RENAUD BROS., INC.

283 Fort Bridgeman Road #2, Vernon, VT 05354 **phone (802) 257-7383**
fax (802) 257-7308

June 8, 2016

Ann Gammell, PE
Vermont Agency of Transportation
Southeast Regional Construction Office
P.O. Box 1837
Wilder, VT 05088

RE: **Brattleboro BF 2000 (26) – Work Platform**

Dear Ann:

We have addressed the notes and revised the plans and calculations.

1. Explain how the deck concrete will be removed. If concrete debris will be dropped on to the work platform during deck removal, explain what will be done to make sure that the design load is not exceeded. The work platform is not intended to be a debris platform. The existing deck will be saw cut and removed with excavators in 10'x10' pieces. If large pieces drop to the platform they will be removed immediately however only small debris is expected to fall on the scaffolding. Once an area is opened up and safe for crews to get on the scaffolding it will be cleaned off. If we find the deck is not coming off in big pieces then we will have to adjust our removal procedure.

2. Provide breakdown of the dead and live loads. Do these values account for the truss self-weight and railing weight? See calculations attached. Is the load from the demolition of the concrete deck included in the live or dead load? The platform is good for 50psf heavy debris will not be on the word platform. A load summary of the components of the 20psf and the 50 psf loads should be provided to verify that the loads are receiving the appropriate load factors. See calculations attached.

3. The factored load for the bar joists is greater than the factored capacity. It is not clear why the factored load is reduced for comparison to the factored capacity. Please explain. See calculations attached.

4. Provide the reactions at the connection points to the bridge beams. Provide calculations to show that the bridge beams have adequate capacity to support the work platform load including calculations for the stresses in the bottom flange at the connection points. See calculations attached.

5. Will the work platform be removed prior to pouring the deck? The work platform will not be removed prior to placing the deck. The whole purpose of the platform is for installing and removing the deck forms.

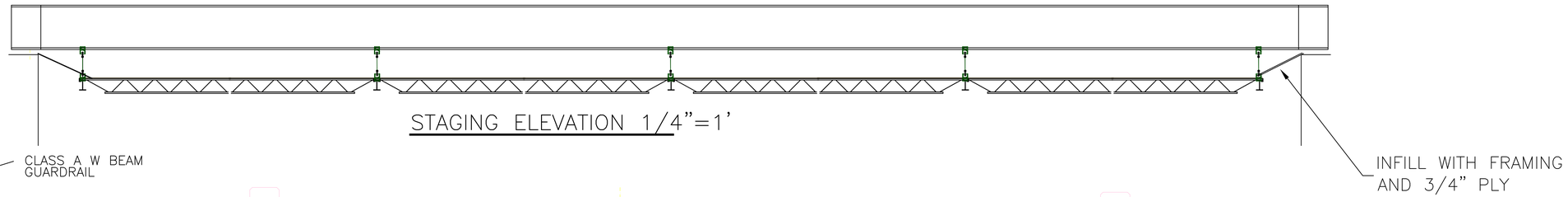
6. The calculations show 2 – 2x10s for the railing resisting the lateral 50 plf, the plan drawing shows these members as 2- 2x6s. See new drawing and calculations.

7. The railing height shown on the plans is less than the 42” standard. See new drawings.

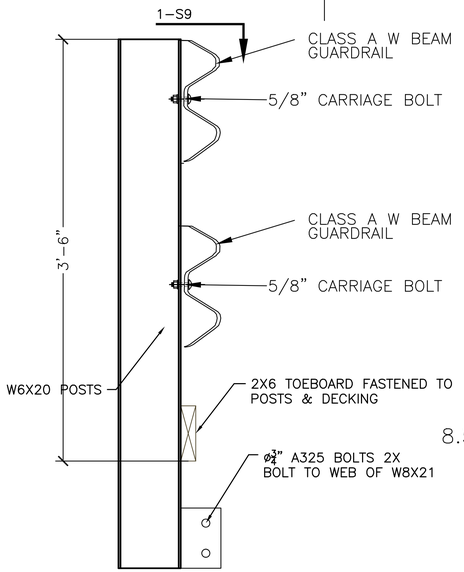
Sincerely,

Renaud Bros., Inc.

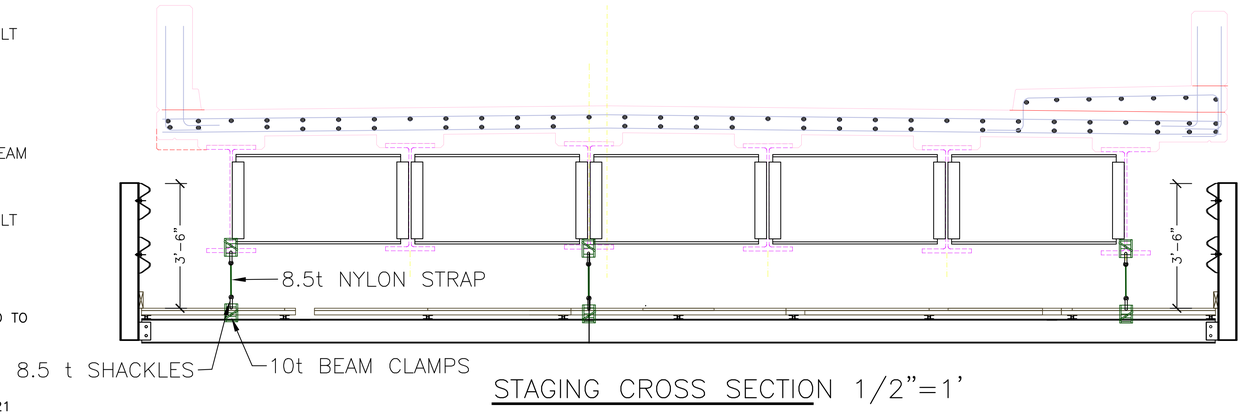
Charlie Ezequelle
Project Manager



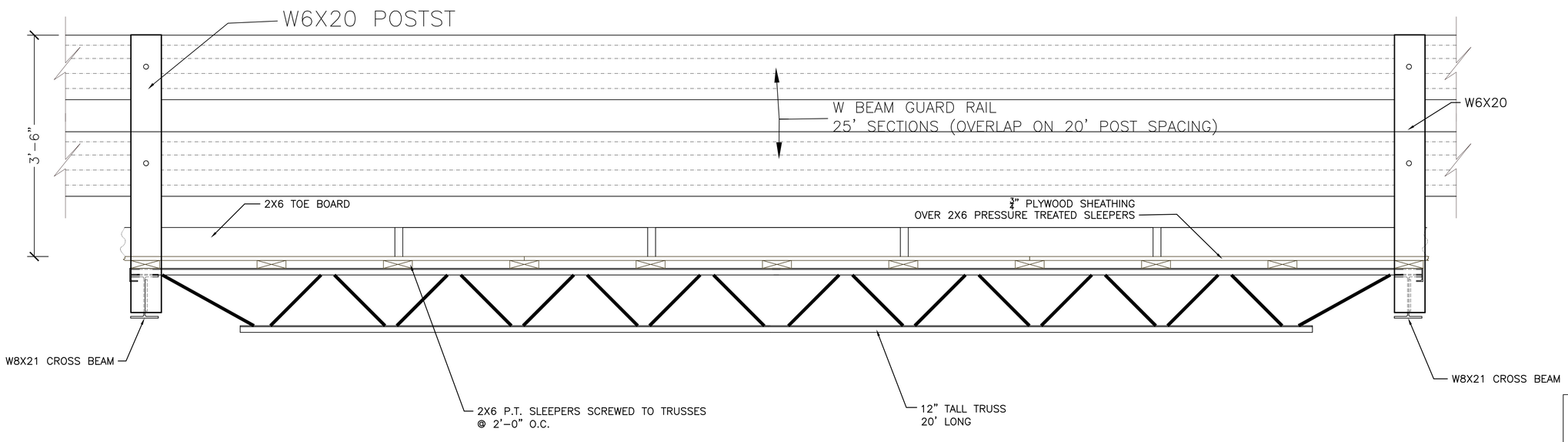
STAGING ELEVATION 1/4"=1'



RAIL POST DETAIL



STAGING CROSS SECTION 1/2"=1'



STAGING PARTIAL ELEVATION NTS

BELL ENGINEERING
CIVIL & ENVIRONMENTAL
17 ECHO COVE WAY-SPOFFORD, NH 03462
(603) 363-9966
bellengineering@myfairpoint.net

REVISIONS:

6-01-2016	modify guard rail member and height

OWNER: BRATTLEBORO BF 2000 (26)
ELLIOT STREET
DECK REPLACEMENT

SHEET TITLE: STAGING PLAN

SCALE: AS NOTED

DATE: MAY 13, 2016

SHEET 1

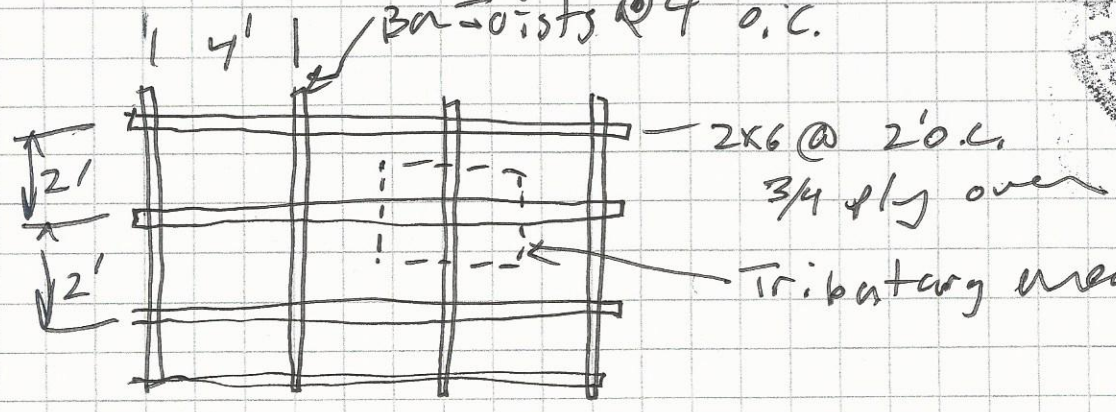
STATE OF VERMONT
RONALD K. BELL
No. 8694
REGISTERED
PROFESSIONAL ENGINEER

BELL ENGINEERING
CIVIL & ENVIRONMENTAL
17 ECHO COVE WAY
SPOFFORD, NEW HAMPSHIRE 03462
(603) 363-9966

PROJECT NAME/ LOCATION: Brattleboro BF 2000 (26)
SHEET #: 1 OF 13
CALCULATED BY: Ron Bell DATE: 5-25-2016
CHECKED BY: DATE:
SCALE:

STATE OF VERMONT
RONALD K. BELL
No. 6934
REGISTERED PROFESSIONAL ENGINEER

Scaffolding Loads:



Bar Joists = 5 #/lin ft × 2 ft = 10 #/8 ft²

2X6 sleeper = 1.5" × 5.5" = 8.25 in² × 48" = 396 in³

396 in³ × 0.023 #/in³ = 9.1 #/4 ft

3/4 Plywood: 8 ft² × 3 #/ft² = 24.0 #/8 ft²

Total per 8 ft² trib area = 43.1 # = 5.4 #/ft²

with hardware assume $\boxed{6.0\ \#/ft^2}$ for conservative estimate

---

Liveloads:

For light duty with no debris on deck + handtools use 25 #/ft²

Check if scaffolding will carry medium duty (50 psf) which would allow some debris.

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CIVIL & ENVIRONMENTAL
17 ECHO COVE WAY
SPOFFORD, NEW HAMPSHIRE 03462
(603) 363-9966

PROJECT NAME/ LOCATION: Brattleboro BF 2000 (26)
SHEET #: 2 OF 13
CALCULATED BY: Ron Bell DATE: 5-25-2010
CHECKED BY: DATE:
SCALE:

Scaffolding total load:

$(1.2)(6.0) + (1.6)(50) = 87.2$ psf

At 4.0' o.c spacing the uniform load per bar joist is 348.8 #/lin ft.

Note the bar joists are older joists in good condition. So to be conservative use 1994 bar joists catalog and convert to LRFD: 241 #/lin ft × 0.9 × 1.65 = 357.9 #/lin ft

357.9 >> 348.8 OK (see attached table + LRFD conversion guidance)

Following pages

STATE OF VERMONT
RONALD K. BELL
No. 6994
REGISTERED
PROFESSIONAL ENGINEER

3/13

STANDARD LOAD TABLE
OPEN WEB STEEL JOISTS, K-SERIES

Based on a Maximum Allowable Tensile Stress of 30,000 psi

Adopted by the Steel Joist Institute November 4, 1985; Revised to May 2, 1994 - Effective September 1, 1994.

The black figures in the following table give the TOTAL safe uniformly distributed load-carrying capacities, in pounds per linear foot, of K-Series Steel Joists. The weight of DEAD loads, including the joists, must be deducted to determine the LIVE load-carrying capacities of the joists. The load table may be used for parallel chord joists installed to a maximum slope of 1/2 inch per foot.

The figures shown in RED in this load table are the LIVE loads per linear foot of joist which will produce an approximate deflection of 1/360 of the span. LIVE loads which will produce a deflection of 1/240 of the span may be obtained by multiplying the figures in RED by 1.5. In no case shall the TOTAL load capacity of the joists be exceeded.

The approximate joist weights per linear foot shown in these tables do not include accessories.

The approximate moment of inertia of the joist, in inches^4 is: $I = 26.767(W_{LL})(L^3)(10^{-6})$, where W_{LL} = RED figure in the Load Table and L = (Span - .33) in feet.

For the proper handling of concentrated and/or varying loads, see Section 5.5 in the Recommended Code of Standard Practice.

Where the joist span is equal to or greater than the span corresponding to the RED shaded area shown in the load table, the row of bridging nearest the mid span of the joist shall be installed as bolted diagonal bridging. Hoisting cables shall not be released until this bolted diagonal bridging is completely installed.

JOIST DESIGNATION	8K1	10K1	12K1	12K3	12K5	14K1	14K3	14K4	14K6	16K2	16K3	16K4	16K5	16K6	16K7	16K9
DEPTH (IN.)	8	10	12	12	12	14	14	14	14	16	16	16	16	16	16	16
APPROX. WT. (lbs./ft.)	5.1	5.0	5.0	5.7	7.1	5.2	6.0	6.7	7.7	5.5	6.3	7.0	7.5	8.1	8.6	10.0
SPAN (ft.) ↓																
8	550 550															
9	550 550															
10	550 480	550 550														
11	532 377	550 542														
12	444 288	550 455	550 550	550 550	550 550											
13	377 225	479 363	550 510	550 510	550 510											
14	324 179	412 289	500 425	550 463	550 463	550 550	550 550	550 550	550 550							
15	281 145	358 234	434 344	543 428	550 434	511 475	550 507	550 507	550 507							
16	246 119	313 192	380 282	476 351	550 396	448 390	550 467	550 467	550 467	550 550	550 550	550 550	550 550	550 550	550 550	550 550
17		277 159	336 234	420 291	550 366	395 324	495 404	550 443	550 443	512 488	550 526	550 526	550 526	550 526	550 526	550 526
18		246 134	299 197	374 245	507 317	352 272	441 339	530 397	550 408	456 409	508 456	550 490	550 490	550 490	550 490	550 490
19		221 113	268 167	335 207	454 269	315 230	395 287	475 336	550 383	408 347	455 386	547 452	550 455	550 455	550 455	550 455
20		199 97	241 142	302 177	409 230	284 197	356 246	428 287	525 347	368 297	410 330	493 386	550 426	550 426	550 426	550 426
21			218 123	273 153	370 198	257 170	322 212	388 248	475 299	333 255	371 285	447 333	503 373	548 405	550 406	550 406
22			199 106	249 132	337 172	234 147	293 184	353 215	432 259	303 222	337 247	406 289	458 323	498 351	550 385	550 385
23			181 93	227 116	308 150	214 128	268 160	322 188	395 226	277 194	308 216	371 252	418 282	455 307	507 339	550 363
24			166 81	208 101	282 132	196 113	245 141	295 165	362 199	254 170	283 189	340 221	384 248	418 269	465 298	550 346
25						180 100	226 124	272 145	334 175	234 150	260 167	313 195	353 219	384 238	428 263	514 311
26						166 88	209 110	251 129	308 156	216 133	240 148	289 173	326 194	355 211	395 233	474 276
27						154 79	193 98	233 115	285 139	200 119	223 132	268 155	302 173	329 188	366 208	439 246
28						143 70	180 88	216 103	265 124	186 106	207 118	249 138	281 155	306 168	340 186	408 220
29										173 95	193 106	232 124	261 139	285 151	317 167	380 198
30										161 86	180 96	216 112	244 126	266 137	296 151	355 178
31										151 78	168 87	203 101	228 114	249 124	277 137	332 161
32										142 71	158 79	190 92	214 103	233 112	259 124	311 147

STEEL JOIST INSTITUTE MEMBER

4/13

The following method may be used to convert the Steel Joist Institute's Specifications for use in Load and Resistance Factor Design (LRFD)

Method:

WU = 1.65 Wsji, or Wsji = WU/1.65

Where, WU = ultimate joist capacity
Wsji = SJI Load Table Load (black figure)

Load tables for LRFD can be obtained directly from the current SJI Load Tables by using the formula:

Wn = Wsji x 0.9 x 1.65

Where, Wn = normal joist capacity
0.9 = Resistance Factor (ø)

"K" Series Example:

Given: WU = 1.2 WD + 1.6 WL
Problem: Select a joist from the current load tables for Wsji ≥ Wu/1.65 (ø)

L = 40 ft.
WD = 50 plf
WL = 150 plf
Use Roof Live load deflection ≤ L/240

WU = 1.2 x 50 + 1.6 x 150 = 300 plf

Wsji ≥ 300/(1.65 x 0.9) = 202 plf

Select 22K6: Wsji @ 40 ft. span = 207 plf > 202 plf. O'K'

From Table

Deflection Live Load ≤ L/240

WsjiLL = 1.5 x 111 = 166 plf > 150 plf O'K'

L/360 to L/240 conversion

The above procedure outlines the specification of a "K" Series Joist to support a uniform gravity load util LRFD. When loads other than uniform gravity loads (such as wind uplift loads, concentrated loads, end mom or non-uniform loads) are a design consideration, the Specifying Professional shall clearly indicate on structural drawings whether these loads are factored or unfactored. To remain consistent with established L design procedures it is recommended that factored loads be specified.

The above procedure is also applicable to the LH/DLH Series Joists and Joist Girders.

BELL ENGINEERING
CIVIL & ENVIRONMENTAL
17 ECHO COVE WAY
SPOFFORD, NEW HAMPSHIRE 03462
(603) 363-9966

PROJECT NAME/ LOCATION: Brattleboro 2000 (26)
SHEET #: 5 OF 13
CALCULATED BY: Ron Bell DATE: 5-25-2016
CHECKED BY: DATE:
SCALE:

CHECK GUARD RAIL W-BEAM

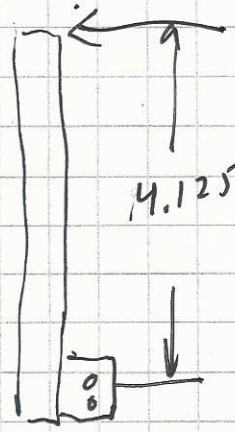
$W_u = 1.6\ W_L = (1.6)(50\ \#/ft) = 80\ \#/ft$

$M_u = \frac{w_u l^2}{8} = \frac{(80)(20)^2}{8} = 4\ \text{Kip ft}$

$S_{req} = \frac{M_u}{\phi F_y} = \frac{4\ \text{Kips-ft}\ (12)}{(0.9)(50)} = 1.06\ in^3$

S of a class A W Beam = 1.39 in^3 OK

CHECK COLUMN BEAM CONNECTION



$(50\ \#/ft)(20')(1.6) = 1600\ \#$

$M = 6600 - .167P - .167P = 0$

$-3.34P = -6600$

$P = 1976$

From Table 7-1 a 3/4 Group A Bolt in Single Shear has a shear capacity of 22.5 KIPS >>> 2 KIPS OK

Tensil load on Bolts in Post = 1976/2 = 988 #/Bolt which is much less than 29.8 From Table 2 OK

STATE OF VERMONT
RONALD K. BELL
No. 8934
REGISTERED PROFESSIONAL ENGINEER

BELL ENGINEERING
CIVIL & ENVIRONMENTAL
17 ECHO COVE WAY-SPOFFORD, NH 03462
(603) 363-9966

Project Title: Brattleboro 2006 (24)
Engineer: Ron Bell
Project Descr:
Project ID:

GUARD RAIL POSTS 6/13

Printed: 8 JUN 2016, 8:02AM

Steel Beam

File = c:\Users\Ron\DOCUME~1\ENERCA~1\EL19D6~1.EC6
ENERCALC, INC. 1983-2013, Build:6.13.8.31, Ver:6.13.12.31

Lic. # : KW-06009396 Licensee : Bell Engineering

Description : W6x20 post @ 20' o.c.

CODE REFERENCES

Calculations per AISC 360-05, IBC 2006, CBC 2007, ASCE 7-05
Load Combination Set : ASCE 7-05

Material Properties

Analysis Method : Load Resistance Factor Design
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling
Bending Axis : Major Axis Bending
Load Combination ASCE 7-05

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi

STATE OF VERMONT RONALD K. BELL No. 8894 REGISTERED PROFESSIONAL ENGINEER

Span = 4.0 ft

W6x20

Applied Loads

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

Load(s) for Span Number 1
Point Load : L = 1.0 k @ 0.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio = 0.114 : 1
Section used for this span W6x20
Mu : Applied 6.400 k-ft
Mn * Phi : Allowable 56.250 k-ft
Load Combination +1.20D+0.50Lr+1.60L+1.60H
Location of maximum on span 4.000 ft
Span # where maximum occurs Span # 1

Maximum Shear Stress Ratio = 0.033 : 1
Section used for this span W6x20
Vu : Applied 1.60 k
Vn * Phi : Allowable 48.360 k
Load Combination +1.20D+0.50Lr+1.60L+1.60H
Location of maximum on span 0.000 ft
Span # where maximum occurs Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.031 in	Ratio =	3,114
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.031 in	Ratio =	3115
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.0308	0.000		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum		1.000
L Only		1.000
D+L		1.000

Steel Section Properties : W6x20

Depth	=	6.200	in	I xx	=	41.40	in^4	J	= 0.240 in^4
Web Thick	=	0.260	in	S xx		13.40	in^3	Cw	= 113.00 in^6
Flange Width	=	6.020	in	R xx	=	2.660	in		
Flange Thick	=	0.365	in	Zx	=	15.000	in^3		
Area	=	5.870	in^2	I yy	=	13.300	in^4		
Weight	=	19.981	plf	S yy	=	4.410	in^3	Wno	= 8.780 in^2
Kdesign	=	0.615	in	R yy	=	1.500	in	Sw	= 4.820 in^4
K1	=	0.563	in	Zy	=	6.720	in^3	Qf	= 3.070 in^3
rts	=	1.700	in	rT	=	1.640	in	Qw	= 7.380 in^3
Ycg	=	3.100	in						

BELL ENGINEERING
CIVIL & ENVIRONMENTAL
17 ECHO COVE WAY-SPOFFORD, NH 03462
(603) 363-9966

Project Title: Brattleboro 2000 (26)
Engineer:
Project Descr:
Project ID: 7113

Printed: 8 JUN 2016, 8:02AM

Steel Beam

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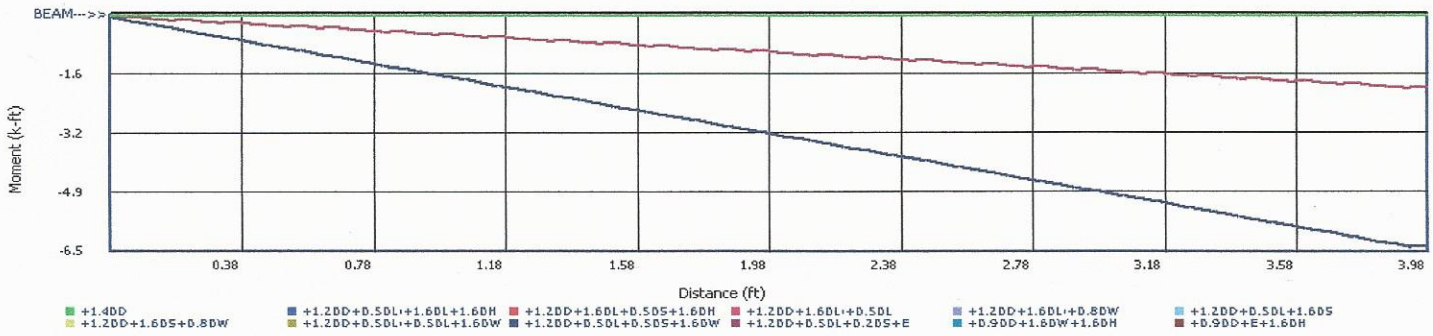
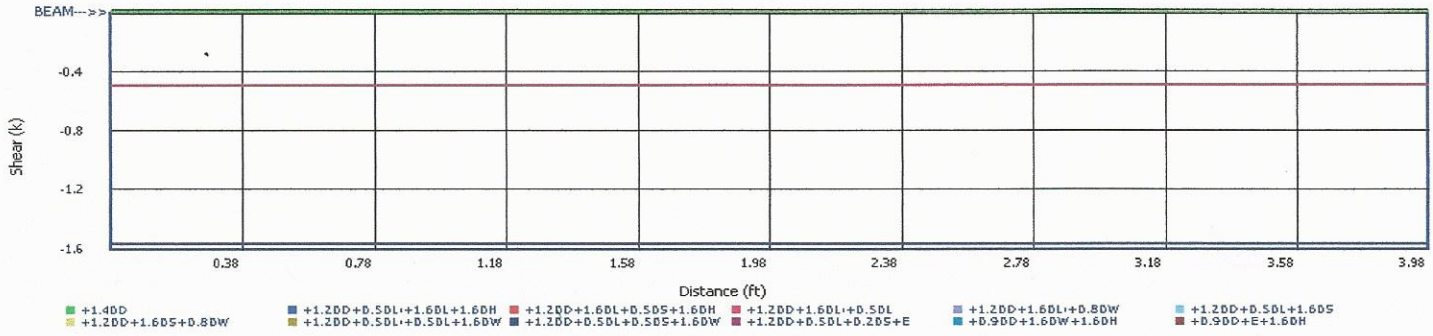
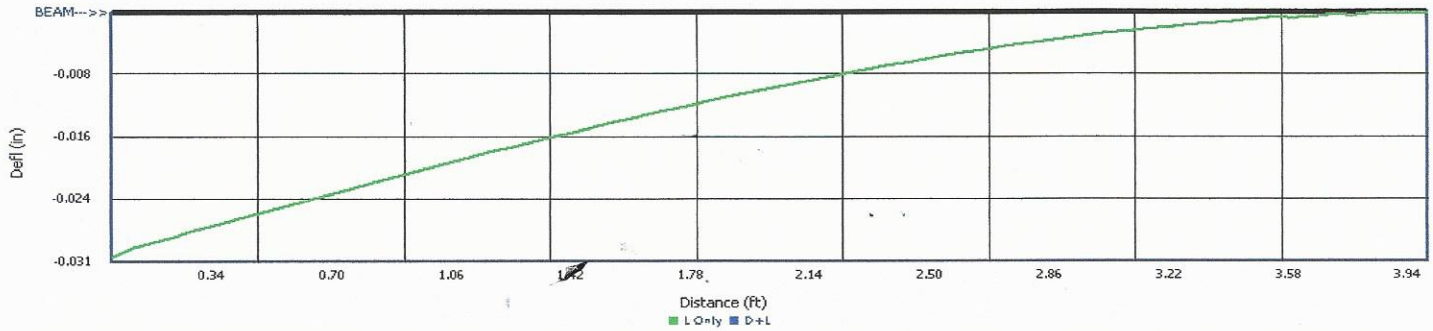
ENERCALC, INC. 1983-2013, Build:6.13.8.31, Ver:6.13.12.31

Lic. # : KW-06009396 | Licensee : Bell Engineering

Description : W6x20 post @ 20' o.c.







STATE OF VERMONT
RONALD K. BELL
No. 6594
REGISTERED PROFESSIONAL ENGINEER

BELL ENGINEERING
CIVIL & ENVIRONMENTAL
17 ECHO COVE WAY
SPOFFORD, NEW HAMPSHIRE 03462
(603) 363-9966

PROJECT NAME/ LOCATION: Brattleboro 2000 (26)
SHEET #: 8 OF 12
CALCULATED BY: Ron Bell DATE: 5-25-2016
CHECKED BY: DATE:
SCALE:

CHECK TO MAKE SURE THAT EXISTING BRIDGE BEAMS WILL CARRY LOAD:
CONCRETE DECK LOADS CALCULATED BY CALCULATING CROSS SECTIONAL TRIBUTARY AREA ABOVE BEAMS (SEE ENERCAL CALCULATIONS) NOTE: PAVEMENT IS TO BE REMOVED BEFORE LOADING SCAFFOLDING

No form work shall be placed on the scaffolding until the existing bridge deck is removed

BENDING + SHEAR OK

CHECK FLANGE LOCAL BENDING:

AISC eq. J10-1 $R_n = 6.25 t^2 F_{ys}$

WORST CASE IS CENTER BEAM FOR SCAFFOLDING POINT LOAD:

$$P_u = (1.2)(1.5\ \text{KIPS}) + (1.6)(12.5\ \text{KIPS}) = 21.8\ \text{KIPS}$$

$$R_n = (6.25)(1.35)^2\left(36\ \frac{\text{Kips}}{\text{in}^2}\right)$$

$$= 410.1\ \text{KIPS}$$

$\phi R_n = (0.9)(410.1\ \text{KIPS}) = 369.1$ KIPS WHICH IS MUCH GREATER THAN 21.8 KIPS OK

STATE OF VERMONT
RONALD K. BELL
No. 8994
REGISTERED PROFESSIONAL ENGINEER

BELL ENGINEERING
CIVIL & ENVIRONMENTAL
17 ECHO COVE WAY-SPOFFORD, NH 03462
(603) 363-9966

Project Title: BRATTLEBORO 2000 (26)
Engineer: RONALD K. BELL, PE
Project Descr: BELOW DECK STAGING

Project ID: 2016-026

CENTER BEAM

~~7/12~~ 9/13

Printed: 25 MAY 2016, 12:24PM

Steel Beam

File = C:\Users\Ron\DOCUME~1\ENERCA~1\ELLIOT~2.EC6
ENERCALC, INC. 1983-2013, Build:6.13.8.31, Ver:6.13.12.31

Lic. # : KW-06009396 Licensee : Bell Engineering

Description : EXISTING 36X245 CENTER BEAM

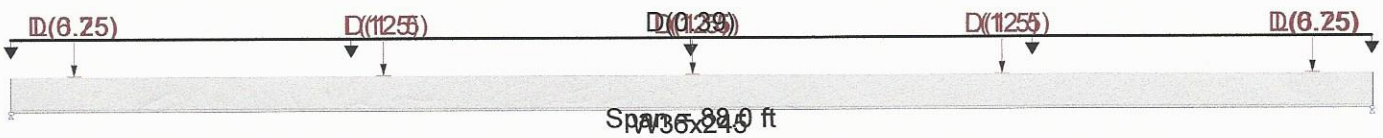
CODE REFERENCES

Calculations per AISC 360-05, IBC 2006, CBC 2007, ASCE 7-05
Load Combination Set : ASCE 7-05

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling
Bending Axis : Major Axis Bending
Load Combination ASCE 7-05

Fy : Steel Yield : 36.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

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

Uniform Load : D = 0.390 k/ft, Tributary Width = 1.0 ft, (CONC. DECK)
Point Load : D = 0.750 k @ 4.10 ft, (DEAD LOAD STAGING)
Point Load : D = 1.50 k @ 24.10 ft, (DEAD LOAD STAGING)
Point Load : D = 1.50 k @ 44.10 ft, (DEAD LOAD STAGING)
Point Load : D = 1.50 k @ 64.10 ft, (DEAD LOAD STAGING)
Point Load : D = 0.750 k @ 84.10 ft, (DEAD LOAD STAGING)
Point Load : L = 6.250 k @ 4.10 ft, (LIVE LOAD STAGING)
Point Load : L = 12.50 k @ 24.10 ft, (LIVE LOAD STAGING)
Point Load : L = 12.50 k @ 44.10 ft, (LIVE LOAD STAGING)
Point Load : L = 12.50 k @ 64.10 ft, (LIVE LOAD STAGING)
Point Load : L = 6.250 k @ 84.10 ft, (LIVE LOAD STAGING)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.578 : 1	Maximum Shear Stress Ratio =	0.109 : 1
Section used for this span	W36x245	Section used for this span	W36x245
Ma : Applied	1,048.820 k-ft	Va : Applied	45.224 k
Mn / Omega : Allowable	1,814.371 k-ft	Vn/Omega : Allowable	415.642 k
Load Combination	+D+L+H	Load Combination	+D+L+H
Location of maximum on span	44.000 ft	Location of maximum on span	88.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	1.727 in	Ratio =	611
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	3.071 in	Ratio =	344
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	3.0713	44.440		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	45.096	45.224
D Only	20.153	20.167
L Only	24.943	25.057
D+L	45.096	45.224

STATE OF VERMONT
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CIVIL & ENVIRONMENTAL
17 ECHO COVE WAY-SPOFFORD, NH 03462
(603) 363-9966

Project Title: BRATTLEBORO 2000 (26)
Engineer: RONALD K. BELL, PE
Project Descr: BELOW DECK STAGING

Project ID: 2016-026

~~8/12~~ 10/13

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Licensee : Bell Engineering

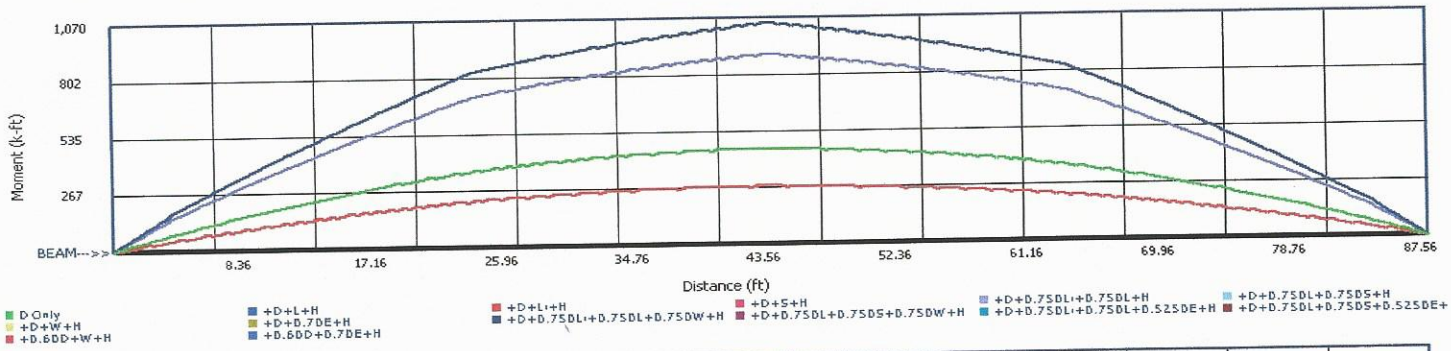
Steel Beam

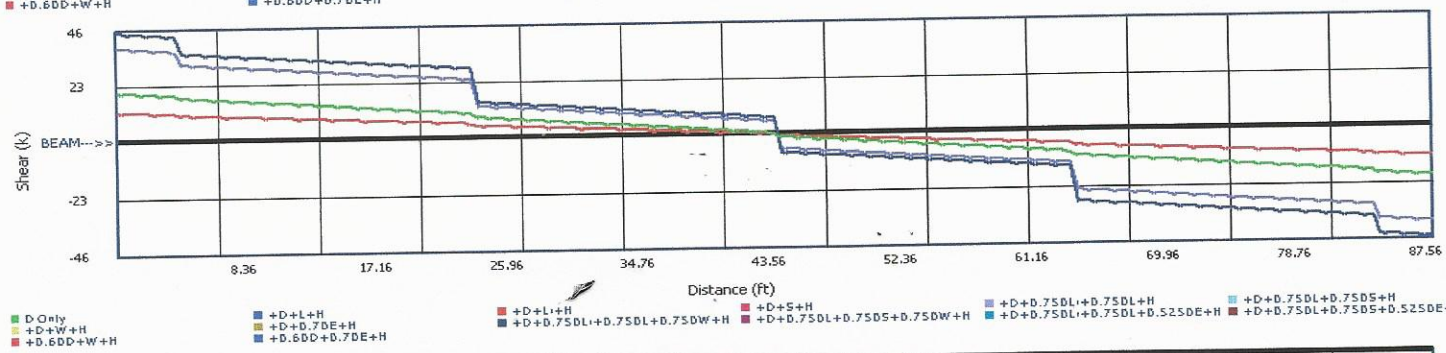
Lic. # : KW-06009396

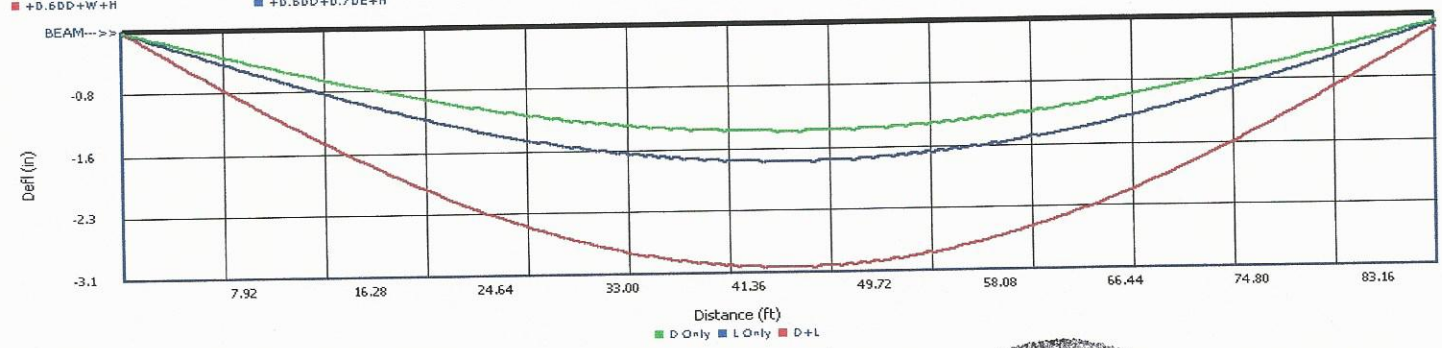
Description : EXISTING 36X245 CENTER BEAM

Steel Section Properties : W36x245

Depth	=	36.080 in	I xx	=	16,100.00 in^4	J	=	34.600 in^4
Web Thick	=	0.800 in	S xx		895.00 in^3	Cw	=	#####.## in^6
Flange Width	=	16.510 in	R xx	=	15.000 in			
Flange Thick	=	1.350 in	Zx	=	1010.000 in^3			
Area	=	72.100 in^2	I yy	=	1,010.000 in^4			
Weight	=	245.428 plf	S yy	=	123.000 in^3	Wno	=	143.000 in^2
Kdesign	=	2.300 in	R yy	=	3.750 in	Sw	=	799.000 in^4
K1	=	1.625 in	Zy	=	190.000 in^3	Qf	=	184.000 in^3
rts	=	4.372 in	rT	=	4.320 in	Qw	=	505.000 in^3
Ycg	=	18.040 in						







STATE OF VERMONT
RONALD K. BELL
No. 6984
REGISTERED PROFESSIONAL ENGINEER

BELL ENGINEERING
CIVIL & ENVIRONMENTAL
17 ECHO COVE WAY-SPOFFORD, NH 03462
(603) 363-9966

Project Title: BRATTLEBORO 2000 (26)
Engineer: RONALD K. BELL, PE
Project Descr: BELOW DECK STAGING
Project ID: 2016-026

END BEAM

11/13

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Steel Beam

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Lic. # : KW-06009396 Licensee : Bell Engineering

Description : EXISTING 36X245 END BEAM

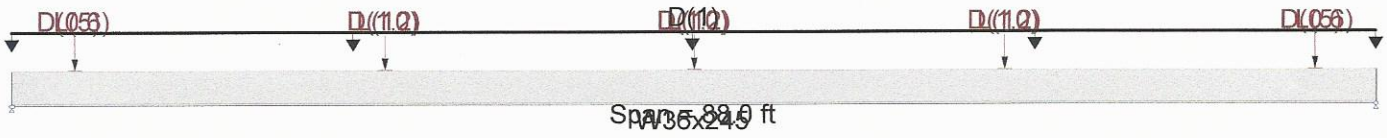
CODE REFERENCES

Calculations per AISC 360-05, IBC 2006, CBC 2007, ASCE 7-05
Load Combination Set : ASCE 7-05

Material Properties

Analysis Method : Load Resistance Factor Design
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling
Bending Axis : Major Axis Bending
Load Combination ASCE 7-05

Fy : Steel Yield : 36.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 1.0 k/ft, Tributary Width = 1.0 ft, (CONC DECK AND RAILING)
Point Load : D = 0.60 k @ 4.10 ft, (DEAD LOAD STAGING)
Point Load : D = 1.20 k @ 24.10 ft, (DEAD LOAD STAGING)
Point Load : D = 1.20 k @ 44.10 ft, (DEAD LOAD STAGING)
Point Load : D = 1.20 k @ 64.10 ft, (DEAD LOAD STAGING)
Point Load : D = 0.60 k @ 84.10 ft, (DEAD LOAD STAGING)
Point Load : L = 5.0 k @ 4.10 ft, (STAGING LIVE LOAD)
Point Load : L = 10.0 k @ 24.10 ft, (STAGING LIVE LOAD)
Point Load : L = 10.0 k @ 44.10 ft, (STAGING LIVE LOAD)
Point Load : L = 10.0 k @ 64.10 ft, (STAGING LIVE LOAD)
Point Load : L = 5.0 k @ 84.10 ft, (STAGING LIVE LOAD)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.837 : 1	Maximum Shear Stress Ratio =	0.162 : 1
Section used for this span	W36x245	Section used for this span	W36x245
Mu : Applied	2,282.938 k-ft	Vu : Applied	100.718 k
Mn * Phi : Allowable	2,727.000 k-ft	Vn * Phi : Allowable	623.46 k
Load Combination	+1.20D+0.50Lr+1.60L+1.60H	Load Combination	+1.20D+0.50Lr+1.60L+1.60H
Location of maximum on span	44.000 ft	Location of maximum on span	88.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	1.382 in	Ratio =	763
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	5.176 in	Ratio =	204
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	5.1762	44.440		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	77.148	77.250
D Only	57.193	57.204
L Only	19.955	20.045
D+L	77.148	77.250

STATE OF VERMONT
RONALD K. BELL
No. 6[illegible]
REGISTERED PROFESSIONAL ENGINEER

BELL ENGINEERING
CIVIL & ENVIRONMENTAL
17 ECHO COVE WAY-SPOFFORD, NH 03462
(603) 363-9966

Project Title: BRATTLEBORO 2000 (26)
Engineer: RONALD K. BELL, PE
Project Descr: BELOW DECK STAGING
Project ID: 2016-026

12/13

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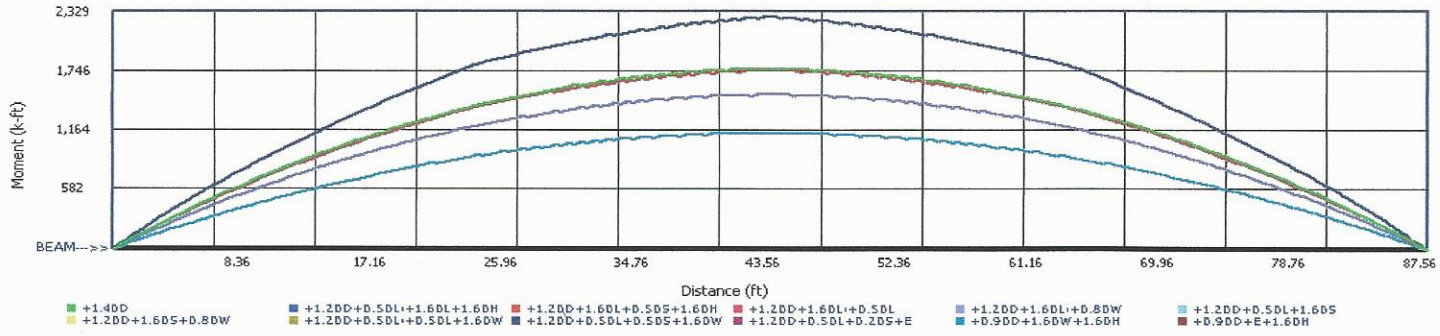

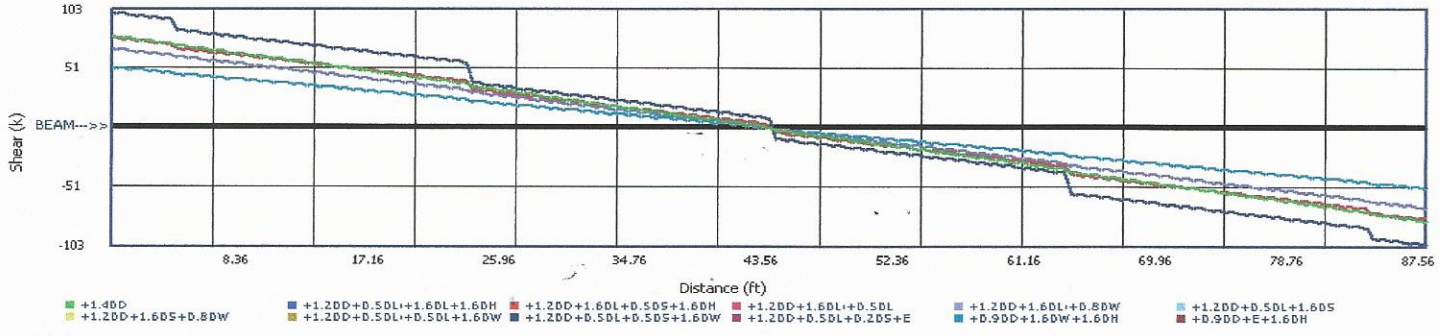

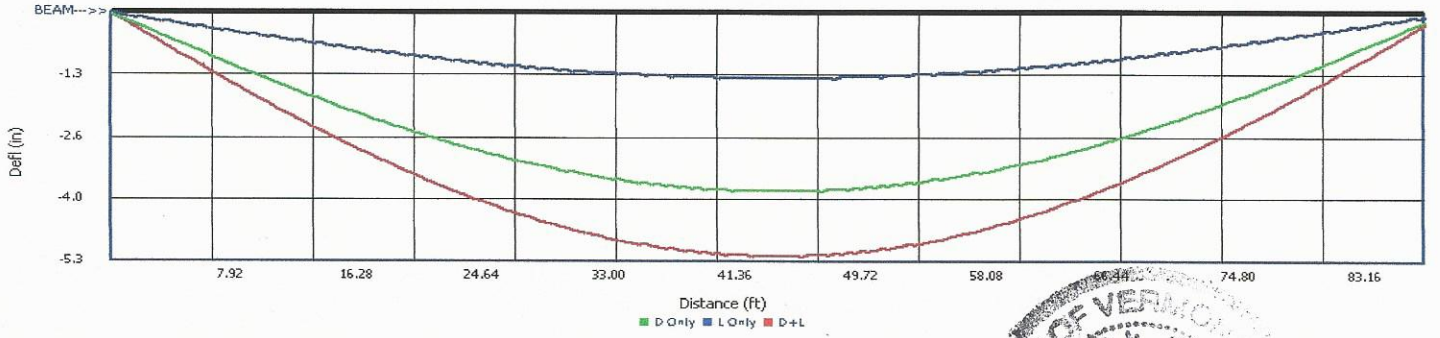

STATE OF VERMONT
RONALD K. BELL
No. 6924
REGISTERED
PROFESSIONAL ENGINEER

BELL ENGINEERING
CIVIL & ENVIRONMENTAL
17 ECHO COVE WAY
SPOFFORD, NEW HAMPSHIRE 03462
(603) 363-9966

PROJECT NAME/ LOCATION: Brattleboro BF 2000 (26)
SHEET #: ~~11~~ 13 OF ~~12~~ 13
CALCULATED BY: Ron Bell DATE: 5-25-16
CHECKED BY: DATE:
SCALE:

CHECK the web local yielding:

Since the point of application > d distance from beam end use eq AISC J10-2:

$$R_n = (5K + N) F_y t_w$$

$$= \cancel{(5)(1.625)(36)}$$

$$= [(5)(1.62) + 6''](36)(1.35) = 685.3 \text{ KIPS}$$

$$\phi R_n = 1.0 \times 685.3 = 685.3 \text{ KIPS} \gg 21.8 \text{ KIPS OK}$$

STATE OF VERMONT
RONALD K
No. 6994
REGISTERED
PROFESSIONAL ENGINEER