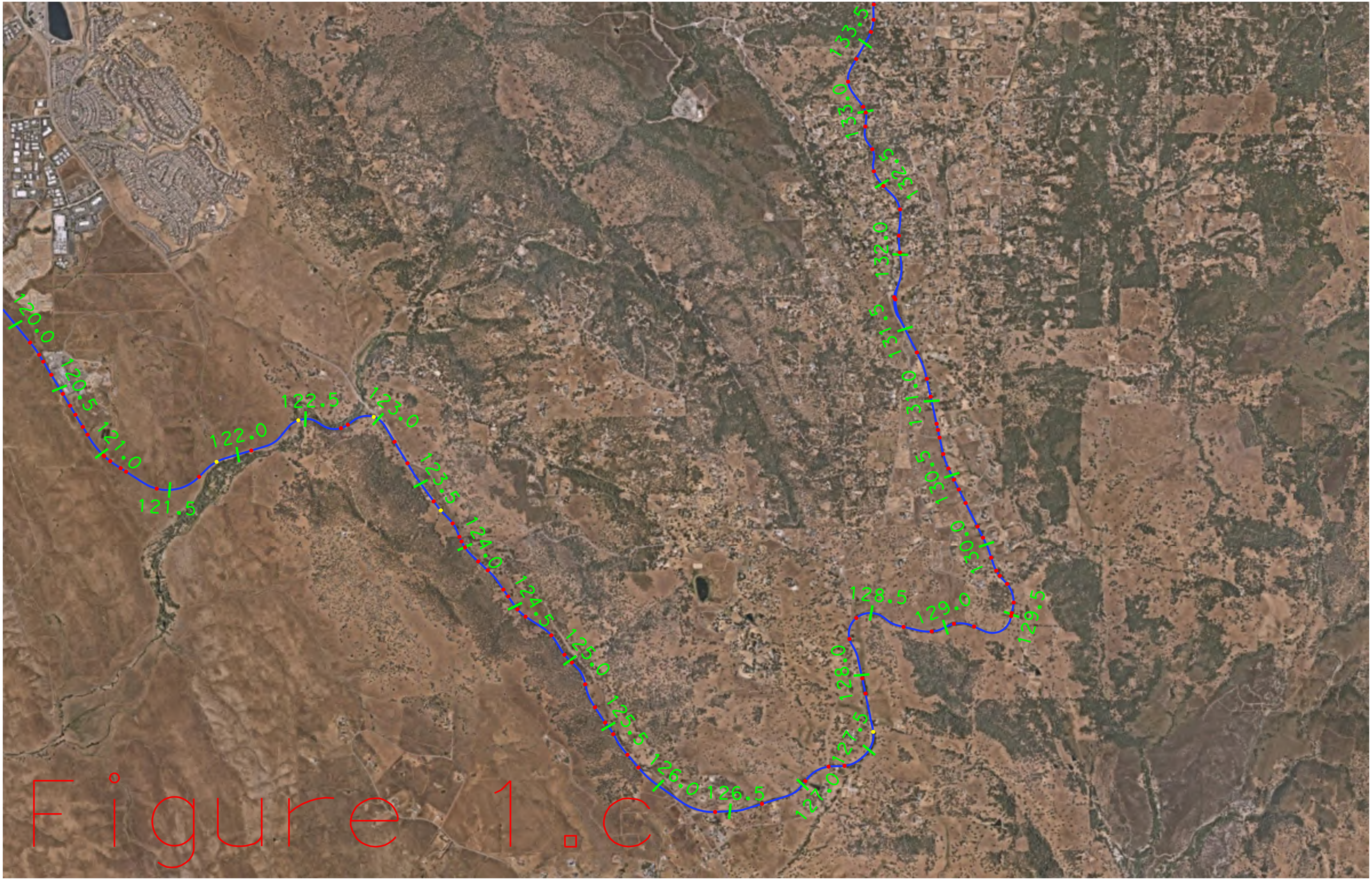
Post-Mile	Coordinates	Bridge Type	Length	No. of spans	Work Recommendation	Estimated Cost ($)	Date Inspected
113.72	38.668881°N 121.146636°W.	Timber Girder	29'-7"	2	Replace tie keepers and cross bracing	$ 5,300.00	11/21/2023
113.81	38.669182°N 121.145042°W	Timber Girder	15'-0"	3	-	-	11/21/2023
114.00	38.668135°N 121.142182°W	Steel Rolled Beam	30'-3"	2	Clean and paint rolled beams	$ 3,400.00	11/21/2023
119.51	38.612753°N 121.069855°W	PC RC Box Girder	128'-0"	5	Clean Expansion Joints, Repair spalling at piers, and replace missing joint seal covers	$ 22,000.00	11/10/2023
121.87	38.594896°N 121.039697°W	Timber Girder	15'-0"	1	Replace broken girder	$ 15,000.00	12/22/2023
122.47	38.598722°N 121.030126°W	Steel Girder	68'-7"	1	Option 1) Clean and paint girders	$ 260,000.00	1/3/2024
122.47	38.598722°N 121.030126°W	Steel Girder	68'-7"	1	Option 2) Spot blast specific corroded areas	$ 52,000.00	1/4/2024
122.97	38.598964°N 121.021353°W	Timber Girder	15'-9"	1	Replace railroad ties	$ 15,900.00	11/10/2023
123.72	38.590106°N 121.013670°W	Timber Girder	5'-0"	1	-	$ -	12/22/2023
127.63	38.568813°N 120.963637°W	Steel Rolled Beam	19'-1"	2	Clean and paint rolled beams. (Remove all lead paint)	$ 5,700.00	11/15/2023
138.11	38.675126°N 120.910521°W	Timber Girder	15'-0"	1	-	-	11/15/2023
138.22	38.676703°N 120.909992°W	Timber Girder	15'-0"	1	-	-	11/15/2023
141.09	38.678903°N 120.876754°W	Timber Girder	50'-10"	4	Replace Bent Cap,add missing column, Replace cross brace	$ 21,200.00	11/15/2023
141.74	38.683618°N 120.866979°W	Timber Girder	15'-2"	1	Replace railroad ties	$ 6,800.00	11/15/2023
142.23	38.684134°N 120.858179°W	Timber Girder	4'-6"	1	-	$ -	11/15/2023
					Total Cost	$ 407,300.00	



PM 111.1
PM 144.8
Figure 1.a
Figure 1.b
Figure 1.c
Figure 1.d
Figure 1.e
LEGEND:
Culvert
Bridge

PM 111.1
111.5
112.0
112.5
113.0
113.5
114.0
114.5
115.0
115.5
116.0
116.5
117.0
117.5
118.0
118.5
119.0
119.5

Figure 1.b



120.0
120.5
121.0
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Figure 1.c

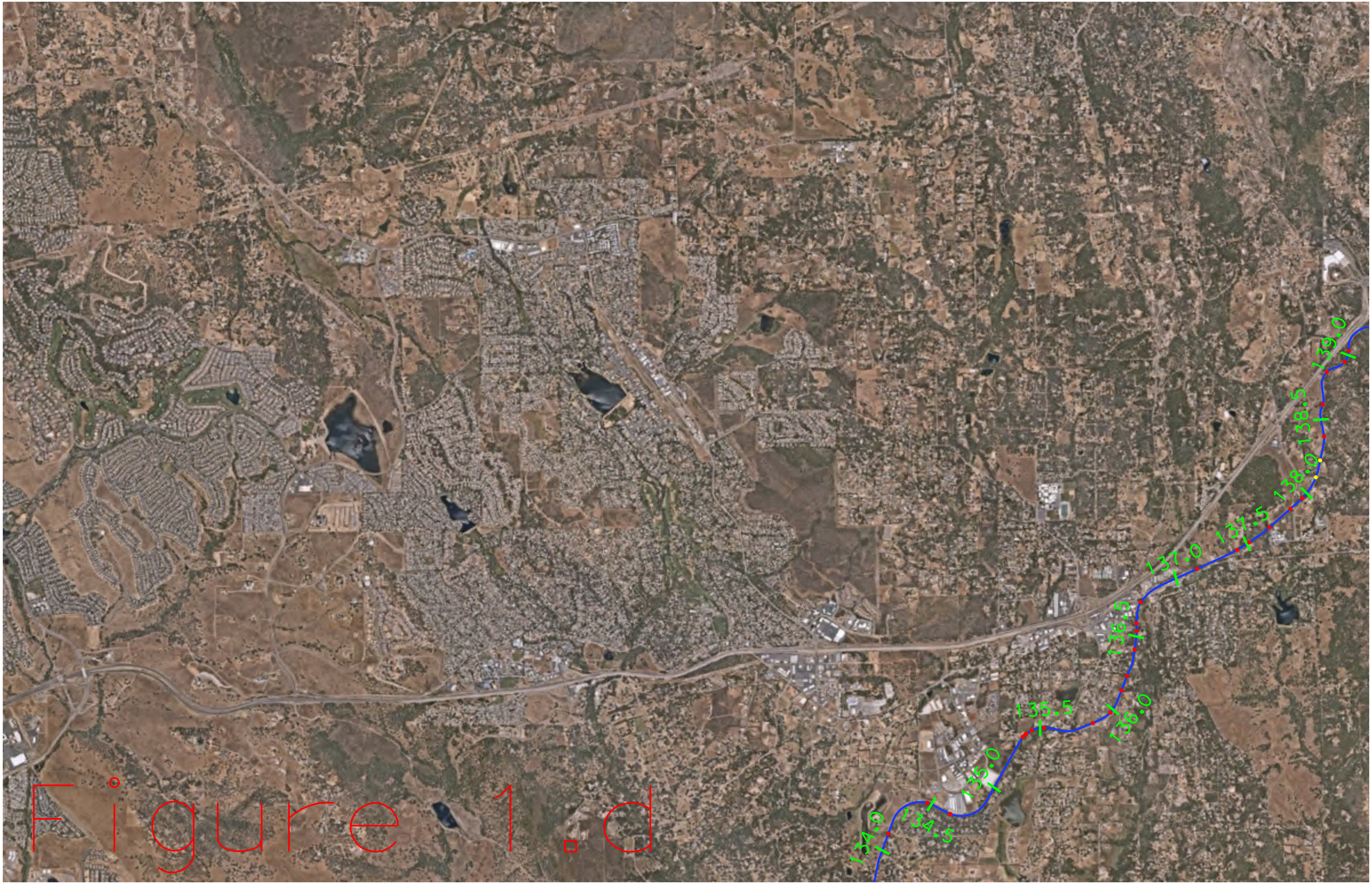
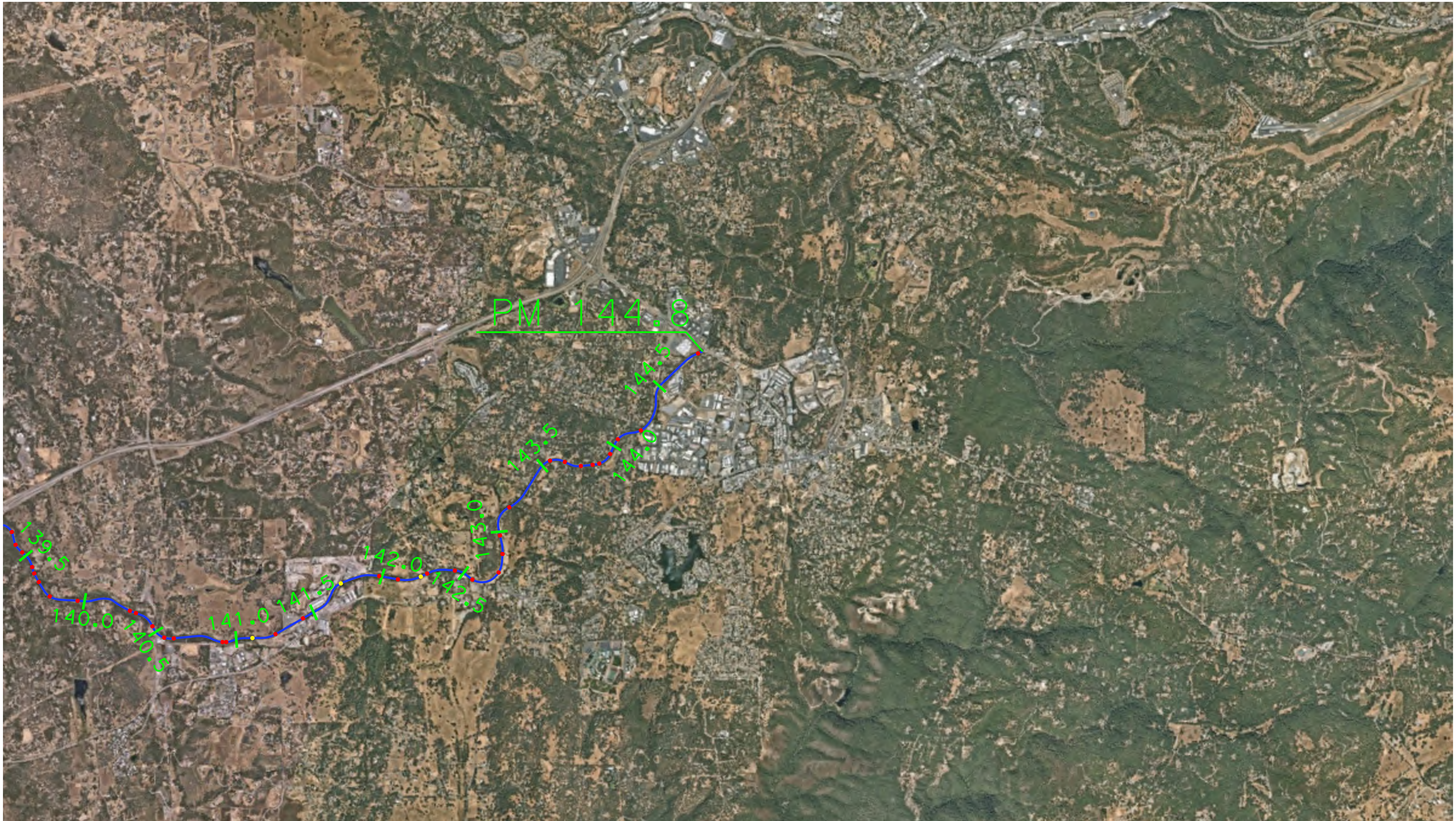


Figure 1.d


PM 144.8
144.5
144.0
143.5
143.0
142.5
142.0
141.5
141.0
140.5
140.0
139.5
Figure 1.e

PM 113.72

This bridge is a 2-span timber girder bridge with a total length of 29’-7”. The width of the bridge was measured to be 14’-2”. The bridge has a superelevation and horizontal curve to the right, with a 5% slope on the railroad ties. The railroad ties on the superstructure are in poor condition as most of the ties have external damages. Railroad tie keepers are in critical condition, as they are highly rotted and damaged. The timber girders are in good shape as no damages were observed. The timber beams sit on a sill beam on a concrete abutment seat. The exposed height of the concrete abutment was measured to be 9’-6”. Spalling was observed underneath the seats of abutment 2(page 69). A large crack in the backwall at abutment 2 was observed and measured to be 1.75” wide with exposed aggregate. Abutment 1 has an exposed footing due to scour from the small channel. The pier on the bridge is also in fair condition. The dimensions of the bent cap were measured to be 14’-2” long, 1’-2” high, and 13¾” wide. No damages were observed at the bent cap. The pier has 5 circular columns that were measured to be 4’-2” in circumference. 4 out of the 5 columns are battered, two of them having a ¾”/ft batter, 1 1½”/ft batter, and a 2⅞”/ft batter. A timber beam sits beneath these columns that share the same dimensions as the bent cap. The height from the bottom timber beam to the bent cap was measured to be 7’-9”. The timber beam sits on 4 small concrete footings that were measured to be 2’-2” and 1’-0”.

Overall, the bridge was rated to be in fair condition and is serviceable. Since each span was measured to be 14’-9½”, this bridge falls into the typical 15'-0" span rating. Please see cost estimate below.

Estimated cost($): $5,300

Photo 1: Top view of bridge looking upstation. Pedestrian path located nearby.

Photo 2: Close up view of track. Shown are the railroad ties, tie keepers, bent cap, and steel rail. Also shown is Abutment 2.

Photo 3: Bridge elevation view, left side of track. Abutment 1 (Right) and Abutment 2 (Left) are shown.

Photo 4: Close up of bridge on the right side. Railroad ties are in poor condition and timber girder are in good condition. Tie keepers are in critical condition. Bolts are exposed.

Photo 3: Pier 2 shown. Pier has 5 circular columns sitting on a timber beam. Beam sits on 4 concrete rectangular footings.

Photo 4: Close up view of Timber beam sitting on a concrete footing at Pier 2.

Photo 5: Timber beam to Abutment 1 connection. Beam sits on a 4"x12" sill beam. Crack shown at backwall at abutment 1.

Photo 6: Pier 2 typical section.

Photo 7: View of Abutment 2. Beams sit on 4"x 12" sill beam. A small amount of spalling shown at Abutment seat. Spalling also seen at headwall.

Photo 8: Closeup of concrete spalling at left side of Abutment 2.

Photo 9: Closeup of concrete spalling at right side of Abutment 2.

Photo 10: View of Abutment 1. No spalling observed at the abutment seat. A small crack can be seen underneath seat.

Photo 11: Closeup of left side of track. Tie keeper shown which is in critical condition. Also shown are Abutment 2 and small retaining wall.

Photo 12: Right side of track. Tie keeper in critical condition.

the material in this chapter is written with regard to typical North American Railroad Timber Trestles and other timber structures mentioned herein with

- spans up to 16 feet;
- Standard Gage Track;
- Norman North American passenger and freight equipment;
- Speeds of freight trains up to 80 mph and passenger trains up to 90 mph;

Chapter 7 - Timber Structure

Part 2 - Design of Wood Railway Bridges and Trestles for Railway Loading

Section 2.4 - Designing for Engineered Wood Products

assume timber used in the bridge is: Douglas-fir Grade No.1 (AREMA page 7-2-24)

Table 7-2-9. allowable unit stresses for stress graded lumber - railroad loading (visual grading)

F_b = 1215 psi $F_{c\perp}$ = 380 psi F_V = 150 psi

table 7-2-3. Applicability of Adjustment Factors				
design properties	temperature	beam Stability	Volumn	
	C_T	C_L	C_V	
$F_b' = F_b$*(...)*(...)*(...)	1.0	1.0	C_V	*bending*
$F_{c\perp}' = F_{c\perp}$*(...)*(...)*(...)	1.0	none	none	*tension*
$F_v' = F_v$*(...)*(...)*(...)	1.0	none	none	*shear*

Note: C_V is [Volumn Factor];

allowable bending stresses of glulam are affected by geometry and size. Generally, larger sizes have a correspondingly lower allowable bending stress than smaller members. To account for this behavior, a volumn factor, C_V, shall be applied. C_V shall not exceed 1.0 and is computed as follows:

$$C_V = \left(\frac{5.125}{b}\right)^p \left(\frac{12}{d}\right)^p \left(\frac{21}{l}\right)^p \leq 1.0$$

where:

32	b = width of bending member in [**inches**]. For multiple piece width layups, b = width of widest piece in the layup, For practical purposes, b ≤ 10.75 in.;
18	d = depth of bending member in [**inches**];
15	l = length of bending member between points of zero moment in [**feet**];
1/10	p = 1/20 for Southern pine and 1/10 for other species; select ==> other species

compute C_V base on above values C_V = 0.82691 -

1 *not glulam*

beam allowable stresses:

bending stress:	F_b'	=	1215	psi
compression stress ⊥ to grain:	$F_{c\perp}'$	=	380	psi
shear stress:	F_v'	=	150	psi

timber beam section properties:

cross sectional area:	A	=	576	in^2
moment of inertia:	I_{xx}	=	15552	in^4

section modulus:	S_{XX}	=	1728	in^3	
radius of gyration:	r_{XX}	=	5.2	in	
unit weight of timber material:	ω_{timber}	=	60	lb/ft^3	
self weight (dead load) of timber beam:	ω_{beam_DL}	=	0.24	kip/ft	
timber tie beam under rail - width:	W_{tie}	=	0.833	ft	
timber tie beam under rail - depth:	D_{tie}	=	0.667	ft	
timber tie beam under rail - length:	L_{tie}	=	4.5	ft	
total number of ties on the beam:	N_{tie}	=	11	each	
self weight (dead load) of tie beams:	ω_{tie_DL}	=	0.12	kip/ft	
sectional area of tie brace (highlight yellow):	A_{brace}	=	0.23	ft^2	
self weight (dead load) of tie brace:	ω_{brace_DL}	=	0.02	kip/ft	
track (rails and fastenings) one side:	ω_{track_DL}	=	0.1	kip/ft	
total distributed dead load on beam:	ω_{total}	=	0.48	kip/ft	<<-- input
maximum moment from self weight DL:	M_{DL}	=	13.3307	kip*ft	<<-- from CSi
maximum moment from train LL:	M_{LL}	=	161.3	kip*ft	<<-- Goal Seek
maximum shear from self weight DL:	V_{DL}	=	4.489	kip	<<-- from CSi
maximum shear from train LL:	V_{LL}	=	81.1	kip	<<-- Goal Seek
maximum compression from self weight DL:	p_{DL}	=	13.35	psi	
maximum compression from train LL:	p_{LL}	=	365.1	psi	<<-- Goal Seek

beam stress demands:

bending stress demand:	F_{b_u}	=	1214	psi
comp. stress ⊥ to grain demand:	$F_{c\perp\_u}$	=	379	psi
shear stress demand:	F_{v_u}	=	149	psi

set stress demand 1 psi less than stress capacity:

$\blacktriangle_b$	=	-1	psi	<<-- Goal Seek
$\blacktriangle_{c\perp}$	=	-1	psi	<<-- Goal Seek
$\blacktriangle_v$	=	-1	psi	<<-- Goal Seek

frame forces from E80 train live load: 80

M_{LL}	=	189.3417	kip*ft
V_{LL}	=	90.4275	kip
p_{LL}	=	390	psi

rating factors for all demand V.S. capacity:

RF_b	=	0.85	N/A
$RF_{c\perp}$	=	0.94	N/A
RF_v	=	0.90	N/A

capacity V.S. demand:

bending:	☺☻☺ this train is good on bridge
compression stress ⊥ to grain:	☺☻☺ this train is good on bridge
shear:	☺☻☺ this train is good on bridge

the maximum train axial load can go through this bridge is:	68	kip	*<<-- Train Rating*
rating factor for E80 train is:	0.85	N/A	*<<-- Rating Factor*

Section 2.3.8 - Impact (2013) R(2022)

The dynamic increment of load due to the effects of speed, roll and track irregularities is not well established for timber structures. Its totaleffect is setimated to be less than the increased strength of timber for the short cumulative duration of loading to which railroad bridges are subjected in service, and is taken into consideration in the derivation of allowable working stresses for design.

- According to above section, it's unnecessary to take impact and rolling effect into consideration.

PM 113.81

This 3-span timber bridge has a length of 45 feet, with each span measuring as 15’-0”. The bridge has two concrete abutments and 2 concrete piers. The bridge has railroad ties, tie keepers, and walkway/post supports. The bridge is on a slight curve due to the alignment of the track. The railroad ties are all in fair condition, none of the ties have external damages or signs of moisture damage. The railroad ties are spaced at 1’-3” with the walkway and post supports installed in between. The walkway supports are also in good condition. The timber girders are semi continuous across the length of the bridge. A total of 8 girders were counted at this location, with 4 girders on left and right side of the bridge, measured at 9’-0” wide from left end to right end. The beams sit on timber sill beams at both the abutments and the piers.

The abutments at were measured to be 17’-0” wide with the seat measuring to be 10’-0”. Both abutments have a 1”/ft batter. The general condition of the abutments was fair as minor cracks at the abutments wall were observed. Both abutments had a width of 1’-8”. The abutments had an average exposed height of 1’-5” with a seat height of 1’-8”. Wingwalls were observed at both abutments as well. The piers at the bridge have typical dimensions at 2’-0” wide, 1’-6” tall, and a length of 15’-1”. Each side of the pier wall has a batter of ⅜”/ft. Overall conditions of the piers are fair since only minor damages were observed such as hairline cracks. A moderate amount of vegetation was observed in between piers 2 and 3. A date of 1920 was observed at Abutment 1, indicating that the substructure is around 104 years old.

Overall, the condition of the bridge is fair since none of the observations made were critical to the bridge’s parameters for rating/serviceability. Since the spans of the bridge are 15’-0”, this falls into the standard bridge rating mentioned earlier for bridge ratings. Due to the parameters and observations, this bridge is serviceable.

Photo 1: Top view of bridge looking upstation.

Photo 2: Closeup image of top view with walkway. Walkway supports, rails, tie keeper, and track shown.

Photo 3: Bridge elevation view, shown is the right side of the track. Pier 2 and Pier 3 shown.

Photo 4: Abutment 1 shown. Large rock seen at the bottom of the left hand side. Abutment is on a superelevation. Walkway supports, Ties, and beams are shown.

Photo 5: Right side of Abutment 1. A date of 1920 is shown indicating when the bridge was built.

Photo 6: View of Abutment 1 seat.

Photo 7: Closeup view of Pier 2. Semi continuous beams meet at this pier. Beams are sitting on sill beams.

Photo 8: Left side of Abutment 2.

Photo 11: View of Pier 3.

Photo 12: Closeup of Pier 3. Concrete spalling observed on the battered wall.

the material in this chapter is written with regard to typical North American Railroad Timber Trestles and other timber structures mentioned herein with

- spans up to 16 feet;
- Standard Gage Track;
- Norman North American passenger and freight equipment;
- Speeds of freight trains up to 80 mph and passenger trains up to 90 mph;

Chapter 7 - Timber Structure

Part 2 - Design of Wood Railway Bridges and Trestles for Railway Loading

Section 2.4 - Designing for Engineered Wood Products

assume timber used in the bridge is: Douglas-fir Grade No.1 (AREMA page 7-2-24)

Table 7-2-9. allowable unit stresses for stress graded lumber - railroad loading (visual grading)

F_b = 1215 psi $F_{c\perp}$ = 380 psi F_V = 150 psi

table 7-2-3. Applicability of Adjustment Factors				
design properties	temperature	beam Stability	Volumn	
	C_T	C_L	C_V	
$F_b' = F_b*(...)*(...)*(...)$	1.0	1.0	C_V	bending
$F_{c\perp}' = F_{c\perp}*(...)*(...)*(...)$	1.0	none	none	tension
$F_v' = F_v*(...)*(...)*(...)$	1.0	none	none	shear

Note: C_V is [Volumn Factor];

allowable bending stresses of glulam are affected by geometry and size. Generally, larger sizes have a correspondingly lower allowable bending stress than smaller members. To account for this behavior, a volumn factor, C_V, shall be applied. C_V shall not exceed 1.0 and is computed as follows:

$$C_V = \left(\frac{5.125}{b}\right)^p \left(\frac{12}{d}\right)^p \left(\frac{21}{l}\right)^p \leq 1.0$$

where:

32	b = width of bending member in [inches]. For multiple piece width layups, b = width of widest piece in the layup, For practical purposes, b ≤ 10.75 in.;
18	d = depth of bending member in [inches];
15	l = length of bending member between points of zero moment in [feet];
1/10	p = 1/20 for Southern pine and 1/10 for other species; select ==> other species

compute C_V base on above values C_V = 0.82691 -

1 not glulam

beam allowable stresses:

bending stress:	F_b'	=	1215	psi
compression stress ⊥ to grain:	$F_{c\perp}'$	=	380	psi
shear stress:	F_v'	=	150	psi

timber beam section properties:

cross sectional area:	A	=	576	in^2
moment of inertia:	I_{xx}	=	15552	in^4

section modulus:	S_{XX}	=	1728	in^3	
radius of gyration:	r_{XX}	=	5.2	in	
unit weight of timber material:	ω_{timber}	=	60	lb/ft^3	
self weight (dead load) of timber beam:	ω_{beam_DL}	=	0.24	kip/ft	
timber tie beam under rail - width:	W_{tie}	=	0.833	ft	
timber tie beam under rail - depth:	D_{tie}	=	0.667	ft	
timber tie beam under rail - length:	L_{tie}	=	4.5	ft	
total number of ties on the beam:	N_{tie}	=	45	each	
self weight (dead load) of tie beams:	ω_{tie_DL}	=	0.16	kip/ft	
sectional area of tie brace (highlight yellow):	A_{brace}	=	0.23	ft^2	
self weight (dead load) of tie brace:	ω_{brace_DL}	=	0.02	kip/ft	
track (rails and fastenings) one side:	ω_{track_DL}	=	0.1	kip/ft	
total distributed dead load on beam:	ω_{total}	=	0.52	kip/ft	<<-- input
maximum moment from self weight DL:	M_{DL}	=	11.649	kip*ft	<<-- from CSi
maximum moment from train LL:	M_{LL}	=	163.1	kip*ft	<<-- by changing
maximum shear from self weight DL:	V_{DL}	=	4.676	kip	<<-- from CSi
maximum shear from train LL:	V_{LL}	=	81.1	kip	<<-- by changing
maximum compression from self weight DL:	p_{DL}	=	13.9	psi	
maximum compression from train LL:	p_{LL}	=	364.2	psi	<<-- by changing
beam stress demands:					
bending stress demand:	F_{b_u}	=	1214	psi	
comp. stress ⊥ to grain demand:	$F_{c\perp\_u}$	=	379	psi	
shear stress demand:	F_{v_u}	=	149	psi	
set stress demand 1 psi less than stress capacity:					
	$\blacktriangle_b$	=	-1	psi	<<-- Goal Seek
	$\blacktriangle_{c\perp}$	=	-1	psi	<<-- Goal Seek
	$\blacktriangle_v$	=	-1	psi	<<-- Goal Seek
frame forces from E80 train live load:			80		
	M_{LL}	=	199.34	kip*ft	
	V_{LL}	=	89.88	kip	
	p_{LL}	=	380	psi	
rating factors for all demand V.S. capacity:					
	RF_b	=	0.82	N/A	
	$RF_{c\perp}$	=	0.96	N/A	
	RF_v	=	0.90	N/A	

capacity V.S. demand:

bending:	☺☻☺ this train is good on bridge
compression stress ⊥ to grain:	☺☻☺ this train is good on bridge
shear:	☺☻☺ this train is good on bridge

the maximum train axial load can go through this bridge is:	65	kip	<<-- *Train Rating*
rating factor for E80 train is:	0.82	N/A	<<-- *Rating Factor*

Section 2.3.8 - Impact (2013) R(2022)

The dynamic increment of load due to the effects of speed, roll and track irregularities is not well established for timber structures. Its totaleffect is setimated to be less than the increased strength of timber for the short cumulative duration of loading to which railroad bridges are subjected in service, and is taken into consideration in the derivation of allowable working stresses for design.

- According to above section, it's unnecessary to take impact and rolling effect into consideration.

PM 114.0

This location has a two span steel girder bridge that crosses over Willow Creek. The total length of this bridge was measured to be 30’-3”, with the two spans being 15’-3” and 15’-0” respectively. The grade of the steel was shown to be ASTM A992. The girders are a W 16x7x45 lbs/ft shape. The girders also have a web diaphragm that is 2” thick, 1‘ deep, and is hollow. Condition of the steel girders are in very good condition. No signs or rust or corrosion was observed. The railroad ties of the bridge are also in very good condition. No signs of damage were observed during inspection. The ties are spaced out at 18 inches. The Substructure consists of 2 concrete abutments and 1 timber pier. The two abutments are 1’-8” wide and are 3 feet tall (exposed height). The condition of the abutments are good. Abutment 1 does have a crack on the left-hand side near the abutment seat. The bent cap is also in fair condition. No signs of damage were observed during inspection. The cross-sectional dimensions of the bent cap are 14” wide by 13” high. Overall, the condition of this bridge upon inspection is good. Per structural evaluation, the bridge is serviceable. The steel girders do have some minor paint chips across the entire span. Dokken recommends repainting the steel girders to prevent rust in the future. Cost estimate for the work is provided below.

Estimated Cost ($): $3,400

Photo 1: Top view of bridge with track, looking up station.

Photo 2: Bridge elevation view, west side of bridge over Willow Creek.

Photo 3: Pier 2 view from west side of bridge

Photo 4: Close up view of steel girder, rail road ties, tie keeper, and rail track. Rail road ties and steel girder (ASTM A992) are in good condition. Tie keeper is relatively new.

Photo 5: Abutment 1 closeup with a diagonal crack on left hand side.

Photo 6: Typical steel girder to timber bent connection (Pier 2)

Photo 7: Steel girder to Abutment connection

Photo 8: Top view of track above Pier 2

allowable normal rating stress shall be based on either the minimum yield strength or the minimum ultimate tensile strength of the material as determined from tests or records. In the absence thereof:

material	F_y (psi)	F_u (psi)
wrought iron	25000	45000
bessemer steel	30000	50000
open-hearth steel	30000	60000
silicon steel	45000	62000
nickle steel	50000	90000

table from AREMA 7.3.3.3 Allowable Stresses for Normal Rating

allowable unit stresses resulting from the loads and forces described in the preceding articles are shown in Table 15-1-11 as supplemented by Table 15-7-2.

table 15-7-2. Supplemental Allowable Stresses for Structure Steel and Fasteners for Use in Normal Rating

Component	allowable stress (psi)	
tension in gross section of floorbeam hangers, including bending, using rivets in end connections		$0.40 * F_y$
shear in hand-driven A 502 grade 1 rivets		11000
shear in power-driven A 502 grade 1 rivets (Note 1)		13500
shear in power-driven A 502 grade 2 rivets (Note 1)		20000
bearing on hand-driven A 502 grade 1 rivets		20000
bearing on power-driven A 502 grade 1 rivets (Note 1)		
in single shear		27000
in double shear		36000
bearing on power-driven A 502 grade 2 rivets, on connected material with a yield point of F_y (Note 1)		
in single shear, the lesser of	40000	$0.75 * F_y$
in double shear, the lesser of	50000	$1.0 * F_y$
shear in A325 bolts		17000
shear in A490 bolts		21000

Note 1: Rivets driven by pneumatically or electrically operated hammers are considered power-driven.

beam allowable stresses:

bending stress:	F_b'	=	50000	psi
shear stress:	F_v'	=	50000	psi

bridge info:

two 15' long spans on total 4 W 16 X 45 steel girders.

W 16 X 45 steel girder section properties:

cross sectional area:	A	=	13.3	in^2
moment of inertia:	I_{XX}	=	586	in^4
section modulus:	S_{XX}	=	72.795	in^3
radius of gyration:	r_{XX}	=	6.6378	in

unit weight of timber material:	ω_{timber}	=	60	lb/ft^3

timber tie beam under rail - width:	W_{tie}	=	0.75	ft	
timber tie beam under rail - depth:	D_{tie}	=	0.5833	ft	
timber tie beam under rail - length:	L_{tie}	=	10	ft	
total number of ties on the beam:	N_{tie}	=	20	each	
self weight (dead load) of tie beams:	ω_{tie_DL}	=	0.18	kip/ft	
sectional area of tie brace:	A_{brace}	=	0.23	ft^2	
self weight (dead load) of tie brace:	ω_{brace_DL}	=	0.02	kip/ft	
track (rails and fastenings) (total 2):	ω_{track_DL}	=	0.2	kip/ft	
total distributed dead load on beam:	ω_{total}	=	0.4	kip/ft	<<-- input

frame forces from E80 train live load:			80	
	M_{LL}	=	162.33	kip*ft
	V_{LL}	=	53.80	kip

AREMA Chapter 15 Steel Structures

Section 1.3.5 Impact Load (2007) R(2021)

c. impact load due to vertical effects, expressed as a percentage of live load applied at each rail, shall be determined by the applicable formula below:

(1) percentage of live load for rolling equipment without hammer blow (freight and passernger cars, and locomotives other than steam):

(a) for span length less than 80 feet: 40-3*L^2/1600

impact	=	39.58	%
addition moment from impact	=	64.25	kip*ft
addition shear from impact	=	21.29	kip

d. impact load dur to rocking effect, RE, is created by the transfer of load from the wheels on one side of a car or locomotive to the other side from periodic lateral rocking of the equipment. RE shall be calculated from loads applied as a vertical force couple, each being 20 percent of the wheel load without impact, acting downward on one rail and upward on the other. The couple shall be applied on each track in the direction that will produce the greatest force in the member under consideration.

RE	=	20	%
addition moment from RE	=	32.47	kip*ft
addition shear from RE	=	10.76	kip

frame force from E80 with impact and RE	M_{LL}	=	259.05	kip*ft
	V_{LL}	=	85.86	kip

section 7.3.3.3 Allowable Stresses for Normal Rating

b. allowable unit stresses resulting from the loads and forces described in the preceeding articles are shown in Table 15-1-11 as supplemented by Table 15-7-2.

Table 15-1-11. Structural Steel, Fasteners and Pins

tension in extreme fibers of rolled shapes, girders and built-up sections, subject to bending, net section: = 0.55*Fy ------ unit: (psi)

shear in webs for rolled beams and plate girders, gross section: = 0.35*Fy ------ unit: (psi)

beam allowable stresses:

bending stress:	F_b'	=	27500	psi	
shear stress:	F_v'	=	17500	psi	
maximum moment from self weight DL:	M_{DL}	=	49.3492	kip*ft	<<-- from CSi
maximum allowable moment from train LL:	M_{LL_allow}	=	617.9	kip*ft	<<-- by chenging
maximum shear from self weight DL:	V_{DL}	=	16.716	kip	<<-- from CSi
maximum allowable shear from train LL:	V_{LL_allow}	=	914.2	kip	<<-- by chenging

beam stress demands:

bending stress demand:	F_{b_u}	=	27499	psi	
shear stress demand:	F_{v_u}	=	17499	psi	

set stress demand 1 psi less than stress capacity:

$\blacktriangle_b$	=	-1	psi	<<-- Goal Seek
$\blacktriangle_v$	=	-1	psi	<<-- Goal Seek

rating factors for all demand V.S. capacity:

	RF_b	=	2.39	N/A	
	RF_v	=	10.65	N/A	
the maximum train axial load can go through this bridge is:			190	kip	<<-- Train Rating
rating factor for E80 train is:			2.39	N/A	<<-- Rating Factor

PM 119.51

Coordinates for this bridge are 38.612753°N 121.069855°W. This location is a 5-span concrete girder bridge with a length of 128 feet and a width of 17’-1½”. This bridge has two abutments with 4 piers. The span length from Abutment 1 to Pier 2 was measured to be 8’-0” while the rest of the spans were measured to be 30’-0”. This bridge was built in 1967, however Abutment 1 appears to be older. During observation/field measurements, the bridge appeared to be in good condition as no major damages were seen to the superstructure and substructure. The superstructure consists of a concrete slab in Span 1 and side-by-side concrete box girders in Spans 2-5. The expansion joints were filled with ballast due to missing joint covers. This was observed along the entire length of the bridge. Another observation was that there were no end diaphragms on the bridge. On the substructure, seismic restrainers were seen at each pier. Specifically at pier 2, spalling was observed on the north side of the pier. The spalling was at the left and right end of the pier, measured to be 1-9” wide by 1’-11” tall and 1’-3” wide and 1’-7” tall respectively. A fair amount of vegetation growth was observed at the creek. Railroad ties and tracks appear to be in fair condition. Due to the conditions of the bridge and the bridge rating, this bridge is serviceable. Dokken recommends maintenance work on this bridge due to the ballast in the joints and spalling at the piers. An estimated cost is provided for the recommended work.

Estimated Cost ($): $22,00

Photo 1: Top view of bridge looking up station.

Photo 2: Bridge elevation view from east side of track, Pier 3 & 4 shown. Typical span is 30 feet.

Photo 3: Elevation view of Abutment 1 and Pier 2, View from east side of the track. Abutment 1 is older than rest of the bridge.

Photo 4: Close up of Span 1 (8'-0"). Shown are Abutment 1 and Pier 2.

Photo 5: Closeup view of Abutment 1 and concrete slab connection. Shown are seismic restrainers.

Photo 6: View of Pier 3.

Photo 7: Typical seismic restrainer to Pier connection. Spalling at connection observed at most locations.

Photo 8: View of Abutment 5. Seismic restrainer shown.

Photo 9: Right corner of Abutment 5. A date of 1967 is shown in the picture indicating year bridge was built.

Photo 10: Typical joint opening between box girder spans. Ballast was found to be inside of these joints. Picture shown is the end of span 2. Joints are missing ballast cover.

Photo 11: Side view of concrete box culvert.

Photo 12: Loose ballast at an open joint

section 2.35 Shear

section 2.35.2 Permissible Shear Stress (2010) R(2022)

b. shear stress carried by conc. v_c, for members subject to shear and flexure only, may be computed by:

$$v_c = 1.9 \cdot \sqrt{f_c'} + 2500 \cdot \rho_w \cdot \frac{V_u \cdot d}{M_u} \qquad \text{EQ 2-46}$$

$$v_c = 0.16 \cdot \sqrt{f_c'} + 17 \cdot \rho_w \cdot \frac{V_u \cdot d}{M_u} \qquad \text{EQ 2-46M}$$

but v_c shall not exceed 3.5*sqrt(f'c) (or 0.29*sqrt(f'c) in metric). The quantity shall not be taken greater than 1.0, where M_u is the factored moment occruing simultaneously with V_u at the section considered.

compressive stress not found for AREMA, use 3000 psi from AASHTO 1961

f_c'	=	3000	psi	

d can be computed as 0.8*H

d	=	24	inch	

use the definiation of ρ_v in AASHTO to calculate ρ_w in EQ 2-46

@ begin of "A", shear reinforcement is #5 @ 9" with 4 legs per standard plan, tot 2 piece

ρ_w	=	0.171775	-	

shear @ begin of "A" from CSiBridge

V	=	278.039	kip	demand

moment @ begin of "A" from CSiBridge

M	=	218.3811	kip/ft	

allowable shear stress per EQ 2-46	v_c	=	192	psi	capacity
		=			
allowable shear force in conc.	V_c	=	187	kip	

AASHTO 5.7.3.3 - Nominal Shear Resistance

steel reinforcement yield strength not found for AREMA, assume 40 ksi;

f_y	=	40	ksi	

shear resistance provided by transverse reinforcement - V_s

per AASHTO Eqn. 5.7.3.3-4

V_s	=	283	kip	capacity

effective shear depth per AASHTO section 5.7.2.8

d_v	=	25.65	in	

compare demand and capacity

this bridge is good for E80 train live load

@ begin of span, shear reinforcement is #5 @ 3-1/2" with 4 legs per standard plan, tot 2 piece

ρ_w = 0.441707 -

shear @ begin of "A" from CSiBridge

V = 333.015 kip demand

moment @ begin of "A" from CSiBridge

M = 0.01 kip/ft

allowable shear stress per EQ 2-46 v_c = 192 psi capacity

=

allowable shear force in conc. V_c = 187 kip

AASHTO 5.7.3.3 - Nominal Shear Resistance

steel reinforcement yield strength not found for AREMA, assume 40 ksi;

f_y = 40 ksi

shear resistance provided by transverse reinforcement - V_s

per AASHTO Eqn. 5.7.3.3-4

V_s = 727 kip capacity

effective shear depth per AASHTO section 5.7.2.8

d_v = 25.65 in

compare demand and capacity

this bridge is good for E80 train live load

PM 121.87

This location is a single 15-foot span timber girder bridge with a width of 8 feet. This bridge has two 7’-6” high abutments. During observation/field measurements, it was noticed that one of the girders is in poor condition due to its damage from water infiltration. This girder will need to be addressed before any train or other vehicle is used. All other timber girders were observed to be in fair condition. The two concrete abutments are in fair condition as no large cracks, spalling, or any other major damages were observed. There is a fair amount of vegetation at the creek underneath of the bridge. A substantial amount of trash and debris was also seen. For the bridge rating, Dokken did not include the damaged girder in the calculations. Conservatively, the deadloads remain the same as the other timber bridges to determine the rating factor. Due to the damaged girder, Dokken recommends replacing and installing a new one and clearing out the vegetation in the creek. The estimated cost of work is provided below.

Estimated Cost ($): $15,000

Photo 1: Top view of bridge with track, looking upstation

Photo 2: Close up view of top of bridge.

Photo 3: View of top of bridge showing condition of railroad ties, steel track, and tie keeper

Photo 4: Elevation view of bridge from the right side of the track.

Photo 5: Bottom view of bridge girders. Fourth girder on the right side is broken, critical condition.

Photo 6: Closeup view of damaged girder.

Photo 7: Bridge elevation view from left side of track.

Photo 8: View of channel from left side of track

the material in this chapter is written with regard to typical North American Railroad Timber Trestles and other timber structures mentioned herein with

- spans up to 16 feet;
- Standard Gage Track;
- Norman North American passenger and freight equipment;
- Speeds of freight trains up to 80 mph and passenger trains up to 90 mph;

Chapter 7 - Timber Structure

Part 2 - Design of Wood Railway Bridges and Trestles for Railway Loading

Section 2.4 - Designing for Engineered Wood Products

assume timber used in the bridge is: Douglas-fir Grade No.1 (AREMA page 7-2-24)

Table 7-2-9. allowable unit stresses for stress graded lumber - railroad loading (visual grading)

F_b = 1215 psi $F_{c\perp}$ = 380 psi F_V = 150 psi

table 7-2-3. Applicability of Adjustment Factors				
design properties	temperature C_T	beam Stability C_L	Volumn C_V	
$F_b' = F_b*(...)*(...)*(...)$	1.0	1.0	C_V	bending
$F_{c\perp}' = F_{c\perp}*(...)*(...)*(...)$	1.0	none	none	tension
$F_v' = F_v*(...)*(...)*(...)$	1.0	none	none	shear

Note: C_V is [Volumn Factor];

allowable bending stresses of glulam are affected by geometry and size. Generally, larger sizes have a correspondingly lower allowable bending stress than smaller members. To account for this behavior, a volumn factor, C_V, shall be applied. C_V shall not exceed 1.0 and is computed as follows:

$$C_V = \left(\frac{5.125}{b}\right)^p \left(\frac{12}{d}\right)^p \left(\frac{21}{l}\right)^p \leq 1.0$$

where:

32	b = width of bending member in [**inches**]. For multiple piece width layups, b = width of widest piece in the layup, For practical purposes, b ≤ 10.75 in.;
18	d = depth of bending member in [**inches**];
15	l = length of bending member between points of zero moment in [**feet**];
1/10	p = 1/20 for Southern pine and 1/10 for other species; select ==> other species

compute C_V base on above values C_V = 0.82691 -

1 not glulam

beam allowable stresses:

bending stress:	F_b'	=	1215	psi
compression stress ⊥ to grain:	$F_{c\perp}'$	=	380	psi
shear stress:	F_v'	=	150	psi

timber beam section properties:

cross sectional area:	A	=	576	in^2
moment of inertia:	I_{xx}	=	11664	in^4

section modulus:	S_{XX}	=	1296	in^3	
radius of gyration:	r_{XX}	=	5.2	in	
unit weight of timber material:	ω_{timber}	=	60	lb/ft^3	
self weight (dead load) of timber beam:	ω_{beam_DL}	=	0.24	kip/ft	
timber tie beam under rail - width:	W_{tie}	=	0.833	ft	
timber tie beam under rail - depth:	D_{tie}	=	0.667	ft	
timber tie beam under rail - length:	L_{tie}	=	4.5	ft	
total number of ties on the beam:	N_{tie}	=	11	each	
self weight (dead load) of tie beams:	ω_{tie_DL}	=	0.12	kip/ft	
sectional area of tie brace (highlight yellow):	A_{brace}	=	0.23	ft^2	
self weight (dead load) of tie brace:	ω_{brace_DL}	=	0.02	kip/ft	
track (rails and fastenings) one side:	ω_{track_DL}	=	0.1	kip/ft	
total distributed dead load on beam:	ω_{total}	=	0.48	kip/ft	<<-- input
maximum moment from self weight DL:	M_{DL}	=	13.53	kip*ft	<<-- from CSi
maximum moment from train LL:	M_{LL}	=	117.5	kip*ft	<<-- Goal Seek
maximum shear from self weight DL:	V_{DL}	=	3.61	kip	<<-- from CSi
maximum shear from train LL:	V_{LL}	=	81.9	kip	<<-- Goal Seek
maximum compression from self weight DL:	p_{DL}	=	10.73	psi	
maximum compression from train LL:	p_{LL}	=	367.6	psi	<<-- Goal Seek
beam stress demands:					
bending stress demand:	F_{b_u}	=	1214	psi	
comp. stress ⊥ to grain demand:	$F_{c\perp\_u}$	=	379	psi	
shear stress demand:	F_{v_u}	=	149	psi	
set stress demand 1 psi less than stress capacity:					
	$\blacktriangle_b$	=	-1	psi	<<-- Goal Seek
	$\blacktriangle_{c\perp}$	=	-1	psi	<<-- Goal Seek
	$\blacktriangle_v$	=	-1	psi	<<-- Goal Seek
frame forces from E80 train live load:			80		
	M_{LL}	=	250	kip*ft	
	V_{LL}	=	476	kip	
	p_{LL}	=	80	psi	
rating factors for all demand V.S. capacity:					
	RF_b	=	0.47	N/A	
	$RF_{c\perp}$	=	0.77	N/A	
	RF_v	=	1.02	N/A	

capacity V.S. demand:

bending:	☺☻☺ this train is good on bridge
compression stress ⊥ to grain:	☺☻☺ this train is good on bridge
shear:	☺☻☺ this train is good on bridge

the maximum train axial load can go through this bridge is:	37	kip	<<-- Train Rating
rating factor for E80 train is:	0.47	N/A	<<-- Rating Factor

Section 2.3.8 - Impact (2013) R(2022)

The dynamic increment of load due to the effects of speed, roll and track irregularities is not well established for timber structures. Its totaleffect is setimated to be less than the increased strength of timber for the short cumulative duration of loading to which railroad bridges are subjected in service, and is taken into consideration in the derivation of allowable working stresses for design.

- According to above section, it's unnecessary to take impact and rolling effect into consideration.

PM 123.72

Coordinates for this bridge are 38.590106°N 121.013670°W. This bridge is a single span bridge with a length of 5 feet. The bridge has 18” depth timber girders running along the whole width of the bridge. The girders are in fair condition. At the time of the inspection, the girders were observed to be damped/moist. No signs of moisture damaged were observed, however. The entire track is filled with ballast. Two small timber retaining walls were seen on both sides to retain the ballast. The substructure of this bridge includes two concrete abutments that are both 3’-0” in height and 14” wide. Abutment 2 has a date of 1928 which we assumed was when the bridge was built. Both abutments have a ½” longitudinal crack at the center. Abutment two also has a fair amount of spalling at the middle and the top. Dimensions of the railroad ties are typical.

Photo 1: Top view of bridge with track, looking upstation

Photo 2: Closeup view of track

Photo 3: Elevation view of bridge, right side of track

Photo 4: Inside of bridge. Abutment 1 and 2 shown

Photo 5: Timber beams along width of the bridge

Photo 6: Elevation of bridge, left side of track

Photo 7: Diagonal crack on Abutment 2

Photo 8: Second crack on Abutment 2

Photo 9: Abutment 1 diagonal crack

Photo 10: Concrete spalling at Abutment 1

bridge info:

Timber bridge, two layers of timber plates functioning as beam, each layer has 15 of 3" X 12" timber plates;
Bridge span length = 3', 1.5' of ballast over timber plate;

the material in this chapter is written with regard to typical North American Railroad Timber Trestles and other timber structures mentioned herein with

- spans up to 16 feet;
- Standard Gage Track;
- Norman North American passenger and freight equipment;
- Speeds of freight trains up to 80 mph and passenger trains up to 90 mph;

Chapter 7 - Timber Structure
Part 2 - Design of Wood Railway Bridges and Trestles for Railway Loading
Section 2.4 - Designing for Engineered Wood Products

assume timber used in the bridge is: Douglas-fir Grade No.1 (AREMA page 7-2-24)

Table 7-2-9. allowable unit stresses for stress graded lumber - railroad loading (visual grading)

F_b = 1215 psi $F_{c\perp}$ = 380 psi F_V = 150 psi

table 7-2-3. Applicability of Adjustment Factors				
design properties	temperature	beam Stability	Volumn	
	C_T	C_L	C_V	
$F_b' = F_b*(...)*(...)*(...)$	1.0	1.0	C_V	*bending*
$F_{c\perp}' = F_{c\perp}*(...)*(...)*(...)$	1.0	none	none	*tension*
$F_v' = F_v*(...)*(...)*(...)$	1.0	none	none	*shear*

Note: C_V is [Volumn Factor];

allowable bending stresses of glulam are affected by geometry and size. Generally, larger sizes have a correspondingly lower allowable bending stress than smaller members. To account for this behavior, a volumn factor, C_V, shall be applied. C_V shall not exceed 1.0 and is computed as follows:

$$C_V = \left(\frac{5.125}{b}\right)^p \left(\frac{12}{d}\right)^p \left(\frac{21}{l}\right)^p \leq 1.0$$

where:

180	b = width of bending member in [inches]. For multiple piece width layups, b = width of widest piece in the layup, For practical purposes, b ≤ 10.75 in.;
3	d = depth of bending member in [inches];
3	l = length of bending member between points of zero moment in [feet];
1/10	p = 1/20 for Southern pine and 1/10 for other species; select ==> other species

compute C_V base on above values C_V = 0.97759 -
1 *not glulam*

beam allowable stresses:

bending stress:	F_b'	=	1215	psi
compression stress ⊥ to grain:	$F_{c\perp}'$	=	380	psi
shear stress:	F_v'	=	150	psi

timber beam section properties:

cross sectional area:	A	=	540	in^2	
moment of inertia:	I_{XX}	=	405	in^4	
section modulus:	S_{XX}	=	270	in^3	
radius of gyration:	r_{XX}	=	0.87	in	
unit weight of timber material:	ω_{timber}	=	60	lb/ft^3	
self weight (dead load) of timber beam:	ω_{beam_DL}	=	0.45	kip/ft	<<-- total 2 layers
timber tie beam under rail - width:	W_{tie}	=	0.833	ft	
timber tie beam under rail - depth:	D_{tie}	=	0.667	ft	
timber tie beam under rail - length:	L_{tie}	=	8	ft	
total number of ties on the beam:	N_{tie}	=	3	each	
self weight (dead load) of tie beams:	ω_{tie_DL}	=	0.27	kip/ft	
sectional area of ballast:	$A_{ballast}$	=	23.4	ft^2	
unit weight of ballast:	$\omega_{ballast}$	=	101	lb/ft^3	
self weight (dead load) of ballast:	$\omega_{ballast}$	=	2.37	kip/ft	
track (rails and fastenings) total:	ω_{track_DL}	=	0.2	kip/ft	
total distributed dead load on beam:	ω_{total}	=	3.29	kip/ft	<<-- input
maximum moment from self weight DL:	M_{DL}	=	3.7013	kip*ft	<<-- from CSi
maximum moment from train LL:	M_{LL}	=	50.9	kip*ft	<<-- Goal Seek
maximum shear from self weight DL:	V_{DL}	=	4.935	kip	<<-- from CSi
maximum shear from train LL:	V_{LL}	=	154.9	kip	<<-- Goal Seek
maximum compression from self weight DL:	p_{DL}	=	2.61	psi	
maximum compression from train LL:	p_{LL}	=	376.4	psi	<<-- Goal Seek

beam stress demands:

bending stress demand:	F_{b_u}	=	1214	psi
comp. stress ⊥ to grain demand:	$F_{c\perp\_u}$	=	379	psi
shear stress demand:	F_{v_u}	=	149	psi

set stress demand 1 psi less than stress capacity:

$\blacktriangle_b$	=	-1	psi	<<-- Goal Seek
$\blacktriangle_{c\perp}$	=	-1	psi	<<-- Goal Seek
$\blacktriangle_v$	=	-1	psi	<<-- Goal Seek

frame forces from E80 train live load: 80

M_{LL}	=	60	kip*ft
V_{LL}	=	80	kip
p_{LL}	=	22	psi

rating factors for all demand V.S. capacity:

RF_b	=	0.85	N/A
$RF_{c\perp}$	=	17.11	N/A
RF_v	=	1.94	N/A

capacity V.S. demand:

bending:	☺☻☺ this train is good on bridge
compression stress ⊥ to grain:	☺☻☺ this train is good on bridge
shear:	☺☻☺ this train is good on bridge

the maximum train axial load can go through this bridge is:	67	kip	<<-- Train Rating
rating factor for E80 train is:	0.85	N/A	<<-- Rating Factor

Section 2.3.8 - Impact (2013) R(2022)

The dynamic increment of load due to the effects of speed, roll and track irregularities is not well established for timber structures. Its totaleffect is setimated to be less than the increased strength of timber for the short cumulative duration of loading to which railroad bridges are subjected in service, and is taken into consideration in the derivation of allowable working stresses for design.

- According to above section, it's unnecessary to take impact and rolling effect into consideration.

PM 122.47

This steel girder bridge spans across deer creek off Latrobe Road. The dimensions of the bridge were measured to be 68'-7" in length and 20'-1" total width. The depth of the two built up, riveted, steel girders was measured to be 8'-3¾". The girders have ½" web plates with double-angle flanges and bottom flange cover plates. There are double angle longitudinal stiffeners approximately one foot from the top of girder. The girders are connected by cross frames and top and bottom lateral bracing consisting of angles and double angles. The steel girders are in fair condition, with failing paint and light to moderate rust throughout and localized pack rust at upper lateral bracing connections resulting in section loss of up to 1/16" from several top lateral angles and connection plates near the ends of the span. The bridge has a name plate indicating it was built in 1913. The current state of the bridge is serviceable.

The substructure consisted of two concrete abutments. The dimensions of the abutments were measured to be 28'-7½" in height, 30'-0" in length, and 10'-0" wide. The abutments have a ¾"/ft batter. A ½" vertical crack was observed at abutment 1 on the right side of the abutment (Photo 3) No other damages to the substructure were observed during inspection.

Two options are recommended for maintenance of this bride. The first option is cleaning and painting the whole bridge girder due to the rust observed throughout. The second option is spot blasting specific corroded areas inside of the girders. Both cost estimates are provided below

Option 1 estimate cost($): $260,000

Option 2 estimate cost($): $52,000

Photo 1: Top view of bridge looking upstation

Photo 2: Bridge elevation view, right side of bridge. Abutment 1 & 2 shown.

Photo 3: Closeup view of Abutment 1, right side of bridge. Large crack can be seen at the edge of the concrete headwall.

Photo 4: Closeup view of steel girder, railroad ties, and walkway supports.

Photo 5: Closeup view of railroad ties,walkway supports, steel track. All are in good/fair condition.

Photo 6: Channel view

Photo 7: Inside steel girders. Cross lateral bracing shown.

Photo 8: Steel Connection

Photo 9: Steel girder connection to concrete seat abutment

Photo 10: Gusset Plate Connection

Photo 11: Vertical Gusset Plate Connection

Photo 12: Gusset Plate Connection Pack Rust Corrosion

Photo 13: Closeup view corroded connection.

Photo 14: Steel Girder to Concrete Abutment Connection

beam allowable stresses:

bending stress:	F_b'	=	30000	psi	
shear stress:	F_v'	=	30000	psi	

section 7.3.3.3 Allowable Stresses for Normal Rating

b. allowable unit stresses resulting from the loads and forces described in the preceeding articles are

Table 15-1-11. Structural Steel, Fasteners and Pins

tension in extreme fibers of rolled shapes, girders and built-up sections, subject to bending, net

shear in webs for rolled beams and plate girders, gross section: = 0.35*Fy ------ unit: (psi)

beam allowable stresses:

bending stress:	F_b'	=	16500	psi	
shear stress:	F_v'	=	10500	psi	

steel girder section properties:

cross sectional area:	A	=	115.0625	in^2	
moment of inertia:	I_{XX}	=	181229	in^4	
section modulus:	S_{XX}	=	3400	in^3	
radius of gyration:	r_{XX}	=	39.68	in	

unit weight of timber material:	ω_{timber}	=	60	lb/ft^3	
timber tie beam under rail - width:	W_{tie}	=	0.833	ft	
timber tie beam under rail - depth:	D_{tie}	=	1	ft	
timber tie beam under rail - length:	L_{tie}	=	10	ft	
total number of ties on the beam:	N_{tie}	=	58	each	
dead load of tie beams over 70' span:	ω_{tie_DL}	=	0.42	kip/ft	
assume additional wood on the structure		=	0	%	
track (rails and fastenings):	ω_{track_DL}	=	0.2	kip/ft	
distributed steel element dead load:	ω_{steel_DL}	=	0.90	kip/ft	

total distributed dead load on beam:	ω_{total}	=	1.52	kip/ft	<<-- input
maximum moment from self weight DL:	M_{DL}	=	465.5	kip*ft	<<-- from CSi
maximum moment from train LL:	M_{LL}	=	8034.1	kip*ft	<<-- Goal Seek
maximum shear from self weight DL:	V_{DL}	=	26.6	kip	<<-- from CSi
maximum shear from train LL:	V_{LL}	=	3425.1	kip	<<-- Goal Seek

beam stress demands:

bending stress demand:	F_{b_u}	=	29999	psi	
shear stress demand:	F_{v_u}	=	29999	psi	

set stress demand 1 psi less than stress capacity:

$\blacktriangle_b$	=	-1	psi	<<-- Goal Seek
$\blacktriangle_v$	=	-1	psi	<<-- Goal Seek

frame forces from E80 train live load: 80

M_{LL}	=	3415	kip*ft	<<-- from CSi
V_{LL}	=	221.0285	kip	<<-- from CSi

AREMA Chapter 15 Steel Structures

Section 1.3.5 Impact Load (2007) R(2021)

c. impact load due to vertical effects, expressed as a percentage of live load applied at each rail, shall be determined by the applicable formula below:

(1) percentage of live load for rolling equipment without hammer blow (freight and passernger cars, and locomotives other than steam):

(a) for span length less than 80 feet: 40-3*L^2/1600

impact	=	30.8125	%
addition moment from impact	=	1052.247	kip*ft
addition shear from impact	=	68.10441	kip

d. impact load dur to rocking effect, RE, is created by the transfer of load from the wheels on one side of a car or locomotive to the other side from periodic lateral rocking of the equipment. RE shall be calculated from loads applied as a vertical force couple, each being 20 percent of the wheel load without impact, acting downward on one rail and upward on the other. The couple shall be applied on each track in the direction that will produce the greatest force in the member under consideration.

RE	=	20	%
addition moment from RE	=	683	kip*ft
addition shear from RE	=	44.2057	kip

frame force from E80 with impact and RE

M_{LL}	=	5150.247	kip*ft
V_{LL}	=	333.3386	kip

rating factors for all demand V.S. capacity:

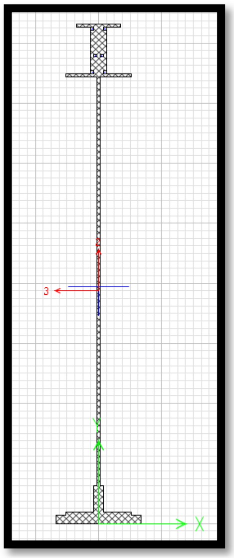
RF_b	=	1.56	N/A
RF_v	=	10.28	N/A

capacity V.S. demand:

bending: ☺☻☺ this train is good on bridge

shear: ☺☻☺ this train is good on bridge

the maximum train axial load can go through this bridge is:	124	kip	<<-- Train Rating
rating factor for E80 train is:	1.56	N/A	<<-- Rating Factor

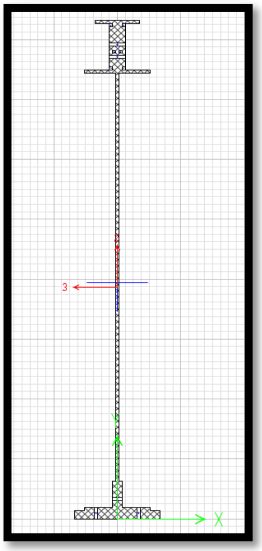


Section Name: mid bridge

Properties

Cross-section (axial) area	122.7188	Section modulus about 3 axis (top)	3705.0959
Moment of Inertia about 3 axis	197375.21	Section modulus about 3 axis (bottom)	4246.5712
Moment of Inertia about 2 axis	811.6939	Section modulus about 2 axis (left)	101.4617
Product of Inertia about 2-3	0.	Section modulus about 2 axis (right)	101.4617
Torsional constant	131.5445	Warping Constant (Cw)	N/C
Shear area in 2 direction	50.2008	Plastic modulus about 3 axis	7672.6529
Shear area in 3 direction	62.0491	Plastic modulus about 2 axis	319.8991
CG offset in 3 direction	2.665E-15	Radius of Gyration about 3 axis	40.1043
CG offset in 2 direction	-3.3963	Radius of Gyration about 2 axis	2.5718
Shear Center Offset (x3)*	N/C		
Shear Center Offset (x2)*	N/C		

* Value is not used in analysis



Section Name: mid bridg - net

Properties

Cross-section (axial) area	115.0625	Section modulus about 3 axis (top)	3400.2081
Moment of Inertia about 3 axis	181229.45	Section modulus about 3 axis (bottom)	3901.5619
Moment of Inertia about 2 axis	763.5108	Section modulus about 2 axis (left)	95.4388
Product of Inertia about 2-3	0.	Section modulus about 2 axis (right)	95.4388
Torsional constant	77.7918	Warping Constant (Cw)	N/C
Shear area in 2 direction	50.217	Plastic modulus about 3 axis	7087.8262
Shear area in 3 direction	51.0778	Plastic modulus about 2 axis	295.6641
CG offset in 3 direction	2.665E-15	Radius of Gyration about 3 axis	39.6869
CG offset in 2 direction	-3.4245	Radius of Gyration about 2 axis	2.576
Shear Center Offset (x3)*	N/C		
Shear Center Offset (x2)*	N/C		

* Value is not used in analysis

PM 122.97

This bridge location is another single span timber bridge. This bridge was built over a small creek along the corridor. This bridge is on a slight horizontal curve. The bridge span was measured on both sides, measuring at 15’-9½” on the right and 16’-½” on the left. The superstructure of the bridge is in fair condition. The timber beams appear to be free of any damage such as cracking and rot. It was observed however that all the railroad ties and tie keepers on the bridge are rotted. This will require replacement. The railroad ties’ measurements are 8.5” wide, 6.5” tall, and 8’-6” in length. Railroad ties were also measured to have 18” spacing (typical for a timber bridge). The two abutments are made of dry stack rock. Longitudinal timber beams suspended from the superstructure act as struts to resist inward movement of the abutments These two abutments are in fair condition. It was noted that embankment erosion was observed at both abutments. Please refer to photo eight. The railroad ties, tie keepers, and the erosion of the embankment are recommended to be addressed due to their conditions. The estimated cost of maintenance is provided below.

Estimated cost ($): $15,900.

Photo 1: Top view of bridge looking up station

Photo 2: Bridge elevation view from left side of track.

Photo 3: Bridge elevation view from right side of track.

Photo 4: Close up view of track. Shown are the railroad ties, timber girders, tie keeper, and steel track.

Photo 5: Closeup of railroad ties from abutment 1. Ties are in poor condition due to age and cracks observed. Tie keepers are also in poor condition.

Photo 6: Tie keeper located on the right side of the track at abutment 2. Tie keeper is in critical condition due to age and timber falling apart.

Photo 7: Left side view of abutment 2. Stone abutment, in fair condition.

Photo 8: Embankment erosion on right hand side of Abutment 1.

the material in this chapter is written with regard to typical North American Railroad Timber Trestles and other timber structures mentioned herein with

- spans up to 16 feet;
- Standard Gage Track;
- Norman North American passenger and freight equipment;
- Speeds of freight trains up to 80 mph and passenger trains up to 90 mph;

Chapter 7 - Timber Structure

Part 2 - Design of Wood Railway Bridges and Trestles for Railway Loading

Section 2.4 - Designing for Engineered Wood Products

assume timber used in the bridge is: Douglas-fir Grade No.1 (AREMA page 7-2-24)

Table 7-2-9. allowable unit stresses for stress graded lumber - railroad loading (visual grading)

F_b = 1215 psi $F_{c\perp}$ = 380 psi F_V = 150 psi

table 7-2-3. Applicability of Adjustment Factors				
design properties	temperature	beam Stability	Volumn	
	C_T	C_L	C_V	
$F_b' = F_b*(...)*(...)*(...)$	1.0	1.0	C_V	bending
$F_{c\perp}' = F_{c\perp}*(...)*(...)*(...)$	1.0	none	none	tension
$F_v' = F_v*(...)*(...)*(...)$	1.0	none	none	shear

Note: C_V is [Volumn Factor];

allowable bending stresses of glulam are affected by geometry and size. Generally, larger sizes have a correspondingly lower allowable bending stress than smaller members. To account for this behavior, a volumn factor, C_V, shall be applied. C_V shall not exceed 1.0 and is computed as follows:

$$C_V = \left(\frac{5.125}{b}\right)^p \left(\frac{12}{d}\right)^p \left(\frac{21}{l}\right)^p \leq 1.0$$

where:

32	b = width of bending member in [inches]. For multiple piece width layups, b = width of widest piece in the layup, For practical purposes, b ≤ 10.75 in.;
18	d = depth of bending member in [inches];
15	l = length of bending member between points of zero moment in [feet];
1/10	p = 1/20 for Southern pine and 1/10 for other species; select ==> other species

compute C_V base on above values C_V = 0.82691 -

1 not glulam

beam allowable stresses:

bending stress:	F_b'	=	1215	psi
compression stress ⊥ to grain:	$F_{c\perp}'$	=	380	psi
shear stress:	F_v'	=	150	psi

timber beam section properties:

cross sectional area:	A	=	576	in^2
moment of inertia:	I_{xx}	=	15552	in^4

section modulus:	S_{XX}	=	1728	in^3	
radius of gyration:	r_{XX}	=	5.2	in	
unit weight of timber material:	ω_{timber}	=	60	lb/ft^3	
self weight (dead load) of timber beam:	ω_{beam_DL}	=	0.24	kip/ft	
timber tie beam under rail - width:	W_{tie}	=	0.833	ft	
timber tie beam under rail - depth:	D_{tie}	=	0.667	ft	
timber tie beam under rail - length:	L_{tie}	=	4.5	ft	
total number of ties on the beam:	N_{tie}	=	11	each	
self weight (dead load) of tie beams:	ω_{tie_DL}	=	0.12	kip/ft	
additional timber beam under bridge					
section area:	A_{add}	=	1	ft^2	
element length:	L_{add}	=	23.45	ft	
dead load of this element:	ω_{add_DL}	=	0.09	kip/ft	
sectional area of tie brace (highlight yellow):	A_{brace}	=	0.23	ft^2	
self weight (dead load) of tie brace:	ω_{brace_DL}	=	0.02	kip/ft	
track (rails and fastenings) one side:	ω_{track_DL}	=	0.1	kip/ft	
total distributed dead load on beam:	ω_{total}	=	0.57	kip/ft	<<-- input
maximum moment from self weight DL:	M_{DL}	=	16.04	kip*ft	<<-- from CSi
maximum moment from train LL:	M_{LL}	=	158.7	kip*ft	<<-- Goal Seek
maximum shear from self weight DL:	V_{DL}	=	4.275	kip	<<-- from CSi
maximum shear from train LL:	V_{LL}	=	81.1	kip	<<-- Goal Seek
maximum compression from self weight DL:	p_{DL}	=	12.71	psi	
maximum compression from train LL:	p_{LL}	=	365.7	psi	<<-- Goal Seek
beam stress demands:					
bending stress demand:	F_{b_u}	=	1214	psi	
comp. stress ⊥ to grain demand:	$F_{c\perp\_u}$	=	379	psi	
shear stress demand:	F_{v_u}	=	149	psi	
set stress demand 1 psi less than stress capacity:					
	$▲_b$	=	-1	psi	<<-- Goal Seek
	$▲_{c\perp}$	=	-1	psi	<<-- Goal Seek
	$▲_v$	=	-1	psi	<<-- Goal Seek
frame forces from E80 train live load:			80		
	M_{LL}	=	250	kip*ft	
	V_{LL}	=	80	kip	
	p_{LL}	=	476	psi	

rating factors for all demand V.S. capacity:

RF_b	=	0.63	N/A
$RF_{c\perp}$	=	0.77	N/A
RF_v	=	1.01	N/A

capacity V.S. demand:

bending:	☺☻☺ this train is good on bridge
compression stress ⊥ to grain:	☺☻☺ this train is good on bridge
shear:	☺☻☺ this train is good on bridge

the maximum train axial load can go through this bridge is:	50	kip	<<-- Train Rating
rating factor for E80 train is:	0.63	N/A	<<-- Rating Factor

Section 2.3.8 - Impact (2013) R(2022)

The dynamic increment of load due to the effects of speed, roll and track irregularities is not well established for timber structures. Its totaleffect is setimated to be less than the increased strength of timber for the short cumulative duration of loading to which railroad bridges are subjected in service, and is taken into consideration in the derivation of allowable working stresses for design.

- According to above section, it's unnecessary to take impact and rolling effect into consideration.

PM 127.63

A two-span steel girder bridge is located at this post mile. The total length of the bridge is 19'-1" between centerlines of bearings. Width of the bridge was measured to be 9'-1". The general condition of the bridge is fair. The railroad ties are in fair condition. This bridge has railing, and walkway supports that are failing. Railing posts are out of plumb. The steel 10 rolled steel beams have light to moderate rust. The paint condition ranges from a fair to failed. The steel girders were measured at 10" deep with 4.5" wide flanges and a web thickness of 5/8". These dimensions are consistent with a S 10x25 shape. The beams are placed in two groups of 5 side-by-side with an out-to-out width of 7'-2". The lengths of the beams were measured at 10 feet each. Vegetation is growing through the bridge from the ground. The condition and capacity of the superstructure indicate this bridge is serviceable.

The substructure consists of two concrete abutments and one pier. The exposed height of the Abutment 1 is 6'-0". Abutment 1 has an exposed footing that sticks out 8.5". Abutment 1 has a seat width of 9'-3". The abutment's wall has a 1"/ft batter. Abutment 2 has a 2'-6" exposed wall height. Both abutments are in fair condition. The pier has an exposed height of 5'-8". The length of the of the pier was measured to be 8'-1". Like the abutments, no damages were observed during inspection.

Estimated cost ($): $5,700

Photo 1: Top view of bridge looking upstation.

Photo 2: Bridge elevation view, left side of track.

Photo 3: Closeup view of steel girder connection at Pier 2. Steel in moderate/poor condition due to rust. Paint on the beams is in fair – failed condition throughout. Two bolts missing at plate connection. 2 span bridge.

Photo 4: Vegetation growth underneath the bridge at abutment 1. Vegetation goes through the bridge as shown in Photo number 1.

Photo 5: Close up of a damaged walkway support on left hand side of bridge, span 1.

Photo 6: Closeup of damage post near Abutment 1. All other posts are in poor/critical condition.

bridge info:

two 9.5' long spans (both span simple supported) on total 10 of S10X25.4 steel girders (5 on each side)

beam allowable stresses:

bending stress:	F_b'	=	30000	psi
shear stress:	F_v'	=	30000	psi

section 7.3.3.3 Allowable Stresses for Normal Rating

b. allowable unit stresses resulting from the loads and forces described in the preceeding articles are

Table 15-1-11. Structural Steel, Fasteners and Pins

tension in extreme fibers of rolled shapes, girders and built-up sections, subject to bending, net

shear in webs for rolled beams and plate girders, gross section: = 0.35*Fy ------ unit: (psi)

beam allowable stresses:

bending stress:	F_b'	=	16500	psi
shear stress:	F_v'	=	10500	psi

S10X25.4 steel girder section properties:

cross sectional area:	A	=	7.45	in^2	
moment of inertia:	I_{XX}	=	123	in^4	
section modulus:	S_{XX}	=	24.6	in^3	
radius of gyration:	r_{XX}	=	4.07	in	
unit weight of steel section:	ω_{girder}	=	25.4	lb/ft	
unit weight of timber material:	ω_{timber}	=	60	lb/ft^3	
timber tie beam under rail - width:	W_{tie}	=	0.75	ft	
timber tie beam under rail - depth:	D_{tie}	=	0.5833	ft	
timber tie beam under rail - length:	L_{tie}	=	9	ft	
total number of ties on the beam:	N_{tie}	=	20	each	
self weight (dead load) of tie beams:	ω_{tie_DL}	=	0.25	kip/ft	
sectional area of tie brace:	A_{brace}	=	0.23	ft^2	
self weight (dead load) of tie brace:	ω_{brace_DL}	=	0.02	kip/ft	
track (rails and fastenings) (total 2):	ω_{track_DL}	=	0.2	kip/ft	
total distributed dead load on beam:	ω_{total}	=	0.724	kip/ft	<<-- input

frame forces from E80 train live load: 80

	M_{LL}	=	190.77	kip*ft
	V_{LL}	=	131.61	kip

AREMA Chapter 15 Steel Structures

Section 1.3.5 Impact Load (2007) R(2021)

c. impact load due to vertical effects, expressed as a percentage of live load applied at each rail, shall be determined by the applicable formula below:

(1) percentage of live load for rolling equipment without hammer blow (freight and passernger cars, and locomotives other than steam):

(a) for span length less than 80 feet: 40-3*L^2/1600

impact	=	39.83	%
addition moment from impact	=	75.99	kip*ft
addition shear from impact	=	52.42	kip

d. impact load dur to rocking effect, RE, is created by the transfer of load from the wheels on one side of a car or locomotive to the other side from periodic lateral rocking of the equipment. RE shall be calculated from loads applied as a vertical force couple, each being 20 percent of the wheel load without impact, acting downward on one rail and upward on the other. The couple shall be applied on each track in the direction that will produce the greatest force in the member under consideration.

RE	=	20	%
addition moment from RE	=	38.15	kip*ft
addition shear from RE	=	26.32	kip

frame force from E80 with impact and RE	M_{LL}	=	304.91	kip*ft	
	V_{LL}	=	210.35	kip	
maximum moment from self weight DL:	M_{DL}	=	7.9175	kip*ft	<<-- from CSi
maximum allowable moment from train LL:	M_{LL_allow}	=	330.3	kip*ft	<<-- by changing
maximum shear from self weight DL:	V_{DL}	=	4.272	kip	<<-- from CSi
maximum allowable shear from train LL:	V_{LL_allow}	=	777.8	kip	<<-- by changing

beam stress demands:

bending stress demand:	F_{b_u}	=	16499	psi
shear stress demand:	F_{v_u}	=	10499	psi

set stress demand 1 psi less than stress capacity:

$\blacktriangle_b$	=	-1	psi	<<-- Goal Seek
$\blacktriangle_v$	=	-1	psi	<<-- Goal Seek

rating factors for all demand V.S. capacity:

RF_b	=	1.08	N/A	
RF_v	=	3.70	N/A	
the maximum train axial load can go through this bridge is:		86	kip	<<-- Train Rating
rating factor for E80 train is:		1.08	N/A	<<-- Rating Factor

bridge rating analysis result:

this bridge is good for E-80 Train live load

PM 138.11

A single span timber bridge was found at this post mile location. The total length of the bridge was measured to be 15’-0” with a 12’-0” abutment seat width. Dimensions of the railroad ties are typical (9’ in length and spaced out at 1’-6”). Bridge was seen to be on a slight curve. The conditions of both the superstructure and substructure are in fair condition. No signs of concerning damages were observed at the timber beams, headwalls, or railroad ties. Railroad ties were observed to be undermined at the approach to the bridge. The substructure has two concrete abutments at an average height of 6’-9”. Width of the abutments are 1’-1”. Both abutments have a 1⅛” /ft batter. Both abutments’ footings were exposed. Poor consolidation was seen at the base of top of footing. Lastly, small retaining walls with steel piles were seen behind the abutments retaining ballast. These retaining walls are in good condition.

Photo 1: Top view of bridge looking upstation.

Photo 2: Bridge elevation view, right side of track. Bridge length was measured to be 15 feet. Abutment 1 and 2 are shown.

Photo 3: Abutment 1 typical section view. Seat width was measured to be 12 feet wide. Abutment 1 wall batter was determined to be 1 ⅛"/ft

Photo 4: Closeup view of top of bridge. Railroad ties are in good condition. No tie keepers observed. Steel track is in fair condition.

Photo 5: Exposed footing at Abutment 1. Footing sticks out approximately 6 inches.

Photo 6: Closeup of left side of Abutment 1. Crack observed slightly above abutment seat.

the material in this chapter is written with regard to typical North American Railroad Timber Trestles and other timber structures mentioned herein with

- spans up to 16 feet;
- Standard Gage Track;
- Norman North American passenger and freight equipment;
- Speeds of freight trains up to 80 mph and passenger trains up to 90 mph;

Chapter 7 - Timber Structure

Part 2 - Design of Wood Railway Bridges and Trestles for Railway Loading

Section 2.4 - Designing for Engineered Wood Products

assume timber used in the bridge is: Douglas-fir Grade No.1 (AREMA page 7-2-24)

Table 7-2-9. allowable unit stresses for stress graded lumber - railroad loading (visual grading)

F_b = 1215 psi $F_{c\perp}$ = 380 psi F_V = 150 psi

table 7-2-3. Applicability of Adjustment Factors				
design properties	temperature	beam Stability	Volumn	
	C_T	C_L	C_V	
$F_b' = F_b*(...)*(...)*(...)$	1.0	1.0	C_V	bending
$F_{c\perp}' = F_{c\perp}*(...)*(...)*(...)$	1.0	none	none	tension
$F_v' = F_v*(...)*(...)*(...)$	1.0	none	none	shear

Note: C_V is [Volumn Factor];

allowable bending stresses of glulam are affected by geometry and size. Generally, larger sizes have a correspondingly lower allowable bending stress than smaller members. To account for this behavior, a volumn factor, C_V, shall be applied. C_V shall not exceed 1.0 and is computed as follows:

$$C_V = \left(\frac{5.125}{b}\right)^p \left(\frac{12}{d}\right)^p \left(\frac{21}{l}\right)^p \leq 1.0$$

where:

32 b = width of bending member in [inches]. For multiple piece width layups, b = width of widest piece in the layup, For practical purposes, b ≤ 10.75 in.;

18 d = depth of bending member in [inches];

15 l = length of bending member between points of zero moment in [feet];

1/10 p = 1/20 for Southern pine and 1/10 for other species; select ==> other species

compute C_V base on above values C_V = 0.82691 -

1 not glulam

beam allowable stresses:

bending stress:	F_b'	=	1215	psi
compression stress ⊥ to grain:	$F_{c\perp}'$	=	380	psi
shear stress:	F_v'	=	150	psi

timber beam section properties:

cross sectional area:	A	=	576	in^2
moment of inertia:	I_{xx}	=	15552	in^4

section modulus:	S_{XX}	=	1728	in^3	
radius of gyration:	r_{XX}	=	5.2	in	
unit weight of timber material:	ω_{timber}	=	60	lb/ft^3	
self weight (dead load) of timber beam:	ω_{beam_DL}	=	0.24	kip/ft	
timber tie beam under rail - width:	W_{tie}	=	0.833	ft	
timber tie beam under rail - depth:	D_{tie}	=	0.667	ft	
timber tie beam under rail - length:	L_{tie}	=	4.5	ft	
total number of ties on the beam:	N_{tie}	=	11	each	
self weight (dead load) of tie beams:	ω_{tie_DL}	=	0.12	kip/ft	
sectional area of tie brace (highlight yellow):	A_{brace}	=	0.23	ft^2	
self weight (dead load) of tie brace:	ω_{brace_DL}	=	0.02	kip/ft	
track (rails and fastenings) one side:	ω_{track_DL}	=	0.1	kip/ft	
total distributed dead load on beam:	ω_{total}	=	0.48	kip/ft	<<-- input
maximum moment from self weight DL:	M_{DL}	=	13.53	kip*ft	<<-- from CSi
maximum moment from train LL:	M_{LL}	=	161.2	kip*ft	<<-- Goal Seek
maximum shear from self weight DL:	V_{DL}	=	3.61	kip	<<-- from CSi
maximum shear from train LL:	V_{LL}	=	81.9	kip	<<-- Goal Seek
maximum compression from self weight DL:	p_{DL}	=	10.73	psi	
maximum compression from train LL:	p_{LL}	=	367.6	psi	<<-- Goal Seek
beam stress demands:					
bending stress demand:	F_{b_u}	=	1214	psi	
comp. stress ⊥ to grain demand:	$F_{c\perp\_u}$	=	379	psi	
shear stress demand:	F_{v_u}	=	149	psi	
set stress demand 1 psi less than stress capacity:					
	$\blacktriangle_b$	=	-1	psi	<<-- Goal Seek
	$\blacktriangle_{c\perp}$	=	-1	psi	<<-- Goal Seek
	$\blacktriangle_v$	=	-1	psi	<<-- Goal Seek
frame forces from E80 train live load:			80		
	M_{LL}	=	250	kip*ft	
	V_{LL}	=	80	kip	
	p_{LL}	=	476	psi	
rating factors for all demand V.S. capacity:					
	RF_b	=	0.64	N/A	
	$RF_{c\perp}$	=	0.77	N/A	
	RF_v	=	1.02	N/A	

capacity V.S. demand:

bending:	☺☻☺ this train is good on bridge
compression stress ⊥ to grain:	☺☻☺ this train is good on bridge
shear:	☺☻☺ this train is good on bridge

the maximum train axial load can go through this bridge is:	51	kip	<<-- Train Rating
rating factor for E80 train is:	0.64	N/A	<<-- Rating Factor

Section 2.3.8 - Impact (2013) R(2022)

The dynamic increment of load due to the effects of speed, roll and track irregularities is not well established for timber structures. Its totaleffect is setimated to be less than the increased strength of timber for the short cumulative duration of loading to which railroad bridges are subjected in service, and is taken into consideration in the derivation of allowable working stresses for design.

- According to above section, it's unnecessary to take impact and rolling effect into consideration.

PM 138.22

This bridge is similar to PM 138.11 The total length of the bridge is 15'-2" with a width of 15'-3" at the abutment seat. Both the superstructure and substructure are in good/fair condition as no concerning damages were observed during inspection. Timber beams do not have any external damage. No signs of moisture were observed. Railroad ties are also in fair condition. No signs of concerning damages were observed. Substructure does not have any concerning damages either. Abutment 1 does have an exposed footing, abutment 2 does not. A date of 1920 was seen at abutment 1, meaning that the bridge substructure was built 104 years ago. The abutments have wingwalls that are 6 feet wide, inclined at 33 degrees. The abutments also have a 1⅛" /ft batter. Overall, due to the bridge rating, this bridge is in fair condition and is serviceable.

Photo 1: Top view of bridge looking upstation.

Photo 2: Bridge elevation view, right side of bridge. The single span of bridge is 15 feet. Slight vegetation through channel on both ends.

Photo 3: Typical abutment section view. Timber girders are in fair condition. Timber seat beams are also in fair condition. Exposed abutment height is 2.75 feet.

Photo 4: Girder to abutment 1 seat connection. Shown are the timber girders, railroad ties, sill beams, and headwalls.

Photo 5: Elevation view of bridge on the left side of the track.

Photo 6: Closeup view of railroad ties and steel track.

the material in this chapter is written with regard to typical North American Railroad Timber Trestles and other timber structures mentioned herein with

- spans up to 16 feet;
- Standard Gage Track;
- Norman North American passenger and freight equipment;
- Speeds of freight trains up to 80 mph and passenger trains up to 90 mph;

Chapter 7 - Timber Structure
Part 2 - Design of Wood Railway Bridges and Trestles for Railway Loading
Section 2.4 - Designing for Engineered Wood Products

assume timber used in the bridge is: Douglas-fir Grade No.1 (AREMA page 7-2-24)

Table 7-2-9. allowable unit stresses for stress graded lumber - railroad loading (visual grading)

F_b = 1215 psi $F_{c\perp}$ = 380 psi F_V = 150 psi

table 7-2-3. Applicability of Adjustment Factors				
design properties	temperature	beam Stability	Volumn	
	C_T	C_L	C_V	
$F_b' = F_b$*(…)*(…)*(…)	1.0	1.0	C_V	bending
$F_{c\perp}' = F_{c\perp}$*(…)*(…)*(…)	1.0	none	none	tension
$F_v' = F_v$*(…)*(…)*(…)	1.0	none	none	shear

Note: C_V is [Volumn Factor];

allowable bending stresses of glulam are affected by geometry and size. Generally, larger sizes have a correspondingly lower allowable bending stress than smaller members. To account for this behavior, a volumn factor, C_V, shall be applied. C_V shall not exceed 1.0 and is computed as follows:

$$C_V = \left(\frac{5.125}{b}\right)^p \left(\frac{12}{d}\right)^p \left(\frac{21}{l}\right)^p \leq 1.0$$

where:

32	b = width of bending member in [inches]. For multiple piece width layups, b = width of widest piece in the layup, For practical purposes, b ≤ 10.75 in.;
18	d = depth of bending member in [inches];
15	l = length of bending member between points of zero moment in [feet];
1/10	p = 1/20 for Southern pine and 1/10 for other species; select ==> other species

compute C_V base on above values C_V = 0.82691 -
1 not glulam

beam allowable stresses:

bending stress:	F_b'	=	1215	psi
compression stress ⊥ to grain:	$F_{c\perp}'$	=	380	psi
shear stress:	F_v'	=	150	psi

timber beam section properties:

cross sectional area:	A	=	576	in^2
moment of inertia:	I_{xx}	=	15552	in^4

section modulus:	S_{XX}	=	1728	in^3	
radius of gyration:	r_{XX}	=	5.2	in	
unit weight of timber material:	ω_{timber}	=	60	lb/ft^3	
self weight (dead load) of timber beam:	ω_{beam_DL}	=	0.24	kip/ft	
timber tie beam under rail - width:	W_{tie}	=	0.833	ft	
timber tie beam under rail - depth:	D_{tie}	=	0.667	ft	
timber tie beam under rail - length:	L_{tie}	=	4.5	ft	
total number of ties on the beam:	N_{tie}	=	11	each	
self weight (dead load) of tie beams:	ω_{tie_DL}	=	0.12	kip/ft	
sectional area of tie brace (highlight yellow):	A_{brace}	=	0.23	ft^2	
self weight (dead load) of tie brace:	ω_{brace_DL}	=	0.02	kip/ft	
track (rails and fastenings) one side:	ω_{track_DL}	=	0.1	kip/ft	
total distributed dead load on beam:	ω_{total}	=	0.48	kip/ft	<<-- input
maximum moment from self weight DL:	M_{DL}	=	13.53	kip*ft	<<-- from CSi
maximum moment from train LL:	M_{LL}	=	161.2	kip*ft	<<-- Goal Seek
maximum shear from self weight DL:	V_{DL}	=	3.61	kip	<<-- from CSi
maximum shear from train LL:	V_{LL}	=	81.9	kip	<<-- Goal Seek
maximum compression from self weight DL:	p_{DL}	=	10.73	psi	
maximum compression from train LL:	p_{LL}	=	367.6	psi	<<-- Goal Seek
beam stress demands:					
bending stress demand:	F_{b_u}	=	1214	psi	
comp. stress ⊥ to grain demand:	$F_{c\perp\_u}$	=	379	psi	
shear stress demand:	F_{v_u}	=	149	psi	
set stress demand 1 psi less than stress capacity:					
	$▲_b$	=	-1	psi	<<-- Goal Seek
	$▲_{c\perp}$	=	-1	psi	<<-- Goal Seek
	$▲_v$	=	-1	psi	<<-- Goal Seek
frame forces from E80 train live load:			80		
	M_{LL}	=	250	kip*ft	
	V_{LL}	=	80	kip	
	p_{LL}	=	476	psi	
rating factors for all demand V.S. capacity:					
	RF_b	=	0.64	N/A	
	$RF_{c\perp}$	=	0.77	N/A	
	RF_v	=	1.02	N/A	

capacity V.S. demand:

bending:	☺☻☺ this train is good on bridge
compression stress ⊥ to grain:	☺☻☺ this train is good on bridge
shear:	☺☻☺ this train is good on bridge

the maximum train axial load can go through this bridge is:	51	kip	<<-- Train Rating
rating factor for E80 train is:	0.64	N/A	<<-- Rating Factor

Section 2.3.8 - Impact (2013) R(2022)

The dynamic increment of load due to the effects of speed, roll and track irregularities is not well established for timber structures. Its totaleffect is setimated to be less than the increased strength of timber for the short cumulative duration of loading to which railroad bridges are subjected in service, and is taken into consideration in the derivation of allowable working stresses for design.

- According to above section, it's unnecessary to take impact and rolling effect into consideration.

PM 141.09

The length of this bridge is 50’-10” from face to face of the backwalls at the abutments. The bridge has a total of 4 spans, measuring at 15’-1”, 14’-11”, 15’-3”, and 14’-6”, respectively, from Abutment 1 to Abutment 5. The bridge is on tangent track. Like the other timber bridges along the corridor, the bridge has two concrete abutments. The superstructure is in fair to poor condition. There is one tie on the bridge that is in poor condition. The tie keepers and timber girders are in fair condition. The timber girders are semi-continuous along the span of the bridge with girders continuous over two spans and adjacent beams staggered.

The substructure is in fair to poor condition overall. Each pier has a 12”x14” bent cap that is 12’-0” long. The bent caps at Piers 2 and 3 are in poor/critical condition due to rot. Bent 2 is in critical condition due to the condition of the bent cap and one of the timber columns being disconnected from the bent cap and another completely missing. The diagonal timber braces at this location have large cracks. All other piers have 5 supporting columns underneath the bent caps. All the columns at the bents are battered except for the middle one. The batter of the piles is 2”/ft or 1⅛” /ft. Abutment 1 has a total height of 6’-9” (1’-9” seat height and 5’-0” exposed height). A 3/16” wide crack was observed at this abutment. The abutment footing is undermined by about 2 feet. The abutment’s right wingwall also has a 0.5” crack on the right-hand side. Abutment 5 has a 3’-9” exposed height, with a 1’-9” seat height. Condition of Abutment 5 is fair, as it is also undermined. A fair amount of vegetation surrounds the bridge. While we are only rating the superstructure, we need to consider the condition of the substructure. In this case, it is partially failed (two bents). Therefore, due to undermining of the abutments and failed columns at two piers, the condition of the bridge is poor and not serviceable. An estimated cost of work is provided below.

Estimated cost($): $ 21,200

Photo 1: Top view of bridge looking upstation.

Photo 2: Bridge elevation at left side of track. Pier 2 and 3 shown.

Photo 3: Closeup view of pier 2. One column is missing. Bent cap is charred and in poor condition. Column restrainers are in critical condition due to large crack.

Photo 4: Close up of bent cap at Pier 2. Bent cap is clearly charred from a past fire. Columns also appear to be affected.

Photo 5: Other view of Pier 2

Photo 6: Bridge elevation view on right hand side. Pier 3 is shown. Pier 3 is in fair condition. Timber beams are semi continuous.

Photo 8: Abutment 5 with footing exposed and slightly undermined. Timber sills are in fair condition.

Photo 9: Timber girder to abutment 5 connection.

The material in this chapter is written with regard to typical North American Railroad Timber Trestles and other timber structures mentioned herein with

- spans up to 16 feet;
- Standard Gage Track;
- Norman North American passenger and freight equipment;
- Speeds of freight trains up to 80 mph and passenger trains up to 90 mph;

Chapter 7 - Timber Structure

Part 2 - Design of Wood Railway Bridges and Trestles for Railway Loading

Section 2.4 - Designing for Engineered Wood Products

assume timber used in the bridge is: Douglas-fir Grade No.1 (AREMA page 7-2-24)

Table 7-2-9. allowable unit stresses for stress graded lumber - railroad loading (visual grading)

F_b = 1215 psi $F_{c\perp}$ = 380 psi F_V = 150 psi

table 7-2-3. Applicability of Adjustment Factors				
design properties	temperature C_T	beam Stability C_L	Volumn C_V	
$F_b' = F_b*(...)*(...)*(...)$	1.0	1.0	C_V	bending
$F_{c\perp}' = F_{c\perp}*(...)*(...)*(...)$	1.0	none	none	tension
$F_v' = F_v*(...)*(...)*(...)$	1.0	none	none	shear

Note: C_V is [Volumn Factor];

allowable bending stresses of glulam are affected by geometry and size. Generally, larger sizes have a correspondingly lower allowable bending stress than smaller members. To account for this behavior, a volumn factor, C_V, shall be applied. C_V shall not exceed 1.0 and is computed as follows:

$$C_V = \left(\frac{5.125}{b}\right)^p \left(\frac{12}{d}\right)^p \left(\frac{21}{l}\right)^p \leq 1.0$$

where:

32	b = width of bending member in [inches]. For multiple piece width layups, b = width of widest piece in the layup, For practical purposes, b ≤ 10.75 in.;
18	d = depth of bending member in [inches];
15	l = length of bending member between points of zero moment in [feet];
1/10	p = 1/20 for Southern pine and 1/10 for other species; select ==> other species

compute C_V base on above values C_V = 0.82691 -

1 not glulam

beam allowable stresses:

bending stress:	F_b'	=	1215	psi
compression stress ⊥ to grain:	$F_{c\perp}'$	=	380	psi
shear stress:	F_v'	=	150	psi

timber beam section properties:

cross sectional area:	A	=	576	in^2
moment of inertia:	I_{xx}	=	15552	in^4

section modulus:	S_{XX}	=	1728	in^3	
radius of gyration:	r_{XX}	=	5.2	in	
unit weight of timber material:	ω_{timber}	=	60	lb/ft^3	
self weight (dead load) of timber beam:	ω_{beam_DL}	=	0.24	kip/ft	
timber tie beam under rail - width:	W_{tie}	=	0.75	ft	
timber tie beam under rail - depth:	D_{tie}	=	0.5833	ft	
timber tie beam under rail - length:	L_{tie}	=	4.5	ft	
total number of ties on the beam:	N_{tie}	=	10	each	
self weight (dead load) of tie beams:	ω_{tie_DL}	=	0.08	kip/ft	
sectional area of tie brace (highlight yellow):	A_{brace}	=	0.23	ft^2	
self weight (dead load) of tie brace:	ω_{brace_DL}	=	0.02	kip/ft	
track (rails and fastenings) one side:	ω_{track_DL}	=	0.1	kip/ft	
total distributed dead load on beam:	ω_{total}	=	0.44	kip/ft	<<-- input
maximum moment from self weight DL:	M_{DL}	=	49.7655	kip*ft	<<-- from CSi
maximum moment from train LL:	M_{LL}	=	125.0	kip*ft	<<-- Goal Seek
maximum shear from self weight DL:	V_{DL}	=	22.48	kip	<<-- from CSi
maximum shear from train LL:	V_{LL}	=	63.0	kip	<<-- Goal Seek
maximum compression from self weight DL:	p_{DL}	=	176.85	psi	
maximum compression from train LL:	p_{LL}	=	201.3	psi	<<-- Goal Seek
beam stress demands:					
bending stress demand:	F_{b_u}	=	1214	psi	
comp. stress ⊥ to grain demand:	$F_{c\perp\_u}$	=	379	psi	
shear stress demand:	F_{v_u}	=	149	psi	
set stress demand 1 psi less than stress capacity:					
	$\blacktriangle_b$	=	-1	psi	<<-- Goal Seek
	$\blacktriangle_{c\perp}$	=	-1	psi	<<-- Goal Seek
	$\blacktriangle_v$	=	-1	psi	<<-- Goal Seek
frame forces from E80 train live load:			80		
	M_{LL}	=	155.39	kip*ft	
	p_{LL}	=	272	psi	
	V_{LL}	=	84.69	kip	
rating factors for all demand V.S. capacity:					
	RF_b	=	0.80	N/A	
	$RF_{c\perp}$	=	0.74	N/A	
	RF_v	=	0.74	N/A	

capacity V.S. demand:

bending:	☺☻☺ this train is good on bridge
compression stress ⊥ to grain:	☺☻☺ this train is good on bridge
shear:	☺☻☺ this train is good on bridge

the maximum train axial load can go through this bridge is:	59	kip	<<-- *Train Rating*
rating factor for E80 train is:	0.74	N/A	<<-- *Rating Factor*

Section 2.3.8 - Impact (2013) R(2022)

The dynamic increment of load due to the effects of speed, roll and track irregularities is not well established for timber structures. Its totaleffect is setimated to be less than the increased strength of timber for the short cumulative duration of loading to which railroad bridges are subjected in service, and is taken into consideration in the derivation of allowable working stresses for design.

- According to above section, it's unnecessary to take impact and rolling effect into consideration.

PM 141.74

Coordinates for this bridge are 38.683618°N 120.866979°W. This location also has a timber bridge that has a span length of 15’-2” from centerlines of abutment 1 & 2. The abutment seats have a width of 15’-4”. The cross-sectional dimensions of the timber girder are 8” x 16”. There are a total of 8 timber girders. The timber girders are in good condition since no major damages were observed such as cracks or deterioration. No tie keepers were seen. Railroad ties are also in poor condition as some appear to be rotting due to water intrusion. Railroad ties are spaced out at 1’-6”. The timber girders sit on timber sill beams at the abutment’s seat. Abutments 1 and 2 have an exposed height of 4’-6” with an additional 1’-9” seat height. Abutments 1 and 2 also have a width of 1’-1”. Both abutments also have a 1⅛”/ft batter. Abutment 2 does have two cracks at the right wingwall. One of them is a ⅛” horizontal crack and the other is a ⅙” vertical crack. Overall, this bridge is in fair condition as no critical damages to the bridge were observed. The bridge rating for this bridge is serviceable. Due to the condition of the railroad ties, Dokken recommends replacing the ties on the bridge. An estimated cost is provided below.

Estimated cost ($): $6,800.

Photo 1: Top view of bridge looking upstation.

Photo 2: Close up view of top of bridge. Railroad ties are in poor condition. Span did not have tie keepers.

Photo 3: Top view of left side of abutment 1. Shown are the abutment seat and the corner of the abutment and wingwall.

Photo 4: Bridge elevation view, left side of the track. Abutment 1(right) and Abutment 2 (left) shown. Span of bridge equals to 15 feet.

Photo 5: Abutment 1 view. Height of abutment measured as 4'-6". Wall has a batter of 1 ⅛"/ft. (Abutment 2 similar). Abutment 1 has a 3° super elevation and Abutment 2 has a 4° super elevation.

Photo 6: Abutment 2 section. Wall is in fair condition. Timber girders are also in fair condition.

Photo 7: Abutment 2 right wingwall. Has a longitudinal 1/8" crack.

Photo 8: Right side elevation showing timber girders and rail road ties. Ties have unequal spacing. Ties and beams are in fair condition as no signs of cracking or decay were observed.

the material in this chapter is written with regard to typical North American Railroad Timber Trestles and other timber structures mentioned herein with

- spans up to 16 feet;
- Standard Gage Track;
- Norman North American passenger and freight equipment;
- Speeds of freight trains up to 80 mph and passenger trains up to 90 mph;

Chapter 7 - Timber Structure

Part 2 - Design of Wood Railway Bridges and Trestles for Railway Loading

Section 2.4 - Designing for Engineered Wood Products

assume timber used in the bridge is: Douglas-fir Grade No.1 (AREMA page 7-2-24)

Table 7-2-9. allowable unit stresses for stress graded lumber - railroad loading (visual grading)

F_b = 1215 psi $F_{c\perp}$ = 380 psi F_V = 150 psi

table 7-2-3. Applicability of Adjustment Factors				
design properties	temperature	beam Stability	Volumn	
	C_T	C_L	C_V	
$F_b' = F_b*(...)*(...)*(...)$	1.0	1.0	C_V	*bending*
$F_{c\perp}' = F_{c\perp}*(...)*(...)*(...)$	1.0	none	none	*tension*
$F_v' = F_v*(...)*(...)*(...)$	1.0	none	none	*shear*

Note: C_V is [Volumn Factor];

allowable bending stresses of glulam are affected by geometry and size. Generally, larger sizes have a correspondingly lower allowable bending stress than smaller members. To account for this behavior, a volumn factor, C_V, shall be applied. C_V shall not exceed 1.0 and is computed as follows:

$$C_V = \left(\frac{5.125}{b}\right)^p \left(\frac{12}{d}\right)^p \left(\frac{21}{l}\right)^p \leq 1.0$$

where:

32	b = width of bending member in [**inches**]. For multiple piece width layups, b = width of widest piece in the layup, For practical purposes, b ≤ 10.75 in.;
18	d = depth of bending member in [**inches**];
15	l = length of bending member between points of zero moment in [**feet**];
1/10	p = 1/20 for Southern pine and 1/10 for other species; select ==> other species

compute C_V base on above values C_V = 0.82691 -

1 *not glulam*

beam allowable stresses:

bending stress:	F_b'	=	1215	psi
compression stress ⊥ to grain:	$F_{c\perp}'$	=	380	psi
shear stress:	F_v'	=	150	psi

timber beam section properties:

cross sectional area:	A	=	576	in^2
moment of inertia:	I_{xx}	=	15552	in^4

section modulus:	S_{XX}	=	1728	in^3	
radius of gyration:	r_{XX}	=	5.2	in	
unit weight of timber material:	ω_{timber}	=	60	lb/ft^3	
self weight (dead load) of timber beam:	ω_{beam_DL}	=	0.24	kip/ft	
timber tie beam under rail - width:	W_{tie}	=	0.833	ft	
timber tie beam under rail - depth:	D_{tie}	=	0.667	ft	
timber tie beam under rail - length:	L_{tie}	=	4.5	ft	
total number of ties on the beam:	N_{tie}	=	11	each	
self weight (dead load) of tie beams:	ω_{tie_DL}	=	0.12	kip/ft	
sectional area of tie brace (highlight yellow):	A_{brace}	=	0.23	ft^2	
self weight (dead load) of tie brace:	ω_{brace_DL}	=	0.02	kip/ft	
track (rails and fastenings) one side:	ω_{track_DL}	=	0.1	kip/ft	
total distributed dead load on beam:	ω_{total}	=	0.48	kip/ft	<<-- input
maximum moment from self weight DL:	M_{DL}	=	13.53	kip*ft	<<-- from CSi
maximum moment from train LL:	M_{LL}	=	161.2	kip*ft	<<-- Goal Seek
maximum shear from self weight DL:	V_{DL}	=	3.61	kip	<<-- from CSi
maximum shear from train LL:	V_{LL}	=	81.9	kip	<<-- Goal Seek
maximum compression from self weight DL:	p_{DL}	=	10.73	psi	
maximum compression from train LL:	p_{LL}	=	367.6	psi	<<-- Goal Seek
beam stress demands:					
bending stress demand:	F_{b_u}	=	1214	psi	
comp. stress ⊥ to grain demand:	$F_{c\perp\_u}$	=	379	psi	
shear stress demand:	F_{v_u}	=	149	psi	
set stress demand 1 psi less than stress capacity:					
	$\blacktriangle_b$	=	-1	psi	<<-- Goal Seek
	$\blacktriangle_{c\perp}$	=	-1	psi	<<-- Goal Seek
	$\blacktriangle_v$	=	-1	psi	<<-- Goal Seek
frame forces from E80 train live load:			80		
	M_{LL}	=	250	kip*ft	
	V_{LL}	=	80	kip	
	p_{LL}	=	476	psi	
rating factors for all demand V.S. capacity:					
	RF_b	=	0.64	N/A	
	$RF_{c\perp}$	=	0.77	N/A	
	RF_v	=	1.02	N/A	

capacity V.S. demand:

bending:	☺☻☺ this train is good on bridge
compression stress ⊥ to grain:	☺☻☺ this train is good on bridge
shear:	☺☻☺ this train is good on bridge

the maximum train axial load can go through this bridge is:	51	kip	<<-- Train Rating
rating factor for E80 train is:	0.64	N/A	<<-- Rating Factor

Section 2.3.8 - Impact (2013) R(2022)

The dynamic increment of load due to the effects of speed, roll and track irregularities is not well established for timber structures. Its totaleffect is setimated to be less than the increased strength of timber for the short cumulative duration of loading to which railroad bridges are subjected in service, and is taken into consideration in the derivation of allowable working stresses for design.

- According to above section, it's unnecessary to take impact and rolling effect into consideration.

PM 142.23

At this location, a timber bridge was located. The span of this bridge is 4’-6” from CL of the bents at Abutments 1 and 2. The cross-sectional dimensions of the timber girders are 8”x16”. The condition of the girders is fair as no signs of concerning damages were observed. The railroad ties are also in fair condition. The ties on this bridge are spaced out at 1’-2”. The substructure consists of a 13.5” by 11.5” bent cap with a length of 11’-11”. The bent cap at abutments 1 and 2 are in fair condition. Each bent has 5 columns with an approximate 3 foot of spacing. The outer columns are battered at 2¾”/ft. The exposed heights of these columns are 3 feet and are approximately 1 foot wide. Behind the bent caps are timber lagging that have a cross sectional dimension of 3” x 12”. It was observed that the lagging at Abutment 1 is in poor condition since it is lightly damages and fill has been sneaking underneath. The overall condition is fair and is serviceable. However, the lagging at abutment 1 should be fixed to prevent further damages.

Photo 1: Top view of bridge looking upstation.

Photo 2: Close up view of track. Ties are in fair condition. Some minor damages observed due to age of ties. Steel track is also in fair condition.

Photo 3: Bridge elevation view, photo shown to the right of track. Bridge length is 4'-6".

Photo 4: Closeup of the right side of Abutment 2. Timber lagging is in fair condition. Struts are in fair condition due to age.

Photo 3: Closeup of Bent 1 near Abutment 1. One circular timber column, rest are square columns.

Photo 4: Close up view of timber lagging at Abutment 1. Lagging is in poor condition due to damage and movement. A small amount of embankment fill can be seen pushing through lagging.

Photo 5: Closeup of railroad ties, timber girder, and bents. All are in fair condition.

Photo 6: Bent 2 at Abutment 2. All columns are square.