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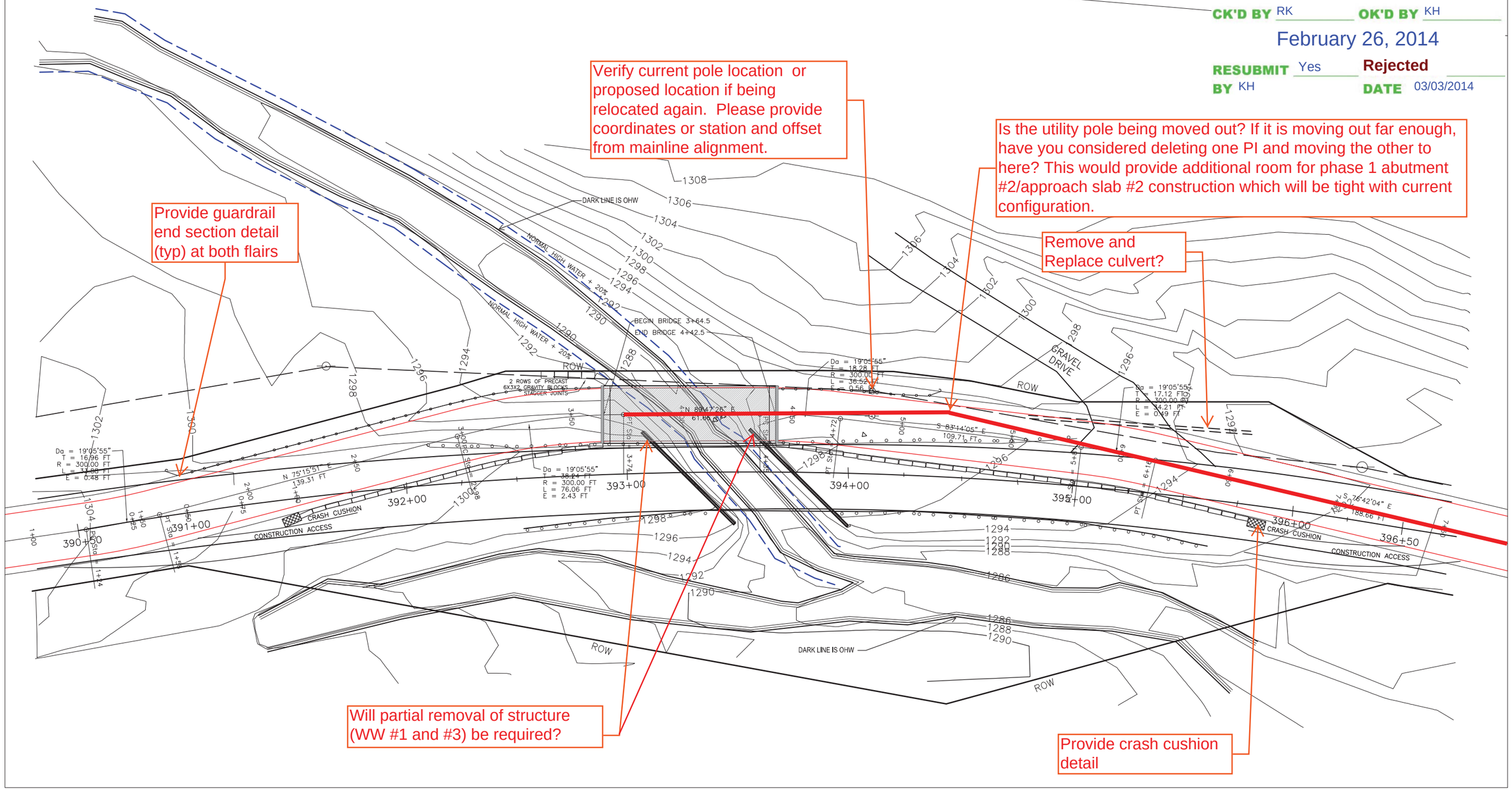
February 26, 2014

RESUBMIT Yes Rejected

BY KH DATE 03/03/2014



PLAN OF TEMPORARY BRIDGE, ABUTMENTS & BYPASS ROAD
BRF 010-1(43)
ROUTE 9
MARLBORO, VT

DATE: FEB. 20, 2014 SHEET: 1

STATE OF VERMONT RONALD K. BELL No. 6004 REGISTERED PROFESSIONAL ENGINEER

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

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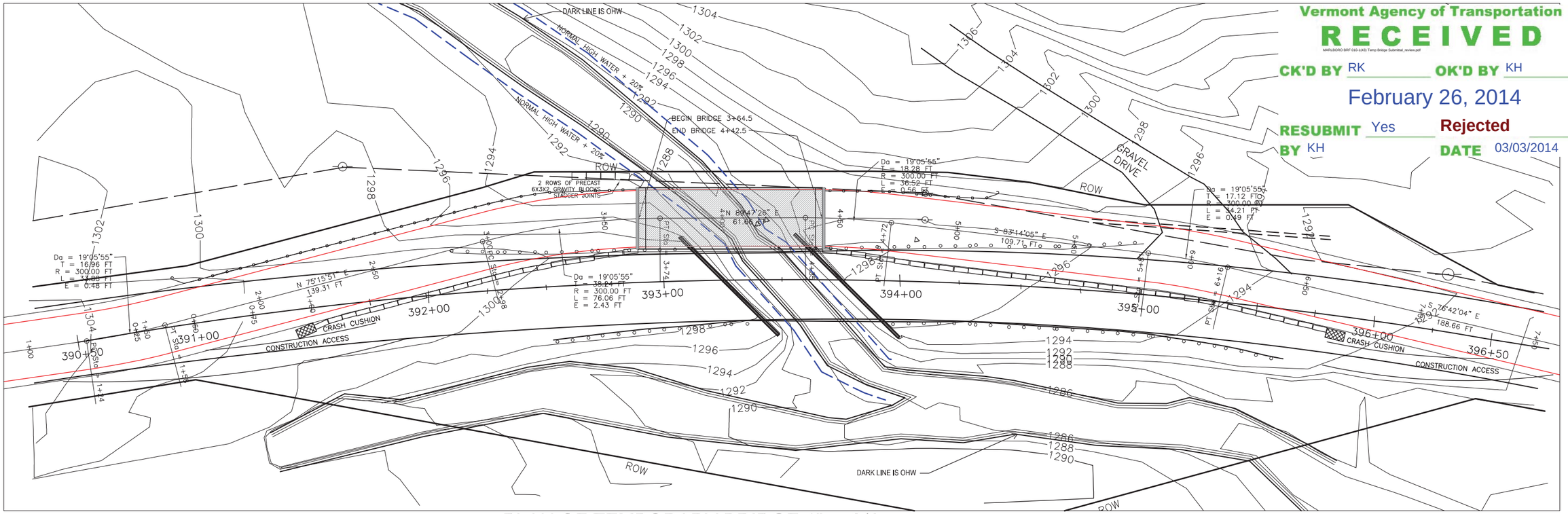
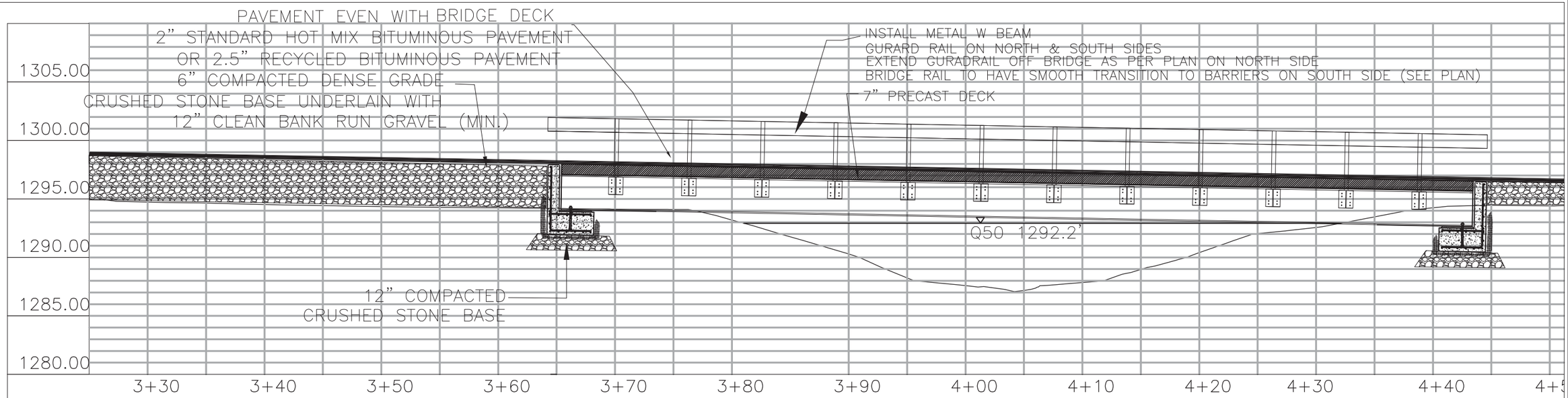


PLAN OF TEMPORARY BRIDGE 1" = 20'



PROFILE OF TEMPORARY BRIDGE 1" = 4'

PLAN AND PROFILE OF TEMPORARY BRIDGE, ABUTMENTS & BYPASS ROAD

RENAUD BROS. INC.
BRF 010-1(43)
ROUTE 9
MARLBORO, VT

DATE: 2-20-2014

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

SHEET 2

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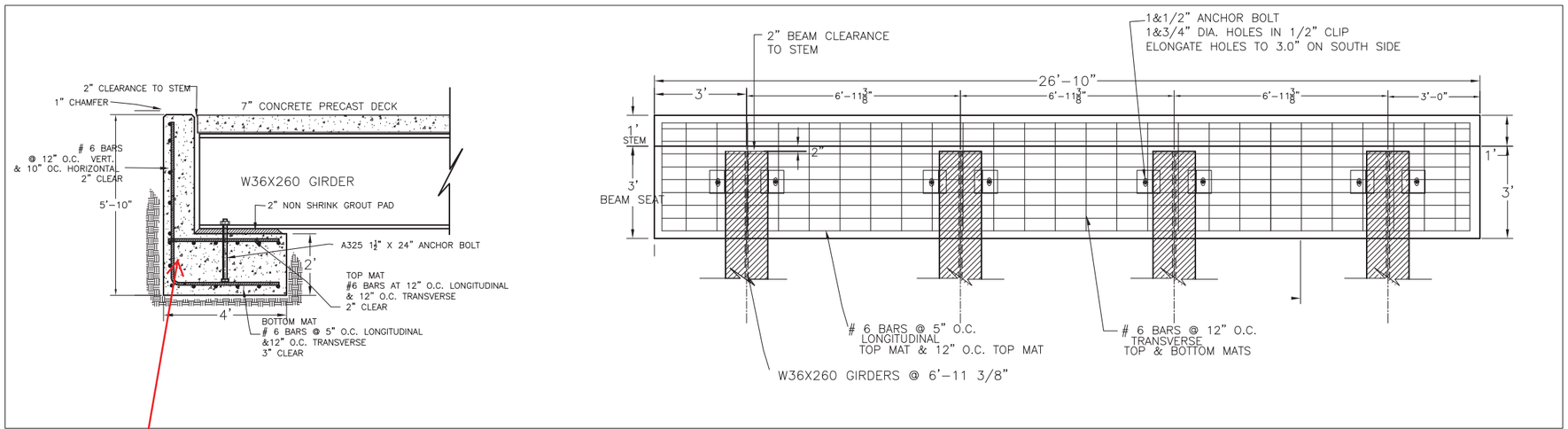
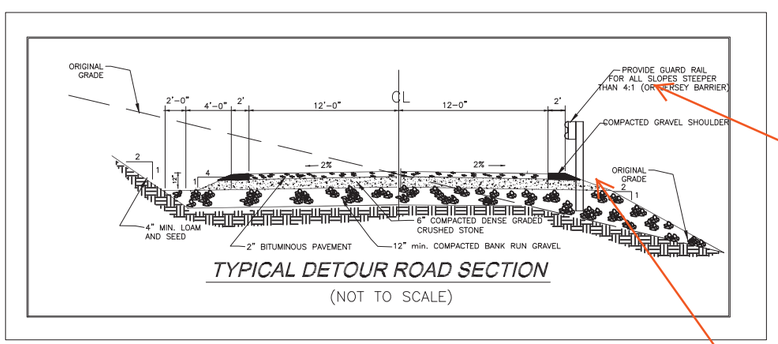
Provide abutment material notes

Provide Bridge seat elevations and bottom of substructure elevations.



ABUTMENT DETAILS 1" = 2'

Is this going to be built in one piece as shown?



TYPICAL DETOUR ROAD SECTION

(NOT TO SCALE)

NOTE: SEE PLANS BY RENAUD BROTHERS CONSTRUCTION FOR CROSS SECTION AND PLAN VIES OF TEMPORARY BRIDGE.

Provide w-beam guardrail dimensions (height of rail, post length, post section, etc.) and jersey barrier dimensions if used.

Verify that adequate buffer is provided for guardrail system.

STATE OF VERMONT RONALD K. RUEL No. 8004 PROFESSIONAL ENGINEER

DETAILS: TEMPORARY BRIDGE

BRF 010-1(43)
ROUTE 9
MARLBORO, VT

DATE: 2-20-2014	

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

SHEET 3

Vermont Agency of Transportation

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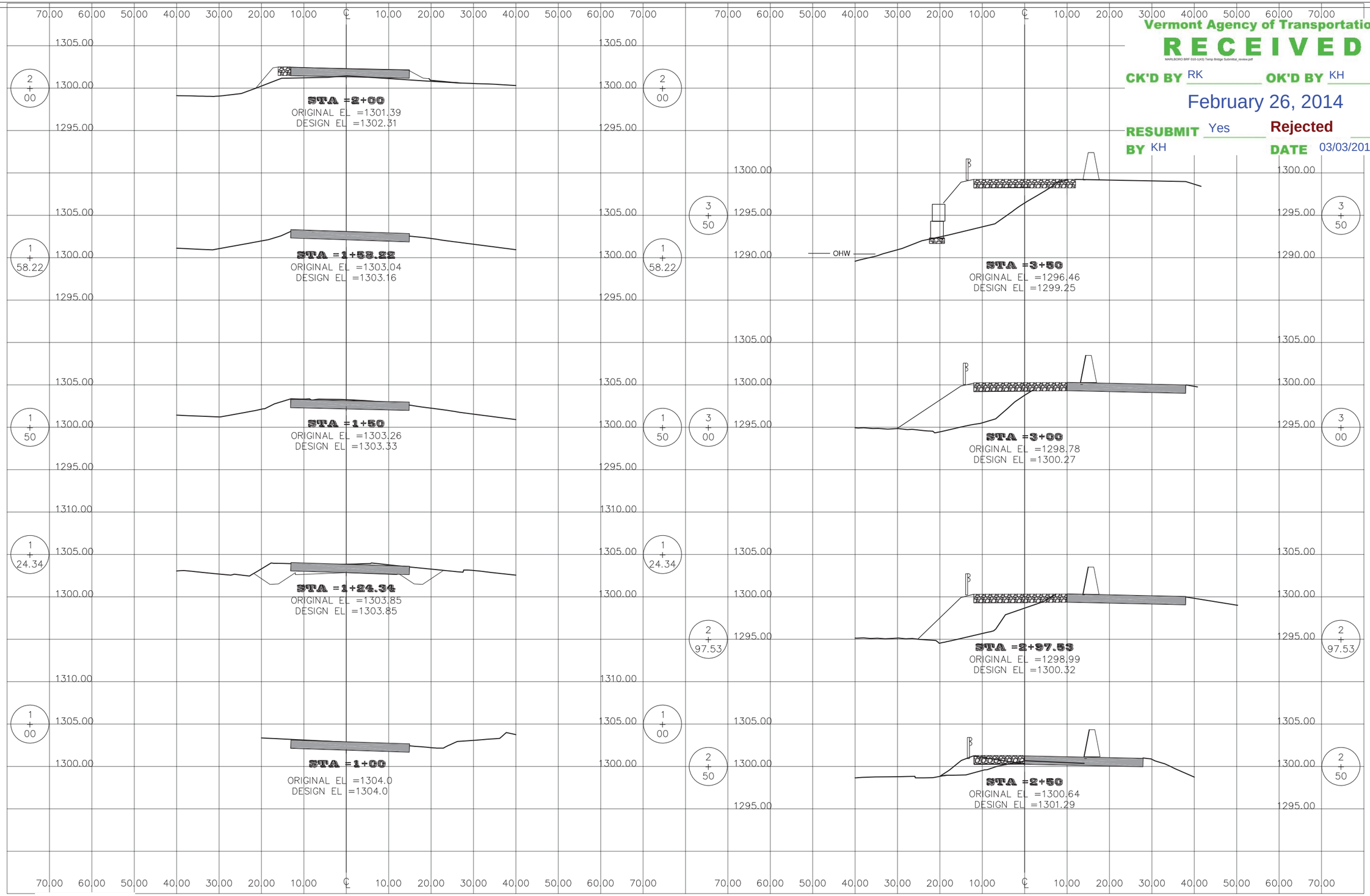

STATE OF VERMONT RONALD R. REED No. 8204 PROFESSIONAL ENGINEER

CROSS SECTIONS

RENAUD BROS. INC.
BRF 010-1(43)
ROUTE 9
MARLBORO, VT

DATE: 2-20-2014	

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

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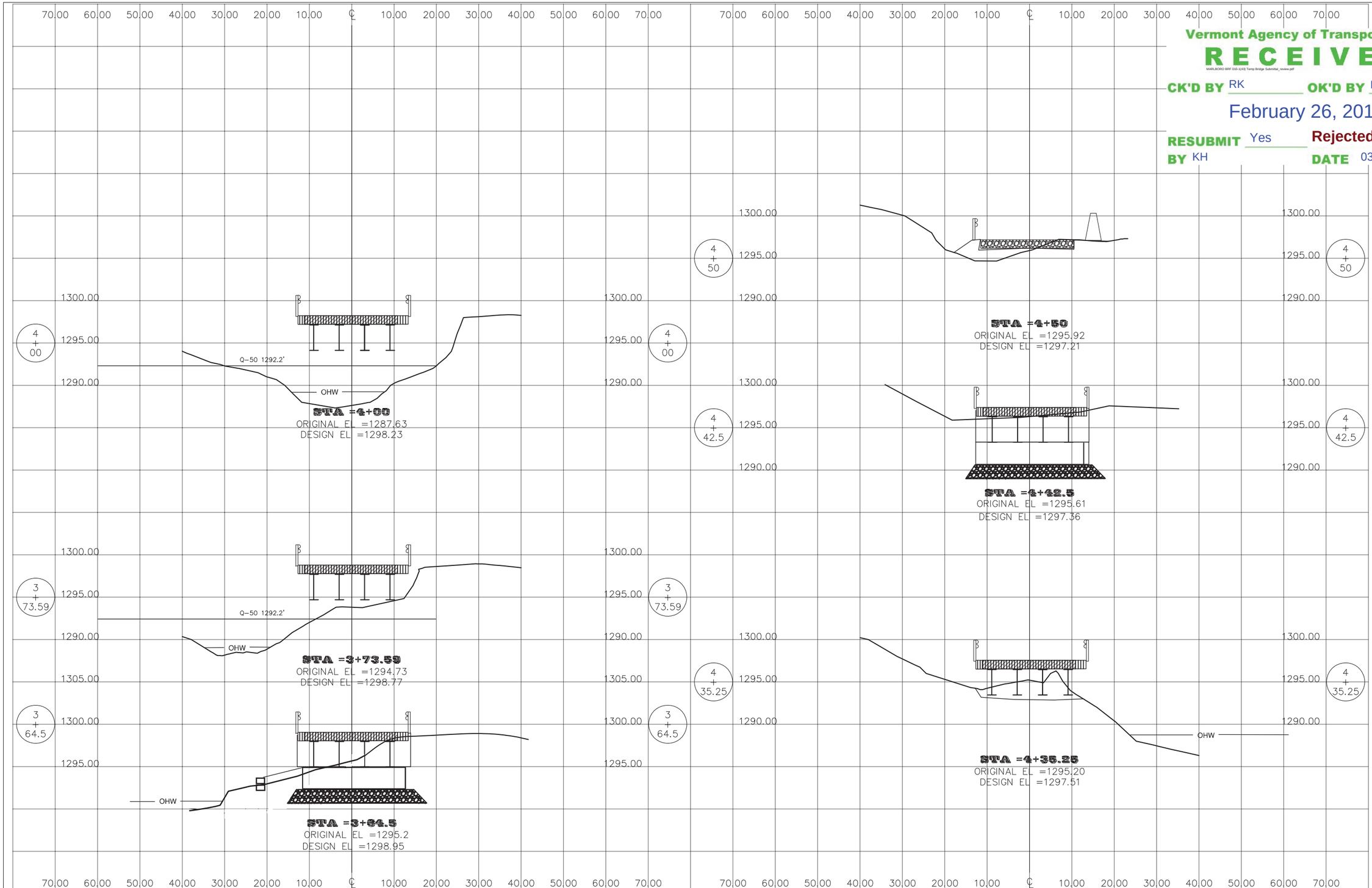

STATE OF VERMONT RONALD A. RUEL No. 8001 REGISTERED PROFESSIONAL ENGINEER

CROSS SECTIONS

RENAUD BROS. INC.
BRF 010-1(43)
ROUTE 9
MARLBORO, VT

DATE: 2-03-2014	

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

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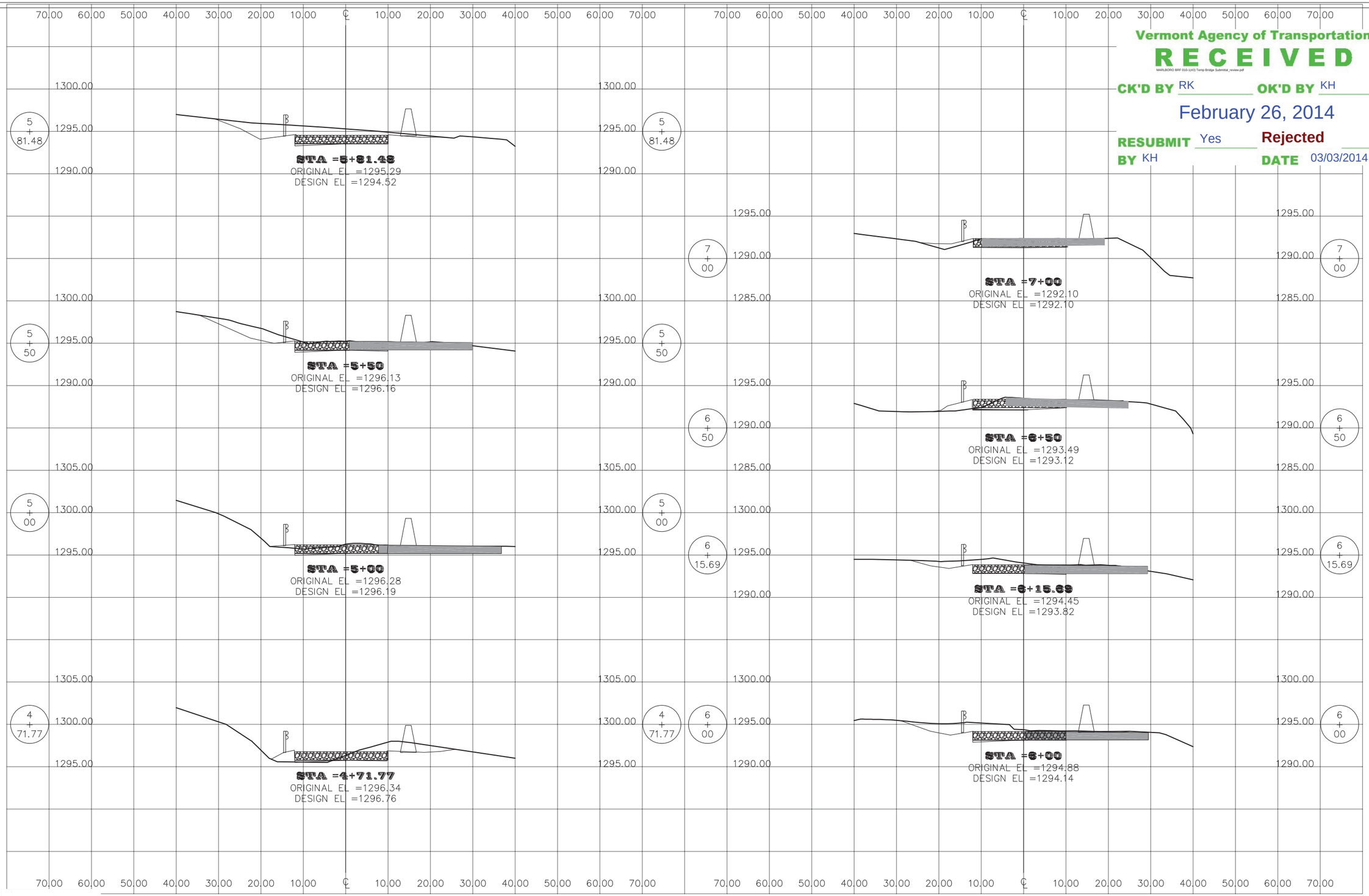

STATE OF VERMONT RONALD K. BELL No. 6204 REGISTERED PROFESSIONAL ENGINEER

CROSS SECTIONS

RENAUD BROS. INC.
BRF 010-1(43)
ROUTE 9
MARLBORO, VT

DATE: 2-20-2014

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

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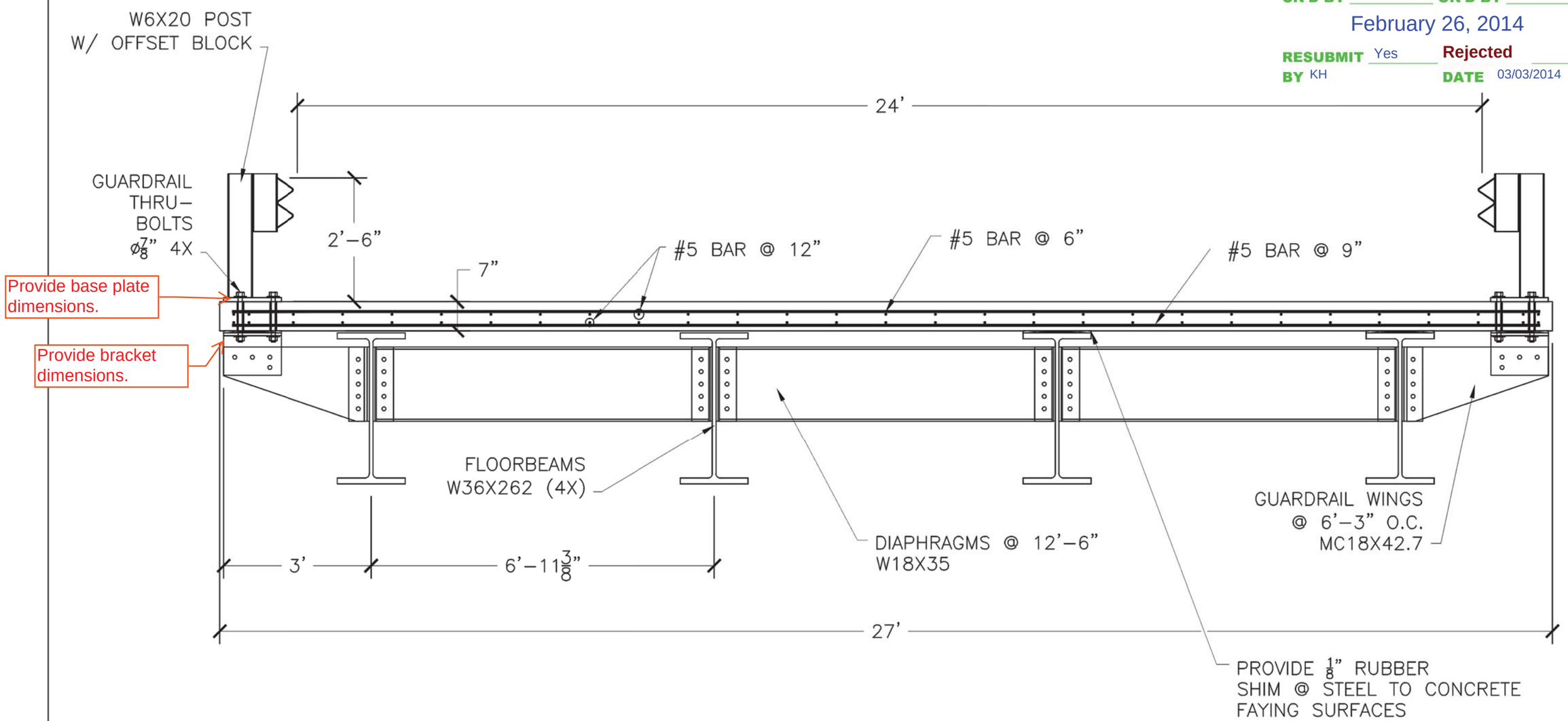
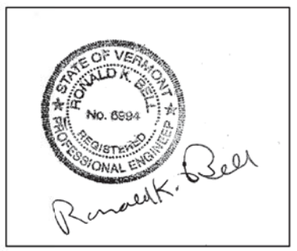


TEMP. BRIDGE SECTION

N.T.S.



MATERIAL NOTES:
1) REBAR TO BE GR. 60 MIN.
2) CONCRETE TO BE 5,000 PSI MIN.

DATE:	2/21/2014
DRAWN BY:	A.D.
CHK'D BY:	C.E.

REV.		
NO.	DATE	COMMENTS

JOB NAME:	MARLBORO BRO 1442(35)
JOB NO:	
SHEET NAME:	TEMP. BRIDGE SECTION

RENAUD BROS., INC.
STEEL SALES & FABRICATION
285 FT. BRIDGMAN RD. VERNON VT. 05354
PH. (802) 297-7585 FAX (802) 297-7508

SHEET: 1 OF 3

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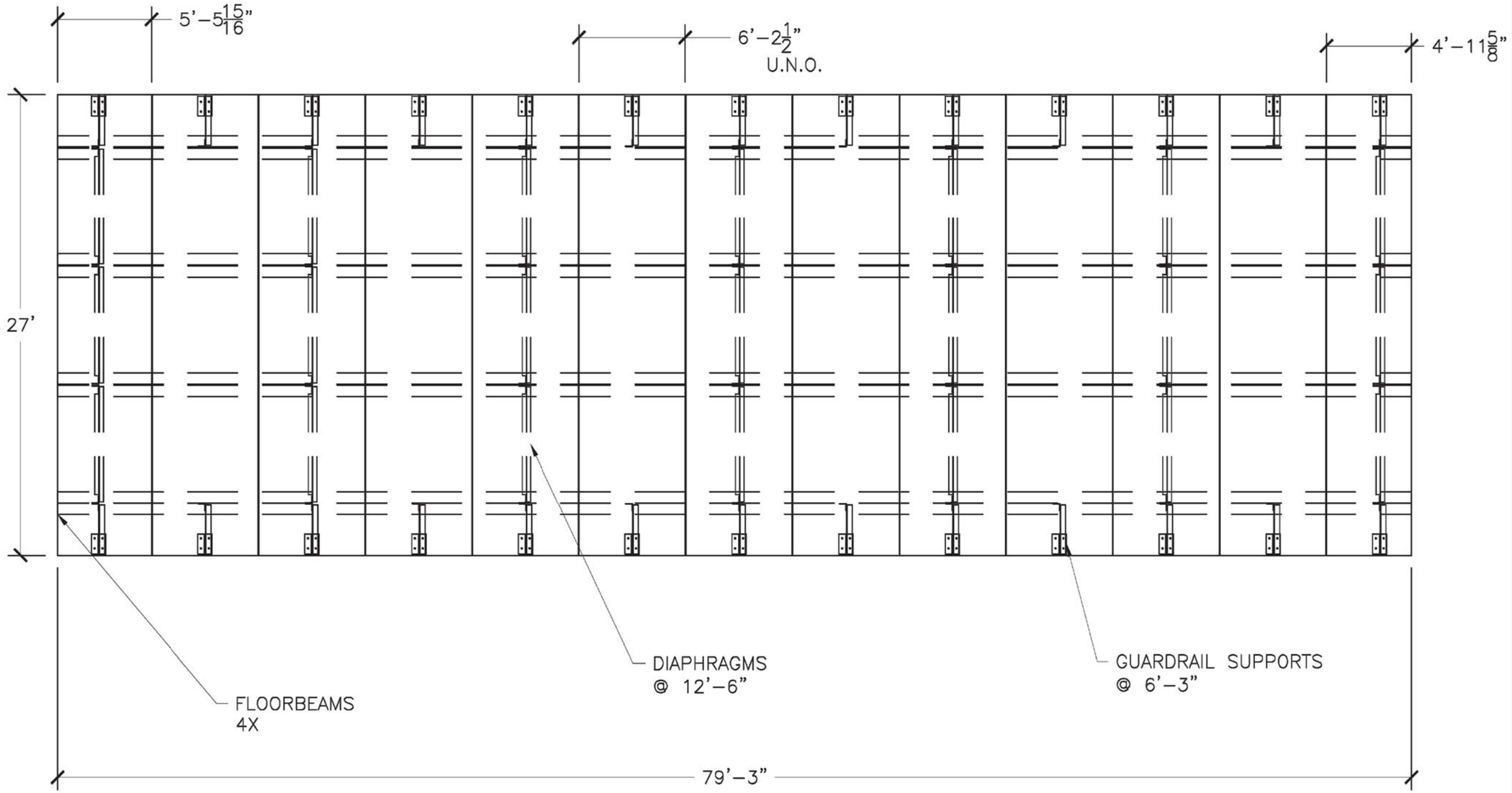

BRIDGE FRAMING PLAN
NOT TO SCALE

Provide connection details for precast slabs.

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BY KH DATE 03/03/2014

STATE OF VERMONT
RONALD K. BELL
No. 6694
REGISTERED
PROFESSIONAL ENGINEER

		REV.		
		NO.	DATE	COMMENTS
DATE:	2/21/2014			
DRAWN BY:	A.D.			
CHK'D BY:	C.E.			

JOB NAME: MARLBORO BRO 1442(35)
JOB NO:
SHEET NAME: BRIDGE PLAN
RENAUD BROS. INC.
SHEET: 2 OF 3

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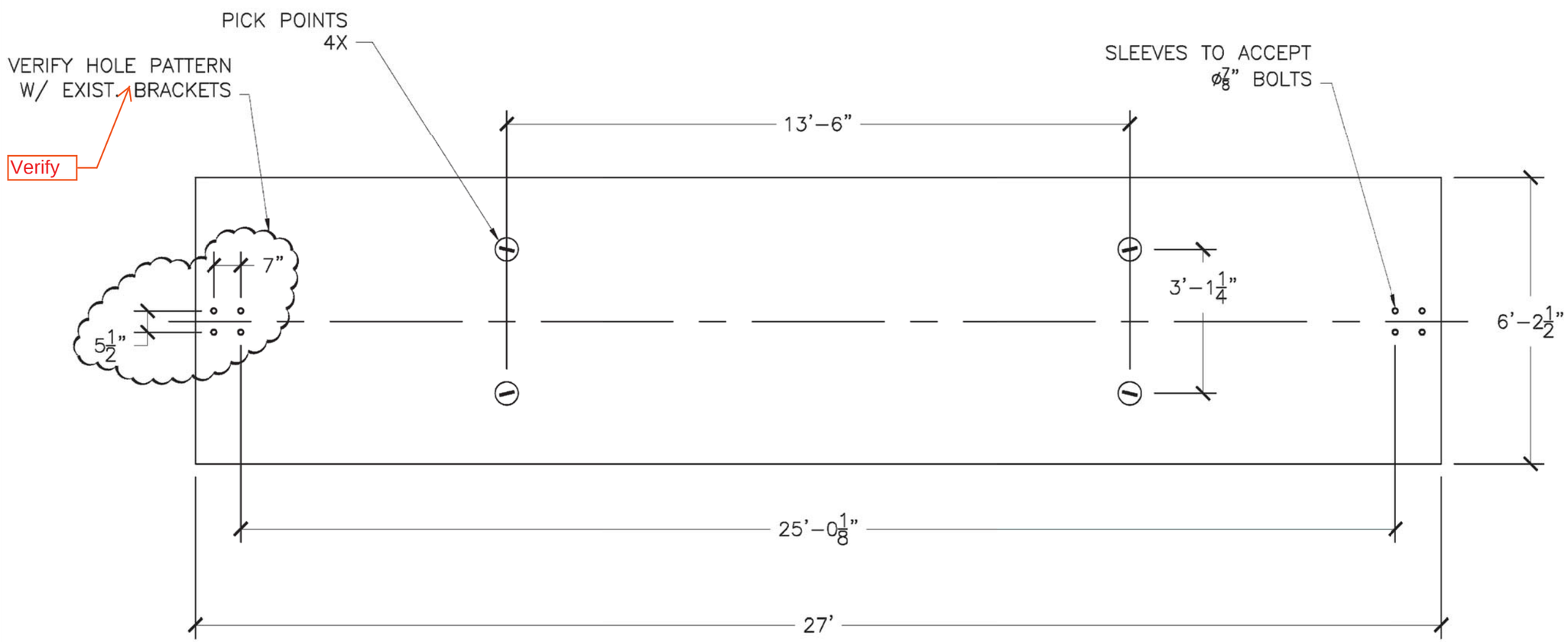
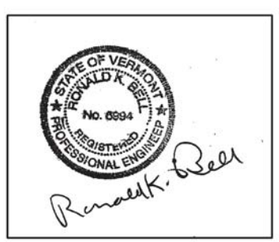


SLAB PLAN–VIEW
NOT TO SCALE



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DATE:	2/21/2014
DRAWN BY:	A.D.
CHK'D BY:	C.E.

REV.		
NO.	DATE	COMMENTS

JOB NAME:	MARLBORO BRO 1442(35)
JOB NO:	
SHEET NAME:	SLAB PLAN

RENAUD BROS. INC.
285 FT. BRIDGMAN RD. VERNON, VT. 05354
P.L.(802) 297-7585 FAX (802) 297-7508

SHEET: 3 OF 3

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NC DECK

Marlboro

010-1 (43)

PROJECT NAME/ LOCATION: Bratt. BRP ~~1442(35)~~
SHEET #: 1 OF 9
CALCULATED BY: Ron Bell DATE: 2-20-14
CHECKED BY: DATE:
SCALE: NTS

Please include calculations for bridge rail posts and anchorage with the loading specified in 528.04(c)

TEMPORARY DECK DESIGN

STATE OF VERMONT RONALD K. BELL No. 6034 REGISTERED PROFESSIONAL ENGINEER

AASHTO LRFD
9.7.5 - PRECAST DECK ON GIRDERS

It is not clear how the precast deck panels will be connected. Please clarify.

MIN. DEPTH = 7.0 in.

9.7.2.5 - REINFORCEMENT REQUIREMENTS:
MIN. BOT LAYER = 0.27 in²/ft
MIN. TOP LAYER = 0.18 in²/ft

FOR ABSOLUTE MIN. 7.0 in. DECK & 2.0 INCH TOP COVER
1.0 INCH BOT "
TO PROVIDE A REINFORCED CORE = 4.0 in.

C9.7.2.5 - Commentary: tests indicate that 0.2% reinforcement in each of four layers based on effective depth d satisfies strength requirements. However, the conservative value of 0.3% of gross area is specified for better crack control. For 7" total depth provide approx 0.25% of total gross area for bottom positive reinforcement min.

NOTE AASHTO LRFD ALLOWS 3 METHODS OF DECK DESIGN FOR PRIMARY REINFORCEMENT ⊥ TO BRIDGE DECK. THIS DESIGN USES THE APPROXIMATE ELASTIC METHOD IN WHICH THE DECK IS DESIGNED FOR FLEXURE RESISTANCE & CONTROL OF CRACKING.

SHEAR DESIGN + FATIGUE + FRACTURE DESIGN IS NOT REQUIRED PER C 4.6.2.1.6

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(603) 363-9966

BRF 010-1 (43)

PROJECT NAME/ LOCATION: ~~BRO 1412 (35)~~
SHEET #: 2 OF 9
CALCULATED BY: Ron Bell DATE: 2-20-14
CHECKED BY: DATE:
SCALE:

STATE OF VERMONT RONALD R. BELL No. 6034 REGISTERED PROFESSIONAL ENGINEER

FOR PRECAST CONCRETE $f'_c = 5$ KSC

$f_y = 60$ KSC

NOTE: M STRENGTH I GOVERNS THIS DESIGN DUE TO LARGER FACTORS ON LL+IM (1.75 VS 1.0)

BEAM SPACING: 6'-11 3/8"

MIN. THICKNESS

$$w_{DC} = \frac{(0.150\,K)}{ft^3} \times \frac{7}{12}'' \times (1\,ft) = 0.088\,\frac{K}{ft}$$

$$M_{DC} = \frac{0.088\,K/ft \times 6.948^2}{10} = 0.425\ K\text{-}ft$$

FROM APPENDIX 4 $M_{LL+IM} = 5.21$ K-FT

1.25 per AASHTO Tables 3.4.1-1 and 3.4.1-2

$$M_{strength\,I} = (1.0)(.425) + 1.75\,(5.21)$$
$$= 9.543\ K\text{-}ft$$
$$= 114.50\ K\text{-}in.$$

$$\phi M_n = \phi\left[A_s f_y \left(d_s - \frac{1}{2}\,\frac{A_s f_y}{0.85 f'_c b}\right)\right] \geq M_{str.\,I}$$

$\phi = 0.9$

$f_s = f_y = 60$ KSC

$f'_c = 5$ KSC

$d_s = 7'' - 1.0$ cover $- 0.5 \times 0.625$ in bar
$= 5.687''$

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BRF 010-1-(43)

PROJECT NAME/ LOCATION: ~~BRF 14422(35)~~
SHEET #: 3 OF 9
CALCULATED BY: Ron Beck DATE: 2-20-14
CHECKED BY: DATE:
SCALE:

SOLVE FOR As for Positive Reinforcement

BY TRIAL AND ERROR: TRY #5 bar @ 9" o.c.

$$(12/9)(3.14)\left(\frac{.625}{2}\right)^2 = 0.409 \text{ in}^2$$

$$0.9\left[(0.409 \times 60)\left(5.687 - \frac{1}{2}\,\frac{0.409 \times 60}{0.85 \times 5 \times 12}\right)\right]$$

$$= 120.289 \text{ K-in} > 114.50 \text{ K-in} \quad \text{OK}$$

BUT TRY #5 Bars @ 10" o.c.

$$A = (12/10)(3.14)\left(\frac{.625}{2}\right)^2 = 0.368$$

$$0.9\left[((0.368)(60))\left(5.687 - \frac{1}{2}\,\frac{0.368 \times 60}{(0.85)(5)(12)}\right)\right]$$

$$= 120.789 > 114.50 \quad \text{OK}$$

TRY 12" o.c. As = 0.307

$$0.9\left[((0.307)(60))\left(5.687 - \frac{1}{2}\,\frac{0.307 \times 60}{(0.85)(5)(12)}\right)\right]$$

$$= 101.4 \ll 114.50 \quad \text{NG}$$

$$\boxed{\text{PICK \#5 bars @ 10" o.c.}}$$

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BRF 010-1 (43)

PROJECT NAME/ LOCATION: ~~1442 (35)~~

SHEET #: 4 OF 9

CALCULATED BY: Ron Bell DATE: 2-24-14

CHECKED BY: DATE:

SCALE:

STATE OF VERMONT RONALD K. BELL No. 6694 REGISTERED PROFESSIONAL ENGINEER

CHECK CRACK CONTROL

$$S \le \frac{700\gamma_e}{\beta_s f_s} - 2d_c$$

S = spacing of reinforcement closest to tension face

where: $\beta_s = 1 + \frac{d_c}{0.7(h-d_c)}$

where $d_c = 1.0''$ clear $+ 0.5 \times 0.625$ in bar

$= 1.3125$ in.

$h = 7''$

$\therefore \beta_s = 1 + \frac{1.3125}{(0.7)(7-1.3125)} = 1.330$

$\Downarrow$ 5.688

$\gamma_e = 0.75$

$f_s = \frac{M_{service} I}{A_s j d_s}$

$f_s =$

$K = \sqrt{((.005)(7))^2 + (2)(.005)(7)} - (.005)(7)$

$= 0.036$

$j = 1 - \frac{K}{3} = 1 - \frac{0.036}{3}$

$= 0.988$

Where;

$A_s = 0.368 \text{ in}^2$

$j = 1 - \frac{K}{3}$

$K = \sqrt{(\rho n)^2 + 2\rho n} - \rho n$

$\rho = \frac{A_s}{b d_s} = \frac{0.368 \text{ in}^2}{(12'')(5.687)''}$

$= 0.005$

$n = \frac{E_s}{E_c} = 7$ for

$4.6 \le f'_c \le 6.0$ C.C.10.1.1b

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PROJECT NAME/ LOCATION: BRF 010-1(43) ~~1442(30)~~
SHEET #: 5 OF 9
CALCULATED BY: RonBell DATE: 2-20-14
CHECKED BY: DATE:
SCALE:

$\gamma_{ie} = 0.75$ for decks exposed to water

$M_{SERVICE\ I} = 1.0\,(DC + LL + IM)$

$= 0.425 + 5.21$

$= 5.635$ K-FT

$= 67.620$ K-IN

$$f_s = \frac{67.620 \text{ K-IN}}{(0.368 \text{ in}^2)(0.988)(5.687)''} = 32.203 \text{ KSI}$$

$$S \leq \frac{(700)(0.75)}{(1.330)(32.203} \text{ in} - (2)(1.3125) \text{ in} =$$

$$\leq 12.043 - 2.625 = 9.418$$

$10'' \geq 9.418$ in N.G.

$\therefore$ MODIFY SPACING TO PROVIDE

$\boxed{\#5 \text{ bars @ } 9'' \text{ O.C.}}$

STATE OF VERMONT
RONALD K. BELL
No. 6334
REGISTERED PROFESSIONAL ENGINEER

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(603) 363-9966

BRF 010-1 (43)

PROJECT NAME/ LOCATION: ~~1442 (35)~~

SHEET #: 6 OF 9

CALCULATED BY: Ron Bell DATE: 2-20-14

CHECKED BY: DATE:

SCALE:

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

CHECK MIN. REINFORCEMENT

$M_r > M_{CR}$

$M_{CR} = \gamma_3 \gamma_1 S f_r$ (K-in)

$S = \frac{1}{6} bh^2 = \frac{1}{6}(12'')(7)^2 = 98 \text{ in}^3$

$f_r = 0.25\sqrt{f'_c} = 0.24\sqrt{5.0 \text{ KSI}} = 0.537 \text{ KSI}$

$\gamma_3 = 0.75$ for grade 60.

$\gamma_1 = 1.6$ for non segmental

$M_{CR} = (0.75)(1.6)(98)(0.537) = 63.151$ K-in

$M_r = \phi M_N = 120.289$ (see pg. 3 for trial with #5 @ 9" o.c.)

$120.289 > 63.151$ ∴ OK

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February 26, 2014
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BY KH DATE 03/03/2014

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BRF 010-1(43)

PROJECT NAME/ LOCATION: ~~14112 (35)~~

SHEET #: 7 OF 9

CALCULATED BY: Ron Bell DATE: 2-20-14

CHECKED BY: DATE:

SCALE:

Negative Reinforcement

From positive calculations $w_{DC} = 0.088$ k/ft

Span length: Reduce by ½ flange width $\cong 6.5'$

$M_{DC} = \left(\frac{1}{10}\right)(0.088\ k/ft)(6.5)^2 = 0.372$ k-ft

LL+IM MOMENTS FROM A-4 (Note 4.6.2.1.6

for negative moments taken as ¼ flange width from c/L of support: $\frac{16.55}{4} = 4.38''$ use 3" from table

$\therefore M_{LL+IM} = 5.17$ K-FT

Factored moment $= \eta_i\left[1.0(M_p + 1.75\, M_{LL+IM})\right] = 9.420$ K-ft $= 113.034$ K-in

Design for Moment Capacity

$$\phi M_n = \phi\left[A_s f_y\left(d_s - \frac{1}{2}\frac{A_s f_y}{0.85 f'_c b}\right)\right] \geq M_{strength\,I}(113.034)$$

$\phi = 0.9$

$f_y = 60$ KSI

$f'_c = 5$ KSI

To account for broom finish

$d_s = 7''$ slab $- (2'' + 0.25''$ cover$) - 0.5 \times 0.625$ bar $= 5.063''$

$b = 12$ in

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BRF 010-1(43)

PROJECT NAME/ LOCATION: ~~1442(35)~~

SHEET #: 8 OF 9

CALCULATED BY: Ron Bell DATE: 2-20-14

CHECKED BY: DATE:

SCALE:

TRY, #5 bars @ #5 bars @ 9" O.C. $A_s = 0.409 \text{ in}^2$

$$0.9\left[(0.409 \times 60)\left(5.063 - \left(\tfrac{1}{2}\right)\left(\frac{0.409 \times 60}{(0.85)(5)12}\right)\right)\right]$$

$= 118.342$ K-in > ~~[illegible]~~ 113.034 K-in OK

Check Cracking for Negative Moment

$$s \le \frac{700\gamma_e}{\beta_s f_s} - 2d_c$$

where

$$\beta_s = \frac{1 + d_c}{0.7(h - d_c)}$$

$$\rho = \frac{A_s}{b d_s} = \frac{0.409 \text{ in}^2}{(12 \text{ in})(5.063)}$$

$= 0.007$

where $d_c =$ (2.0 + 0.25) in clear + 0.5 × 0.625

$= 2.563$ in

$h = 7''$

$n = 7$ from C 6.10.1.1b for $4.0 \le f'_c \le 6.0$

$$\therefore \beta_s = 1 + \frac{2.563}{0.7(7 - 2.563)}$$

$= 1.825$

$$K = \sqrt{[(.007)(7.0)]^2 + (2)(.007)(7.0)} - (.007)(7)$$

$= 0.226$

$\gamma_e = 0.75$

$$f_s = \frac{M_{service\ I}}{A_s j d_s}$$

$M_{service\ I} = 1(1.0 M_{DC} + 1.0 M_{LL+IM})$

$= 5.542$ K-ft

$= 66.504$ K-IN

$A_s = 0.409$

$j = 1 - \frac{K}{3}$

$K = \sqrt{(\rho n)^2 + 2\rho n} - \rho n$

$j = 1 - \frac{0.226}{3} = 0.925$

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PROJECT NAME/ LOCATION: BRF 010-1(43) ~~BRO 1442 (35~~
SHEET #: 9 OF 9
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$$f_s = \frac{M_{service}}{A_s j d_s} = \frac{66.504 \text{ K-in}}{(.409 \text{ in}^2)(0.925)(5.063)}$$

$$= 34,720 \text{ KSI}$$

$$s \le \frac{700 \gamma_e}{\beta_s f_s} - 2d_c$$

$$s \le \frac{(700)(0.75)}{(1.825)(34.720)} - (2)(2.563) = 3.158 \text{ in}$$

9" > 3.158" No.6

TRY #5 bars @ 6" c to c spacing. $A_s = 0.62 \text{ in}^2$

$$\rho = \frac{0.62}{(12)(5.063)} = 0.010$$

$$K = \sqrt{[(.01)(7.0)]^2 + (2)(.01)(7)} - (.01)(7) = 0.274$$

$$j = 1 - \frac{K}{3} = 0.909$$

$$f_s = \frac{M_{service}}{A_s j d_s} = \frac{66.504}{(0.62)(0.909)(5.063)} = 23.307 \text{ KSI}$$

$$s \le \frac{(700)(0.75)}{(1.825)(23.307)} - (2)(2.563) = 7.217$$

6" < 7.217 OK

Distribution Reinforcement = 0.67% of Primary as per 9.7.3.2 for bottom Bars

∴ #5 bars @ 12" O.C.

NOTE: Top Longitudinal steel only needs to meet temp. + shrinkage ∴ #5 @12" O.C adequate

BEAM CALCULATIONS:

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PROJECT NAME/ LOCATION: 010-(1)(43)
SHEET #: 1 OF 5
CALCULATED BY: Ron Bell DATE: 2-6-2014
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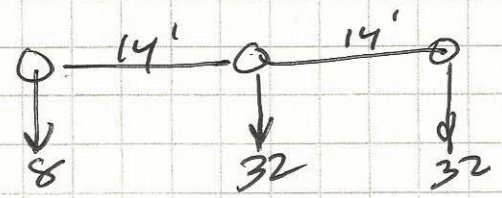
TEMPORARY BRIDGE CALCULATIONS
STEEL BEAMS

BEAMS @ 36x260 WITH 1x14 PLATE ON BOTT FLANGE
SEE SECTION CALCULATION
$A = 90.50 \text{ in}^2$
$S_x = 1317.61 \text{ in}^3$
$Z_x = 1515.0 \text{ in}^3$

DESIGN = HL 93 SPAN FROM ℄ + ℄ BEARING = 76'

DESIGN TRUCK MOMENT > DESIGN TANDEM

DESIGN TRUCK =



FROM BALANCE POINT OF DESIGN TRUCK
LOADS ACT 19.33' (FRONT), 33.77' + 47.33'

LOAD DISTRIBUTION

UNIFORM LANE LOAD = $S/10 = \frac{7.0}{10} = 0.70 \times 0.64 = 0.448$ K/ft

WHEEL LOADS:

FOR EXTERIOR BEAMS (WORST CASE)

TABLE 4.6.2.2.2d-1 USE LEVER RULE

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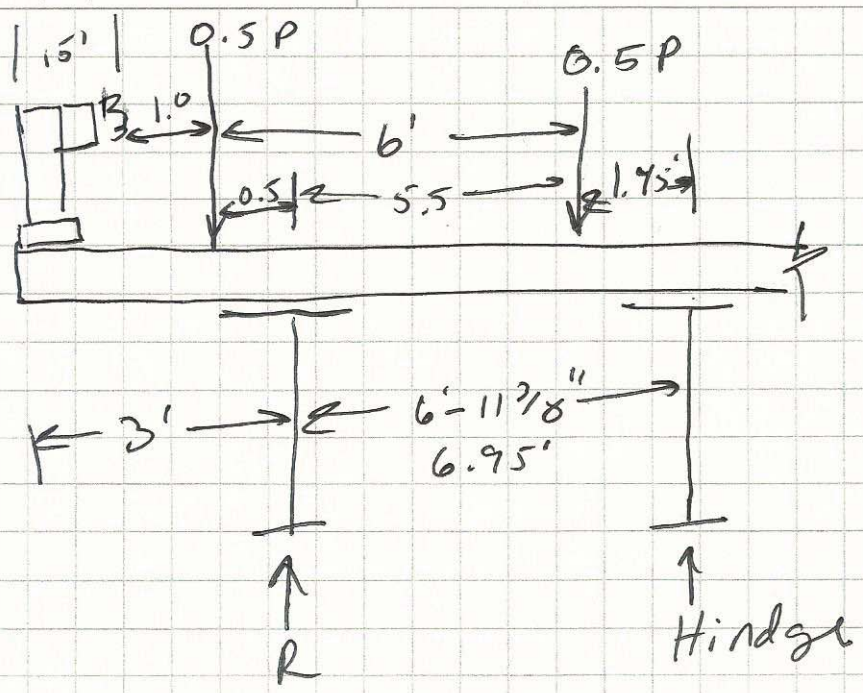


Why are the axle loads (P) divided by 2 here? The applied load for the Lever Rule calculation is already half of the axle load (0.5*P).

$M_{Hindge} = -(1.45)(0.5P) - (7.45' \times 0.5P) + 6.95(R) = 0$

$R = 0.64P$

Dynamic Load Allowance = 33%

VTrans Load Modifier for temp bridge = 0.9

Multiple Lane Factor for one ~~des~~ Lane design = 1.2

1.436

DF

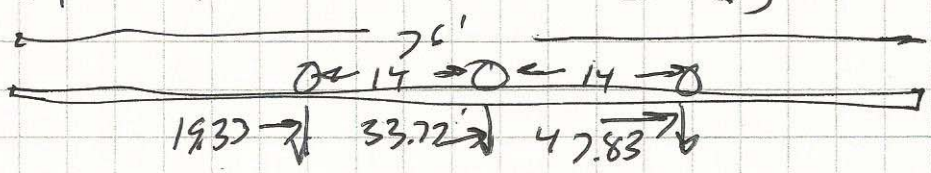
FRONT WHEEL LOAD = (0.64)(8/2 KIPS)(1.436) = 3.676 KIPS

REAR WHEEL LOAD = (0.64)(32/2)(1.436) = 14.705 KIPS

DL ON EXTERIOR BEAM = $(7/12)(150)\left[\left(\frac{6.95}{2}\right)+(3)\right] + 50\,\#/ft$

$= 616.56\,\#/ft = 0.616\,K/ft$

WORST CASE WHEEL LOADS



BASED ON BALANCE POINT OF DESIGN TRUCK

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FOR STRENGTH I DESIGN FACTORS ARE:
DC 1.0
LL+IM = 1.75

1.25 per AASHTO Tables 3.4.1-1 and 3.4.1-2

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CIVIL & ENVIRONMENTAL
17 ECHO COVE WAY-SPOFFORD, NH 03462
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Project Title:
Engineer:
Project Descr:
Project ID:

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General Beam Analysis

File = c:\Users\Ron\DOCUME~1\ENERCA~1\RT9CON~1.EC6
ENERCALC, INC. 1983-2013, Build:6.13.8.31, Ver:6.13.12.31

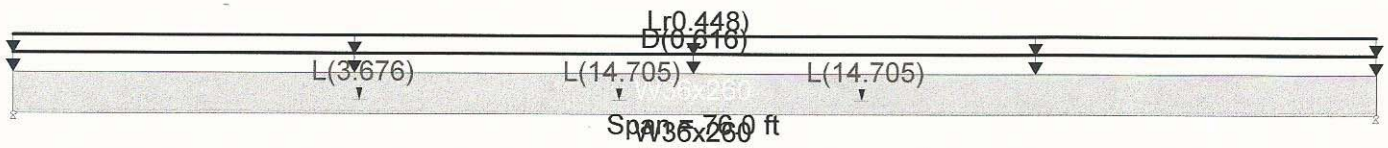
Lic. # : KW-06009396 Licensee : Bell Engineering

Description : --None--

General Beam Properties

Elastic Modulus 29,000.0 ksi

Span #1 Span Length = 76.0 ft Area = 90.50 in^2 Moment of Inertia = 21,408.6 in^4



Applied Loads

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

Beam self weight calculated and added to loads
Load(s) for Span Number 1
Point Load : L = 3.676 k @ 19.330 ft ← FRONT
Point Load : L = 14.705 k @ 33.770 ft REAR
Uniform Load : D = 0.6160 k/ft, Tributary Width = 1.0 ft DEAD
Point Load : L = 14.705 k @ 47.330 ft REAR
Uniform Load : Lr = 0.4480 k/ft, Tributary Width = 1.0 ft ← UNIFORM LANE LOAD

DESIGN SUMMARY

Maximum Bending =	1,880.412 k-ft	Maximum Shear =	82.297 k
Load Combination	+D+1.750Lr+1.750L	Load Combination	+D+1.750Lr+1.750L
Location of maximum on span	35.720ft	Location of maximum on span	76.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

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FROM ENERCAL $M_F = 1880.41$ K-FT

$$M_u = F_y Z_z = \frac{36 \frac{KIPS}{in^2} \times 1515.0 \ in^3}{12 \ in/ft}$$

$$= 4545 \ K\text{-}FT$$

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NO 6904
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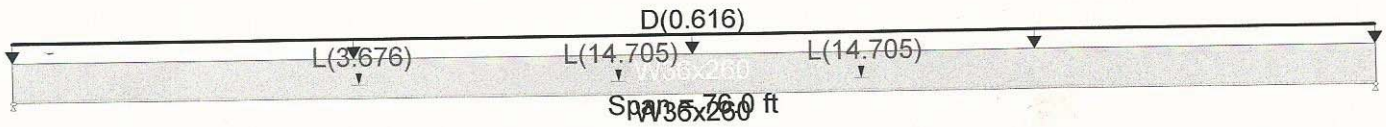
$M_u >>> M_F \therefore OK$

CHECK DEFLECTION

NOTE: PER AASHTO THE UNIFORM LANE LOAD IS NOT INCLUDED IN DEFLECTION CALCS.

General Beam Properties

Elastic Modulus 29,000.0 ksi
Span #1 Span Length = 76.0 ft Area = 90.50 in^2 Moment of Inertia = 21,408.6 in^4



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

Beam self weight calculated and added to loads
Load(s) for Span Number 1
Point Load : L = 3.676 k @ 19.330 ft
Point Load : L = 14.705 k @ 33.770 ft
Uniform Load : D = 0.6160 k/ft, Tributary Width = 1.0 ft
Point Load : L = 14.705 k @ 47.330 ft

DESIGN SUMMARY

Maximum Bending =	1,319.011 k-ft	Maximum Shear =	52.505 k
Load Combination	+D+1.750Lr+1.750L	Load Combination	+D+1.750Lr+1.750L
Location of maximum on span	33.820ft	Location of maximum on span	76.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.782 in
Max Upward L+Lr+S Deflection	0.000 in
Max Downward Total Deflection	1.533 in
Max Upward Total Deflection	0.000 in

$76 \times 12 = 912$

$\frac{912}{0.782} = 1166$ OK

> 777 800

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Project Title: ~~1442126~~ BRF 010-(1)(43)
Engineer: Ron Bell
Project Descr:

Project ID:

5/5

Printed: 25 FEB 2014, 9:15AM

General Section Property Calculator

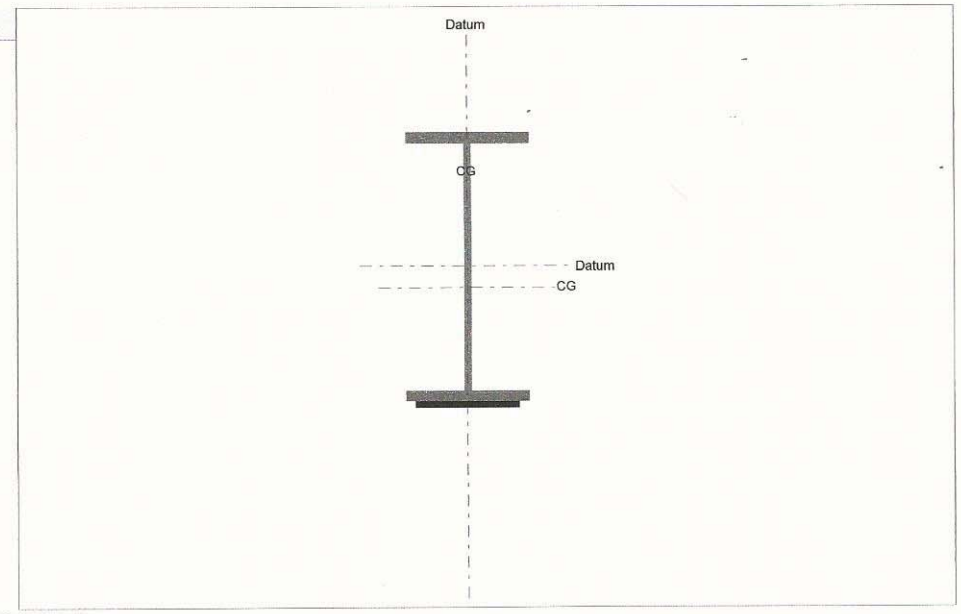
File = c:\Users\Ron\DOCUME~1\ENERCA~1\RT9SEC~1.EC6
ENERCALC, INC. 1983-2013, Build:6.13.8.31, Ver:6.13.12.31

Lic. # : KW-06009396 Licensee : Bell Engineering

Description : 36X260 WITH 1X14 PLATE ON BOT FLANGE

Final Section Properties

Total Area	:	90.50 in^2
Calculated final C.G. distance from Datum :		
X cg Dist.	:	0.0 in
Y cg Dist.	:	-2.882 in
Edge Distances from CG. :		
+X	:	8.275 in
-X	:	-8.275 in
+Y	:	21.012 in
-Y	:	-16.248 in
Ixx =	:	21,408.6 in^4
Iyy =	:	1,318.67 in^4
Sxx : - Y	:	1,317.61 in^3
Sxx : +Y	:	1,018.87 in^3
Syy : - X	:	159.355 in^3
Syy : +X	:	159.355 in^3
r xx	:	15.380 in
r yy	:	3.817 in



General Shapes

Rect : 1 Height = 1.000 in Width = 14.000 in Rotation = 0 dec CCW

Area =	14.000 in^2	Xcg =	0.000 in	Ixx =	1.167 in^4	Sxx =	2.333 in^3	Rxx =	0.289 in
		Ycg =	-18.630 in	Iyy =	228.667 in^4	Syy =	32.667 in^3	Ryy =	4.041 in

Steel Shapes

W36x260 : 1

		Ixx =	17,300.000 in^4	Rotation =	0 dec CCW
Area =	76.500 in^2	Iyy =	1,090.000 in^4	Xcg =	0.000 in
Height	36.260 in	Sxx =	954.220 in^3	Ycg =	0.000 in
Width	16.550 in	Syy =	131.722 in^3		

$Z_x = 1,515.0 \text{ in}^3$

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SUBSTRUCTURE

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PROJECT NAME/ LOCATION: 010-1(43)
SHEET #: 1 OF 4
CALCULATED BY: Ron Bell DATE: 6-20-14
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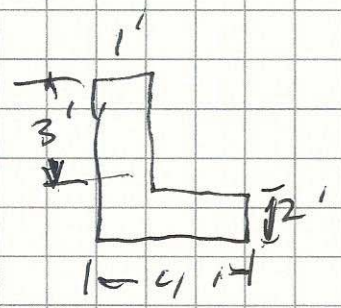
Verify that the appropriate load factor is applied to dead loads per AASHTO Tables 3.4.1-1 and 3.4.1-2

2 TRUCK = 144 KIPS × 1.75 × 1.33 = 335 KIPS
DECK = 27' × (7/12) (76') × (150 lb/ft³) = 179 KIPS
BEAMS + GR = 90 KIPS

604 KIPS



(2×4) + (3×1) (150) (26.83') = 44 KIPS

604 / 2 side + 44 KIPS = 346 KIPS
× 1.5
519 KIPS

519 KIPS / (4' × 26.83') = 4.83 KIPS/ft²

FROM REVIEW OF NORTH SIDE BORING
N_{WORST} = 4 @ 5' below grade.

~~FOR COHERENT~~ FOR GRANULAR SOILS

$q_u = \gamma (D x_1 + B x_2]$ psf γ in (pcf)
D in (ft)
B in (in)

FROM CHART Q calculated to be 15,500 psf

15,500 ~~[illegible]~~ 777 4,830 OK

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Verify that the appropriate resistance factor is applied to bearing resistance per AASHTO Table 10.5.5.2.2-1

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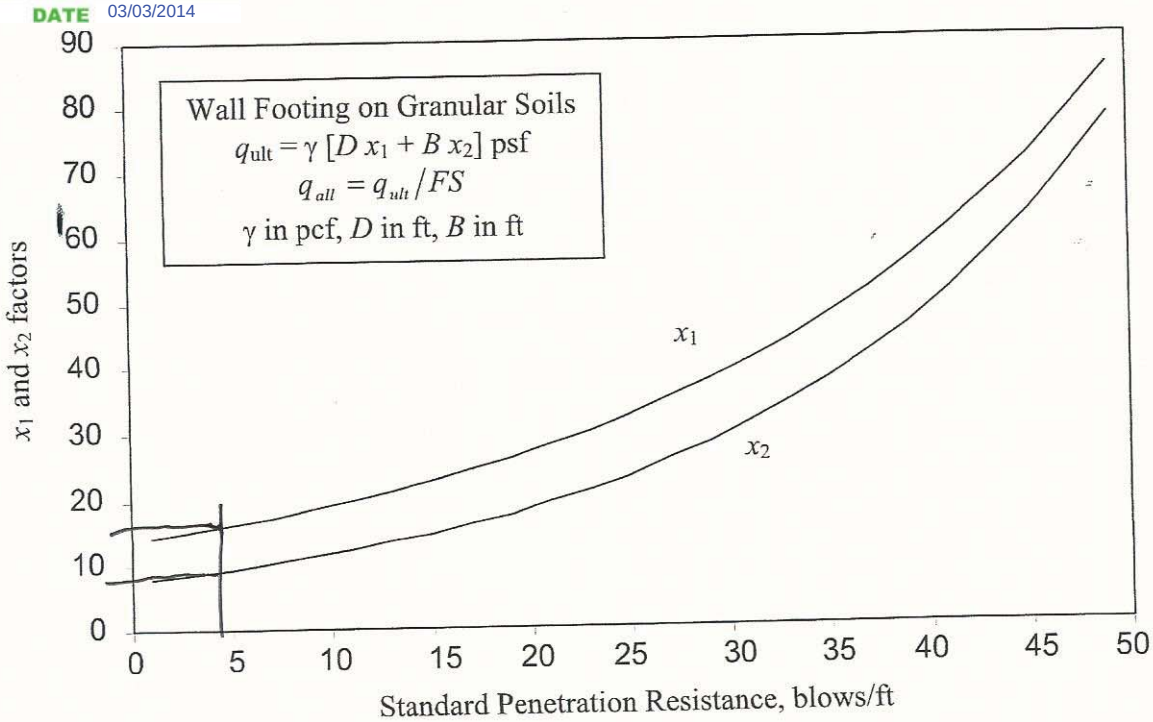

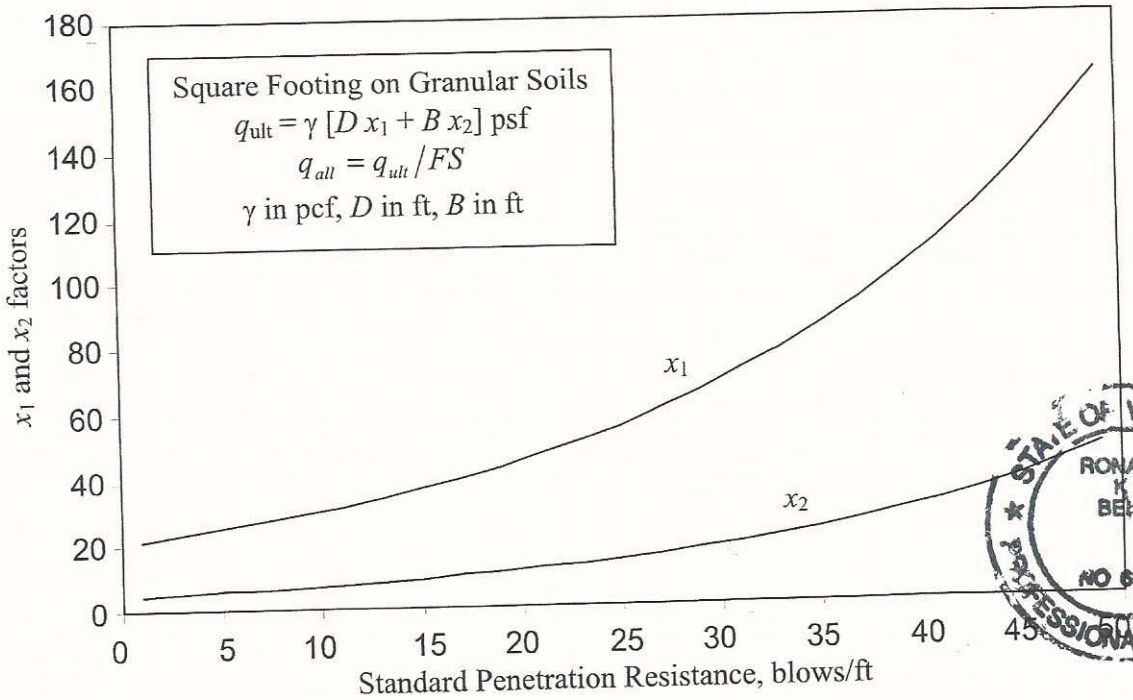

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Note: Settlement was not considered in the graphs.

Bearing Capacity for a Footing on Granular Soils

$q = (120)\,[(4)(60)(15) + (48)(7)]$ 336

47,520 psf / 3 FS =

15,840 psf

This seems high compared to AASHTO C10.6.2.6.1-1 based on soil classification and density listed for B-102 and B-104 on the boring logs. Please verify.

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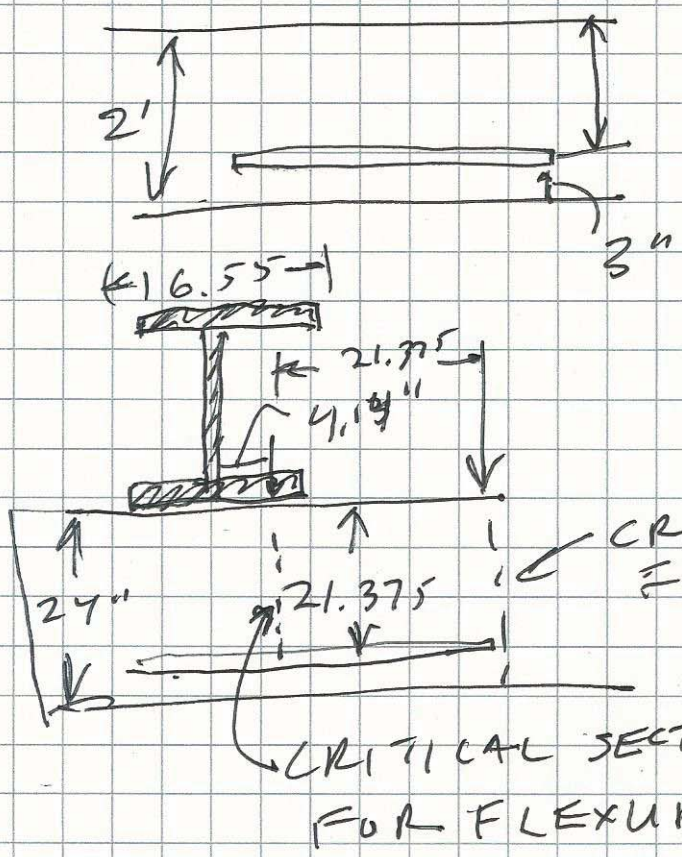
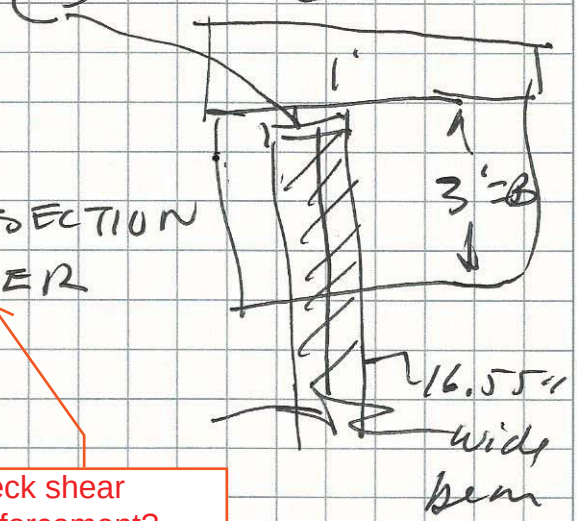
CHECK REINFORCEMENT



$D_s = 24 - 3 + 6/8/2$
$= 21.375''$

(beam spacing)
$L = 7'$
$C = 16.55/12$



Check shear reinforcement?

$$M_{u\,(Approx)} = \frac{q_u B \left(\frac{L}{2} - \frac{C}{2}\right)^2}{2}$$

$$= (4.83)(3)\left(\frac{7}{2} - \frac{1.38}{2}\right)^2$$

$$= 144.4 \text{ KIP-ft}$$

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Flexural Reinforcement

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$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \frac{Mu}{0.383\, bd^2 f'_c}}\right)}{f_y}$$

$$= \frac{(0.85)(4)\left(1 - \sqrt{1 - \frac{144 \times 12 \times 1000}{(0.383)(36)(21.375)^2(4000)}}\right)}{60}$$

$$= \underline{0.004}$$

#6 bars longitudinal @ 5" O.C.

$$\rho = \frac{\pi \left[\left(\frac{6}{8}\right)/2\right]^2 \left(\frac{12}{5}\right)}{12 \times 21.375} = \frac{1.06}{256.5} = .004$$

Verify that the appropriate resistance factor is applied.

.004 = .004 OK

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