PZSE structural ENGINEERS

STRUCTURAL CALCULATIONS FOR

Coleman - Wild Life Lane Bridge Rating

Site Number CVL02122
Site Name Coleman - Wild Life Lane
Site Type Existing Steel / Wood Bridge
Site Location 13083 Wild Life Lane, Grass Valley, CA 95945

DATE: 1/19/2018

PZSE PROJECT NUMBER: 2017-06533 PH02

REGISTERED PROFESSIONAL ENGINEER
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Exp. 3-31-19
STRUCTURAL
STATE OF CALIFORNIA

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A. SCOPE OF WORK:

Per the Authorization by Borges Architectural Group, Inc. the following structural analysis report by PZSE, Inc. - Structural Engineers (PZSE) was performed to rate the existing bridge structure

B. DESIGN ASSUMPTIONS:

1. Bridge was constructed after 1963.
2. All structural members are in good working condition and have been checked for cracks, rust and signs of failure.
3. Construction utilized industry standards and hardware shall have appropriate installation torque.
4. The existing concrete abutments below the existing bridge structure can support the loads from the bridge.

C. BRIDGE DESCRIPTION

The bridge is composed of the following:

Equip No.	Status	Carrier	Qty.	Appurtenance
1	Exisiting	N/A	3	Steel Railroad Box Girders w/ Laminated Timber Deck
2	Exisiting	N/A	2	Concrete Abutments

D. DESIGN LOADING:

The analysis of the system was performed with the following design criteria:

The existing Bridge structure was analyzed for a design load of an HL93 Design truck per the AASHTO Manual for Bridge Evaluation (MBE) 2011

Loading was applied to the structure in accordance with the appropriate provisions from the following codes:

AASHTO MBE 2011 (LRFD Provisions)
AASHTO Bridge Specifications
AISC 14th Edition (LRFD Provisions)

E. METHOD OF ANALYSIS:

The analysis of the existing bridge structure was conducted using the Steel Construction Manual 14th Edition and analysis methods based on the AASHTO Manual for Bridge Evaluation (LRFD Provisions). PZSE used excel to format and compile the analysis for the bridge rating.

F. RESULTS:

No.	Component of System	RF	Controlling Factor
1	Existing Bridge Structure	1.05	Pass

Notes:

1) RF, (Rating Factor) over 1.00 indicates bridge has sufficient capacity
2) Results of the analyis can be seen in the calculations appendix.
3) Rating Factor is based on site conditions at the time of the site investigation and periodic inspections should be conducted on a bi-annual basis per Section 650.311 of the Federal Register.

G. CONCLUSIONS:

The existing bridge structure is adequate to support the AASHTO Legal Loads from the MBE. A Rating Factor of 1.05 indicates that the bridge has approximately 5% excess capacity for the current configuration of load.

H. ANALYSIS PROVISIONS:

The analysis performed and conclusions/recommendations contained herein assumes that the structure and equipment have been properly built and installed per the original drawings and maintained up to the standard of the current code. It also assumes that there is no structural damage to any component.

The results and conclusions contained within the previous sections of this report were determined by the application of current "state of the art" engineering and analysis design procedures. PZSE assumes no obligation to revise any section of this report in the event that such engineering and analysis procedures are modified or revised.

1. All material and work shall conform to the minimum standards of the AASHTO Manual for Bridge Evaluation and the adopted standards referenced therein; as well as of any other regulating agencies having authority & jurisdiction over any portion of the work; and of these structural notes and specifications.

2. All drawings shall be considered part of the contract documents.

3. Any work performed in conflict with the contract documents or any code requirements shall be corrected by the contractor at his own expense and at no expense to the owner, Architect, or Engineer of Record.

4. The contractor shall verify all dimensions, elevations and site conditions before starting work and shall notify PZSE of any discrepancies.

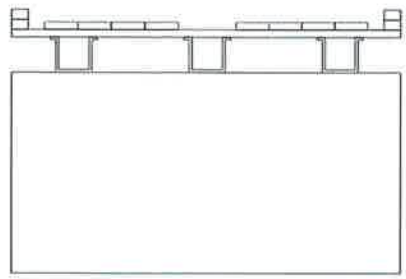
5. All omissions and conflicts between the various elements of the working drawings and/or specifications shall be brought to the attention of PZSE and resolved prior to proceeding with work so involved.

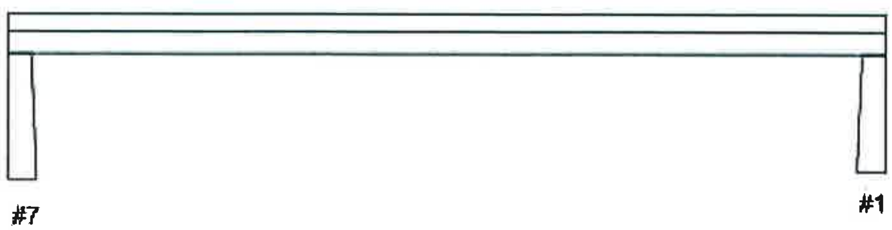
6. Neither the owner nor PZSE will enforce safety measures or regulations. The contractor shall design, construct and maintain all safety devices including shoring and bracing, and shall be solely responsible for conforming to all local, state and federal safety and health standards, laws and regulations. The contract drawings and specifications represent the finished structure and do not indicate methods, procedures or sequence of construction. If a lawsuit is filed by one of the contractor's or subcontractor's employees, or anyone else, the contractor will indemnify, defend and hold the owner and PZSE harmless of any and all such claims.

PROJECT DIMENSIONS

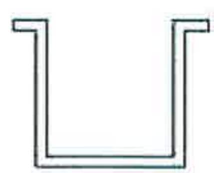
Governing code: ASCE 7-10

Description	Variable	Value	Unit
Bridge Characteristics:			
Bridge Type		Steel Railroad Box Girder	
Decking		Laminated Timber	
Bridge Longitudinal Length	B =	40.5	ft
Bridge Transverse Length	L =	11.8	ft
Deck Overhaging Girders		1.3	ft
Bridge Profiles			





Description	Variable	Value	Unit
Girder Characteristics			
Girder Depth		14	in
Girder Width		13	in
Girder Thickness		1	in
On Center Spacing of Girders		4	ft
Striffeners Present		Yes	
Girder Profile			



Description	Variable	Value	Unit
Abutment Characteristics			
Abutment 1 Height from Grade	h_1 =	6.2	ft
Width of Abutment 1 at Bridge	w_{1b} =	13.5	in
Width of Abutment 1 at Grade	w_{1g} =	16.5	in
Abutment 7 Height from Grade	h_2 =	6.7	ft
Width of Abutment 1 at Top	w_{7b} =	13.5	in
Width of Abutment 1 at Grade	w_{7g} =	16.5	in

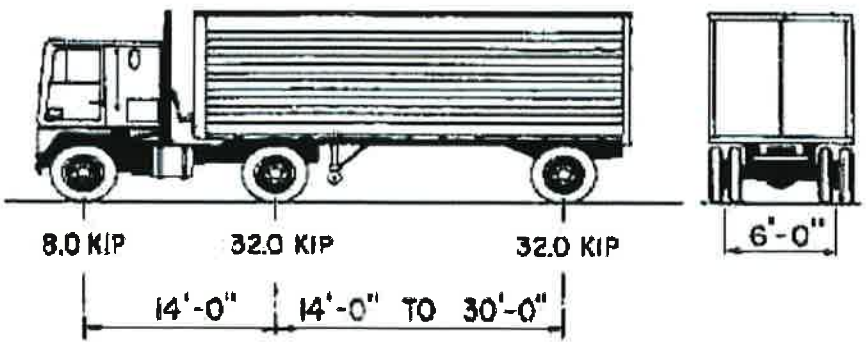
Live Load Determination

Description	Variable	Value	Unit

Live Load Characteristics

The design load per the 2011 AASHTO MBE is the HL-93 Truck. Bridges that rate above one for MBE loading criteria will encompass all of the AASHTO Legal loads and no further analysis is required.

Design Truck Load HL93



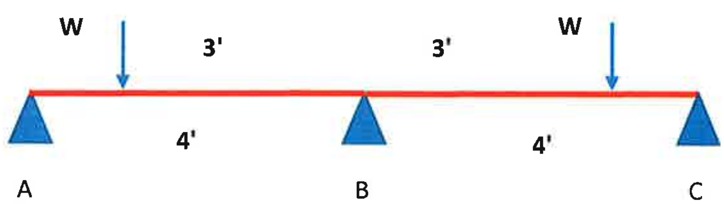
Axel Width	6	ft
Axel Spacing for Worst Case Loading	14	ft

Establishing Live Load Distribution Factor For Bridge

Lever Rule Approach For HL-93 Truck Axel Configuration

For Interior Girder

Interior girder analyzed for truck centerd on bridge



$$R_b = \frac{P \times (4'-3') + P \times (4'-3')}{4}$$

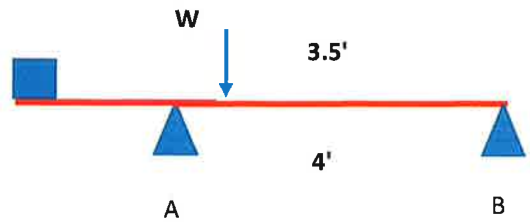
= 0.5 W — W= Weight on Wheel

= 0.25 Lanes — and One Lane (or Axle)= 2 x W

LLDF= 0.25 Lanes for Interior Girder

For Exterior Girder

Exterior Girder Analyzed using Placement of Axel 2'-0" from bridge barrier.



$$R_a = \frac{W \times (4'-0.5')}{4}$$

=	0.875 W	W= Weight on Wheel
=	0.4375 Lanes	and One Lane (or Axle)= 2 x W

LLDF= 0.4375 Lanes for Exterior Girder

Rigid Deck Approach For HL-93 Truck Axel Configuration

$$LLDF\ Exterior = \frac{P}{N_g} + \frac{M\ x\ c}{I_g}$$

Where
- P= Weight on all tires of an axle
- N_g= Number of Girders
- M= Moment Generated by weight of tires about centerline of bridge
- c= Distance from CL to exterior girder
- I_g= Moment of inertia of girders about CL of the bridge

P= 2 x W

N_g= 3

M= (3.5*T)-(.5*T)

= 3 T

c= 4 ft

I_g= $(4.0^2+4.0^2)$

= 32 gir-ft^2

LLDF Exterior=	1.04 W	W= Weight on Wheel
=	0.52 Lanes	and One Lane (or Axle)= 2 x W

Summary of LLDF's and Governing Design LLDF

Interior Girder LLDF Lever Rule	0.25
Exterior Girder LLDF Lever Rule	0.44
Exterior Girder LLDF Rigid Deck Approach	0.52 Case Governs

The lever rule and rigid deck approaches are used to determine the worst case LLDF for Interior and Exterior girders. Since the Interior and Exterior girders have the same section properties, The worst Case LLDF will be used for all girders.

Dead Load Determination

Description	Variable	Value	Unit
Deck Dead Load			
Type of Barrier		4"x6"	Laminated Timber
Barrier Width		4	in
Barrier Depth		6	in
Number of Barriers		2	
Top Layer Surface Type		3"x12"	Laminated Timber
Number of Top Layer Members		4	
Top Layer Member Width		3	in
Top Layer Member Depth		12	in
Deck Substrate Type		3"x12"	Laminated Timber
Substrate Depth		3	in
Substrate Width		141	in
Assumed Specific Gravity		0.5	
Cross Sectional Area of Barriers		0.33	ft^2
Cross Sectional Area of Top Layer		1	ft^2
Cross Sectional Area of Substrate		2.94	ft^2
Deck Load Tributary to Exterior Girders			
Weight of Deck on exterior girder		72.17	plf
Girder Load			
Area of Girder		43	in^2
		0.30	ft^2
Average Density of Steel		490	lb/ft^3
Number of Girders		1	
Weight of Girders		146	plf
Total Dead Load			
Weight of Deck		72	plf
Weight of Girders		146	plf
Total Dead Load		218	plf
Design Dead Load		220	plf

Section Properties: Section1_Girder.dxf

Section Information:

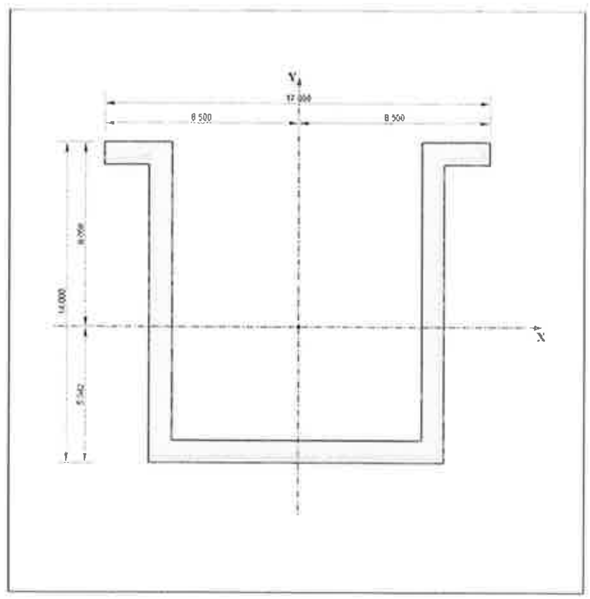
Material Type	=	General	
Shape Type	=	Arbitrary	
Number of Shapes	=	1	

Basic Properties:

Total Width	=	17.000	in
Total Height	=	14.000	in
Centroid, Xo	=	29.463	in
Centroid, Yo	=	10.875	in
X-Bar (Right)	=	8.500	in
X-Bar (Left)	=	8.500	in
Y-Bar (Top)	=	8.058	in
Y-Bar (Bot)	=	5.942	in
Max Thick	=	17.000	in

Equivalent Properties:

Area, Ax	=	43.000	in^2
Inertia, Ixx	=	1044.19	in^4
Inertia, Iyy	=	1347.58	in^4
Inertia, Ixy	=	0.000	in^4
Sx (Top)	=	129.58	in^3
Sx (Bot)	=	175.73	in^3
Sy (Left)	=	158.54	in^3
Sy (Right)	=	158.54	in^3
rx	=	4.928	in
ry	=	5.598	in
Plastic Zx	=	189.38	in^3
Plastic Zy	=	228.25	in^3
Torsional J	=	14.560	in^4
As-xx Def	=	1.000	
As-yy Def	=	1.000	
As-xx Stress	=	1.000	
As-yy Stress	=	1.000	



Section Diagram

Capacity Calculation

Description	Variable	Value	Unit	
FLEXURE				
LRFD Factor for Bending	Φ_b=	0.9		(F6 AISC 14th Ed)

Yeilding (F6 AISC 14th Ed)

$$Mn = Mp = Fy * Zy \leq 1.6\, Fy * Sy$$ (F6-1 AISC 14th Ed)

Yeild Strength	F_y=	36	ksi	
Plastic Modulus	Z_y=	189.36	in^3	(RISA Section)
Section Modulus	S_y=	129.58	in^3	(RISA Section)
	Fy * Zy=	6816.96	kip-in	
	1.6*Fy*Sy=	7463.81	kip-in	

6817 kip-in < 7463.81 Kips

6817 kip-in Governs

Flange Local Buckling (F6 AISC 14th Ed)

Width to Thickness Ration Calculation

Flange Width	b=	14	in	
Flange Thickness	t=	1	in	
Modulus of Elasticity	E=	29000	ksi	
Yeild Strength	F_y=	36	ksi	
Slenderness Ratio	λ=	14		Table B4.1b AISC 14th ed
Compact Ratio Limit	λ_p=	10.7853		Table B4.1b AISC 14th ed
Non Compact Ration Limit	λ_r=	28.3823		Table B4.1b AISC 14th ed

$\lambda > \lambda_p$

$\lambda < \lambda_r$ Therefore Non Compact

$$Mn = (Mp - (Mp - .7FySY) * (\lambda - \lambda_p / (\lambda_r - \lambda_p))$$

M_n= 6443.38 kip-in

Moment Capacity

Governing Flexural Capacity	M_n=	6443.38	kip-in	
		536.949	kip-ft	
	$M_n * \Phi_b$=	483.254	kip-ft	(F6 AISC 14th Ed)

SHEAR

LRFD Factor for Shear	Φ_v=	0.9		(G5 AISC 14th Ed)

Shear (G5 AISC 14th Ed)

$$Vn = 0.6 * Fy * Aw * Cv$$ (G2-1 AISC 14th ed)

Aw=	28	
h=	14	inches
h/t_w=	14	
k_v=	5	
SQRT(kv*E/Fy)=	63.46	
1.1*SQRT(kv*E/Fy)=	69.8113	
1.37*SQRT(kv*E/Fy)=	86.9467	
C_v=	1	
V_n=	604.8	kips

Shear Capacity

$V_n*\Phi_v$=	544.32	kips	(G5 AISC 14th Ed)

Bridge Rating Calculation

Description	Variable	Value	Unit

Inventory Level Loading

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_{LL})(LL + IM)}$$

(6A.4.2.1-1)

Flexure

Description	Variable	Value	Unit
Capacity	C=	483.25	kip-ft
Dead Load Factor	γ_D=	1.25	
Dead Load Effect	D=	45.1	kip-ft
Live Load Factor	γ_{LL}=	1.35	
Live Load Effect	LL=	302	kip-ft
Condition Factor	φ_c=	1	
System Factor	φ_s=	1	
	RF=	1.0464	

Shear

Description	Variable	Value	Unit
Capacity	C=	544.32	Kips
Dead Load Factor	γ_D=	1.25	
Dead Load Effect	D=	4.5	Kips
Live Load Factor	γ_{LL}=	1.35	
Live Load Effect	LL=	23.073	Kips
Condition Factor	φ_c=	0.95	
System Factor	φ_s=	1	
	RF=	16.423	

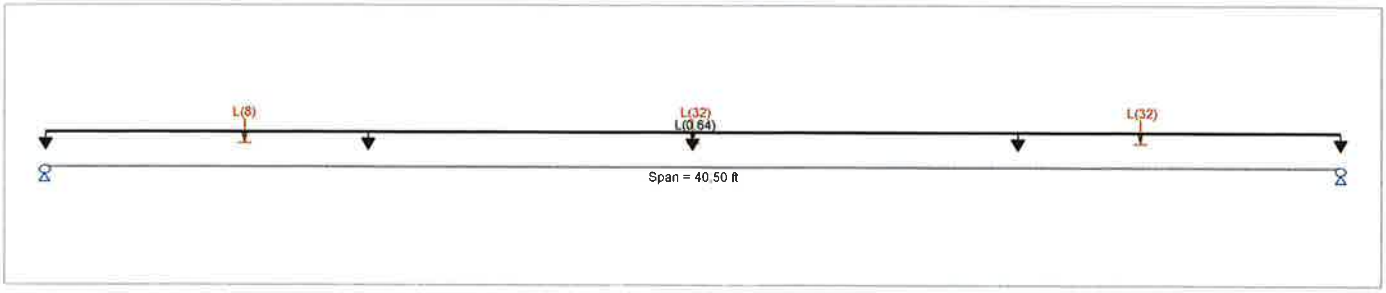
General Beam Analysis

Description : Unfactored Live Load Calculation

General Beam Properties

Elastic Modulus 29,000.0 ksi

Span #1 Span Length = 40.50 ft Area = 10.0 in^2 Moment of Inertia = 100.0 in^4



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Load(s) for Span Number 1
- Point Load : L = 8.0 k @ 6.250 ft
- Point Load : L = 32.0 k @ 20.250 ft
- Point Load : L = 32.0 k @ 34.250 ft
- Uniform Load : L = 0.640 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Maximum Bending =	580.220 k-ft	**Maximum Shear =**	57.256 k
Load Combination	+D+L+H	Load Combination	+D+L+H
Location of maximum on span	20.250ft	Location of maximum on span	40.500 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	54.977 in	8
Max Upward Transient Deflection	0.000 in	0
Max Downward Total Deflection	54.977 in	8
Max Upward Total Deflection	0.000 in	0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
L Only	1	54.9771	20.655		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	40.664	57.256
Overall MINimum	30.498	42.942
+D+H		
+D+L+H	40.664	57.256
+D+Lr+H		
+D+S+H		
+D+0.750Lr+0.750L+H	30.498	42.942
+D+0.750L+0.750S+H	30.498	42.942
+D+0.60W+H		
+D+0.70E+H		
+D+0.750Lr+0.750L+0.450W+H	30.498	42.942
+D+0.750L+0.750S+0.450W+H	30.498	42.942
+D+0.750L+0.750S+0.5250E+H	30.498	42.942
+0.60D+0.60W+0.60H		
+0.60D+0.70E+0.60H		
D Only		
Lr Only		
L Only	40.664	57.256
S Only		
W Only		
E Only		
H Only		

General Beam Analysis

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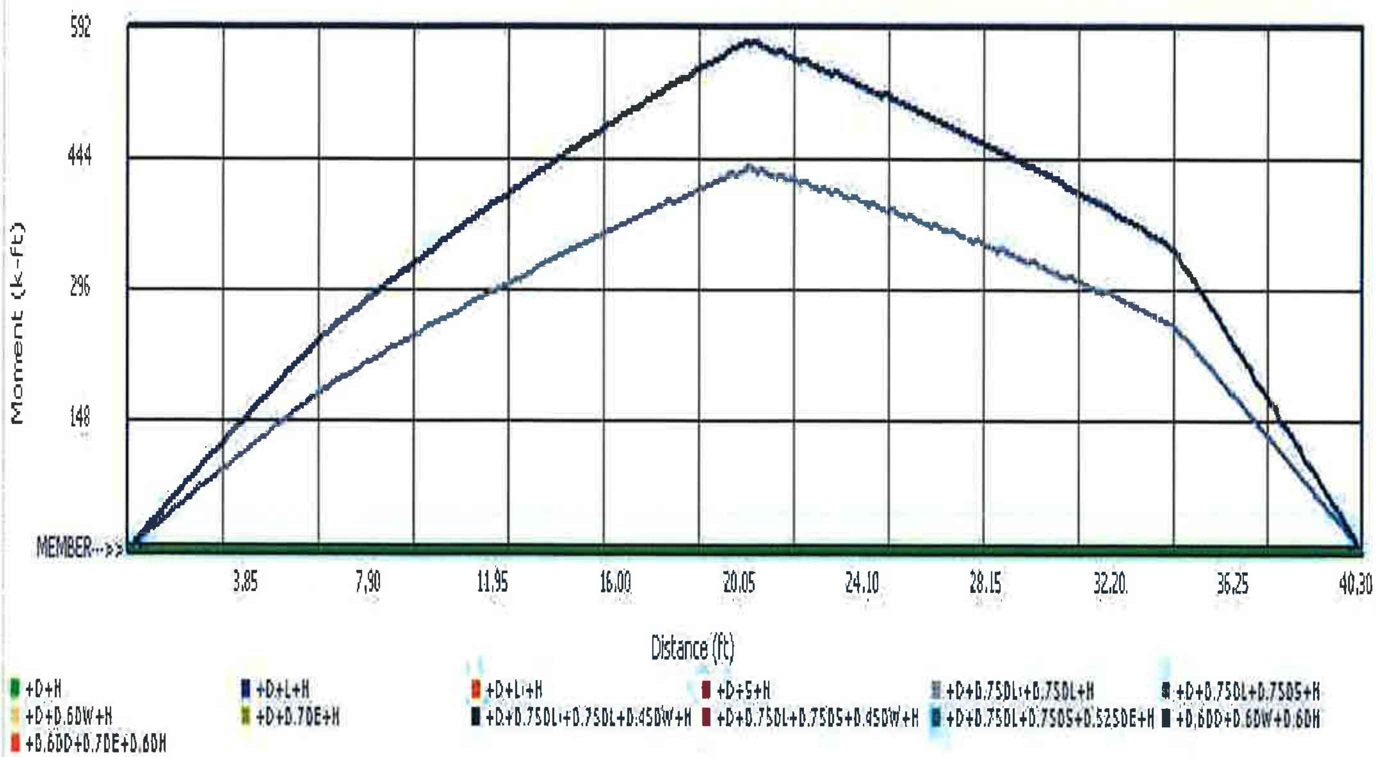
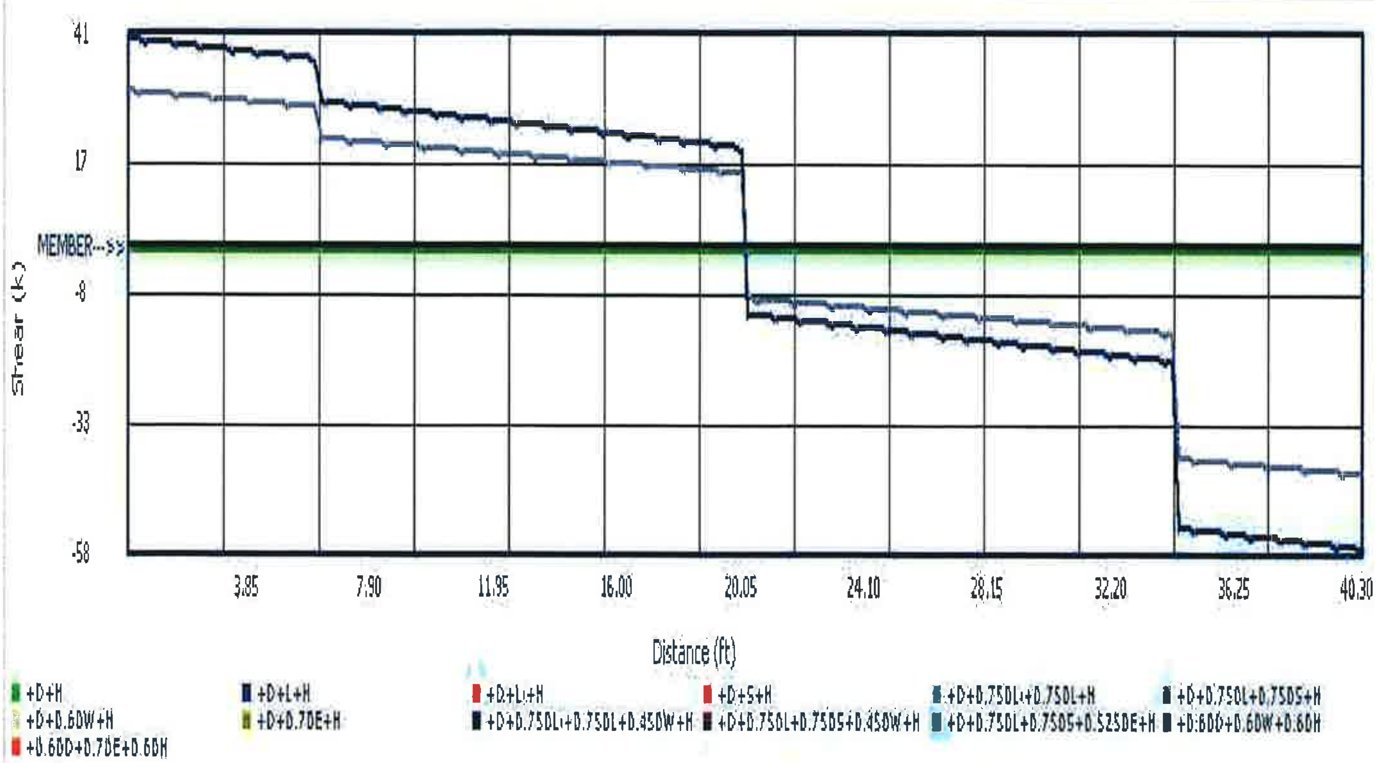
Description : Unfactored Live Load Calculation





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General Beam Analysis

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