DESIGN NOTES

FOR

STATE ROUTE 45 (OLD HICKORY BLVD.) OVER I-65

DAVIDSON COUNTY

BRIDGE ID NUMBER: 19I00650081
FEDERAL PROJECT NUMBER: IM-65-3(108)
STATE CONSTRUCTION NUMBER: 19012-3154-44
CONSTRUCTION CONTRACT NUMBER: A049

DESIGN SPECIFICATIONS: LRFD (2004, 3rd EDITION)

SPANS: 165'0" – 165'0"

OUT TO OUT WIDTH: 86'0"

WELDED STEEL PLATE GIRDER (54" WEB)

BEAM SPACING: 11'0"

BRIDGE RAIL: STD-1-1

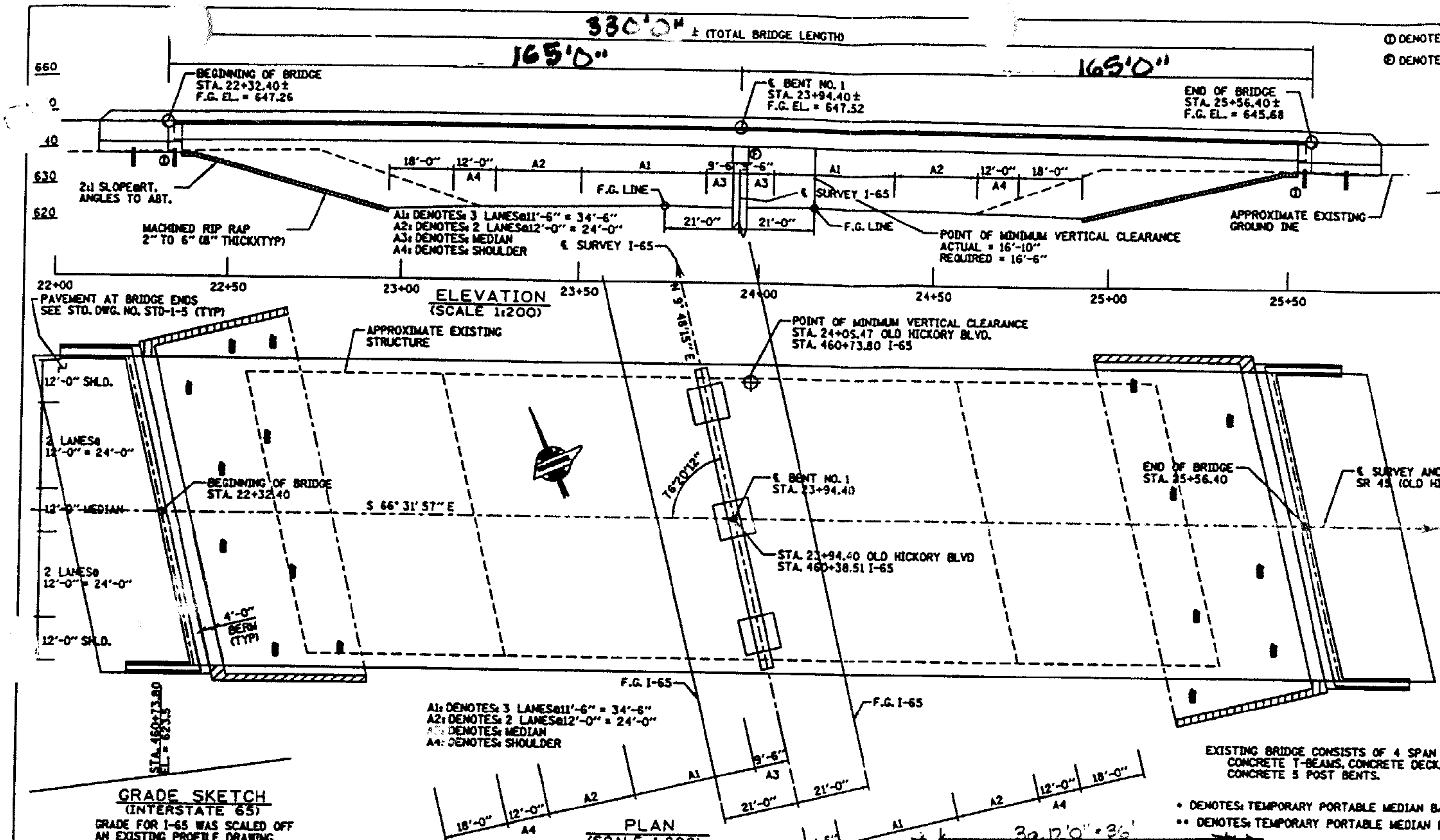
Date: August 24, 2004

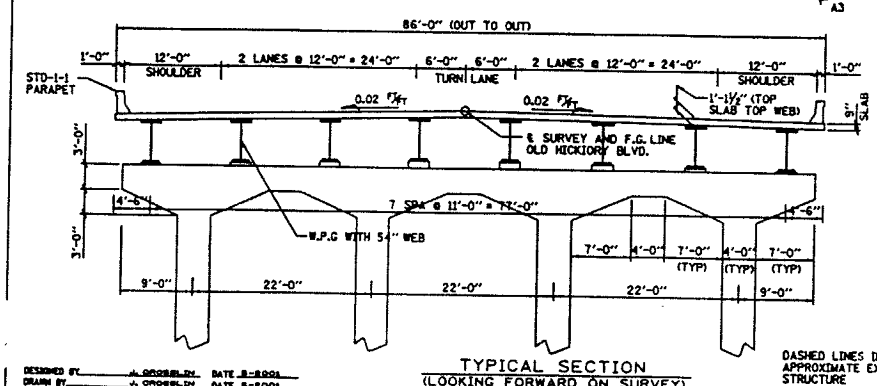
DESIGNED BY: WHP

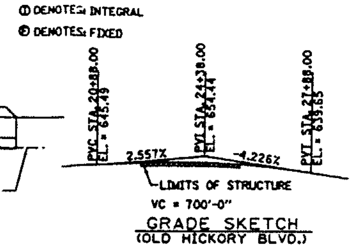
TENNESSEE DEPARTMENT OF TRANSPORTATION

DIVISION STRUCTURES DIRECTOR: EDWARD P. WASSERMAN

INDEX OF SHEETS







CONST. NO.		
PROJECT NO.	YEAR	SHEET NO.

REVISIONS			
NO.	DATE	BY	BRIEF DESCRIPTION

GENERAL NOTES

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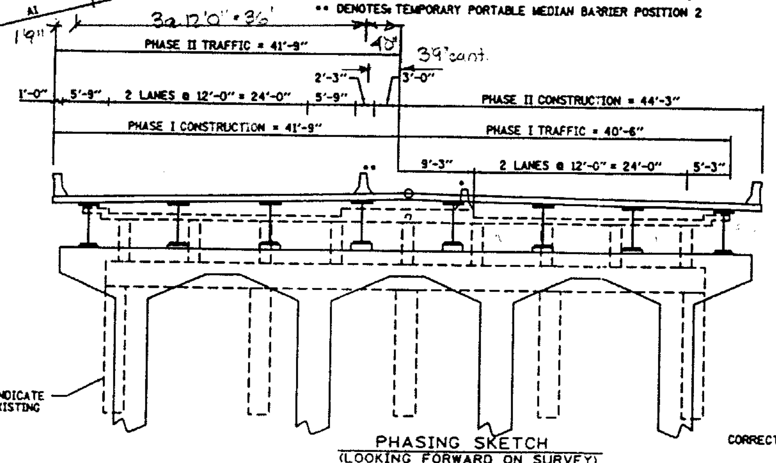
2) LOADING: HS20-44. (DEAD LOADS TO INCLUDE 33 LB/FT² FOR FUTURE OVERLAY.)
3) DESIGN SPECIFICATIONS: AASHTO 1996 EDITION WITH ADDENDA INCLUDING THE STANDARD SPECIFICATIONS FOR SEISMIC DESIGN OF HIGHWAY BRIDGES. (SEISMIC PERFORMANCE CATEGORY "A" WITH ACCELERATION COEFFICIENT 0.075).
4) CONCRETE: TO BE CLASS "A", f'c = 3000 psi. BRIDGE DECK CLASS "D", f'c = 4000 psi.
5) REINFORCING STEEL: SHALL BE ASTM A615 GRADE 60 UNLESS NOTED OTHERWISE. SEE SECTION 604 AND 907 OF THE STANDARD SPECIFICATIONS.
6) SUPERSTRUCTURE SHALL CONSIST OF 2 SPAN CONTINUOUS STEEL WELDED PLATE GIRDER (54" WEB), WITH 4 POST BENTS, AND COMPOSITE CONCRETE DECK SLAB.
7) USE STD-1-1 PARAPET
8) BRIDGE DECK SURFACE FINISH: TO BE IN ACCORDANCE WITH NOTE "C" IN ARTICLE 604.[illegible] OF THE STANDARD SPECIFICATIONS.
9) MACHINED RIP-RAP: MACHINED RIP-RAP FOR SLOPE PROTECTION SHALL BE 6" TO 8" IN SIZE UNIFORMLY GRADED AND MEET THE REQUIREMENTS OF SECTION 709 OF THE STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION AND SHALL BE MEASURED AND PAID FOR UNDER ROADWAY ITEM 709-05.05 [illegible]
10) TEXTURE COATING: TO BE MOUNTAIN GREY (36440) FOR ALL CONCRETE SURFACES EXCEPT INSIDE FACE AND TOP OF PARAPET, WHICH ARE TO BE PAINTED WHITE (37886).
11) ALL STRUCTURAL STEEL IS TO BE WEATHERING STEEL.
12) EXCAVATION SHALL BE BASED ON FINAL PROFILE AT ABUTMENTS AND BENTS.

EXISTING BRIDGE CONSISTS OF 4 SPAN (58'-6", 72'-6", 72'-6", 58'-6"), CONCRETE T-BEAMS, CONCRETE DECK, STUB ABUTMENTS, AND CONCRETE 5 POST BENTS.

* DENOTES: TEMPORARY PORTABLE MEDIAN BARRIER POSITION 1
** DENOTES: TEMPORARY PORTABLE MEDIAN BARRIER POSITION 2



2021 ADT =
84'-0" ROADWAY WITH STD-1-1 PARAPET
DESIGN SPEED =

STATE OF TENNESSEE
DEPARTMENT OF TRANSPORTATION

STATE ROUTE 45
(OLD HICKORY BLVD.)
OVER INTERSTATE 65
BRIDGE I.D. NO. XXXX
STATION 23+94.40
DAVIDSON COUNTY
2001

CORRECT Edward P. Wasserman

DESIGNED BY	J. GROSELIN	DATE 8-2001
DRAWN BY	J. GROSELIN	DATE 8-2001
SUPERVISED BY	G. DIETKAS	DATE 8-2001
CHECKED BY	G. DIETKAS	DATE 8-2001

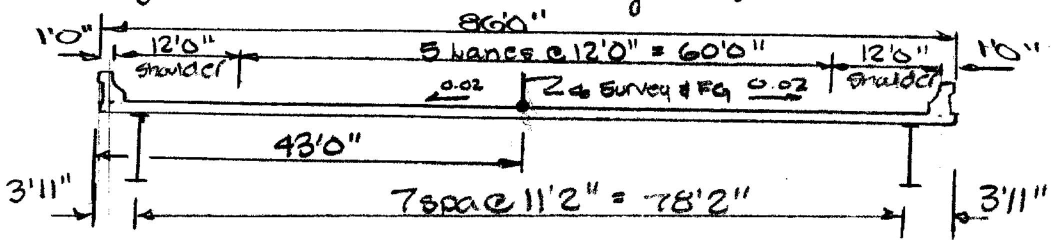
COUNTY: Davidson CROSSING: I-65: Old Hickory Blvd. to Vietnam Veterans

NOTES

Three bridges, noise barriers, & Retaining Walls

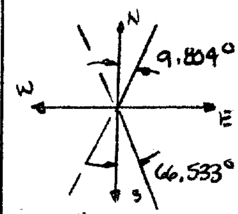
Bridge NO. 1, Old Hickory Blvd / I-65
(19-I-65 - 17.32)

Typical Section (Old Hickory Blvd) use WPG



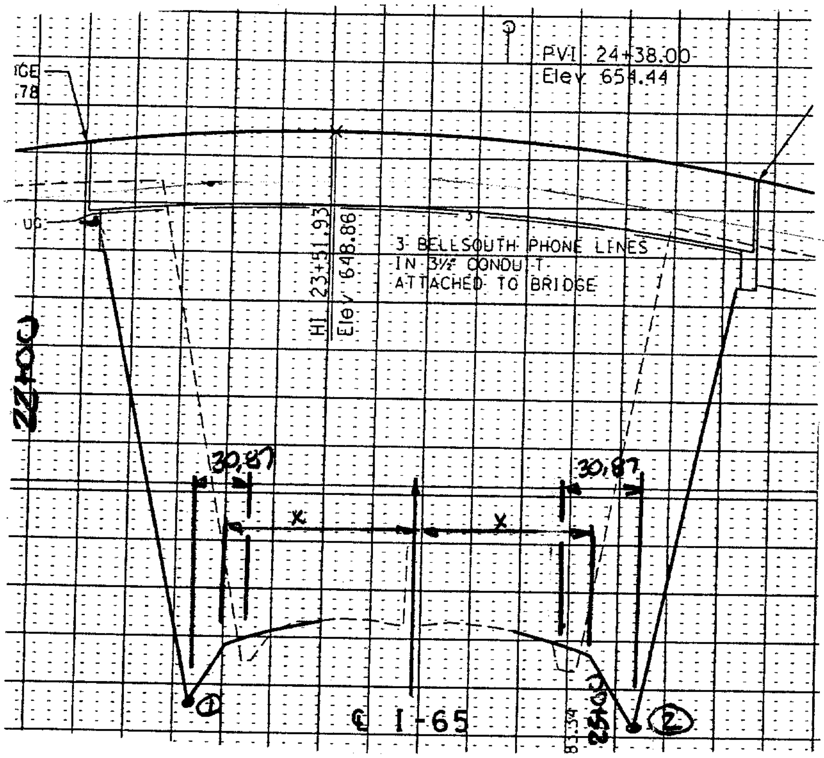
Skew

OHB:
S 66°31'57" E
I-65
N 9°48'15" E
Skew = 76.337°



Typical Section (I-65)

12'0" Shoulder - 12'0" Ramp - 2 Lanes @ 12'0" - 3 Lanes @ 11'6" - 19'0" median - 3 Lanes @ 11'6" - 2 Lanes @ 12'0" - 12'0" Ramp - 12'0" Shoulder



VC (OHB)
PI: 24+38.00
elev: 654.44
PC: 20+88.00
elev: 645.49
PT: 27+88.00
elev: 639.65
G1: +2.56%
G2: -4.23%
VCL: 700.00

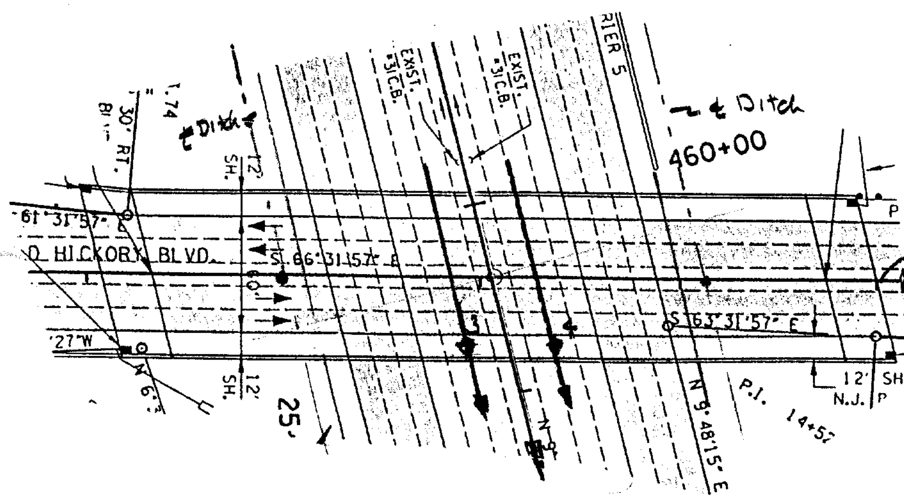
SE trans
Sta 24+83.34 NC
360' trans
Sta 28+43.34
(0.070)

Intersection Point:
23+94.397 OHB
460+38.507 I-65

Dim X

$$x = \frac{(12+12+2(12)+3(11.5)+9.5)}{\sin 76.337}$$

x = 94.679'



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**** Bridge Length Program Input Data ****

JOB DESCRIPTION...OHB / I-65

P.C. Station 20.8800

P . . Elevation 645.490

Grade 1 . . . . 2.560

Grade 2 . . . . -4.230

Length of vertical curve in stations . . . 7.00

Skew angle 76.3370

Left slope 2.00

Left slope break station 22.8120

Left slope break elevation 618.7300

Right slope 2.00

Right slope break station. 25.0760

Right slope break elevation. 619.030

**** Bridge Length Program Output Data ****

Left slope intersection point 22.2088

Right slope intersection point. 25.6450

**** Structure depths used in calculations ****

Slab depth	9.50	inches
Filler depth	1.500	inches
Bearing device depth	0.250	inches
Beam depth	63.000	inches
Berm width	4.000	feet
Abutment depth	3.000	feet
Abutment width	3.000	feet

Beginning of bridge . . . . Station 2232.5857

End of bridge . . . . Station 2552.7937

VERTICAL CURVE - VERSION 1.0 - 04/13/88

Old Hickory Blvd. over I-65
STATION Station 23+94.397
Davidson COUNTY
. URED BY whp DATE 03-15-2001

PC STA. = 20.88000
PC EL. = 645.490

G1 = 2.56000
G2 = -4.23000
V = 7.00000
K = -0.48500

STA. = 22.32586	FG. EL. = 648.177	BOB	.648/149.186 = 0.004
STA. = 23.81772	FG. EL. = 648.825	Point 3	
STA. = 23.94397	FG. EL. = 648.781	℄ Pier	
STA. = 24.26024	FG. EL. = 648.602	point 4	
STA. = 25.52794	FG. EL. = 646.911	EOB	1.691/126.770 = 0.013
STA. = 27.88000	FG. EL. = 639.645	PT	

COUNTY: Davidson CROSSING: Old Hickory Blvd over I-65 | NOTES

Bridge No. 1: (min Vert Clearance)

Point 3:

OHB: Sta $23+94.397-\left(\frac{9.5+12}{\sin 76.337}\right)+\left(\frac{39.08}{\tan 76.337}\right) = 23+81.772$

elev $648.825 - 39.08(0.02) = 648.043$

I-65: Sta $460+38.507+(39.08/\sin 76.337)-(9.5+12/\tan 76.337) = 460+73.502$

elev 624.0

Min Super-structure depth = $648.043-624.0-16.5 = 7.543$ ft

Point 4

OHB: Sta $23+94.397+\left(\frac{9.5+12}{\sin 76.337}\right)+\left(\frac{39.08}{\tan 76.337}\right) = 24+26.024$

elev $648.602-39.08(0.02) = 647.820$

I-65: Sta $460+38.507+(39.08/\sin 76.337)+(9.5+12/\tan 76.337) = 460+83.954$

elev 624.2

Min super-structure depth = $647.82-624.2-16.5 = 7.12$ ft

Approx. Bridge limits:

Point 1:

OHB: $23+94.397-94.679-18.521 = 22+81.197$

I-65: $460+38.507-(68.5+30)/\tan 76.337 = 460+14.563$

elev: $623.5-(3(11.5)0.02)-24(0.025)-12(0.04)-3.0 = 618.73$

Point 2:

OHB: $23+94.397+94.679+18.521 = 25+07.597$

I-65: $460+38.507+(68.5+30)/\tan 76.337 = 460+62.451$

elev: $623.8-(3(11.5)0.02)-24(0.025)-12(0.04)-3.0 = 619.63$

span lengths:

BOB Sta 22+32.5857

EOB Sta 25+52.7937

℄ Pier Sta 23+94.397

Span No 1 = $2394.397-2232.586 = 161.81$

Span No. 2 = $2552.794-2394.397 = 158.40$

Required Super-structure depth = $9.5+1.5+54+3 = 68$ in

= 5.667 ft

amount Grade can be lowered = $7.12-5.667 = 1.453'$

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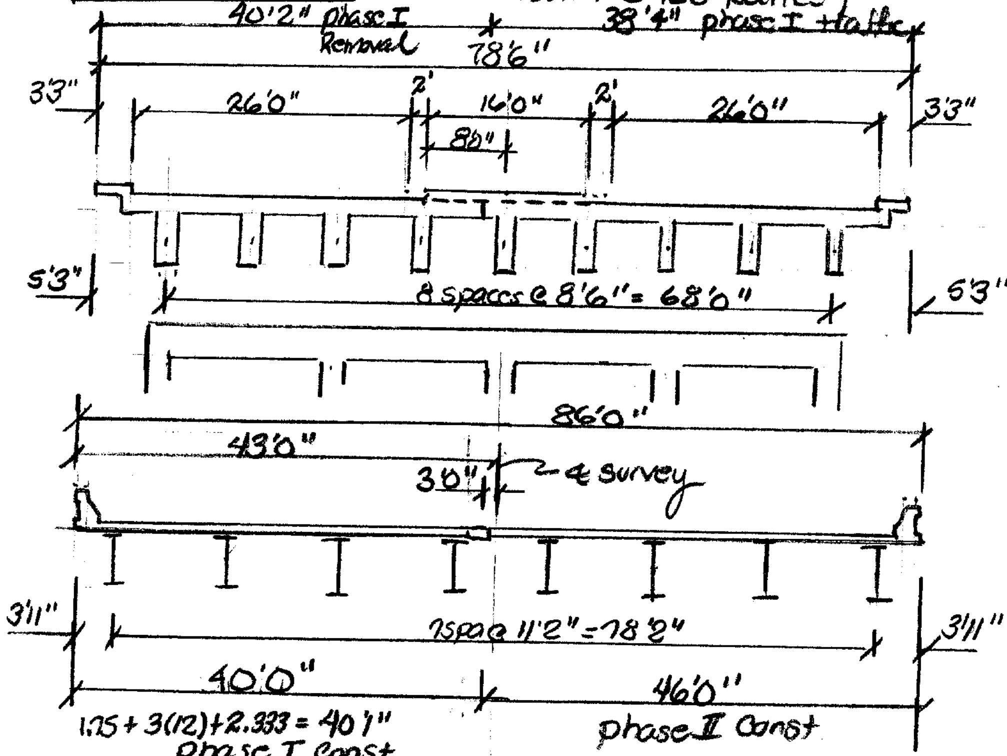
COUNTY: Davidson CROSSING: Old Hickory Blvd over I-65

Phase Const. (maintain 3-12'0" lanes)



use Double Hammerhead on proposed structure

NOTES

3(12) + 2.33 = 38'4"
26 + 2 + 8 = 36'0"

note: Bar couplers will be required

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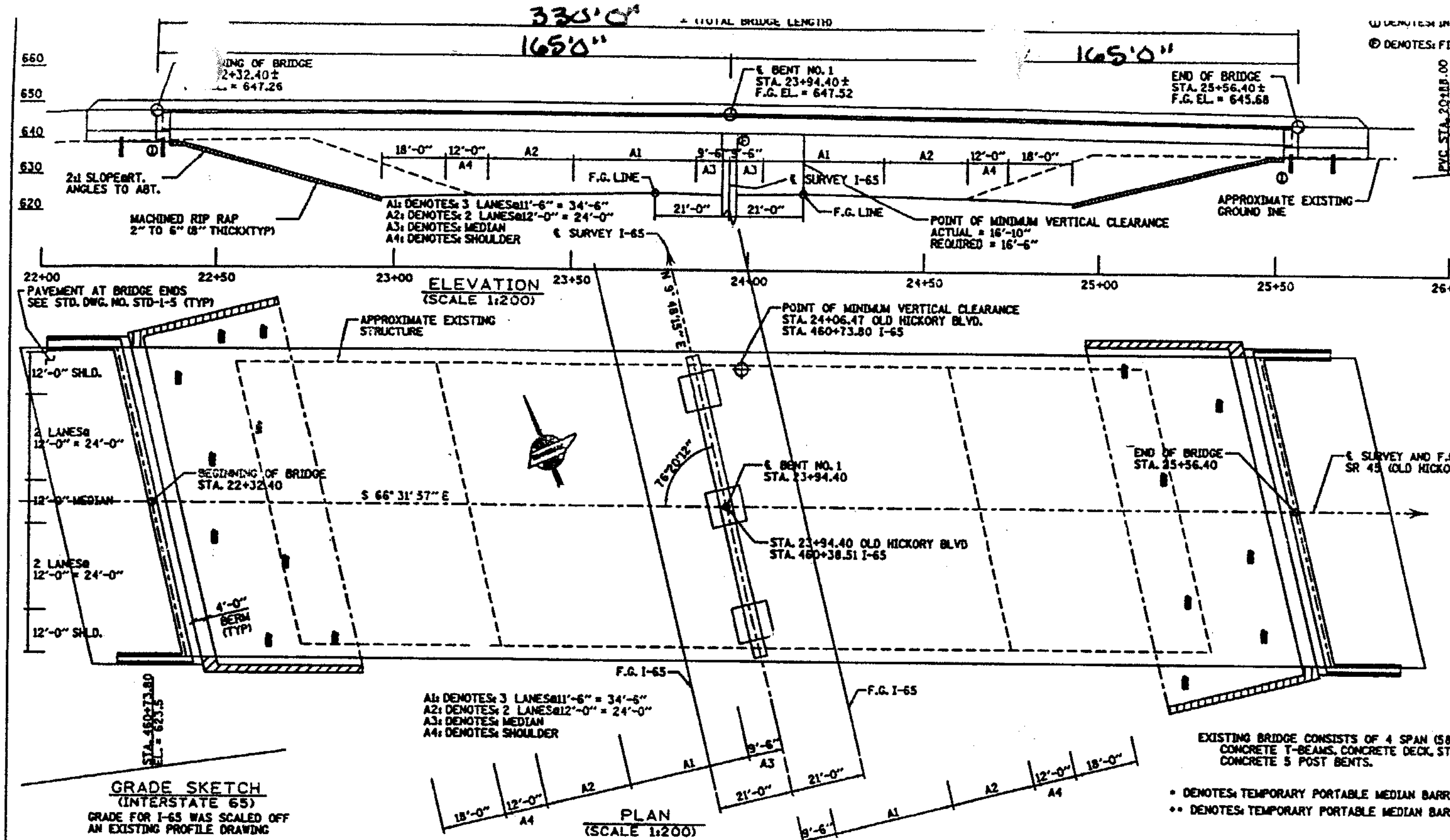
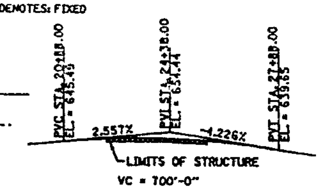


① DENOTES: INTEGRAL
Ⓕ DENOTES: FIXED



GRADE SKETCH
(OLD HICKORY BLVD.)

CONST. NO.		YEAR	SHEET NO.
PROJECT			

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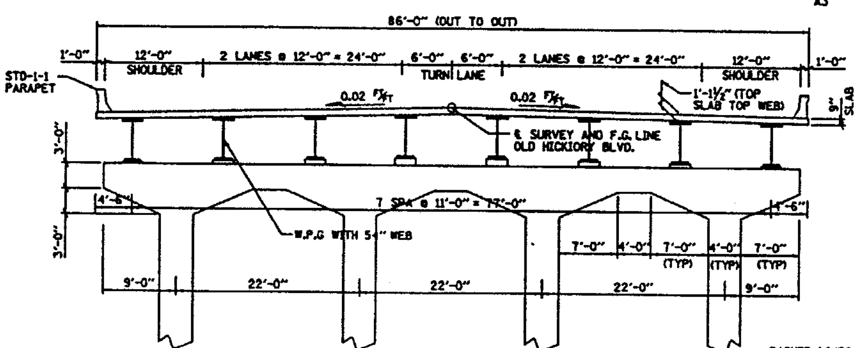
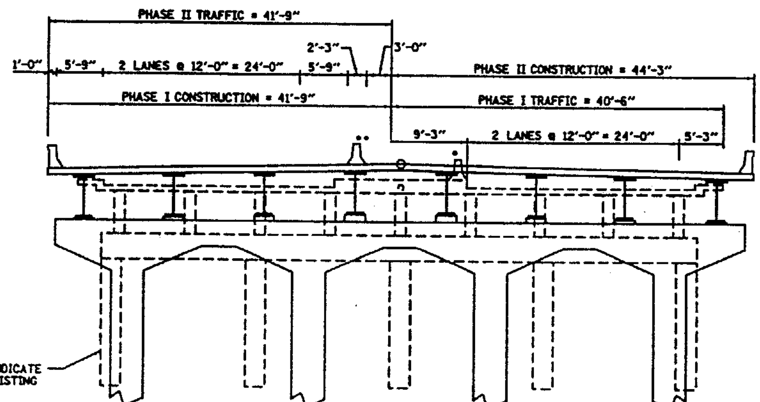


TYPICAL SECTION
(LOOKING FORWARD ON SURVEY)



PHASING SKETCH
(LOOKING FORWARD ON SURVEY)

2021 ADT =
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DESIGN SPEED =

STATE OF TENNESSEE
DEPARTMENT OF TRANSPORTATION

STATE ROUTE 45
(OLD HICKORY BLVD.)
OVER INTERSTATE 65
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STATION 23+94.40
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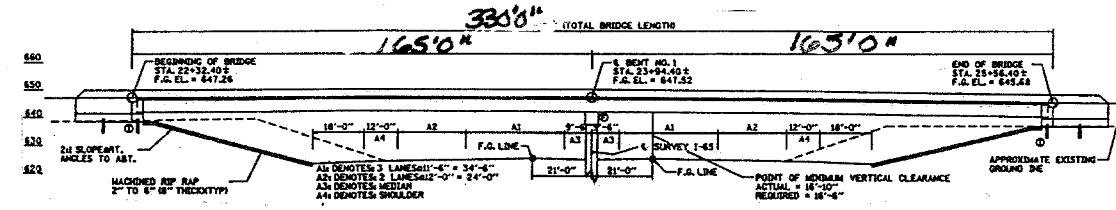
CORRECT Edward P. Wasserman

DESIGNED BY	J. GROSSLIN	DATE	8-2001
DRAWN BY	J. GROSSLIN	DATE	8-2001
SUPERVISED BY	G. DIETERS	DATE	8-2001
CHECKED BY	G. DIETERS	DATE	8-2001

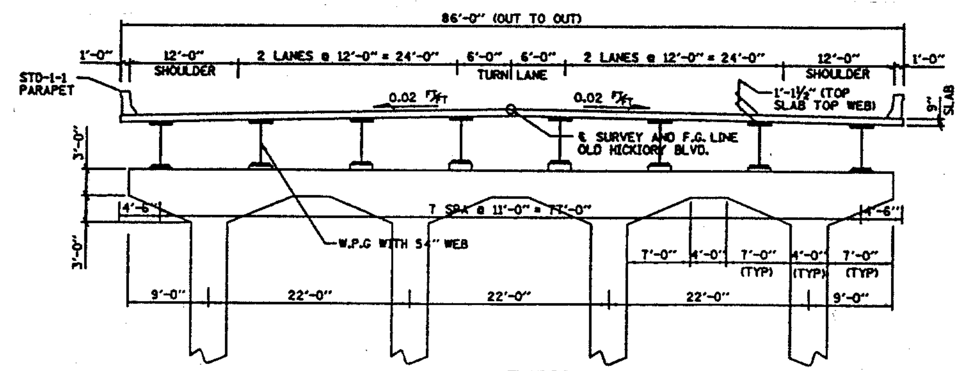
DESIGNED BY	DATE	PROJECT SR45 (Old Hickory Blvd.) / I-65 Davidson County	LRFD Design	PAGE ___ OF ___
CHECKED BY	DATE	SUBJECT		COUNTY

Proposed Bridge: (From Preliminary Calculations)



Skew: 76°20'13"
intersection Station: 23+94.40 (Old Hickory Blvd.)
460+38.51 (I-65)

Stub-Integral abutments;
Fixed-piers



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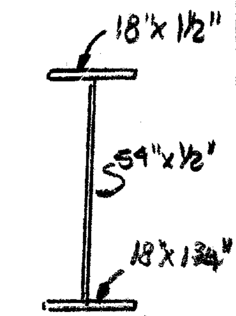


avg Girder (approx.)

Uniform loads:

non-composite:

Slab: (86.0')(.75')(.150)/8 Girders = 1.21 K/ft-Girder

filler: (18")(1.5")(1/144)(0.150) = 0.03 K/ft-Girder

total Slab: 1.24 K/ft-Girder

Composite:

Wearing surface: (0.033 K/ft²)(86-1.75(2))/8 Girders = 0.34 K/ft-Girder

Bridge rail: (0.112 cy/ft)(27 cf/cy)(0.150 lbs/cf)(2)/8 Girders = 0.11 K/ft-Girder

approx. Dead load of Girder:

avg top flg. 18" x 1.5", avg Bot flg. 18" x 1.75", avg web 54" x .5"

.32+.10=.42 K/ft avg. Girder Cross-sectional weight: [(1.5 x .125)+(4.5 x .042)+(1.5 x .146)] 490 #/cf = .29 K/ft

.10 is a future weight allowance

Cross-frames, splice plates, and etc. @ 10% girder weight

detail weight = .10(0.29) = .03 K/ft-girder

Total avg. Girder weight = 0.32 K/ft-girder

Total Non composite weight = 1.24 K/ft-Girder + 0.42 K/ft-girder = 1.66 K/ft-girder

Total composite weight = 0.34 K/ft-Girder + 0.11 K/ft-girder = 0.45 K/ft-girder

Distribution of live loads per lane for moment interior girders

art. 4.6.2.2.2b interior Beams w/ Concrete Decks

Table 4.6.2.2.2b-1

Type of Beams	Applicable Cross-Section from Table 4.6.2.2.1-1	Distribution Factors	Range of Applicability
Concrete Deck, Filled Grid, or Partially Filled Grid on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Sections	a, e, k and also i, j if sufficiently connected to act as a unit	One Design Lane Loaded: $0.06+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$ Two or More Design Lanes Loaded: $0.075+\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$	$3.5 \le S \le 16.0$ $4.5 \le t_s \le 12.0$ $20 \le L \le 240$ $N_b \ge 4$

<u>Distribution of live loads (cont.)</u> (Moment only)
int. Beam

S = Beam spacing (ft)
L = Span of Beam (ft)
t_s = depth of Concrete Slab (in)
e_g = distance between centers of gravity of basic beam and concrete deck (in)
I = moment of inertia of Beam (in⁴)

K_g = long. stiffness parameter (in⁴)
N_b = number of girders
A = area of girder
E_B = modulus of elasticity of Beam mat.
E_D = modulus of elasticity of deck mat.

$$K_g = n(I + Ae_g^2)\ ;\ n = E_B/E_D = 9$$

Girder Inertia Calculation: (Reference line at center of web)

Component		Area	y	A(y)	Ay²	I_o	Iref
18" x 1.5"	flange	27.0	27.75	749.25	20,791.7	5.06	20,796.8
54" x 0.5"	web	27.0	0	0	0	6561	6561
18" x 1.75"	flange	31.5	-27.88	-878.22	24,484.8	8.04	24,492.8
		85.5		-128.97			51,850.6

$y = -128.97/85.5 = -1.51$ in
$I_{ref} = I_{beam} + Ad^2,\ I_{beam} = 51{,}850.6 - 85.5(1.51)^2 = 51{,}655.7$ in⁴
$e_g = (54)(0.5) + 1.51 + 1.50 + 9.0(0.5) = 34.51$ in
$A = 85.5$ in²
$K_g = 9(51{,}655.71 + 85.5(34.51)^2) = 1{,}381{,}330$ in⁴
$L = 165$ ft

one Design Lane loaded:
$$LLDF = 0.06 + \left(\frac{11.0}{14}\right)^{.4}\left(\frac{11.0}{165.0}\right)^{.3}\left(\frac{1{,}381{,}330}{12.0(165)(9.0)^3}\right)^{.1} = 0.401 \text{ Lanes/Girder}$$

two or more Design lanes loaded:
$$LLDF = .075 + \left(\frac{11.0}{9.5}\right)^{.6}\left(\frac{11.0}{165}\right)^{.2}\left(\frac{1{,}381{,}330}{12.0(165)(9.0)^3}\right)^{.1} = 0.708 \text{ Lanes/Girder}$$

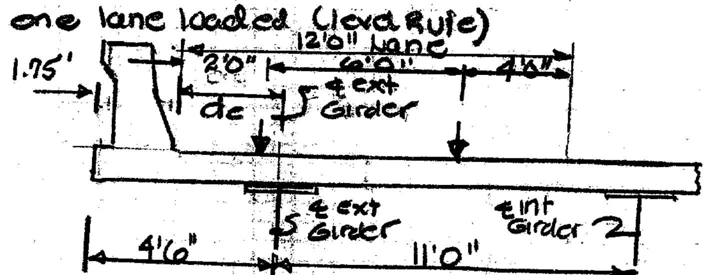
Distribution of live loads per lane for <u>moment</u> exterior beams
art. 4.6.2.2.2d Exterior Beams.

Table 4.6.2.2.2d-1

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	One Design Lane Loaded	Two or More Design Lanes Loaded	Range of Applicability
Concrete Deck, Filled Grid, or Partially Filled Grid on Steel or Concrete Beams; Concrete T-Beams, T and Double T Sections	a, e, k and also i, j if sufficiently connected to act as a unit	Lever Rule	$g = e\, g_{interior}$ $e = 0.77 + \frac{d_e}{9.1}$	$-1.0 \le d_e \le 5.5$

de = distance between the center of exterior beam and interior edge of curb or traffic barrier

one lane loaded (Lever Rule)



de = 4.5 - 1.75 = 2.75

art. 3.6.1.1.2
multi-presence factor
one lane loaded, m = 1.20
$R = (1.0 + 5.75/11.0) = 1.523$ lines of wheels/Girder
$LLDF = 1.523\left(\frac{1\text{ lane}}{2\text{ wheels}}\right)(1.2) = .914$ Lanes/Girder

Distribution of live loads (cont.) (moment only ext. Beam)

two or more lanes loaded

$g = e(g_{interior})$

$e = 0.77 + \frac{d_e}{9.1} = 0.77 + \frac{2.75}{9.1} = 1.072$

$g = 1.072(0.708) = 0.759$ Lanes/Girder

Skewed Bridges: Art 4.6.2.2.2e

$\theta = 90° - 76.337° = 13.663°$

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	Any Number of Design Lanes Loaded	Range of Applicability
Concrete Deck, Filled Grid, or Partially Filled Grid on Steel or Concrete Beams; Concrete T-Beams, T or Double T Sections	a, e, k and also i, j if sufficiently connected to act as a unit	$1 - c_1(\tan\theta)^{1.5}$ $c_1 = 0.25\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5}$ If $\theta < 30°$ then $c_1 = 0.0$ If $\theta > 60°$ use $\theta = 60°$	$30° \le \theta \le 60°$ $3.5 \le S \le 16.0$ $20 \le L \le 240$ $N_b \ge 4$

Table 4.6.2.2.2e-1 Reduction of Load Distribution Factors for moment in long. Beams on skewed supports

If $\theta < 30°$, then $c_1 = 0.0$ therefore $1 - (0.0)(\tan 13.663)^{1.5} = 1.0$ no reduction

art. 4.6.2.2.3
Distribution of live loads for shear (Int. Beam)

Table 4.6.2.2.3a-1 - Distribution of Live Load per Lane for Shear in Interior Beams

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	One Design Lane Loaded	Two or More Design Lanes Loaded	Range of Applicability
Concrete Deck, Filled Grid, or Partially Filled Grid on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Sections	a, e, k and also i, j if sufficiently connected to act as a unit	$0.36 + \frac{S}{25.0}$	$0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^{2.0}$	$3.5 \le S \le 16.0$ $20 \le L \le 240$ $4.5 \le t_s \le 12.0$ $10{,}000 \le K_g \le 7{,}000{,}000$ $N_b \ge 4$

one lane loaded:

$LLDF = 0.36 + S/25.0 = 0.36 + 11.0/25.0 = 0.800$ Lanes/Girder

two Lanes loaded:

$LLDF = 0.2 + S/12 - (S/35)^{2.0} = 0.2 + 11.0/12 - (11.0/35)^2 = 1.018$ Lanes/Girder

Art. 4.6.2.2.3b Exterior Beams, Shear

Table 4.6.2.2.3b-1 - Distribution of Live Load per Lane for Shear in Exterior Beams

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	One Design Lane Loaded	Two or More Design Lanes Loaded	Range of Applicability
Concrete Deck, Filled Grid, or Partially Filled Grid on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Beams	a, e, k and also i, j if sufficiently connected to act as a unit	Lever Rule	$g = e\, g_{interior}$ $e = 0.6 + \frac{d_e}{10}$	$-1.0 \le d_e \le 5.5$

Distribution of loads (cont.) (Shear, ext. Girder)

one lane loaded (lever rule)

$g = 0.914$ lanes/Girder (same as g for moment)

two or more design lanes loaded

$g = e\,(g_{int.})$

$e = 0.6 + d_e/10 = 0.6 + 2.75/10 = 0.875$

$g = 0.875(1.018) = 0.891$ lanes/Girder

art. 4.6.2.2.3c Skewed Bridges

$\theta = 13.663°$

Table 4.6.2.2.3c-1 - Correction Factors for Load Distribution Factors for Support Shear of the Obtuse Corner

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	Correction Factor	Range of Applicability
Concrete Deck, Filled Grid, or Partially Filled Grid on Steel or Concrete Beams; Concrete T-Beams, T- and Double T Section	a, e, k and also i, j if sufficiently connected to act as a unit	$1.0 + 0.20\left(\frac{12.0Lt_s^3}{K_g}\right)^{0.3}\tan\theta$	$0° \le \theta \le 60°$ $3.5 \le S \le 16.0$ $20 \le L \le 240$ $N_b \ge 4$

Correction factor $= 1.0 + 0.20\left(\frac{12.0(165)(9)^3}{1{,}381{,}330.0}\right)^{.3}(\tan 13.663) = 1.061$

Summary of Live Load Distribution factors

(LLDF of deflection
$(84/12)(1/8 \text{ Girders})(.65) = .569$
12' traffic lane — reduction

Moment:

Beam	one lane loaded	two or more lanes loaded
ext.	0.914	0.759
int.	0.401	0.708

Shear

Beam	one lane loaded	two or more lanes loaded
ext.	0.914(1.061) = 0.970	0.891(1.061) = 0.945
int.	0.800(1.061) = 0.849	1.018(1.061) = 1.080

effective slab width (Art. 4.6.2.6)

Int. Beams

1/4 effective span = 1/4(116)(12) = 345 in — end of Girder to CL inflection

⇒ 12 x slab thickness + 1/2 width of Top flg = 12(9) + 18/2 = 117 in

Beam spacing = 11.00(12) = 132 in

controlling int. Beam = 117 in

ext. Beam: (1/2 effect slab int. Beam + least of Below

1/8 (effective span = 1/8(116)(12) = 174 in

⇒ 6 x slab thickness + 1/4 width top flg = 58.5 in

The width of overhang = 4.5(12) = 54 in

controlling ext. Beam = 117(.5) + 58.5 = 117 in

use 117 in for effective slab width

Dynamic load allowance: (Art. 3.6.2)

Fatigue and Fracture Limit State: $IM = 1.15$

All other limit states: $IM = 1.33$

Article 4.6.2.2.2d Exterior Beams

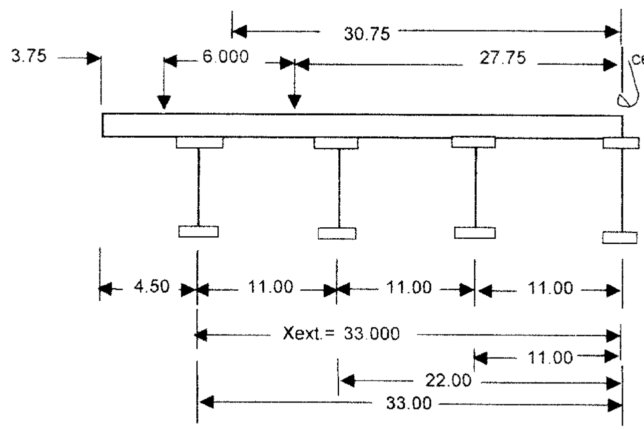
The additional investigation is required because the distribution factor for girders in a multi-girder cross-section, Type "a","e", and "k" in table 4.6.2.2.1-1, was determined without consideration of diaphragm or cross-frames.
The recommended procedure is an interim provision until research provides a better solution.

The procedure outlined in this section is the same as the conventional approximation for loads on piles.

R = (NL / Nb) +((Xext)(sum e))lanes / ((sum x^2))beams

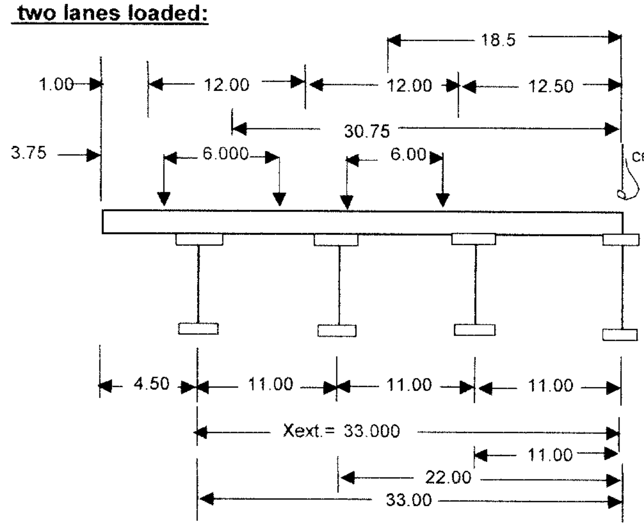
where: R = reaction on exterior beam in terms of lanes
NL = number of loaded lanes under consideration
e = eccentricityj of a design truck or a design lane load from the center of gravity of the pattern of girders (FT)
x = horizontal distance from the center of gravity of the pattern of girders to each girder (FT)
Xext. = horizontal distance from the center of gravity of the pattern of girders to the exterior girder (FT)

one loaded:



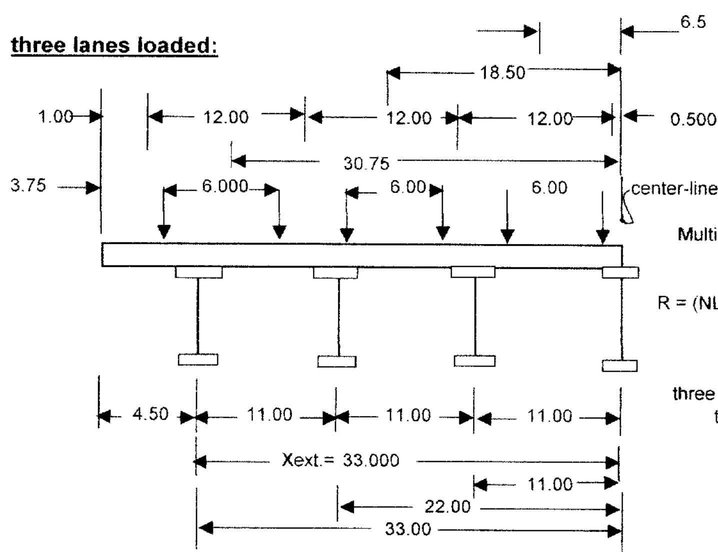
Multi-presense factor for one lane loaded: 1.2
number of lanes: 1
number of beams: 7
R = (NL / Nb) +((Xext)(sum e))lanes / ((sum x^2))beams
Lanes: ((Xext)(sum e)): 1014.750
Beams: ((sum X^2)): 3388.000
NL / Nb : 0.143
one lane loaded:
therefore: R = 0.531 lanes per beam

two lanes loaded:



Multi-presense factor for one lane loaded: 1
number of lanes: 2
number of beams: 7
R = (NL / Nb) +((Xext)(sum e))lanes / ((sum x^2))beams
Lanes: ((Xext)(sum e)): 1625.250
Beams: ((sum X^2)): 3388.000
NL / Nb : 0.286
two lanes loaded:
therefore: R = 0.765 lanes per beam

three lanes loaded:



Multi-presense factor for one lane loaded: 0.85
number of lanes: 3
number of beams: 7
R = (NL / Nb) +((Xext)(sum e))lanes / ((sum x^2))beams
Lanes: ((Xext)(sum e)): 1839.750
Beams: ((sum X^2)): 3388.000
NL / Nb : 0.429
three lanes loaded:
therefore: R = 0.826 lanes per beam

Supporting Component: Steel Beam
Deck Type : CIP/Precast Concrete

Int. Girder

Supporting Component Description
Interior Girder
Top Flange t = 1.5 inch w = 18 inch
Web t = 0.5 inch h = 54 inch
Bottom Flange t = 1.75 inch w = 18 inch
Unit Wgt = 490 pcf
Mod. E = 2.9e+07 psi
Span Length = 162.5 feet
Girder Spacing = 11 feet
Num. Beams = 8

Deck Description
Slab Depth = 9 inch
Pad Depth = 1.5 inch
Sacrificial Depth = 0 inch
Unit Wgt = 150 pcf
f'c = 3000 psi
Eff. Span Length = 162.5 feet
Design Lane Width = 12 feet

Skew Corrections
Distribution factors for moment are corrected for skew
Distribution factors for shear are corrected for skew
Skew Angle = 13.663 deg

Girder Properties
Ax = 85.500e+00 inch^2
Iz = 51.647e+03 inch^4
CG = 27.243e+00 inch
Slab Properties
Eff. Flange Width = 117.000e+00 inch
Mod. E = 3.340e+06 psi

Composite Properties
Ax = 209.912e+00 inch^2
Iz = 117.771e+03 inch^4
CG = 48.506e+00 inch
Unit Wgt = 971.334 pcf
Mod. E = 29.000e+06 psi
n = 8.68081

Distribution Factors
eg = 36.006e+00 inch
Kg = 1.410e+06 inch^4

Strength/Service Limit State
Moment
1 Loaded Lane = 0.463275
2+ Loaded Lanes = 0.711177
Shear
1 Loaded Lane = 0.840199
2+ Loaded Lanes = 1.08349

Fatigue Limit State
gMoment = 0.386063
gShear = 0.700166

Supporting Component: Steel Beam
Deck Type : CIP/Precast Concrete

ext. Girder

Supporting Component Description
Exterior Girder
Top Flange t = 1.5 inch w = 18 inch
Web t = 0.5 inch h = 54 inch
Bottom Flange t = 1.75 inch w = 18 inch
Unit Wgt = 490 pcf
Mod. E = 2.9e+07 psi
Cross Frames are present
Span Length = 162.5 feet
Girder Spacing = 11 feet
Num. Beams = 8
Num. Lanes = 3

Deck Description
Slab Depth = 9 inch
Pad Depth = 1.5 inch
Sacrificial Depth = 0 inch
Overhang = 54 inch
de = 33 inch
Unit Wgt = 150 pcf
f'c = 3000 psi
Eff. Span Length = 162.5 feet
Design Lane Width = 12 feet

Skew Corrections
Distribution factors for moment are corrected for skew
Distribution factors for shear are corrected for skew
Skew Angle = 13.667 deg

Girder Properties
Ax = 85.500e+00 inch^2
Iz = 51.647e+03 inch^4
CG = 27.243e+00 inch
Slab Properties
Eff. Flange Width = 112.500e+00 inch
Mod. E = 3.340e+06 psi

Composite Properties
Ax = 205.246e+00 inch^2
Iz = 116.702e+03 inch^4
CG = 48.171e+00 inch
Unit Wgt = 963.815 pcf
Mod. E = 29.000e+06 psi
n = 8.68081

Distribution Factors
eg = 36.006e+00 inch
Kg = 1.410e+06 inch^4

Strength/Service Limit State
Moment
1 Loaded Lane = 0.963135
2+ Loaded Lanes = 0.788737
Shear

Shear:
1 Loaded Lane = 1.01008
2+ Loaded Lanes = 0.952838

Fatigue Limit State
gMoment = 0.802613
gShear = 0.841737

SPECIFICATIONS

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1.3 DESIGN PHILOSOPHY

1.3.1 General

Bridges shall be designed for specified limit states to achieve the objectives of constructibility, safety, and serviceability, with due regard to issues of inspectability, economy, and aesthetics, as specified in Article 2.5.

Regardless of the type of analysis used, Equation 1.3.2.1-1 shall be satisfied for all specified force effects and combinations thereof.

C1.3.1

The resistance of components and connections is determined, in many cases, on the basis of inelastic behavior, although the force effects are determined by using elastic analysis. This inconsistency is common to most current bridge specifications as a result of incomplete knowledge of inelastic structural action.

1.3.2 Limit States

1.3.2.1 GENERAL

Each component and connection shall satisfy Equation 1 for each limit state, unless otherwise specified. For service and extreme event limit states, resistance factors shall be taken as 1.0, except for bolts, for which the provisions of Article 6.5.5 shall apply. All limit states shall be considered of equal importance.

$$\Sigma \eta_i \gamma_i Q_i \le \varphi R_n = R_r \quad (1.3.2.1\text{-}1)$$

for which:

For loads for which a maximum value of γ_i is appropriate:

$$\eta_i = \eta_D \eta_R \eta_I \ge 0.95 \quad (1.3.2.1\text{-}2)$$

For loads for which a minimum value of γ_i is appropriate:

$$\eta_i = \frac{1}{\eta_D \eta_R \eta_I} \le 1.0 \quad (1.3.2.1\text{-}3)$$

where:

γ_i = load factor: a statistically based multiplier applied to force effects

φ = resistance factor: a statistically based multiplier applied to nominal resistance, as specified in Sections 5, 6, 7, 8, 10, 11, and 12

η_i = load modifier: a factor relating to ductility, redundancy, and operational importance

η_D = a factor relating to ductility, as specified in Article 1.3.3

η_R = a factor relating to redundancy as specified in Article 1.3.4

C1.3.2.1

Equation 1 is the basis of LRFD methodology.

Assigning resistance factor φ = 1.0 to all nonstrength limit states is a temporary measure; development work is in progress.

Ductility, redundancy, and operational importance are significant aspects affecting the margin of safety of bridges. Whereas the first two directly relate to physical strength, the last concerns the consequences of the bridge being out of service. The grouping of these aspects on the load side of Equation 1 is, therefore, arbitrary. However, it constitutes a first effort at codification. In the absence of more precise information, each effect, except that for fatigue and fracture, is estimated as ±5 percent, accumulated geometrically, a clearly subjective approach. With time, improved quantification of ductility, redundancy, and operational importance, and their interaction and system synergy, may be attained, possibly leading to a rearrangement of Equation 1, in which these effects may appear on either side of the equation or on both sides. NCHRP Project 12-36 is currently addressing the issue of redundancy.

The influence of η on the reliability index, β, can be estimated by observing its effect on the minimum values of β calculated in a database of girder-type bridges. For discussion purposes, the girder bridge data used in the calibration of these Specifications was modified by multiplying the total factored loads by η = 0.95, 1.0, 1.05, and 1.10. The resulting minimum values of β for 95 combinations of span, spacing, and type of construction were determined to be approximately 3.0, 3.5, 3.8, and 4.0, respectively.

A further approximate representation of the effect of η values can be obtained by considering the percent of random normal data less than or equal to the mean value plus $\lambda\,\sigma$, where λ is a multiplier, and σ is the standard deviation of the data. If λ is taken as 3.0, 3.5, 3.8, and 4.0, the percent of values less than or equal to the mean value plus $\lambda\,\sigma$ would be about 99.865 percent,

SPECIFICATIONS

= 1.00 for conventional designs and details complying with these Specifications

≥ 0.95 for components and connections for which additional ductility-enhancing measures have been specified beyond those required by these Specifications

For all other limit states:

η_D = 1.00

COMMENTARY

can be considered ductile. Such ductile performance shall be verified by testing.

In order to achieve adequate inelastic behavior the system should have a sufficient number of ductile members and either:

- Joints and connections that are also ductile and can provide energy dissipation without loss of capacity; or
- Joints and connections that have sufficient excess strength so as to assure that the inelastic response occurs at the locations designed to provide ductile, energy absorbing response.

Statically ductile, but dynamically nonductile response characteristics should be avoided. Examples of this behavior are shear and bond failures in concrete members and loss of composite action in flexural components.

Past experience indicates that typical components designed in accordance with these provisions generally exhibit adequate ductility. Connection and joints require special attention to detailing and the provision of load paths.

The Owner may specify a minimum ductility factor as an assurance that ductile failure modes will be obtained. The factor may be defined as:

$$\mu = \frac{\Delta_u}{\Delta_y} \qquad \text{(C1.3.3-1)}$$

where:

Δ_u - deformation at ultimate

Δ_y - deformation at the elastic limit

The ductility capacity of structural components or connections may either be established by full- or large-scale testing or with analytical models based on documented material behavior. The ductility capacity for a structural system may be determined by integrating local deformations over the entire structural system.

The special requirements for energy dissipating devices are imposed because of the rigorous demands placed on these components.

1.3.4 Redundancy

Multiple-load-path and continuous structures should be used unless there are compelling reasons not to use them.

C1.3.4

For each load combination and limit state under consideration, member redundancy classification (redundant or nonredundant) should be based upon the member contribution to the bridge safety. Several

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REFERENCES

Frangopol, D. M., and R. Nakib. "Redundancy in Highway Bridges." Engineering Journal, AISC, Vol. 28, No. 1, 1991, pp. 45-50.

Table 3.4.1-1 - Load Combinations and Load Factors

Load Combination Limit State	DC DD DW EH EV ES **EL**	LL IM CE BR PL LS	WA	WS	WL	FR	TU CR SH	TG	SE	Use One of These at a Time			
										EQ	IC	CT	CV
STRENGTH-I (unless noted)	γ_p	1.75	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-II	γ_p	1.35	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-III	γ_p	-	1.00	1.40	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-IV EH, EV, ES, DW DC ONLY	γ_p 1.5	-	1.00	-	-	1.00	0.50/1.20	-	-	-	-	-	-
STRENGTH-V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
EXTREME EVENT-I	γ_p	γ_{EQ}	1.00	-	-	1.00	-	-	-	1.00	-	-	-
EXTREME EVENT-II	γ_p	0.50	1.00	-	-	1.00	-	-	-	-	1.00	1.00	1.00
SERVICE-I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-
SERVICE-II	1.00	1.30	1.00	-	-	1.00	1.00/1.20	-	-	-	-	-	-
SERVICE-III	1.00	0.80	1.00	-	-	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-
FATIGUE-LL, IM & CE ONLY	-	0.75	-	-	-	-	-	-	-	-	-	-	-

Table 3.4.1-2 - Load Factors for Permanent Loads, γ_p

Type of Load	Load Factor	
	Maximum	Minimum
DC: Component and Attachments	1.25	0.90
DD: Downdrag	1.80	0.45
DW: Wearing Surfaces and Utilities	1.50	0.65
EH: Horizontal Earth Pressure • Active • At-Rest	 1.50 1.35	 0.90 0.90
EL: Locked-in Erection Stresses	1.0	1.0
EV: Vertical Earth Pressure • Retaining Walls and Abutments • Rigid Buried Structure • Rigid Frames • Flexible Buried Structures other than Metal Box Culverts • Flexible Metal Box Culverts	 1.35 1.30 1.35 1.95 1.50	 1.00 0.90 0.90 0.90 0.90
ES: Earth Surcharge	1.50	0.75

'99 '00

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GIRDER ANALYSIS
FOR
AASHTO LRFD LOADS

BTBEAM
Version 2.0

BridgeTech, Inc.
Laramie, Wyoming

If you have questions, contact:

BridgeTech, Inc. at 800-731-5070.

GIRDER ANALYSIS
FOR
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Version 2.0

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```
JOB TITLE:
DATE     :  7/12/2001
TIME     :  9:17. 0


ECHO INPUT FILE ======================

TITLE                   State Route 45 over I-65
COMMENT                 LRFD Design, Live Loads Only composite section
OUTPUT                  0  0  1
SPAN                    163.50  163.50
INBEAM                  1   1.000    1.000    1.000    1.000    1.000    1.000
INBEAM                  2   1.000    1.000    1.000    1.000    1.000    1.000
EXBEAM                  1   1.000    1.000    1.000    1.000    1.000    1.000
EXBEAM                  2   1.000    1.000    1.000    1.000    1.000    1.000
INREACT                 1   1.000    1.000    1.000    1.000    1.000    1.000
INREACT                 2   1.000    1.000    1.000    1.000    1.000    1.000
EXREACT                 1   1.000    1.000    1.000    1.000    1.000    1.000
EXREACT                 2   1.000    1.000    1.000    1.000    1.000    1.000
FIXITY                   1   10
FIXITY                   2   10
INERTIA                  1          1.000
INERTIA                  2          1.000
INERTIA                  3          1.000
INERTIA                  4          1.000
INERTIA                  5          1.000
INERTIA                  6          1.000
INERTIA                  7          1.000
INERTIA                  8          1.000
INERTIA                  9          1.000
INERTIA                  10          1.00
INERTIA                  11          1.00
INERTIA                  12          1.00
INERTIA                  13          1.00
INERTIA                  14          1.00
INERTIA                  16          1.00
INERTIA                  17          1.00
INERTIA                  18          1.00
INERTIA                  19          1.00
INERTIA                  20          1.00
DEAD                     0.000   0.000   0.000   0.000
IMPACT                   1.330   1.330   1.330   1.330   1.150   1.000   1.000
1.
MULTIPLE                 1.000   1.000
GENERAL                  1.00   1.000   1.000   1.000   1.000   1.000
FACTOR                   1.000   1.000   1.000   1.000   1.000   1.000   1.000
1.
CONTROL                 1  1  0  0  0 1  0 0
LIMITS                   1   1   1   1   1

END INPUT FILE ========================


========== ADMINISTRATIVE INFORMATION ==========

JOB TITLE:  State Route 45 over I-65
```

DATE : 7/12/2001
TIME : 9:17. 0

========== SUMMARY OF STRUCTURE INFORMATION ==========

```
          =====================
          ^         ^         ^
  Spans        1         2
  Nodes   1         11        21
Elements    1 to 10     1 to 20
Supports  1          2         3
```

SPAN NO.	LENGTH
1	163.50
2	163.50

MOMENTS OF INERTIA

ELEMENT	INERTIA
1	1.00
2	1.00
3	1.00
4	1.00
5	1.00
6	1.00
7	1.00
8	1.00
9	1.00
10	1.00
11	1.00
12	1.00
13	1.00
14	1.00
15	1.00
16	1.00
17	1.00
18	1.00
19	1.00
20	1.00

```
JOB TITLE:  State Route 45 over I-65
DATE     :  7/12/2001
TIME     :  9:17. 0


OUTPUT OPTION: MINIMAL

EQUATION HEADER PRINTED WITH COMBINATION TABLES


UNITS: IMPERIAL (KIPS, FT)

LOAD FACTOR SUMMARY
  IMPORTANCE FACTOR (STRENGTH/OTHERS)      : 1.00 1.00
  DUCTILITY FACTOR  (STRENGTH/OTHERS)      : 1.00 1.00
  REDUNDANCY FACTOR (STRENGTH/OTHERS)      : 1.00 1.00
  COMP. LOAD FACTORS (MAX/MIN/SERVICE)     : 1.00 1.00 1.00
  WEARING SURF FACTORS (MAX/MIN/SERVICE)   : 1.00 1.00 1.00
  LIVE LOAD FACTOR -- STRENGTH I & II      : 1.00 1.00
  LIVE LOAD FACTOR -- SERVICE  I & II      : 1.00 1.00
  LIVE LOAD FACTOR -- FATIGUE              : 0.00
  TRUCK IMPACT (STR I&II/SER I&II/FATIGUE): 1.33 1.33 1.33 1.33 1.15
  LANE  IMPACT (STR I&II/SER I&II/FATIGUE): 1.00 1.00 1.00 1.00 0.00


CONTROL PARAMETERS
     DATA SCANNER                    : YES
     VARIABLE AXLES SPACING          : YES
     NEGLECT AXLES THAT DO NOT
       CONTRIBUTE TO CRITICAL
       LOAD EFFECT (3.6.1.3.1)       : YES
     DESIGN TRUCK TRAIN              : YES
     INTERIOR OR EXTERIOR GIRDER : BOTH
     ONE OR MULTIPLE LANES LOADED: BOTH
     MAX OR MIN DEAD LOAD FACTORS: BOTH
     STRENGTH I  LIMIT STATE         : YES
     SERVICE  I  LIMIT STATE         : YES
     FATIGUE     LIMIT STATE         : YES
     STRENGTH II LIMIT STATE         : YES
     SERIVCE  II LIMIT STATE         : YES

INTERIOR GIRDER DATA
     COMPONENT DEAD LOAD                           :  0.00
     WEARING SURFACE DEAD LOAD                     :  0.00
     DIST. FACTOR (SHEAR-ONE LANE)       BY SPAN:  1.00 1.00
     DIST. FACTOR (MOMENT-ONE LANE)      BY SPAN:  1.00 1.00
     DIST. FACTOR (DEFL-ONE LANE)        BY SPAN:  1.00 1.00
     DIST. FACTOR (SHEAR-MULTI  LANE)    BY SPAN:  1.00 1.00
     DIST. FACTOR (MOMENT-MULTI LANE)    BY SPAN:  1.00 1.00
     DIST. FACTOR (DEFL-MULTI   LANE)    BY SPAN:  1.00 1.00
     DIST. FACTOR (VERT-ONE LANE)     BY SUPPORT:  1.00 1.00 1.00
     DIST. FACTOR (MOMENT-ONE LANE)   BY SUPPORT:  1.00 1.00 1.00
     DIST. FACTOR (DEFL-ONE LANE)     BY SUPPORT:  1.00 1.00 1.00
     DIST. FACTOR (VERT-MULTILANE)    BY SUPPORT:  1.00 1.00 1.00
     DIST. FACTOR (MOMENT-MULTILANE) BY SUPPORT:  1.00 1.00 1.00
```

DIST. FACTOR (DEFL-MULTILANE) BY SUPPORT: 1.00 1.00 1.00

EXTERIOR GIRDER DATA

COMPONENT DEAD LOAD : 0.00
WEARING SURFACE DEAD LOAD : 0.00
DIST. FACTOR (SHEAR-ONE LANE) BY SPAN: 1.00 1.00
DIST. FACTOR (MOMENT-ONE LANE) BY SPAN: 1.00 1.00
DIST. FACTOR (DEFL-ONE LANE) BY SPAN: 1.00 1.00
DIST. FACTOR (SHEAR-MULTI LANE) BY SPAN: 1.00 1.00
DIST. FACTOR (MOMENT-MULTI LANE) BY SPAN: 1.00 1.00
DIST. FACTOR (DEFL-MULTI LANE) BY SPAN: 1.00 1.00
DIST. FACTOR (VERT-ONE LANE) BY SUPPORT: 1.00 1.00 1.00
DIST. FACTOR (MOMENT-ONE LANE) BY SUPPORT: 1.00 1.00 1.00
DIST. FACTOR (DEFL-ONE LANE) BY SUPPORT: 1.00 1.00 1.00
DIST. FACTOR (VERT-MULTILANE) BY SUPPORT: 1.00 1.00 1.00
DIST. FACTOR (MOMENT-MULTILANE) BY SUPPORT: 1.00 1.00 1.00
DIST. FACTOR (DEFL-MULTILANE) BY SUPPORT: 1.00 1.00 1.00

RESTRAINT DATA

NODE NO.	VERTICAL-ROTATION
1	FIXED-FREE
11	FIXED-FREE
21	FIXED-FREE

COMBINING ACTIONS FOR STRENGTH I LIMIT STATE

========== LOAD COMBINATION REPORT ==========
STRENGTH I LIMIT STATE

CRITICAL ACTIONS FOR THE STRENGTH I LIMIT STATE
GIRDER: INTERIOR

SPAN-%	V+	V-	M+	M-'	D+	D-
1- 0	134.78	-15.52	0.00	0.00	0.135E-07	-0.155E-08
1- 10	113.12	-16.18	1924.53	-253.82	0.340E+07	-0.112E+07
1- 20	92.99	-26.08	3297.78	-507.64	0.640E+07	-0.217E+07
1- 30	74.49	-40.97	4144.01	-761.46	0.871E+07	-0.309E+07
1- 40	57.74	-56.75	4519.60	-1015.28	0.100E+08	-0.380E+07
1- 50	42.83	-73.21	4444.49	-1269.09	0.103E+08	-0.424E+07
1- 60	29.83	-90.15	3960.00	-1522.92	0.940E+07	-0.434E+07
1- 70	18.79	-107.33	3071.93	-1776.73	0.763E+07	-0.404E+07
1- 80	9.76	-124.51	1839.29	-2357.27	0.521E+07	-0.326E+07
1- 90	3.64	-141.42	618.85	-2911.27	0.252E+07	-0.193E+07
1-100	0.00	-157.80	0.00	-4562.64	0.113E-07	0.000E+00
2- 0	157.79	0.00	0.00	-4562.64	0.113E-07	0.000E+00
2- 10	141.41	-3.64	618.85	-2910.67	0.252E+07	-0.193E+07
2- 20	124.50	-9.76	1839.31	-2356.73	0.521E+07	-0.326E+07
2- 30	107.32	-18.79	3071.94	-1776.69	0.763E+07	-0.404E+07
2- 40	90.13	-29.83	3960.03	-1522.89	0.940E+07	-0.434E+07
2- 50	73.21	-42.83	4444.55	-1269.07	0.103E+08	-0.424E+07
2- 60	56.75	-57.74	4519.53	-1015.26	0.100E+08	-0.380E+07
2- 70	40.97	-74.50	4144.00	-761.45	0.871E+07	-0.309E+07

2- 80	26.08	-92.99	3297.95	-507.63	0.640E+07	-0.217E+07
2- 90	16.18	-113.13	1924.62	-253.82	0.340E+07	-0.112E+07
2-100	15.52	-134.80	0.00	0.00	0.445E-08	-0.517E-09
R- 1	134.78	-15.52	0.00	0.00	0.135E-07	-0.155E-08
R- 2	275.29	0.00	0.00	0.00	0.138E-07	0.000E+00
R- 3	134.80	-15.52	0.00	0.00	0.445E-08	-0.517E-09

CRITICAL ACTIONS FOR THE STRENGTH I LIMIT STATE
GIRDER: EXTERIOR

SPAN-%	V+	V-	M+	M-'	D+	D-
1- 0	134.78	-15.52	0.00	0.00	0.135E-07	-0.155E-08
1- 10	113.12	-16.18	1924.53	-253.82	0.340E+07	-0.112E+07
1- 20	92.99	-26.08	3297.78	-507.64	0.640E+07	-0.217E+07
1- 30	74.49	-40.97	4144.01	-761.46	0.871E+07	-0.309E+07
1- 40	57.74	-56.75	4519.60	-1015.28	0.100E+08	-0.380E+07
1- 50	42.83	-73.21	4444.49	-1269.09	0.103E+08	-0.424E+07
1- 60	29.83	-90.15	3960.00	-1522.92	0.940E+07	-0.434E+07
1- 70	18.79	-107.33	3071.93	-1776.73	0.763E+07	-0.404E+07
1- 80	9.76	-124.51	1839.29	-2357.27	0.521E+07	-0.326E+07
1- 90	3.64	-141.42	618.85	-2911.27	0.252E+07	-0.193E+07
1-100	0.00	-157.80	0.00	-4562.64	0.113E-07	0.000E+00
2- 0	157.79	0.00	0.00	-4562.64	0.113E-07	0.000E+00
2- 10	141.41	-3.64	618.85	-2910.67	0.252E+07	-0.193E+07
2- 20	124.50	-9.76	1839.31	-2356.73	0.521E+07	-0.326E+07
2- 30	107.32	-18.79	3071.94	-1776.69	0.763E+07	-0.404E+07
2- 40	90.13	-29.83	3960.03	-1522.89	0.940E+07	-0.434E+07
2- 50	73.21	-42.83	4444.55	-1269.07	0.103E+08	-0.424E+07
2- 60	56.75	-57.74	4519.53	-1015.26	0.100E+08	-0.380E+07
2- 70	40.97	-74.50	4144.00	-761.45	0.871E+07	-0.309E+07
2- 80	26.08	-92.99	3297.95	-507.63	0.640E+07	-0.217E+07
2- 90	16.18	-113.13	1924.62	-253.82	0.340E+07	-0.112E+07
2-100	15.52	-134.80	0.00	0.00	0.445E-08	-0.517E-09
R- 1	134.78	-15.52	0.00	0.00	0.135E-07	-0.155E-08
R- 2	275.29	0.00	0.00	0.00	0.138E-07	0.000E+00
R- 3	134.80	-15.52	0.00	0.00	0.445E-08	-0.517E-09

COMBINING ACTIONS FOR SERVICE I LIMIT STATE

========== LOAD COMBINATION REPORT ==========
SERVICE I LIMIT STATE

CRITICAL ACTIONS FOR THE SERVICE I LIMIT STATE
GIRDER: INTERIOR

SPAN-%	V+	V-	M+	M-'	D+	D-
1- 0	134.78	-15.52	0.00	0.00	0.135E-07	-0.155E-08
1- 10	113.12	-16.18	1924.53	-253.82	0.340E+07	-0.112E+07
1- 20	92.99	-26.08	3297.78	-507.64	0.640E+07	-0.217E+07
1- 30	74.49	-40.97	4144.01	-761.46	0.871E+07	-0.309E+07
1- 40	57.74	-56.75	4519.60	-1015.28	0.100E+08	-0.380E+07

1- 50	42.83	-73.21	4444.49	-1269.09	0.103E+08-0.424E+07
1- 60	29.83	-90.15	3960.00	-1522.92	0.940E+07-0.434E+07
1- 70	18.79	-107.33	3071.93	-1776.73	0.763E+07-0.404E+07
1- 80	9.76	-124.51	1839.29	-2357.27	0.521E+07-0.326E+07
1- 90	3.64	-141.42	618.85	-2911.27	0.252E+07-0.193E+07
1-100	0.00	-157.80	0.00	-4562.64	0.113E-07 0.000E+00
2- 0	157.79	0.00	0.00	-4562.64	0.113E-07 0.000E+00
2- 10	141.41	-3.64	618.85	-2910.67	0.252E+07-0.193E+07
2- 20	124.50	-9.76	1839.31	-2356.73	0.521E+07-0.326E+07
2- 30	107.32	-18.79	3071.94	-1776.69	0.763E+07-0.404E+07
2- 40	90.13	-29.83	3960.03	-1522.89	0.940E+07-0.434E+07
2- 50	73.21	-42.83	4444.55	-1269.07	0.103E+08-0.424E+07
2- 60	56.75	-57.74	4519.53	-1015.26	0.100E+08-0.380E+07
2- 70	40.97	-74.50	4144.00	-761.45	0.871E+07-0.309E+07
2- 80	26.08	-92.99	3297.95	-507.63	0.640E+07-0.217E+07
2- 90	16.18	-113.13	1924.62	-253.82	0.340E+07-0.112E+07
2-100	15.52	-134.80	0.00	0.00	0.445E-08-0.517E-09
R- 1	134.78	-15.52	0.00	0.00	0.135E-07-0.155E-08
R- 2	275.29	0.00	0.00	0.00	0.138E-07 0.000E+00
R- 3	134.80	-15.52	0.00	0.00	0.445E-08-0.517E-09

CRITICAL ACTIONS FOR THE SERVICE I LIMIT STATE
GIRDER: EXTERIOR

SPAN-%	V+	V-	M+	M-'	D+ D-
1- 0	134.78	-15.52	0.00	0.00	0.135E-07-0.155E-08
1- 10	113.12	-16.18	1924.53	-253.82	0.340E+07-0.112E+07
1- 20	92.99	-26.08	3297.78	-507.64	0.640E+07-0.217E+07
1- 30	74.49	-40.97	4144.01	-761.46	0.871E+07-0.309E+07
1- 40	57.74	-56.75	4519.60	-1015.28	0.100E+08-0.380E+07
1- 50	42.83	-73.21	4444.49	-1269.09	0.103E+08-0.424E+07
1- 60	29.83	-90.15	3960.00	-1522.92	0.940E+07-0.434E+07
1- 70	18.79	-107.33	3071.93	-1776.73	0.763E+07-0.404E+07
1- 80	9.76	-124.51	1839.29	-2357.27	0.521E+07-0.326E+07
1- 90	3.64	-141.42	618.85	-2911.27	0.252E+07-0.193E+07
1-100	0.00	-157.80	0.00	-4562.64	0.113E-07 0.000E+00
2- 0	157.79	0.00	0.00	-4562.64	0.113E-07 0.000E+00
2- 10	141.41	-3.64	618.85	-2910.67	0.252E+07-0.193E+07
2- 20	124.50	-9.76	1839.31	-2356.73	0.521E+07-0.326E+07
2- 30	107.32	-18.79	3071.94	-1776.69	0.763E+07-0.404E+07
2- 40	90.13	-29.83	3960.03	-1522.89	0.940E+07-0.434E+07
2- 50	73.21	-42.83	4444.55	-1269.07	0.103E+08-0.424E+07
2- 60	56.75	-57.74	4519.53	-1015.26	0.100E+08-0.380E+07
2- 70	40.97	-74.50	4144.00	-761.45	0.871E+07-0.309E+07
2- 80	26.08	-92.99	3297.95	-507.63	0.640E+07-0.217E+07
2- 90	16.18	-113.13	1924.62	-253.82	0.340E+07-0.112E+07
2-100	15.52	-134.80	0.00	0.00	0.445E-08-0.517E-09
R- 1	134.78	-15.52	0.00	0.00	0.135E-07-0.155E-08
R- 2	275.29	0.00	0.00	0.00	0.138E-07 0.000E+00
R- 3	134.80	-15.52	0.00	0.00	0.445E-08-0.517E-09

COMBINING ACTIONS FOR FATIGUE LIMIT STATE

CRITICAL ACTION RANGE FOR THE FATIGUE LIMIT STATE

GIRDER: EXTERIOR

SPAN-%	DELTA V	DELTA M
1- 0	59.15	0.00
1- 10	51.57	837.90
1- 20	46.33	1435.33
1- 30	45.80	1846.39
1- 40	46.53	2047.25
1- 50	47.47	2098.62
1- 60	48.62	2036.23
1- 70	49.98	1813.82
1- 80	51.97	1467.46
1- 90	54.73	1068.02
1-100	57.58	921.09
2- 0	57.58	920.61
2- 10	54.72	1082.64
2- 20	51.97	1467.52
2- 30	49.98	1813.84
2- 40	48.62	2036.26
2- 50	47.47	2098.66
2- 60	46.53	2047.23
2- 70	45.80	1846.33
2- 80	46.34	1435.88
2- 90	51.57	837.94
2-100	59.15	0.00
R- 1	59.15	0.00
R- 2	61.06	0.00
R- 3	59.15	0.00

========== LOAD COMBINATION REPORT ==========
FATIGUE LIMIT STATE

CRITICAL ACTION RANGE FOR THE FATIGUE LIMIT STATE
GIRDER: INTERIOR

SPAN-%	DELTA V	DELTA M
1- 0	59.15	0.00
1- 10	51.57	837.90
1- 20	46.33	1435.33
1- 30	45.80	1846.39
1- 40	46.53	2047.25
1- 50	47.47	2098.62
1- 60	48.62	2036.23
1- 70	49.98	1813.82
1- 80	51.97	1467.46
1- 90	54.73	1068.02
1-100	57.58	921.09
2- 0	57.58	920.61
2- 10	54.72	1082.64
2- 20	51.97	1467.52
2- 30	49.98	1813.84
2- 40	48.62	2036.26
2- 50	47.47	2098.66
2- 60	46.53	2047.23
2- 70	45.80	1846.33
2- 80	46.34	1435.88
2- 90	51.57	837.94
2-100	59.15	0.00
R- 1	59.15	0.00
R- 2	61.06	0.00
R- 3	59.15	0.00

========== LOAD COMBINATION REPORT ==========
FATIGUE LIMIT STATE

CRITICAL ACTION RANGE FOR THE FATIGUE LIMIT STATE
GIRDER: INTERIOR

SPAN-%	DELTA V	DELTA M
1- 0	0.00	0.00
1- 10	0.00	0.00
1- 20	0.00	0.00
1- 30	0.00	0.00
1- 40	0.00	0.00
1- 50	0.00	0.00
1- 60	0.00	0.00
1- 70	0.00	0.00
1- 80	0.00	0.00
1- 90	0.00	0.00
1-100	0.00	0.00
2- 0	0.00	0.00
2- 10	0.00	0.00
2- 20	0.00	0.00
2- 30	0.00	0.00
2- 40	0.00	0.00
2- 50	0.00	0.00
2- 60	0.00	0.00
2- 70	0.00	0.00
2- 80	0.00	0.00
2- 90	0.00	0.00
2-100	0.00	0.00
R- 1	0.00	0.00
R- 2	0.00	0.00
R- 3	0.00	0.00

CRITICAL ACTION RANGE FOR THE FATIGUE LIMIT STATE
GIRDER: EXTERIOR

SPAN-%	DELTA V	DELTA M
1- 0	0.00	0.00
1- 10	0.00	0.00
1- 20	0.00	0.00
1- 30	0.00	0.00
1- 40	0.00	0.00
1- 50	0.00	0.00
1- 60	0.00	0.00
1- 70	0.00	0.00
1- 80	0.00	0.00
1- 90	0.00	0.00
1-100	0.00	0.00
2- 0	0.00	0.00
2- 10	0.00	0.00
2- 20	0.00	0.00
2- 30	0.00	0.00

2- 40	0.00	0.00
2- 50	0.00	0.00
2- 60	0.00	0.00
2- 70	0.00	0.00
2- 80	0.00	0.00
2- 90	0.00	0.00
2-100	0.00	0.00
R- 1	0.00	0.00
R- 2	0.00	0.00
R- 3	0.00	0.00

COMBINING ACTIONS FOR STRENGTH II LIMIT STATE

========== LOAD COMBINATION REPORT ==========
STRENGTH II LIMIT STATE

CRITICAL ACTIONS FOR THE STRENGTH II LIMIT STATE
GIRDER: INTERIOR

SPAN-%	V+	V-	M+	M-'	D+	D-
1- 0	134.78	-15.52	0.00	0.00	0.135E-07	-0.155E-08
1- 10	113.12	-16.18	1924.53	-253.82	0.340E+07	-0.112E+07
1- 20	92.99	-26.08	3297.78	-507.64	0.640E+07	-0.217E+07
1- 30	74.49	-40.97	4144.01	-761.46	0.871E+07	-0.309E+07
1- 40	57.74	-56.75	4519.60	-1015.28	0.100E+08	-0.380E+07
1- 50	42.83	-73.21	4444.49	-1269.09	0.103E+08	-0.424E+07
1- 60	29.83	-90.15	3960.00	-1522.92	0.940E+07	-0.434E+07
1- 70	18.79	-107.33	3071.93	-1776.73	0.763E+07	-0.404E+07
1- 80	9.76	-124.51	1839.29	-2357.27	0.521E+07	-0.326E+07
1- 90	3.64	-141.42	618.85	-2911.27	0.252E+07	-0.193E+07
1-100	0.00	-157.80	0.00	-4562.64	0.113E-07	0.000E+00
2- 0	157.79	0.00	0.00	-4562.64	0.113E-07	0.000E+00
2- 10	141.41	-3.64	618.85	-2910.67	0.252E+07	-0.193E+07
2- 20	124.50	-9.76	1839.31	-2356.73	0.521E+07	-0.326E+07
2- 30	107.32	-18.79	3071.94	-1776.69	0.763E+07	-0.404E+07
2- 40	90.13	-29.83	3960.03	-1522.89	0.940E+07	-0.434E+07
2- 50	73.21	-42.83	4444.55	-1269.07	0.103E+08	-0.424E+07
2- 60	56.75	-57.74	4519.53	-1015.26	0.100E+08	-0.380E+07
2- 70	40.97	-74.50	4144.00	-761.45	0.871E+07	-0.309E+07
2- 80	26.08	-92.99	3297.95	-507.63	0.640E+07	-0.217E+07
2- 90	16.18	-113.13	1924.62	-253.82	0.340E+07	-0.112E+07
2-100	15.52	-134.80	0.00	0.00	0.445E-08	-0.517E-09
R- 1	134.78	-15.52	0.00	0.00	0.135E-07	-0.155E-08
R- 2	275.29	0.00	0.00	0.00	0.138E-07	0.000E+00
R- 3	134.80	-15.52	0.00	0.00	0.445E-08	-0.517E-09

CRITICAL ACTIONS FOR THE STRENGTH II LIMIT STATE
GIRDER: EXTERIOR

SPAN-%	V+	V-	M+	M-'	D+	D-
1- 0	134.78	-15.52	0.00	0.00	0.135E-07	-0.155E-08

1- 10	113.12	-16.18	1924.53	-253.82	0.340E+07-0.112E+07
1- 20	92.99	-26.08	3297.78	-507.64	0.640E+07-0.217E+07
1- 30	74.49	-40.97	4144.01	-761.46	0.871E+07-0.309E+07
1- 40	57.74	-56.75	4519.60	-1015.28	0.100E+08-0.380E+07
1- 50	42.83	-73.21	4444.49	-1269.09	0.103E+08-0.424E+07
1- 60	29.83	-90.15	3960.00	-1522.92	0.940E+07-0.434E+07
1- 70	18.79	-107.33	3071.93	-1776.73	0.763E+07-0.404E+07
1- 80	9.76	-124.51	1839.29	-2357.27	0.521E+07-0.326E+07
1- 90	3.64	-141.42	618.85	-2911.27	0.252E+07-0.193E+07
1-100	0.00	-157.80	0.00	-4562.64	0.113E-07 0.000E+00
2- 0	157.79	0.00	0.00	-4562.64	0.113E-07 0.000E+00
2- 10	141.41	-3.64	618.85	-2910.67	0.252E+07-0.193E+07
2- 20	124.50	-9.76	1839.31	-2356.73	0.521E+07-0.326E+07
2- 30	107.32	-18.79	3071.94	-1776.69	0.763E+07-0.404E+07
2- 40	90.13	-29.83	3960.03	-1522.89	0.940E+07-0.434E+07
2- 50	73.21	-42.83	4444.55	-1269.07	0.103E+08-0.424E+07
2- 60	56.75	-57.74	4519.53	-1015.26	0.100E+08-0.380E+07
2- 70	40.97	-74.50	4144.00	-761.45	0.871E+07-0.309E+07
2- 80	26.08	-92.99	3297.95	-507.63	0.640E+07-0.217E+07
2- 90	16.18	-113.13	1924.62	-253.82	0.340E+07-0.112E+07
2-100	15.52	-134.80	0.00	0.00	0.445E-08-0.517E-09
R- 1	134.78	-15.52	0.00	0.00	0.135E-07-0.155E-08
R- 2	275.29	0.00	0.00	0.00	0.138E-07 0.000E+00
R- 3	134.80	-15.52	0.00	0.00	0.445E-08-0.517E-09

COMBINING ACTIONS FOR SERVICE II LIMIT STATE

========== LOAD COMBINATION REPORT ==========
SERVICE II LIMIT STATE

CRITICAL ACTIONS FOR THE SERVICE II LIMIT STATE
GIRDER: INTERIOR

SPAN-%	V+	V-	M+	M-'	D+ D-
1- 0	134.78	-15.52	0.00	0.00	0.135E-07-0.155E-08
1- 10	113.12	-16.18	1924.53	-253.82	0.340E+07-0.112E+07
1- 20	92.99	-26.08	3297.78	-507.64	0.640E+07-0.217E+07
1- 30	74.49	-40.97	4144.01	-761.46	0.871E+07-0.309E+07
1- 40	57.74	-56.75	4519.60	-1015.28	0.100E+08-0.380E+07
1- 50	42.83	-73.21	4444.49	-1269.09	0.103E+08-0.424E+07
1- 60	29.83	-90.15	3960.00	-1522.92	0.940E+07-0.434E+07
1- 70	18.79	-107.33	3071.93	-1776.73	0.763E+07-0.404E+07
1- 80	9.76	-124.51	1839.29	-2357.27	0.521E+07-0.326E+07
1- 90	3.64	-141.42	618.85	-2911.27	0.252E+07-0.193E+07
1-100	0.00	-157.80	0.00	-4562.64	0.113E-07 0.000E+00
2- 0	157.79	0.00	0.00	-4562.64	0.113E-07 0.000E+00
2- 10	141.41	-3.64	618.85	-2910.67	0.252E+07-0.193E+07
2- 20	124.50	-9.76	1839.31	-2356.73	0.521E+07-0.326E+07
2- 30	107.32	-18.79	3071.94	-1776.69	0.763E+07-0.404E+07
2- 40	90.13	-29.83	3960.03	-1522.89	0.940E+07-0.434E+07
2- 50	73.21	-42.83	4444.55	-1269.07	0.103E+08-0.424E+07
2- 60	56.75	-57.74	4519.53	-1015.26	0.100E+08-0.380E+07

2- 70	40.97	-74.50	4144.00	-761.45	0.871E+07	-0.309E+07
2- 80	26.08	-92.99	3297.95	-507.63	0.640E+07	-0.217E+07
2- 90	16.18	-113.13	1924.62	-253.82	0.340E+07	-0.112E+07
2-100	15.52	-134.80	0.00	0.00	0.445E-08	-0.517E-09
R- 1	134.78	-15.52	0.00	0.00	0.135E-07	-0.155E-08
R- 2	275.29	0.00	0.00	0.00	0.138E-07	0.000E+00
R- 3	134.80	-15.52	0.00	0.00	0.445E-08	-0.517E-09

CRITICAL ACTIONS FOR THE SERVICE II LIMIT STATE
GIRDER: EXTERIOR

SPAN-%	V+	V-	M+	M-'	D+	D-
1- 0	134.78	-15.52	0.00	0.00	0.135E-07	-0.155E-08
1- 10	113.12	-16.18	1924.53	-253.82	0.340E+07	-0.112E+07
1- 20	92.99	-26.08	3297.78	-507.64	0.640E+07	-0.217E+07
1- 30	74.49	-40.97	4144.01	-761.46	0.871E+07	-0.309E+07
1- 40	57.74	-56.75	4519.60	-1015.28	0.100E+08	-0.380E+07
1- 50	42.83	-73.21	4444.49	-1269.09	0.103E+08	-0.424E+07
1- 60	29.83	-90.15	3960.00	-1522.92	0.940E+07	-0.434E+07
1- 70	18.79	-107.33	3071.93	-1776.73	0.763E+07	-0.404E+07
1- 80	9.76	-124.51	1839.29	-2357.27	0.521E+07	-0.326E+07
1- 90	3.64	-141.42	618.85	-2911.27	0.252E+07	-0.193E+07
1-100	0.00	-157.80	0.00	-4562.64	0.113E-07	0.000E+00
2- 0	157.79	0.00	0.00	-4562.64	0.113E-07	0.000E+00
2- 10	141.41	-3.64	618.85	-2910.67	0.252E+07	-0.193E+07
2- 20	124.50	-9.76	1839.31	-2356.73	0.521E+07	-0.326E+07
2- 30	107.32	-18.79	3071.94	-1776.69	0.763E+07	-0.404E+07
2- 40	90.13	-29.83	3960.03	-1522.89	0.940E+07	-0.434E+07
2- 50	73.21	-42.83	4444.55	-1269.07	0.103E+08	-0.424E+07
2- 60	56.75	-57.74	4519.53	-1015.26	0.100E+08	-0.380E+07
2- 70	40.97	-74.50	4144.00	-761.45	0.871E+07	-0.309E+07
2- 80	26.08	-92.99	3297.95	-507.63	0.640E+07	-0.217E+07
2- 90	16.18	-113.13	1924.62	-253.82	0.340E+07	-0.112E+07
2-100	15.52	-134.80	0.00	0.00	0.445E-08	-0.517E-09
R- 1	134.78	-15.52	0.00	0.00	0.135E-07	-0.155E-08
R- 2	275.29	0.00	0.00	0.00	0.138E-07	0.000E+00
R- 3	134.80	-15.52	0.00	0.00	0.445E-08	-0.517E-09

Code: LRFD First Edition 1994

Span Data

Span 1 Length: 162.500 ft

Section Properties

Location (ft)	Ax (in^2)	Iz (in^4)	Mod. E (psi)	Unit Wgt (pcf)
0.000	1.000e+00	1.000e+00	1.000e+03	1.000e+00

Live Load Distribution Factors

Location	Str/Serv Limit States		Fatigue Limit State	
(ft)	gM	gV	gM	gV
0.000	1.000	1.000	1.000	1.000

Strength Limit State Factors: Ductility 1.00 Redundancy 1.00 Importance 1.00
Service Limit State Factors: Ductility 1.00 Redundancy 1.00 Importance 1.00

Span 2 Length: 162.500 ft

Section Properties

Location (ft)	Ax (in^2)	Iz (in^4)	Mod. E (psi)	Unit Wgt (pcf)
0.000	1.000e+00	1.000e+00	1.000e+03	1.000e+00

Live Load Distribution Factors

Location	Str/Serv Limit States		Fatigue Limit State	
(ft)	gM	gV	gM	gV
0.000	1.000	1.000	1.000	1.000

Strength Limit State Factors: Ductility 1.00 Redundancy 1.00 Importance 1.00
Service Limit State Factors: Ductility 1.00 Redundancy 1.00 Importance 1.00

Support Data

Support 1 Pinned

Support 2 Pinned

Support 3 Pinned

Loading Data

DC Loads
Self Weight Generation Disabled
Traffic Barrier Load 110.000e+00 plf
Span 1 W 1.558e+03 plf from 0.000 ft to 162.500 ft
Span 2 W 1.558e+03 plf from 0.000 ft to 162.500 ft

DW Loads
Utility Load Disabled
Wearing Surface Load 340.000e+00 plf

Live Load Data

Live Load Generation Parameters
Design Tandem : Enabled
Design Truck : 1 rear axle spacing increments
Dual Truck Train : Headway Spacing varies from 49.213 ft to 49.213 ft using 1 increments
Dual Tandem Train: Disabled
Fatigue Truck : Enabled

Live Load Impact
Truck Loads 33.000%
Lane Loads 0.000%
Fatigue Truck 15.000%

Pedestrian Live Load 0.000e+00 plf

Load Factors

Strength I	DC min	0.900	DC max	1.250	DW min	0.650	DW max	1.500	LL	1.750
Service I	DC	1.000	DW	1.000	LL	1.000				
Service II	DC	1.000	DW	1.000	LL	1.300				
Service III	DC	1.000	DW	1.000	LL	0.800				
Fatigue	DC	0.000	DW	0.000	LL	0.750				

Analysis Results

DC Dead Load

Span	Point	Shear(lbs)	Moment(ft-lbs)
1	0	101.643e+03	0.000e+00
1	1	74.538e+03	1.431e+06
1	2	47.433e+03	2.422e+06
1	3	20.328e+03	2.973e+06
1	4	-6.776e+03	3.083e+06
1	5	-33.881e+03	2.752e+06
1	6	-60.986e+03	1.982e+06
1	7	-88.091e+03	770.798e+03
1	8	-115.196e+03	-880.912e+03
1	9	-142.301e+03	-2.973e+06
1	10	-169.406e+03	-5.505e+06
2	0	169.406e+03	-5.505e+06
2	1	142.301e+03	-2.973e+06
2	2	115.196e+03	-880.912e+03
2	3	88.091e+03	770.798e+03
2	4	60.986e+03	1.982e+06
2	5	33.881e+03	2.752e+06
2	6	6.776e+03	3.083e+06
2	7	-20.328e+03	2.973e+06
2	8	-47.433e+03	2.422e+06
2	9	-74.538e+03	1.431e+06
2	10	-101.643e+03	0.000e+00

DW Dead Load

Span	Point	Shear(lbs)	Moment(ft-lbs)
1	0	20.718e+03	0.000e+00
1	1	15.193e+03	291.789e+03
1	2	9.668e+03	493.796e+03
1	3	4.143e+03	606.023e+03
1	4	-1.381e+03	628.468e+03
1	5	-6.906e+03	561.132e+03
1	6	-12.431e+03	404.015e+03
1	7	-17.956e+03	157.117e+03
1	8	-23.481e+03	-179.562e+03
1	9	-29.006e+03	-606.023e+03
1	10	-34.531e+03	-1.122e+06
2	0	34.531e+03	-1.122e+06
2	1	29.006e+03	-606.023e+03

2	2	23.481e+03	-179.562e+03
2	3	17.956e+03	157.117e+03
2	4	12.431e+03	404.015e+03
2	5	6.906e+03	561.132e+03
2	6	1.381e+03	628.468e+03
2	7	-4.143e+03	606.023e+03
2	8	-9.668e+03	493.796e+03
2	9	-15.193e+03	291.789e+03
2	10	-20.718e+03	0.000e+00

Live Load Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-15.597e+03	135.585e+03	0.000e+00	0.000e+00
1	1	-16.243e+03	113.848e+03	-253.455e+03	1.923e+06
1	2	-26.199e+03	93.625e+03	-506.911e+03	3.295e+06
1	3	-41.231e+03	75.038e+03	-760.367e+03	4.140e+06
1	4	-57.131e+03	58.194e+03	-1.013e+06	4.513e+06
1	5	-73.699e+03	43.184e+03	-1.267e+06	4.438e+06
1	6	-90.720e+03	30.077e+03	-1.520e+06	3.955e+06
1	7	-107.963e+03	18.940e+03	-1.774e+06	3.068e+06
1	8	-125.204e+03	9.849e+03	-2.367e+06	1.836e+06
1	9	-142.124e+03	3.606e+03	-2.922e+06	621.839e+03
1	10	-158.480e+03	0.000e+00	-4.083e+06	0.000e+00
2	0	0.000e+00	157.728e+03	-4.083e+06	0.000e+00
2	1	-3.606e+03	142.124e+03	-2.922e+06	621.839e+03
2	2	-9.849e+03	125.204e+03	-2.367e+06	1.836e+06
2	3	-18.940e+03	107.963e+03	-1.774e+06	3.068e+06
2	4	-30.077e+03	90.720e+03	-1.520e+06	3.955e+06
2	5	-43.184e+03	73.699e+03	-1.267e+06	4.438e+06
2	6	-58.194e+03	57.131e+03	-1.013e+06	4.513e+06
2	7	-75.038e+03	41.231e+03	-760.367e+03	4.140e+06
2	8	-93.625e+03	26.199e+03	-506.911e+03	3.295e+06
2	9	-113.848e+03	16.243e+03	-253.455e+03	1.923e+06
2	10	-135.585e+03	15.597e+03	0.000e+00	0.000e+00

Number not correct

Design Tandem + Lane Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-12.711e+03	110.147e+03	0.000e+00	0.000e+00
1	1	-14.272e+03	92.243e+03	-206.561e+03	1.572e+06
1	2	-24.321e+03	75.735e+03	-413.123e+03	2.714e+06
1	3	-35.446e+03	60.696e+03	-619.684e+03	3.435e+06
1	4	-47.508e+03	47.187e+03	-826.246e+03	3.751e+06
1	5	-60.355e+03	35.252e+03	-1.032e+06	3.695e+06
1	6	-73.819e+03	24.921e+03	-1.239e+06	3.293e+06
1	7	-87.715e+03	16.181e+03	-1.445e+06	2.569e+06
1	8	-101.871e+03	9.111e+03	-1.652e+06	1.560e+06
1	9	-115.996e+03	3.606e+03	-2.146e+06	610.184e+03
1	10	-129.928e+03	0.000e+00	-3.107e+06	0.000e+00
2	0	0.000e+00	127.075e+03	-3.107e+06	0.000e+00
2	1	-3.606e+03	115.996e+03	-2.146e+06	610.184e+03
2	2	-9.111e+03	101.871e+03	-1.652e+06	1.560e+06
2	3	-16.181e+03	87.715e+03	-1.445e+06	2.569e+06
2	4	-24.921e+03	73.819e+03	-1.239e+06	3.293e+06
2	5	-35.252e+03	60.355e+03	-1.032e+06	3.695e+06
2	6	-47.187e+03	47.508e+03	-826.246e+03	3.751e+06
2	7	-60.696e+03	35.446e+03	-619.684e+03	3.435e+06
2	8	-75.735e+03	24.321e+03	-413.123e+03	2.714e+06
2	9	-92.243e+03	14.272e+03	-206.561e+03	1.572e+06
2	10	-110.147e+03	12.711e+03	0.000e+00	0.000e+00

Design Truck + Lane Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-15.597e+03	135.585e+03	0.000e+00	0.000e+00
1	1	-16.243e+03	113.848e+03	-253.455e+03	1.923e+06

1	2	-26.199e+03	93.625e+03	-506.911e+03	3.295e+06
1	3	-41.231e+03	75.038e+03	-760.367e+03	4.140e+06
1	4	-57.131e+03	58.194e+03	-1.013e+06	4.513e+06
1	5	-73.699e+03	43.184e+03	-1.267e+06	4.438e+06
1	6	-90.720e+03	30.077e+03	-1.520e+06	3.955e+06
1	7	-107.963e+03	18.940e+03	-1.774e+06	3.068e+06
1	8	-125.204e+03	9.849e+03	-2.027e+06	1.836e+06
1	9	-142.124e+03	3.105e+03	-2.568e+06	621.839e+03
1	10	-158.480e+03	0.000e+00	-3.576e+06	0.000e+00
2	0	0.000e+00	157.728e+03	-3.576e+06	0.000e+00
2	1	-3.105e+03	142.124e+03	-2.568e+06	621.839e+03
2	2	-9.849e+03	125.204e+03	-2.027e+06	1.836e+06
2	3	-18.940e+03	107.963e+03	-1.774e+06	3.068e+06
2	4	-30.077e+03	90.720e+03	-1.520e+06	3.955e+06
2	5	-43.184e+03	73.699e+03	-1.267e+06	4.438e+06
2	6	-58.194e+03	57.131e+03	-1.013e+06	4.513e+06
2	7	-75.038e+03	41.231e+03	-760.367e+03	4.140e+06
2	8	-93.625e+03	26.199e+03	-506.911e+03	3.295e+06
2	9	-113.848e+03	16.243e+03	-253.455e+03	1.923e+06
2	10	-135.585e+03	15.597e+03	0.000e+00	0.000e+00

Dual Truck Train + Lane Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	8	0.000e+00	0.000e+00	-2.367e+06	0.000e+00
1	9	0.000e+00	0.000e+00	-2.922e+06	0.000e+00
1	10	0.000e+00	0.000e+00	-4.083e+06	0.000e+00
2	0	0.000e+00	0.000e+00	-4.083e+06	0.000e+00
2	1	0.000e+00	0.000e+00	-2.922e+06	0.000e+00
2	2	0.000e+00	0.000e+00	-2.367e+06	0.000e+00
2	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Dual Tandem Train + Lane Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00

2	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Fatigue Truck Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-7.678e+03	72.581e+03	0.000e+00	0.000e+00
1	1	-7.678e+03	62.316e+03	-124.769e+03	1.012e+06
1	2	-10.572e+03	52.315e+03	-249.538e+03	1.700e+06
1	3	-19.681e+03	42.705e+03	-374.307e+03	2.130e+06
1	4	-29.744e+03	33.612e+03	-499.076e+03	2.278e+06
1	5	-39.442e+03	25.155e+03	-623.845e+03	2.223e+06
1	6	-48.648e+03	17.463e+03	-748.614e+03	2.013e+06
1	7	-57.237e+03	10.694e+03	-873.383e+03	1.586e+06
1	8	-65.082e+03	5.400e+03	-998.152e+03	993.558e+03
1	9	-72.078e+03	2.214e+03	-1.122e+06	343.262e+03
1	10	-78.046e+03	0.000e+00	-1.247e+06	0.000e+00
2	0	0.000e+00	77.026e+03	-1.247e+06	0.000e+00
2	1	-2.214e+03	72.078e+03	-1.122e+06	343.262e+03
2	2	-5.400e+03	65.082e+03	-998.152e+03	993.558e+03
2	3	-10.694e+03	57.237e+03	-873.383e+03	1.586e+06
2	4	-17.463e+03	48.648e+03	-748.614e+03	2.013e+06
2	5	-25.155e+03	39.442e+03	-623.845e+03	2.223e+06
2	6	-33.612e+03	29.744e+03	-499.076e+03	2.278e+06
2	7	-42.705e+03	19.681e+03	-374.307e+03	2.130e+06
2	8	-52.315e+03	10.572e+03	-249.538e+03	1.700e+06
2	9	-62.316e+03	7.678e+03	-124.769e+03	1.012e+06
2	10	-72.581e+03	7.678e+03	0.000e+00	0.000e+00

Strength I Limit State Envelopes

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	77.651e+03	395.406e+03	0.000e+00	0.000e+00
1	1	48.535e+03	315.199e+03	1.034e+06	5.593e+06
1	2	3.125e+03	237.640e+03	1.614e+06	9.536e+06
1	3	-51.165e+03	162.944e+03	1.739e+06	11.870e+06
1	4	-110.522e+03	94.843e+03	1 409e+06	12.695e+06
1	5	-181.685e+03	40.590e+03	624.565e+03	12.049e+06
1	6	-253.640e+03	-10.331e+03	-614.826e+03	10.005e+06
1	7	-325.983e+03	-57.807e+03	-2.308e+06	6.568e+06
1	8	-398.324e+03	-101.703e+03	-5.514e+06	2.304e+06
1	9	-470.104e+03	-140.614e+03	-9.739e+06	-1.981e+06
1	10	-540.895e+03	-174.910e+03	-15.711e+06	-5.684e+06
2	0	263.554e+03	539.579e+03	-15.711e+06	-5.684e+06
2	1	140.614e+03	470.104e+03	-9.739e+06	-1.981e+06
2	2	101.703e+03	398.324e+03	-5.514e+06	2.304e+06
2	3	57.807e+03	325.983e+03	-2.308e+06	6.568e+06
2	4	10.331e+03	253.640e+03	-614.826e+03	10.005e+06
2	5	-40.590e+03	181.685e+03	624.565e+03	12.049e+06
2	6	-94.843e+03	110.522e+03	1.409e+06	12.695e+06
2	7	-162.944e+03	51.165e+03	1.739e+06	11.870e+06
2	8	-237.640e+03	-3.125e+03	1.614e+06	9.536e+06
2	9	-315.199e+03	-48.535e+03	1.034e+06	5.593e+06
2	10	-395.406e+03	-77.651e+03	0.000e+00	0.000e+00

Service I Limit State Envelopes

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	106.765e+03	257.947e+03	0.000e+00	0.000e+00
1	1	73.489e+03	203.581e+03	1.469e+06	3.646e+06
1	2	30.902e+03	150.728e+03	2.409e+06	6.211e+06
1	3	-16.758e+03	99.511e+03	2.818e+06	7.719e+06
1	4	-65.288e+03	50.036e+03	2.697e+06	8.225e+06

1	5	-114.487e+03	2.397e+03	2.046e+06	7.752e+06
1	6	-164.138e+03	-43.339e+03	865.334e+03	6.341e+06
1	7	-214.010e+03	-87.106e+03	-846.274e+03	3.996e+06
1	8	-263.881e+03	-128.828e+03	-3.428e+06	776.274e+03
1	9	-313.432e+03	-167.701e+03	-6.501e+06	-2.957e+06
1	10	-362.418e+03	-203.937e+03	-10.711e+06	-6.627e+06
2	0	203.937e+03	361.666e+03	-10.711e+06	-6.627e+06
2	1	167.701e+03	313.432e+03	-6.501e+06	-2.957e+06
2	2	128.828e+03	263.881e+03	-3.428e+06	776.274e+03
2	3	87.106e+03	214.010e+03	-846.274e+03	3.996e+06
2	4	43.339e+03	164.138e+03	865.334e+03	6.341e+06
2	5	-2.397e+03	114.487e+03	2.046e+06	7.752e+06
2	6	-50.036e+03	65.288e+03	2.697e+06	8.225e+06
2	7	-99.511e+03	16.758e+03	2.818e+06	7.719e+06
2	8	-150.728e+03	-30.902e+03	2.409e+06	6.211e+06
2	9	-203.581e+03	-73.489e+03	1.469e+06	3.646e+06
2	10	-257.947e+03	-106.765e+03	0.000e+00	0.000e+00

Service II Limit State Envelopes

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	102.086e+03	298.623e+03	0.000e+00	0.000e+00
1	1	68.616e+03	237.735e+03	1.393e+06	4.224e+06
1	2	23.043e+03	178.815e+03	2.257e+06	7.200e+06
1	3	-29.128e+03	122.022e+03	2.590e+06	8.961e+06
1	4	-82.428e+03	67.495e+03	2.393e+06	9.579e+06
1	5	-136.597e+03	15.352e+03	1.666e+06	9.083e+06
1	6	-191.354e+03	-34.316e+03	409.114e+03	7.527e+06
1	7	-246.399e+03	-81.424e+03	-1.378e+06	4.916e+06
1	8	-301.443e+03	-125.873e+03	-4.138e+06	1.327e+06
1	9	-356.069e+03	-166.619e+03	-7.378e+06	-2.770e+06
1	10	-409.962e+03	-203.937e+03	-11.936e+06	-6.627e+06
2	0	203.937e+03	408.984e+03	-11.936e+06	-6.627e+06
2	1	166.619e+03	356.069e+03	-7.378e+06	-2.770e+06
2	2	125.873e+03	301.443e+03	-4.138e+06	1.327e+06
2	3	81.424e+03	246.399e+03	-1.378e+06	4.916e+06
2	4	34.316e+03	191.354e+03	409.114e+03	7.527e+06
2	5	-15.352e+03	136.597e+03	1.666e+06	9.083e+06
2	6	-67.495e+03	82.428e+03	2.393e+06	9.579e+06
2	7	-122.022e+03	29.128e+03	2.590e+06	8.961e+06
2	8	-178.815e+03	-23.043e+03	2.257e+06	7.200e+06
2	9	-237.735e+03	-68.616e+03	1.393e+06	4.224e+06
2	10	-298.623e+03	-102.086e+03	0.000e+00	0.000e+00

Service III Limit State Envelopes

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	109.884e+03	230.830e+03	0.000e+00	0.000e+00
1	1	76.737e+03	180.811e+03	1.520e+06	3.262e+06
1	2	36.142e+03	132.003e+03	2.510e+06	5.552e+06
1	3	-8.512e+03	84.503e+03	2.970e+06	6.891e+06
1	4	-53.862e+03	38.398e+03	2.900e+06	7.322e+06
1	5	-99.747e+03	-6.239e+03	2.300e+06	6.864e+06
1	6	-145.994e+03	-49.355e+03	1.169e+06	5.550e+06
1	7	-192.417e+03	-90.894e+03	-491.436e+03	3.382e+06
1	8	-238.841e+03	-130.798e+03	-2.954e+06	408.924e+03
1	9	-285.007e+03	-168.422e+03	-5.917e+06	-3.081e+06
1	10	-330.722e+03	-203.937e+03	-9.894e+06	-6.627e+06
2	0	203.937e+03	330.120e+03	-9.894e+06	-6.627e+06
2	1	168.422e+03	285.007e+03	-5.917e+06	-3.081e+06
2	2	130.798e+03	238.841e+03	-2.954e+06	408.924e+03
2	3	90.894e+03	192.417e+03	-491.436e+03	3.382e+06
2	4	49.355e+03	145.994e+03	1.169e+06	5.550e+06
2	5	6.239e+03	99.747e+03	2.300e+06	6.864e+06
2	6	-38.398e+03	53.862e+03	2.900e+06	7.322e+06
2	7	-84.503e+03	8.512e+03	2.970e+06	6.891e+06

2	8	-132.003e+03	-36.142e+03	2.510e+06	5.552e+06
2	9	-180.811e+03	-76.737e+03	1.520e+06	3.262e+06
2	10	-230.830e+03	-109.884e+03	0.000e+00	0.000e+00

Fatigue Limit State Envelopes

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-5.758e+03	54.435e+03	0.000e+00	0.000e+00
1	1	-5.758e+03	46.737e+03	-93.576e+03	759.476e+03
1	2	-7.929e+03	39.236e+03	-187.153e+03	1.275e+06
1	3	-14.761e+03	32.029e+03	-280.730e+03	1.598e+06
1	4	-22.308e+03	25.209e+03	-374.307e+03	1.708e+06
1	5	-29.581e+03	18.866e+03	-467.884e+03	1.667e+06
1	6	-36.486e+03	13.097e+03	-561.460e+03	1.510e+06
1	7	-42.928e+03	8.021e+03	-655.037e+03	1.190e+06
1	8	-48.812e+03	4.050e+03	-748.614e+03	745.168e+03
1	9	-54.058e+03	1.660e+03	-842.191e+03	257.446e+03
1	10	-58.534e+03	0.000e+00	-935.768e+03	0.000e+00
2	0	0.000e+00	57.770e+03	-935.768e+03	0.000e+00
2	1	-1.660e+03	54.058e+03	-842.191e+03	257.446e+03
2	2	-4.050e+03	48.812e+03	-748.614e+03	745.168e+03
2	3	-8.021e+03	42.928e+03	-655.037e+03	1.190e+06
2	4	-13.097e+03	36.486e+03	-561.460e+03	1.510e+06
2	5	-18.866e+03	29.581e+03	-467.884e+03	1.667e+06
2	6	-25.209e+03	22.308e+03	-374.307e+03	1.708e+06
2	7	-32.029e+03	14.761e+03	-280.730e+03	1.598e+06
2	8	-39.236e+03	7.929e+03	-187.153e+03	1.275e+06
2	9	-46.737e+03	5.758e+03	-93.576e+03	759.476e+03
2	10	-54.435e+03	5.758e+03	0.000e+00	0.000e+00

Loading Check
State Route 45 (Old Hickory Blvd. over I-65)
Davidson County
Date: 7/10/01

MAX POSITIVE MOMENT CALCULATIONS (LRFD)

MOMENT DUE TO UNIFORM LANE LOAD:

Mw = (W)(X)(3(l) - 4(X)) / 8 FOR BOTH SPANS LOADED

Mw = (W)(X)(7(l) - 8(X)) / 16 , FOR ONE SPAN LOADED

WHERE: W = 0.640 KIP PER LINEAR FOOT
X = 64.000 FEET
l = 162.500 FEET

DETERMINATION OF CONTROLING, Mw,
ONE SPAN LOADED, Mw = 1601.28 'K
BOTH SPANS LOADED, Mw = 1185.28 'K

AT SPAN POINT 1.394 THE Mw = 1601.28 'K

MOMENT DUE TO TRUCK LOAD:

Mp = P(a)(b)(4(l)^2 - a(l + a)) / 4(l^3)

WHERE: Pr = WEIGTH OF THE DESIGN TRUCK REAR AXLE = 32.000 KIPS
Pf = WEIGTH OF THE DESIGN TRUCK FRONT AXLE = 8.000 KIPS
a = DIST FROM ABUTMENT NO. 1 TO CGS OF TRUCK = 64.000 FEET
b = DIST FROM CGS OF TRUCK TO CL PIER = 98.500 FEET
l = SPAN LENGT 162.500 FEET
DISTANCE BETWEEN REAR AXLES = 14.000 FEET
DISTANCE FROM FRONT AXLE TO CGS OF LOAD = 18.667 FEET

MOMENT FROM FRONT 8.00 KIP AXLE:
a = 82.666667 FEET
b = 79.833333 FEET
Mpf = 262.55981 'K

MOMENT FROM FIRST 32.00 KIP AXLE:
a = 68.666667 FEET
b = 93.833333 FEET
Mpr1 = 1078.1396 'K

MOMENT FROM SECOND 32.00 KIP AXLE:
a = 54.666667 FEET
b = 107.83333 FEET
Mpr2 = 1030.3665 'K

THEREFORE: Mp = Mpf + Mpr1 + Mpr2 = 2371.0659 'K AT SPAN POINT 1.394

TOTAL UNFACTORED MOMENT AT THE SECTION:

Mtot = (Mw + (LLDF)(IM)(Mp))

WHERE: LLDF = LIVE LOAD DISTRIBUTION FACTOR FOR MOMENT = 1.000 LANES PER LINE OF GIRDERS
IM = DYNAMIC LOAD ALLOWANCE = 1.33

Mtot 4754.80 'K MOMENT FROM QCom PROGRAM: 4513.00 'K % DIFFERANCE = 5.358 %

TOTAL STRENGTH I MOMENT AT SECTION:

M(strength I) = 0.95(1.75(Mtot)) = 7904.851 'K MOMENT FROM QCom: 7503 'K % DIFFERANCE = 5.356 %

MAX NEGATIVE MOMENT CALCULATIONS (LRFD)

MOMENT DUE TO UNIFORM LANE LOAD:

Mw = (W)(X)(3(l) - 4(X)) / 8 FOR BOTH SPANS LOADED

Mw = (W)(X)(7(l) - 8(X)) / 16 , FOR ONE SPAN LOADED

WHERE: W = 0.640 KIP PER LINEAR FOOT
X = 162.500 FEET
l = 162.500 FEET

DETERMINATION OF CONTROLING, Mw,
ONE SPAN LOADED, Mw = 1056.250 'K
BOTH SPANS LOADED, Mw = 2112.500 'K

AT SPAN POINT 2.000 THE Mw = 2112.5 'K

MOMENT DUE TO FIRST TRUCK LOAD:

Mp = P(a)(b)(l + a) / 4(l^2)

WHERE: Pr = WEIGTH OF THE DESIGN TRUCK REAR AXLE = 32.000 KIPS
Pf = WEIGTH OF THE DESIGN TRUCK FRONT AXLE = 8.000 KIPS
a = DIST FROM ABUTMENT NO. 1 TO CGS OF TRUCK = 96.000 FEET
b = DIST FROM CGS OF TRUCK TO CL PIER = 66.500 FEET
l = SPAN LENGT 162.500 FEET
DISTANCE BETWEEN REAR AXLES = 14.000 FEET
DISTANCE FROM FRONT AXLE TO CGS OF LOAD = 18.667 FEET

MOMENT FROM FRONT 8.00 KIP AXLE:
a = 114.66667 FEET
b = 47.833333 FEET
Mpf = 115.14156 'K

MOMENT FROM FIRST 32.00 KIP AXLE:
a = 100.66667 FEET
b = 61.833333 FEET
Mpr1 = 496.2751 'K

MOMENT FROM SECOND 32.00 KIP AXLE:
a = 86.666667 FEET
b = 75.833333 FEET
Mpr2 = 496.11852 'K

THEREFORE: Mp1 = Mpf + Mpr1 + Mpr2 = 1107.5352 'K AT SPAN POINT 2.000 , FIRST TRUCK CGS LOCATED AT SPAN PT. 1.591

MOMENT DUE TO SECOND TRUCK LOAD IN SPAN NO 2:

Mp = P(a)(b)(l + a) / 4(l^2)

WHERE: Pr = WEIGTH OF THE DESIGN TRUCK REAR AXLE = 32.000 KIPS
Pf = WEIGTH OF THE DESIGN TRUCK FRONT AXLE = 8.000 KIPS
a = DIST FROM ABUTMENT NO. 1 TO CGS OF TRUCK = 91.000 FEET
b = DIST FROM CGS OF TRUCK TO CL PIER = 71.500 FEET
l = SPAN LENGT 162.500 FEET
DISTANCE BETWEEN REAR AXLES = 14.000 FEET
DISTANCE FROM FRONT AXLE TO CGS OF LOAD = 18.667 FEET

MOMENT FROM FRONT 8.00 KIP AXLE:
a = 72.333333 FEET
b = 90.166667 FEET
Mpf = 116.00254 'K

MOMENT FROM FIRST 32.00 KIP AXLE:
a = 86.333333 FEET
b = 76.166667 FEET
Mpr1 = 495.71867 'K

MOMENT FROM SECOND 32.00 KIP AXLE:
a = 100.33333 FEET
b = 62.166667 FEET
Mpr2 = 496.66839 'K

THEREFORE: Mp2 = Mpf + Mpr1 + Mpr2 = 1108.3896 'K AT SPAN POINT 2.000 , SECOND TRUCK CGS LOCATED AT SPAN PT. 2.440

THEREFORE: Mp = Mp1 + Mp2 = 2215.9248 'K

TOTAL UNFACTORED MOMENT AT THE SECTION:

Mtot = 0.90(Mw + (LLDF)(IM)(Mp))

WHERE: LLDF = LIVE LOAD DISTRIBUTION FACTOR FOR MOMENT = 1.000 LANES PER LINE OF GIRDERS
IM = DYNAMIC LOAD ALLOWANCE = 1.33

Mtot 4553.71 'K MOMENT FROM QCon PROGRAM: 4691.00 'K % DIFFERANCE = -2.927 %

TOTAL STRENGTH I MOMENT AT SECTION:

M(strength I) = 0.95(1.75(Mtot)) = 7570.5461 'K MOMENT FROM QCon: 7798.7'K % DIFFERANCE = -2.929 %

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MAX REACTION AT ABUTMENT NO 1 CALCULATIONS (LRFD)

REACTION DUE TO UNIFORM LANE LOAD:
Rw = (6)(w)(l) / 16 FOR BOTH SPANS LOADED
WHERE: W = 0.640 KIP PER LINEAR FOOT
X 0.000 FEET
l = 162.500 FEET
AT SPAN POINT 1.000 THE Rw = 45.5 KIP

Rw = (7)(w)(l) / 16 , FOR ONE SPAN LOADED
DETERMINATION OF CONTROLING, Rw,
ONE SPAN LOADED, Rw = 45.5 KIP
BOTH SPANS LOADED, Rw = 39 KIP

REACTION DUE TO TRUCK LOAD:
Rp = P(b)(4(l)^2 - a(l +a)) / 4(l^3)
WHERE: Pr = WEIGTH OF THE DESIGN TRUCK REAR AXLE = 32.000 KIPS
Pf = WEIGTH OF THE DESIGN TRUCK FRONT AXLE = 8.000 KIPS
l = SPAN LENGT 162.500 FEET
DISTANCE BETWEEN REAR AXLES = 14.000 FEET
DISTANCE FROM FRONT AXLE TO CGS OF LOAD = 18.667 FEET
SINGLE TRUCK HEADING FROM ABUTMENT NO 1 TOWARD PIER NO 1:
a = DIST FROM ABUTMENT NO. 1 TO CGS OF TRUCK = 11.000 FEET
b = DIST FROM CGS OF TRUCK TO CL PIER = 151.500 FEET

REACTION FROM FRONT 8.00 KIP AXLE:	REACTION FROM FIRST 32.00 KIP AXLE:	REACTION FROM SECOND 32.00 KIP AXLE:
a = 29.666667 FEET	a = 15.666667 FEET	a = 1.6666667 FEET
b = 132.83333 FEET	b = 146.83333 FEET	b = 160.83333 FEET
Rpf = 6.187 KIP	Rpr1 = 28.151 KIP	Rpr2 = 31.590 KIP

THEREFORE: Rp = Rpf + Rpr1 + Rpr2 = 65.927 KIP AT SPAN POINT 1.000
SINGLE TRUCK HEADING FROM PIER NO 1 TOWARD ABUTMENT NO 2:
a = DIST FROM ABUTMENT NO. 2 TO CGS OF TRUCK = 8 FEET
b = DIST FROM CGS OF TRUCK TO CL PIER = 154.500 FEET

REACTION FROM FRONT 8.00 KIP AXLE:	REACTION FROM FIRST 32.00 KIP AXLE:	REACTION FROM SECOND 32.00 KIP AXLE:
a = -10.66667 FEET	a = 3.3333333 FEET	a = 17.333333 FEET
b = 173.16667 FEET	b = 159.16667 FEET	b = 145.16667 FEET
Rpf = 0.000 KIP	Rpr1 = 31.180 KIP	Rpr2 = 27.743 KIP

THEREFORE: Rp = Rpf + Rpr1 + Rpr2 = 58.923 KIP AT SPAN POINT 2.951
TOTAL UNFACTORED REACTION AT THE SECTION:
Rtot = (Rw + (LLDF)(IM)(Rp)) CONTROLING Rp = 65.927 KIP
WHERE: LLDF = LIVE LOAD DISTRIBUTION FACTOR FOR SHEAR = 1.000 LANES PER LINE OF GIRDERS
IM = DYNAMIC LOAD ALLOWANCE = 1.33
Rtot 133.18 KIP REACTION FROM QCon. PROGRAM: 134.5 KIP % DIFFERANCE = -0.979 %
TOTAL STRENGTH I REACTION AT SECTION:
R(strength I) = 0.95(1.75(Rtot)) = 221.41668 KIP, REACTION QCon.: 223.6 KIP % DIFFERANCE = -0.976 %

MAX NEGATIVE REACTION AT PIER CALCULATIONS (LRFD)

REACTION DUE TO UNIFORM LANE LOAD:
Rw = (5)(W)(l) / 4 FOR BOTH SPANS LOADED
WHERE: W = 0.640 KIP PER LINEAR FOOT
X 0.000 FEET
l = 162.500 FEET
AT SPAN POINT 1.000 THE Rw = 130 KIP

Rw = (5)(W)(l) / 8 , FOR ONE SPAN LOADED
DETERMINATION OF CONTROLING, Mw,
ONE SPAN LOADED, Rw = 65.000 KIP
BOTH SPANS LOADED, Rw = 130.000 KIP

REACTION DUE TO FIRST TRUCK LOAD IN SPAN NO 1:
Rp = P(a)(2(l^2) + b(l + a)) / 2(l^3)
WHERE: Pr = WEIGTH OF THE DESIGN TRUCK REAR AXLE = 32.000 KIPS
Pf = WEIGTH OF THE DESIGN TRUCK FRONT AXLE = 8.000 KIPS
a = DIST FROM ABUTMENT NO. 1 TO CGS OF TRUCK = 143.800 FEET
b = DIST FROM CGS OF TRUCK TO CL PIER = 18.700 FEET
l = SPAN LENGT 162.500 FEET
DISTANCE BETWEEN REAR AXLES = 14.000 FEET
DISTANCE FROM FRONT AXLE TO CGS OF LOAD = 18.667 FEET

REACTION FROM FRONT 8.00 KIP AXLE:	REACTION FROM FIRST 32.00 KIP AXLE:	REACTION FROM SECOND 32.00 KIP AXLE:
a = 162.46667 FEET	a = 148.46667 FEET	a = 134.46667 FEET
b = 0.0333333 FEET	b = 14.033333 FEET	b = 28.033333 FEET
Rpf = 8.000 KIP	Rpr1 = 31.652 KIP	Rpr2 = 30.654 KIP

THEREFORE: Rp1 = Rpf + Rpr1 + Rpr2 = 70.306 KIP AT SPAN POINT 1.000 , FIRST TRUCK CGS LOCATED AT SPAN PT. 1.885
Calculation for Max. Shear at Pier (one span Loaded)
Max. Shear at Pier: Rtot = 1.33 (Rp1 + Rw) 179.95693 kips Shear at Pier from Qcon. is 160.6 kips % differance = 10.756422 %
Note: QCon program multiplies the shear at the pier by 0.90 which the Spec. only requries the reaction be multi by 0.9
0.9(Rtot.) = 161.96123 kips which is very close to QCon's value of 160.6 kips

REACTION DUE TO SECOND TRUCK LOAD IN SPAN NO 2:
Rp = P(a)(2(l^2) + b(l + a)) / 2(l^3)
WHERE: Pr = WEIGTH OF THE DESIGN TRUCK REAR AXLE = 32.000 KIPS
Pf = WEIGTH OF THE DESIGN TRUCK FRONT AXLE = 8.000 KIPS
a = DIST FROM ABUTMENT NO. 1 TO CGS OF TRUCK = 103.200 FEET
b = DIST FROM CGS OF TRUCK TO CL PIER = 59.300 FEET
l = SPAN LENGT 162.500 FEET
DISTANCE BETWEEN REAR AXLES = 14.000 FEET
DISTANCE FROM FRONT AXLE TO CGS OF LOAD = 18.667 FEET

REACTION FROM FRONT 8.00 KIP AXLE:	REACTION FROM FIRST 32.00 KIP AXLE:	REACTION FROM SECOND 32.00 KIP AXLE:
a = 84.533333 FEET	a = 98.533333 FEET	a = 112.53333 FEET
b = 77.966667 FEET	b = 63.966667 FEET	b = 49.966667 FEET
Rpf = 5.679 KIP	Rpr1 = 25.538 KIP	Rpr2 = 27.927 KIP

THEREFORE: Rp2 = Rpf + Rpr1 + Rpr2 = 59.144 KIP AT SPAN POINT 1.000 , SECOND TRUCK CGS LOCATED AT SPAN PT. 2.365
THEREFORE: Rp = Rp1 + Rp2 = 129.450 KIP
TOTAL UNFACTORED REACTION AT THE PIER NO 1:
Rtot = 0.90(Rw + (LLDF)(IM)(Rp))
WHERE: LLDF = LIVE LOAD DISTRIBUTION FACTOR FOR MOMENT = 1.000 LANES PER LINE OF GIRDERS
IM = DYNAMIC LOAD ALLOWANCE = 1.33
Rtot 271.95 KIP REACTION FROM Qcon. PROGRAM: 320.12 KIP % DIFFERANCE = -15.047 %
TOTAL STRENGTH I REACTION AT PIER NO 1:
R(strength I) = 0.95(1.75(Rtot)) = 452.12029 KIP REACTION FROM Qcon: 532.2 KIP % DIFFERANCE = -15.047 %
Note: QCon Program did not calculate a reaction at the Pier, the only way to get the reaction is to add shears of adjacent spans

LRFD Moments and Shears
Old Hickory Bvld. Over I-65
Davidson County
Date: November 21, 2001

Non-Factored Dead & Live Loads: (Qcon. Program with constant girder section)

note: Live loads account for the 1.33 impact factor

Span Pt.	total dead load (Dc)		Comp.dead load (Dw)		Min. Live Load w/o LLDF		Max. Live Load w/o LLDF	
	Moment	Shear	Moment	Shear	Moment	Shear	Moment	Shear
1.0000	0.0	101.6	0	20.7	0	-15.6	0	135.6
1.1000	1431	74.5	291.8	15.2	-253.5	-16.2	1923	113.8
1.2000	2422	47.4	493.8	9.7	-506.9	-26.2	3295	93.6
1.3000	2973	20.3	606	4.1	-760.4	-41.2	4140	75
1.4000	3083	-6.8	628.5	-1.4	-1013	-57.1	4513	58.2
1.5000	2752	-33.9	561.1	-6.9	-1267	-73.7	4438	43.2
1.6000	1982	-60.9	404	-12.4	-1520	-90.72	3955	30.1
1.7000	771	-88.1	157.1	-18	-1774	-108	3068	18.9
1.8000	-880	-115.2	-179.6	-23.5	-2367	-125.2	1836	9.8
1.9000	-2973	-142.3	-606	-29	-2922	-142.1	621.8	3.6
2.0000	-5505	-169.4	-1122	-34.5	-4083	-158.5	0	0

Non-Factored Dead & Live Loads: (BTBeam with constant girder section)

Note: The live loads account for the 1.33 impact factor
Non-Composite Uniform Dead Load (Dc): 1.5300 Kip-foot (Dead load of Beam and Slab)
Composite Uniform Dead Load (Dc): 0.1380 Kip-foot (Dead load of rails and attachments)
Total uniform Dead load (Dc): 1.6680 Kip-foot (Dead load of Bm, Slab, rails&attachments)
Composite Uniform Dead Load (Dw): 0.3400 Kip-foot (Dead load of Wearing Surface)
Design Span length (L) = 162.50 feet X = location where moment is requested
Moment equation for uniform Dead loads for both spans loaded: Mx = (Wo)(X)(3(L)-4(X)) / 8
Shear equation for uniform Dead loads for both spans loaded: Vx = (3/8)(Wo)(L) - (Wo)(X)

Span Pt.	total dead load Dc		Comp.dead load Dw		Min. Live Load w/o LLDF		Max. Live Load w/o LLDF	
	Moment	Shear	Moment	Shear	Moment	Shear	Moment	Shear
1.0000	0.0	101.6	0.0	20.7	0.0	-15.5	0.0	134.8
1.1000	1431.5	74.5	291.8	15.2	-253.8	-16.2	1924.5	113.1
1.2000	2422.5	47.4	493.8	9.7	-507.6	-26.1	3297.8	93.0
1.3000	2973.1	20.3	606.0	4.1	-761.5	-41.0	4144.0	74.5
1.4000	3083.2	-6.8	628.5	-1.4	-1015.3	-56.8	4519.6	57.7
1.5000	2752.9	-33.9	561.1	-6.9	-1269.1	-73.2	4444.5	42.8
1.6000	1982.1	-61.0	404.0	-12.4	-1522.9	-90.2	3960.0	29.8
1.7000	770.8	-88.1	157.1	-18.0	-1776.7	-107.3	3071.9	18.8
1.8000	-880.9	-115.2	-179.6	-23.5	-2357.3	-124.5	1839.3	9.8
1.9000	-2973.1	-142.3	-606.0	-29.0	-2911.3	-141.5	618.9	3.6
2.0000	-5505.7	-169.4	-1122.3	-34.5	-4562.6	-157.8	0.0	0.0

Live load shear at supports: (from Btbeam program)
abutments: 134.8 kips
Piers: 275.3 kips

location of dead load inflection point: 41.2 ft
(dist. Measure from CL of pier to DL inflection point)

Dead Load (Dc) Shears at supports:
abutments: 101.6 kips
Piers: 338.8 kips

Dead Load (Dw) Shears at supports:
abutments: 20.7 kips
Piers: 69.0 kips

Span Pt.	non-comp.dead load Dc		Comp.dead load Dc		total dead load, Dc	
	Moment	Shear	Moment	Shear	moment	shear
1.0000	0.0	93.2	0.0	8.4	0.0	101.6
1.1000	1313.1	68.4	118.4	6.2	1431.5	74.5
1.2000	2222.1	43.5	200.4	3.9	2422.5	47.4
1.3000	2727.1	18.6	246.0	1.7	2973.1	20.3
1.4000	2828.1	-6.2	255.1	-0.6	3083.2	-6.8
1.5000	2525.1	-31.1	227.8	-2.8	2752.9	-33.9
1.6000	1818.1	-55.9	164.0	-5.0	1982.1	-61.0
1.7000	707.0	-80.8	63.8	-7.3	770.8	-88.1
1.8000	-808.0	-105.7	-72.9	-9.5	-880.9	-115.2
1.9000	-2727.1	-130.5	-246.0	-11.8	-2973.1	-142.3
2.0000	-5050.2	-155.4	-455.5	-14.0	-5505.7	-169.4

LRFD Moments and Shears
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Live Load distribution factors:

Moment:

beam	one-lane loaded	two/more lanes loaded
exterior	0.914	0.759
interior	0.401	0.708

Shear:

beam	one-lane loaded	two/more lanes loaded
exterior	0.97	0.945
interior	0.849	1.08

Article 1.3.2 Limit States

Applied Load Modifiers:

ductility (art.1.3.3)	0.95
redundancy (art.1.3.4)	1
opt. Importance(1.3.5)	1

applied load modifier=(duc.)(red.)(opt.imp.)
applied Load Modifier 0.95
the applied load modifier > or = 0.95

Strength-I Limit State: (Yp)(Dc) + (1.75)(LL)

Load Factor for permanent loads, Yp.

type of load	load factor, Yp	
	Maximum	Minimum
Dc	1.25	0.9
Dw	1.5	0.65

Controlling Live Load Distribution factors:
moment: 0.9140 lanes/girder (ext.bm one lane loaded)
shear: 1.0800 lanes/girder (int.bm two or more lanes loaded)

MOMENTS: (contains applied load modifier)

Span Pt.	non-comp.dead load Dc		Comp.dead load Dw		Live Load		Strenght I moments	
	max.	min.	max.	min.	max.	min.	max. pos.	max. neg.
1.0000	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1.1000	1699.9	1223.9	415.8	180.2	2924.3	-385.7	5040.0	1730.0
1.2000	2876.7	2071.2	703.7	304.9	5011.1	-771.3	8591.5	2809.1
1.3000	3530.5	2542.0	863.6	374.2	6296.9	-1157.1	10691.0	3237.0
1.4000	3661.3	2636.1	895.6	388.1	6867.6	-1542.8	11424.5	3014.1
1.5000	3269.0	2353.7	799.6	346.5	6753.5	-1928.4	10822.2	2140.2
1.6000	2353.7	1694.7	575.7	249.5	6017.3	-2314.1	8946.7	615.3
1.7000	915.3	659.0	223.9	97.0	4667.8	-2699.8	5807.0	-1560.6
1.8000	-1046.1	-753.2	-255.9	-110.9	2794.9	-3582.0	1492.9	-4883.9
1.9000	-3530.5	-2542.0	-863.6	-374.2	940.4	-4423.8	-3453.8	-8817.9
2.0000	-6538.0	-4707.4	-1599.2	-693.0	0.0	-6933.0	-8137.3	-15070.2

SHEARS: (contains applied load modifier)

Span Pt.	non-comp.dead load Dc		Comp.dead load Dw		Live Load		Strenght I shears	
	max.	min.	max.	min.	max.	min.	max. pos.	max. neg.
1.0000	120.7	86.9	29.5	12.8	242.0	-27.8	392.3	122.4
1.1000	88.5	63.7	21.7	9.4	203.1	-29.1	313.2	81.1
1.2000	56.3	40.6	13.8	6.0	167.0	-46.9	237.1	23.2
1.3000	24.1	17.4	5.9	2.6	133.8	-73.6	163.8	-43.5
1.4000	-8.0	-5.8	-2.0	-0.9	103.6	-102.0	93.6	-112.0
1.5000	-40.2	-29.0	-9.8	-4.3	76.8	-131.4	26.8	-181.5
1.6000	-72.4	-52.1	-17.7	-7.7	53.5	-162.0	-36.6	-252.1
1.7000	-104.6	-75.3	-25.6	-11.1	33.8	-192.7	-96.4	-322.9
1.8000	-136.8	-98.5	-33.5	-14.5	17.6	-223.5	-152.7	-393.8
1.9000	-169.0	-121.7	-41.3	-17.9	6.5	-254.1	-203.9	-464.4
2.0000	-201.2	-144.8	-49.2	-21.3	0.0	-283.3	-250.4	-533.7

Strenght I reactions (Kips) : (contains applied load modifier)

location	maxinum	minimum
abutment	392.3	122.4
Pier	958.4	427.7

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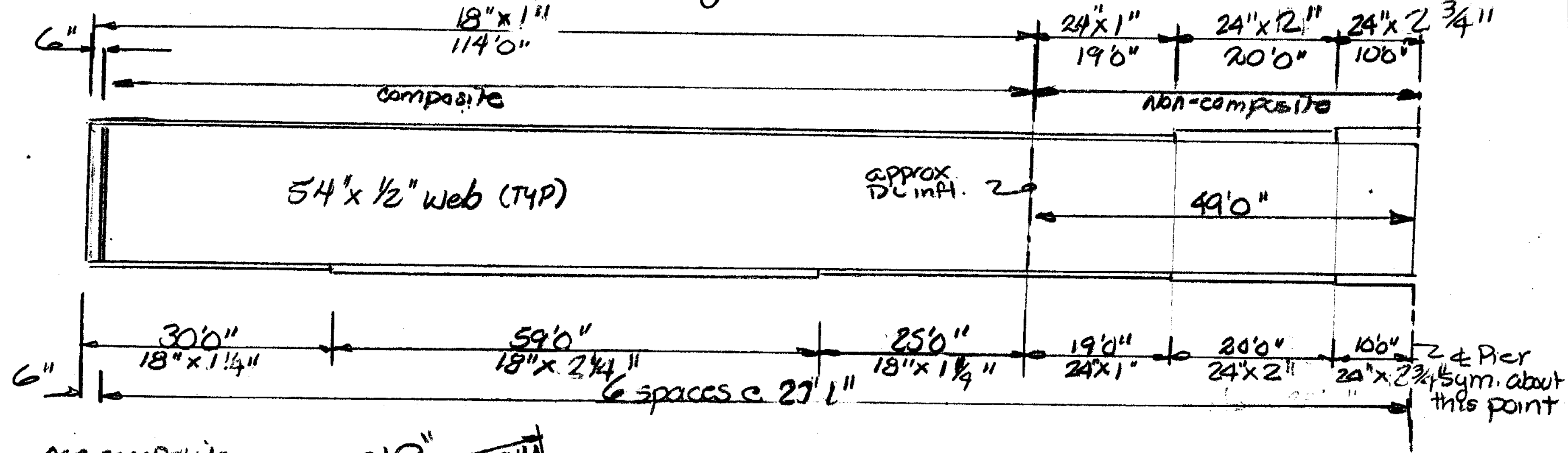
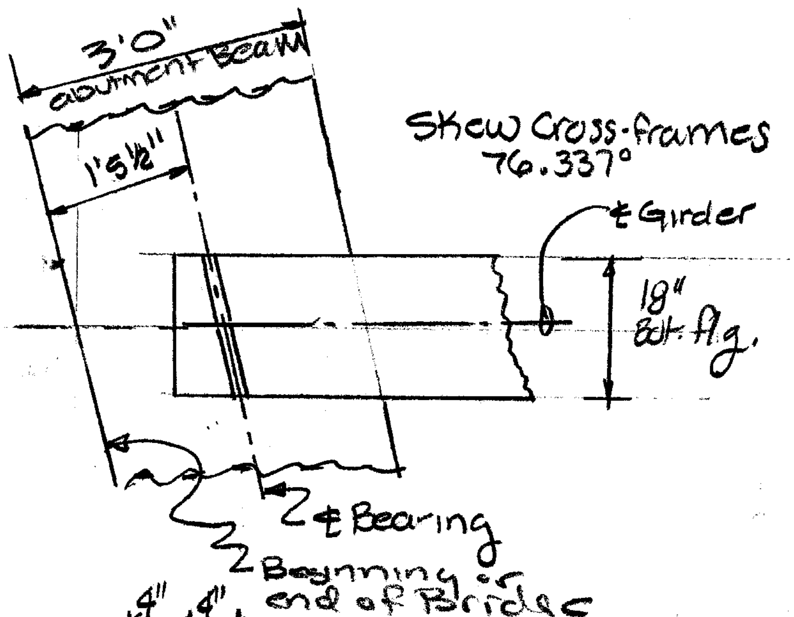
Girder Trail Section

(Girder length = 165' - 1.5/sin 76.337 + 0.5' = 162.96') use 163'



Non-composite Changes in Stiffness

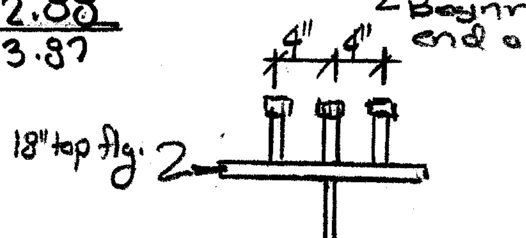
Span pt	Interia
1.0	1.0
1.1	1.0
1.2	1.33
1.3	1.33
1.4	1.33
1.5	1.33
1.6	1.0
1.7	1.34
1.8	2.88
1.9	2.88
2.0	3.87



Skew Cross-frames 76.337°

Composite (n=9) Changes in Stiffness

Span pt.	Interia
1.0	1.0
1.1	1.0
1.2	1.51
1.3	1.51
1.4	1.51
1.5	1.51
1.6	1.00
1.7	1.18
1.8	2.09
1.9	2.09
2.0	2.63



Shear Stud detail

DT-1412

TENNESSEE DEPARTMENT OF TRANSPORTATION DIVISION OF STRUCTURES 1100 J. K. POLK OFFICE NASHVILLE, TENNESSEE 37219

DESIGNED BY	DATE	PROJECT SR 45 / I-65	___ OF ___ PAGE
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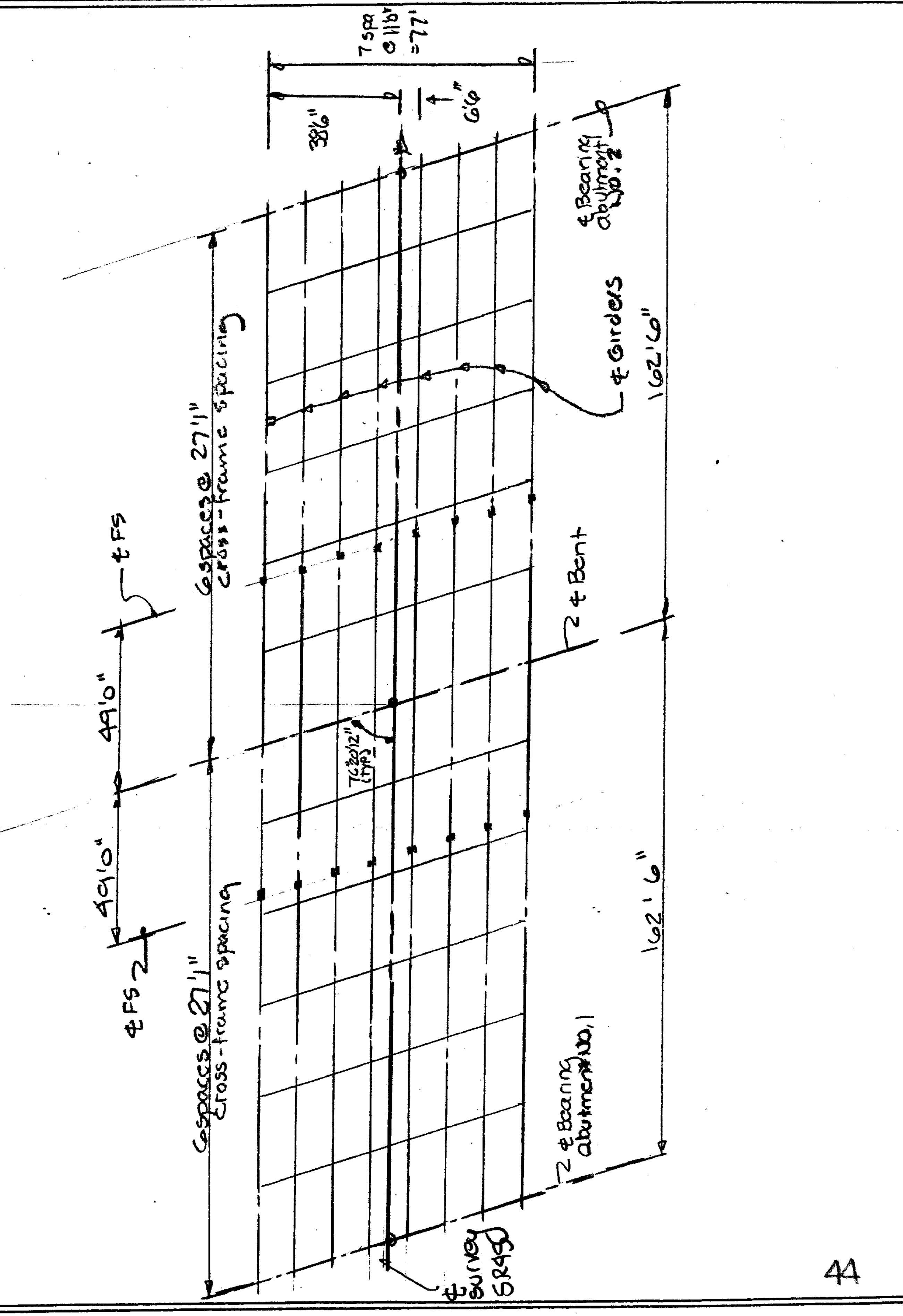
Girder layout



DT-1412

Slab Design
Old Hickory Blvd. Over I-65
Davidson County
Designer: whp
Date: October 16, 2001

Article 9.7, " Concrete Deck Slabs "

Design by the traditional design method (strip methold) in Article 9.7.3

The traditional design method is based on flexure. The live load force effect in the slab may be determined using the approximate methods of Art. 4.6.2.1 or the refined methods of Art. 4.6.3.2.

Distribution Reinforcement:

Reinforcement shall be placed in the secondary direction in the bottom of slabs as a percentage of the primary reinforcement for positive moment as follows:

for primary reinforcement perpendicular to traffic: 220 / (sqrt(S)) <= 67%

where S = the effective span length taken as equal to the effective length specified in article 9.7.2.3 (ft)

Width of equivalent strip of deck:

Where the strip method is used, the extreme positive moment in any deck panel between girders shall be taken to apply to all positive moment regions. Similarly, the extreme negative moment over any beam or girder shall be taken to apply . to all negaive moment regions.

The width of the equivalent strip of a deck may be taken as specified in table 1. But no greater than 144 inches .

Concrete deck with cast-place slab with or without stay-in-place formwork.

equivalent strip width (max. positive) = 26.00 + 6.6(S) = 99.7 inches

equivalent strip width (max. negative) = 26.00 +3.0(S) = 59.5 inches

S = spacing of supporting components (beam spacing) = 11.167 feet

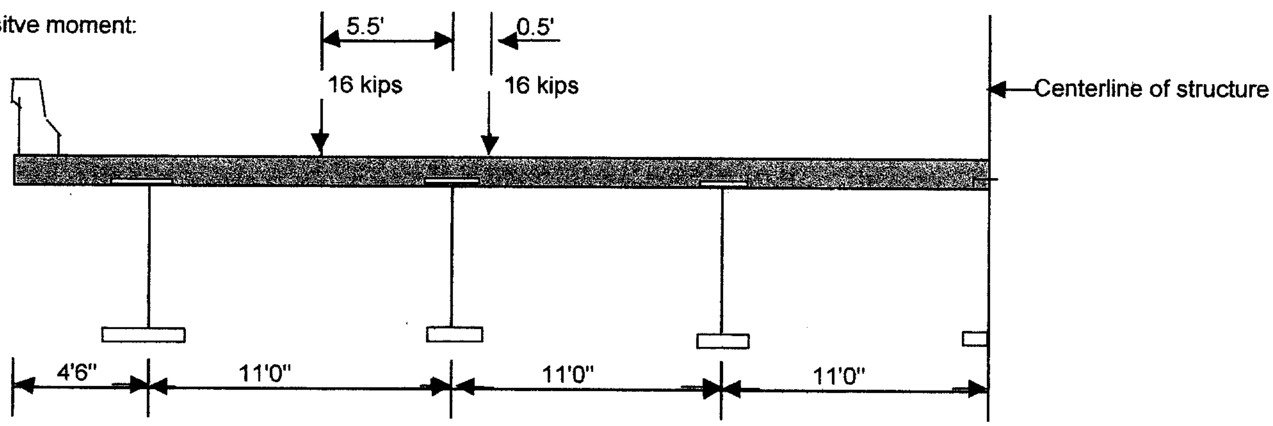
Concrete deck overhangs. (X) = distance from load to point of support = (4.5(12)-(18 in. flg / 4)-36.0)/12 = 1.125 ft

equivalent strip width (max. negative) = 45.00 + 10.0(X) = 56.3 inches

Calculation of force effects, art.4.6.2.1.6

The strips shall be treated as continuous beams. Span length shall be taken as the center-to-center distance between the supporting components. For the purpose of determining force effects in the strip, the supporting components shall be assumed to be infinitely rigid. The wheel loads may be modeled as concentrated loads. The Strips should be analyzed by classical beam theory.

Max. positve moment:



Max.Positive moment slab design (cont.)

Dead loads: 3 inch wearing surface (convert to equilvalent concrete slab) = 3 (108 / 150) = 2.16 inch

9.0 inch concrete slab, weight of concrete = 150 lbs per cubic

Live Loads: Multi-presense factor for single lane load = 1.20

impact for live load = 1.33

Simple beam moment for a uniform load = (w)(L)^2 / 8, for approx. cont. moment multiple by 0.80

Simple beam moment for concentrated load a mid-point = (P)(L) / 4, for approx. cont. moment multiple by 0.80

Max. Live Load Moment:

MLLpos. = (16 kip)(11.0 feet)(1.33)(0.80)(1.20) / ((4)(99.7/12)) = 6.762 foot-kip

Max. Dead Load Moment:

MDLpos. = (9.0in + 2.16in)(.150 k/ft^3)(0.8)(11.0 ft)^2 / (8)(12 in/ft) = 1.688 foot-kip

Slab Design
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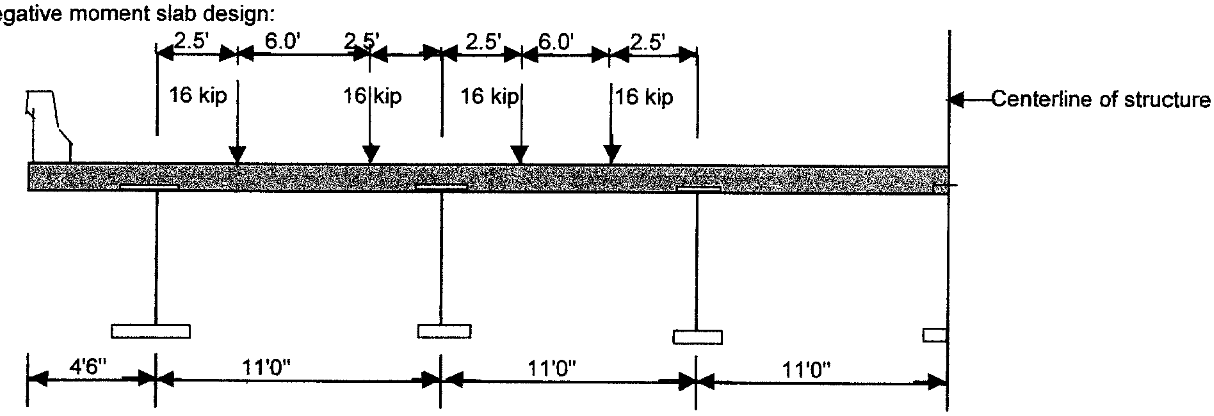
Max. Negative moment slab design:



Max.Negative moment slab design (cont.)

Dead loads: 3 inch wearing surface (convert to equilvalent concrete slab) = 3 (108 / 150) = 2.16 inch
9.0 inch concrete slab, weight of concrete = 150 lbs per cubic

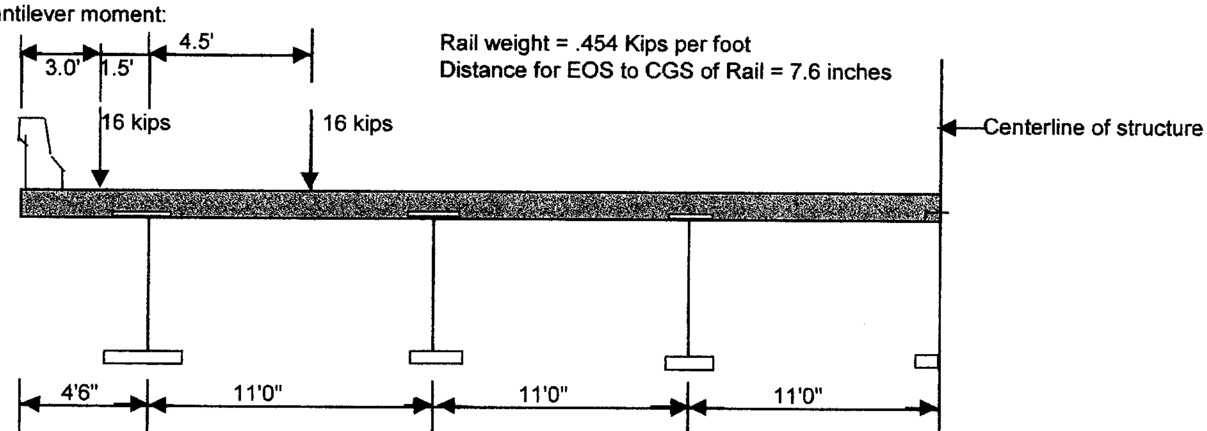
Live Loads: Multi-presense factor for two lanes loaded = 1.00
impact for live load = 1.33

two span beam max. neg. moment at int. support for a uniform load = (w)(L)^2 / 8, for approx. cont. moment multi.by 0.80
two span beam max. neg moment at int. support for a concentrated point load = ((P)(a)(b) / 4(L^2))(L+a),
for approx. continous moment for more than two spans multiple by 0.80

Max. Live Load Moment:
MLLpos. = (16 kip)(1.33)(0.80)(1.00)((2.5)(8.5)(11.0+2.5)+(8.5)(2.5)(11.0+8.5))/((4)(59.5/12)(11.0^2))
MLLpos. = 4.975 foot-kip

Max. Dead Load Moment:
MDLpos. = (9.0in + 2.16in)(.150 k/ft^3)(0.8)(11.0 ft)^2 / (8)(12 in/ft) = 1.688 foot-kip

Max. Cantilever moment:



equivalent strip width (max. negative) = 45.00 + 10.0(X) = 56.3 inches
Clear span of cantilever = 4.5(12) - flange width / 4 = 49.5 inches for 18 inch flange

Dead loads: 3 inch wearing surface (convert to equilvalent concrete slab) = 3 (108 / 150) = 2.16 inch
9.0 inch concrete slab, weight of concrete = 150 lbs per cubic

Live Loads: Multi-presense factor for two lanes loaded = 1.20
impact for live load = 1.33

neg. moment at support of cantilever for a uniform load = (w)(L)^2 /2
neg. moment at support of cantilever for a concentrated point load = (P)(b),

Max. Live Load Moment:
MLLpos. = (16 kip)(1.33)(1.20)(49.5 in - 36.0 in) / 12 (56.3 in/12) = 6.1231972 foot-kip

Max. Dead Load Moment:
MDLpos. Slab = (9.0in + 2.16in)(.150 k/ft^3)(49.5 /12 ft)^2 / (2)(12 in/ft) = 1.187 foot-kip
MDLpos. Rail = (0.454 kip) (49.5 in - 7.6 in) / 12 = 1.585 foot-kip
total MDLpos. = 2.772 foot-kip

Loading Combination: (Strenght I normal vehicular use without wind)
moment = 1.25(DL) + 1.75(LL)

:tored loadings (including 0.95 factor):

Max. Mpos. =	13.246	foot-kip
Max. Mneg. =	10.275	foot-kip over int. supports
Max. Mneg. =	13.472	foot-kip over ext. support

total factored load: Art. 1.3.2
Q = n(sum (Yi)(qi)) nd = 1.05 ductility
n = (nd)(nr)(ni) => 0.95 nr = 0.95 redundancy
n = 1.05(.95)(.95) = .95 ni = 0.95 importance
Mactual = 0.95(Moment from loading combination

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Designer: whp
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'alculation for required reinforcement for slab:

Mr = (phi)(Mn), which phi = 0.90 for flexure and b = 12 inches

Fy = 60 ksi, f'c = 3.0 ksi, d = 9.0 -3.0 = 6.0 inches

Mr = (0.90)(As)(Fy)(d -a / 2) a = depth of equilvalent stress block

a = (As)(Fy) / (0.85)(f'c)(b)

try number 5 bars at 6 inches for slab steel, As = (0.31in^2 / bar)(12 / 6) = 0.62 in^2

which a = 1.216 inches

therefore, Mr = 15.044 foot-kip => 13.472 foot-kip OK

Use number 5 bars at 6 inches both top and bottom mats of steel

Empirical Desigh of Reinforced Concrete Decks: (Art. 9.7.2.2)

Empirical design of reinforced concrete decks may be used if the conditions set forth in Article 9.7.2.4 are satisified

The provisions of this article shall not be applied to overhangs.

Article 9.7.2.4, The empirical design may be used only if the following conditions are satisfied:

a) Cross-frames or diaghragms are used throughout the cross-section at lines of support.
Cross-frames are spaced at 27'1", condition satisfied

b) The supporting components are made of steel and/or concrete
supporting components are welded steel plate girders, condition satisfied

c) The deck is of uniform depth, except for haunches at girder flanges and other local thickenings
uniform 9.0 inch slab is used through out the deck, condition satisified

d) The ratio of the effective length to design depth does not exceed 18.0 and is not less than 6.0.
(11.0)(12) / (9.0) = 14.7 < 18.0 but > 6.0, condition satisfied

e) The deck is fully cast-in-place and water cured
deck cured according to Specifications, condition satisfied

f) Core depth of the slab is not less than 4.0 inches
9.0 in - 2.5 in - 1.0 in = 5.5 in > 4.0 in, condition satisfied

g) The effective length, as specified in Article 9.7.2.3 does not exceed 13.5 feet
effective length = 117 in / 12 = 9.75 ft.< 13.5 ft., condition satisfied

h) The minimum depth of the slab is not less than 7.0 in., excluding a sacrifical wearing surface where applicable
slab depth = 9.0 in. > 7.0 in., conditon satisfied

I) There is an overhang beyound the centerline of the outside girder of at least 5.0 times the depth of the slab:
this condition is satisfied if the overhang is at least 3.0 times the depth of the slab and a structurally continouous concrete barrier is made composite with the overhang.
overhang = (4.5 ft)(12) = 54 in > 9.0 in (5.0) = 45.0 in, condition satisfied

k) The specified 28-day strenght of the deck concrete is not less than 4.0 ksi.
required deck strenght is 4.0 ksit, condition satisfied

L) The deck is made composite with the supporting structural components.
deck is composite with girders, condition satisfied
note: shear studs space at 24 inches over the int. supports will be required to meet this requirement

reinforcement requirements:

Four layers of isotropic reinforcement shall be provided in empricically designed slabs.

Reinforcement shall be located as close to the outside surfaces as permitted by cover requirements

Reinforcement shall be provided in each face of the slab with the outermost layers placed in the direction of the eff.length

The minimum amount of reinforcement shall be 0.27 in^2 per ft.of steel for each bottom layer and 0.18 in^2 per ft. of steel for each top layer. Spacing shall not exceed 18.0 inches

Reinforcing steel shall be grade 60 or better.

Therefore use no. 5 bars spaced at 12 inches, As = (.31 in^2) / 1.0 = 0.31 in^2 per foot > 0.27 in^2 per foot, OK
this will be required for both top and bottom mat, except top mat cantilever reinforcing steel which steel requires the spacing to be number 5 bars spaced at 6 inches.

Slab Design
Old Hickory Blvd. Over I-65
Davidson County
Designer: whp
Date: October 16, 2001

Art. 6.10.3.7 Minimum Negative Flexure Slab Reinforcement:

The total cross-sectional area of the longitudinal reinforcement shall not be less than 1 % of the total cross-sectional area of the slab wherever the long. tensile stress in the slab due to either factored construction loads or Load Combination Service II in Table 3.4.1-1 exceeds (phi)(fr) , where fr is the modulus of rupture of the concrete specified in Art. 5.4.2.6 and (phi) is the appropriate resistance factor for concrete in tension specified in Art. 5.5.4.2.1. The reinforcement used to satisfy this requirement shall have a specified min. yield strenght not less than 60 ksi and a size not exceeding no. 6 bars The required reinforcement shall be placed in two layers uniformly distributed across the slab width, and two-thirds shall be placed in the top layer. The individual bars shall be spaced at intervals not exceeding 6.0 inches within each row. When shear connectors are omitted over the negative flexural region, all longitudinal reinforcement shall be extended into the positive flexural region beyond the shear connectors a distance not less than the development length specified in section 5.

Section 5, Development of Reinforcement:

for No. 11 bar and smaller, lb = 1.25(Ab)(Fy) / (f'c^0.5) but not less than lb = 0.4(db)(Fy)

The size of reinforcement used in the slab is a no. 5 bar, Ab = 0.31in^2 per bar, db = 0.625 in, Fy = 60 kis, f'c = 3 ksi

lb= 13.423 inches, but not less than lb = 15.000 inches <controls

Modification Factors that increase the development length, min. bar spacing = 6.0 inches, epoxy coated steel

For epoxy-coated bars not covered below, use a 1.2 factor

concrete cover = 2.5 inches, 3(db) = 1.875 in. which is less than 2.5 inches therefore no further adjustment

bar spacing = 6.0 inches, 6(db) = 3.75 in. which is less than 6.0 inches therefore no further adjustment

the development length (lb) for no. 5 bar = 1.2(15) = 18 inches

Slab area:

out to out bridge width: 86.00 feet — area of slab = 9288.0 inches^2

slab thichness: 9.00 inches — 1% slab area = 92.88 inches^2

2/3 of 1 % of slab area = 61.92 inches^2 to be uniformly placed in top mat of steel

1/3 of 1 % of slab area = 30.96 inches^2 to be uniformly placed in bottom mat of steel

use no. 5 bars for slab steel, number of bars required in top mat of steel = 200 bars

number of bars required in bot mat of steel = 100 bars

for top mat use no. 5 bars spaced at 6 inches, number of bars = 171 bars

the 171 bars in the top should be adquate to meet the requirement, the only thing that could happen is that there may be a few more cracks in the slab.

for bottom mat, need to check distribution reinforcement requirement:

for priminary reinforcement perpendicular to traffic: 220 / (S^0.5) <= 67 %

S = effective span length = beam spacing - 0.5(flange width) = 10.25 ft.

220 / (S^0.5) = 68.716455 % therefore use 67 %

If empircal design method is used: slab reinforcement is no. 5's spaced at 12 inches, except canitlevers

if conventional design method is used: slab reinforcement is no. 5's spaced at 6 inches

use empircal design method, As requied = 0.67(0.31 in^2 per foot) = 0.21 in^2 per foot

required spacing = (0.31 in^2 per foot) / (0.21 in^2 per foot) = 1.48 ft or 18 inches

use 12 inch spacing

number of bars in the bottom mat of steel = (11 bars per bay)(8 bays) + (2 cantilevers)(6 bars per canitlever)

number of bars in the bottom mat of steel = 100 bars, which is equal to 100 bars requied OK

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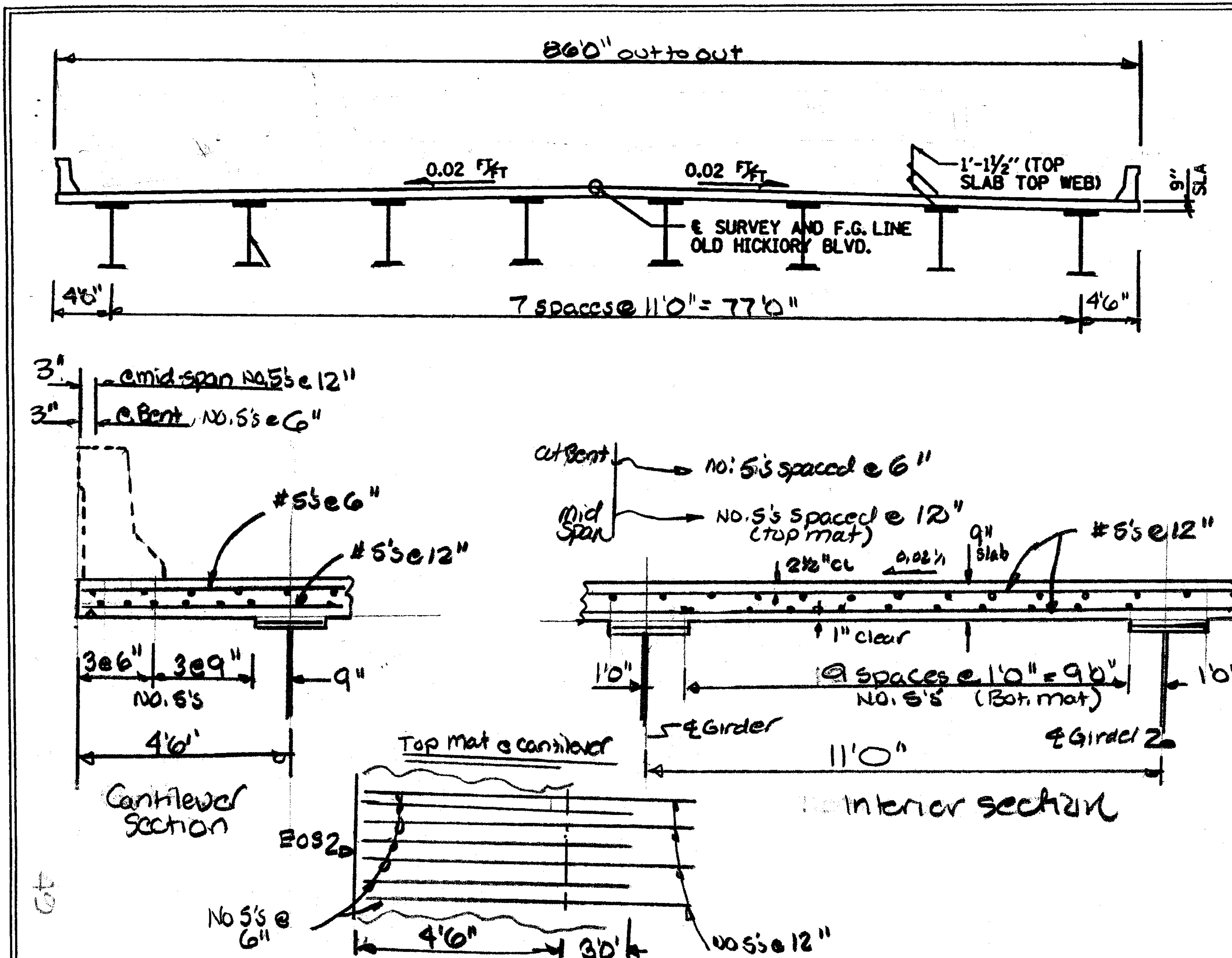

DT-1412

SECTION PROPERTIES FOR WELDED PLATE GIRDERS
Old Hickory Blvd. (SR-45) over I-65
Davidson County
Prepared by WHP
Date: July 30, 2003
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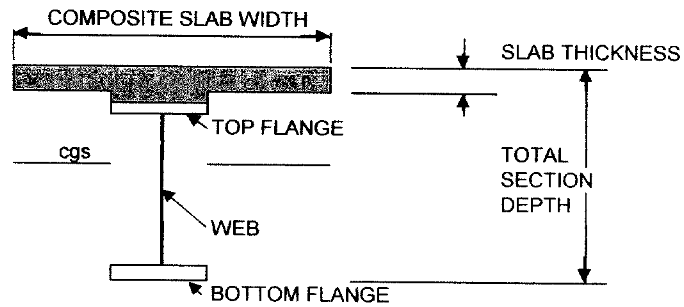


EXTERIOR BEAM
STRENGTH I LOADS

MOMENT DUE DL SLAB & BEAM:	2642 'k
MOMENT DUE DL WEARING AND RAIL:	646 'k
MOMENT DUE LIVE LOAD:	4594 'k
MOMENTS TAKEN AT SPAN POINT:	1.18 'k

REQUIRED INPUT: (inches)

COMPOSITE SLAB WIDTH:	117.00
SLAB THICKNESS:	9.00
FILLER:	1.50
TOP FLANGE WIDTH:	18.00
TOP FLANGE THICKNESS:	1.00
WEB DEPTH:	54.00
WEB THICKNESS:	0.50
BOTTOM FLANGE WIDTH:	18.00
BOTTOM FLG THICKNESS:	1.25
TOTAL DEPTH OF SECTION	66.75

NON-COMPOSITE PROPERTIES

(referance line in at bottom of bottom flange)

	Area	Y x Area	Y^2 x Area	Io	I ref
SLAB	0.00	0.00	0.0	0.00	0.00
TOP FLANGE	18.00	1003.50	55945.1	1.50	55946.63
WEB	27.00	762.75	21547.7	6561.00	28108.69
BOTTOM FLANGE	22.50	14.06	8.8	2.93	11.72
TOTALS	67.50	1780.31			84067.03

Ycgs= 26.38 in
I @ CGS= 37111.29 in^4

S slab= 0.00 in^3
S tf= 1242.22 in^3
S bf= 1407.06 in^3

COMPOSITE PROPERIES N=27

(reference line at the cgs of girder)

	AREA	Y X AREA	Y^2 x area	Io	I ref
SLAB	39	1399.1	50193.6	263.3	50456.9
GIRDER	67.5	0.0	0.00	37111.3	37111.3
TOTALS	106.5	1399.1			87568.1

Ycgs= 13.1 in
I @ cgs= 69187.4 in^4

S slab= 2540.1 in^3
S tf= 4133.6 in^3
S bf= 1751.0 in^3

COMPOSITE PROPERIES N=9

(reference line at the cgs of girder)

	AREA	Y X AREA	Y^2 x Area	Io	I ref
SLAB	117.0	4197.4	150580.8	789.8	151370.6
GIRDER	67.5	0.0	0	37111.3	37111.3
TOTALS	184.5	4197.4			188481.9

Ycgs= 22.75 in
I @ cgs= 92991.59 in^4

S slab= 5276.1 in^3
S tf= 13051.5 in^3
S bf= 1893.0 in^3

YIELD MOMENT CALULATIONS

Yield Moment at span pt 1.180
Applied Moment: (Strenght 1)
DLnc: 2642 'K
DLcomp 646 'K
LLcomp: 4594 'K
Total Actual Moment: 7882 'K

ADcomp IS THE ADDED MOMENT APPLIED TO THE COMPOSITE SECTION SO THAT THE STEEL WILL REACH ITS YIELD MOMENT.
Fy = YEILD STRESS = 50 KSI

ACTUAL STRESS:
Fslab= 13.50 KSI
Ftf= 31.62 KSI
Fbf= 56.08 KSI

Fy = DLnc / Snc + DLcomp / Scomp n=27 + ADcomp / Scomp N=9
SLOVE EQUATION FOR ADcomp
Top Flange: ADcomp = 24583 'K
Bottom Flange: ADcomp = 3635 'K CONTROLS

Modulus of Elasticity:
Concrete, Ec = 33,000((wc)^1.5)(f'c)^.5
wc = 0.150 kcf and f'c = 3 ksi
Ec = 3320.56 ksi
Steel, Es = 29,000 ksi

My = YEILD MOMENT = DLnc + DLcomp + ADcomp
My = 6923 'k > 7882 'K plastic section

If Moment is elastic, Dc = ((fdlnc + fdlcomp + fllcomp) / (fdlnc/Csteel + fdlcomp/C3n +fllcomp/Cn)) - tf

Csteel =	29.875 inches	fdlnc =	25.522 ksi
C3n =	16.738 inches	fdlcomp =	1.875 ksi
Cn =	7.125 inches	fllcomp =	4.224 ksi

Therefore, Dc = 24.947 inches

Article 6.10.1.10 Flange Reduction Factors

6.10.1.10 Hybrid Factor, Rh

All steel in the web, top and bottom flanges are Grade 50W. Therefore Rh = 1.0

6.10.1.10.2 Web Load-Shedding Factor, Rb

The section is composite and is in positive flexure and the web satisfies the requirement of Aritcle 6.10.2.1.1, D / tw <= 150

(**Note**: Ariticle 6.10.2.1.1 has been satisfiied which D / tw = 108 which is less than 150, then Rb = 1.0 and preceeding calculations for Rb need not apply)

Or If the web is longitudinally stiffened

Or If 2(Dc) / tw <= (lamda)rw

then , Rb, shall be taken as 1.0

Otherwise:

Rb = 1 - (awc / (1200 + 300(awc))(2Dc / tw - (lamda)rw) <= 1.00

in which:

(lamda)rw = limiting slenderness ratio for a noncompact web

5.7 (sqrt(E / Fyc)) = 137.27

awc = ratio of two times the web area in compression to the area of the compression flange

awc = 2(Dc)(tw) / (bfc)(tfc) = 1.39

Dc = depth of web in compression in the elastic range (in)

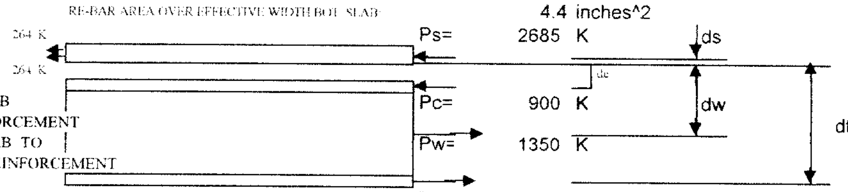
2(Dc) / tw = 99.787 (lamda)rw = 137.27

therfore: 2(Dc) / tw <= (lamda)rw then , Rb, shall be taken as 1.0

Calculation for Plastic Moment:

ASSUME 2 1/2" CLEAR TO TOP REINFORCEMENT
AND 1" CLEAR FOR BOTTOM REINFORCEMENT

RE-BAR AREA OVER EFFECTIVE WIDTH TOP SLAB 4.4 inches^2
RE-BAR AREA OVER EFFECTIVE WIDTH BOT SLAB 4.4 inches^2



Crt=DISTANCE FROM TOP OF SLAB
TO CENTER OF TOP SLAB REINFORCEMENT
Cbt=DISTANCE FROM TOP OF SLAB TO
TO CENTER OF BOTTOM SLAB REINFORCEMENT

Crt= 3.4375 IN
Cbt= 7.0625 IN

CALCULATION OF Y AND Mp FOR POSITIVE BENDING SECTIONS
APPENDIX D (D6.1 TABLE D6.1-1)

CASE NO.	PNA LOCATION	CONDITION	Y AND Mp FORMULAS
1	IN THE WEB	Pt+Pw>Pc+Ps+Prb+Prt	Y=(D/2)(((Pt-Pc-Ps-Prt-Prb)/Pw)+1) Mp=(Pw/2D)(Y^2+(D-Y)^2)+(Ps)(ds)+(Prt)(drt)+(Prb)(drb)+(Pc)(dc)+(Pt)(dt)
2	IN THE TOP FLANGE	Pt+Pw+Pc>Ps+Prb+Prt	Y=(tc/2)(((Pw+Pt-Ps-Prt-Prb)/Pc)+1) Mp=(Pc/2tc)(Y^2+(tc-Y)^2)+(Ps)(ds)+(Prt)(drt)+(Prb)(drb)+(Pw)(dw)+(Pt)(dt)
3	SLAB, BELOW Prb	Pt+Pw+Pc>(Crb/ts)Ps+Prb+Prt	Y=(ts)((Pc+Pw+Pt-Prt-Prb)/Ps) Mp=((Y^2)(Ps)/2ts)+(Prt)(drt)+(Prb)(drb)+(Pw)(dw)+(Pt)(dt)+(Pc)(dc)
4	SLAB, AT Prb	Pt+Pw+Pc+Prb>(Crb/ts)Ps+Prt	Y=Crb Mp=((Y^2)(Ps)/2ts)+(Prt)(drt)+(Pw)(dw)+(Pt)(dt)+(Pc)(dc)
5	SLAB, ABOVE Prb	Pt+Pw+Pc+Prb>(Crt/ts)Ps+Prt	Y=(ts)((Pc+Pw+Pt-Prt-Prb)/Ps) Mp=((Y^2)(Ps)/2ts)+(Prt)(drt)+(Prb)(drb)+(Pw)(dw)+(Pt)(dt)+(Pc)(dc)

DETERMINE THE LOCATION OF PNA:

CASE	Y	ds	d	dw	dt	drt	dbt	Mp	controling case
CASE NO 1. PNA IN THE WEB	0.00	7.00	0.50	27.00	54.63	8.06	4.44	10037	VOID
CASE NO 2.PNA IN THE TOP FLA	**0.09**	**6.09**	**0.41**	**27.91**	**55.54**	**7.15**	**3.53**	**9975**	**controls**
CASE NO 3.PNA IN THE SLAB BEL	9.54	5.04	-1.46	28.96	56.58	6.10	2.48	9774	VOID
CASE NO 4.PNA AT Prb	7.06	2.56	3.94	31.44	59.06	3.63	0.00	10069	VOID
CASE NO 5.PNA IN THE SLAB ABO	11	6.81	0.31	27.19	54.81	7.87	4.25	10078	VOID

FOR CASE NO 1: Y IS MEASURED FROM TOP OF WED TO PNA
FOR CASE NO. 2: Y IS MEASURED FROM TOP OF TOP FLANGE TO PNA
FOR CASE NOS. 3, 4, & 5: Y IS MEASURED FROM TOP OF SLAB TO PNA

PLASTIC MOMENT, Mp= 9975 'K
Y= 0.09 inches, PNA falls in the top flange
therefore Dcp = 0.000 inches, since PNA falls in the top flange

Check Ductiltity Requirement:

A) Art. 6.10.7.3
Dp <= 0.42 (Dt)
Where:
Dp = distance from the top of the concrete slab to the neutral axis of the composite section at the plastic moment (in)
Dp = 10.59 in
Dt= total depth of the composite section (in)
Dt = 66.75 in
0.42(Dt) = 28.035 in
therefore: 10.59 <= 28.04 OK

Art. 6.10.2.1.1 web proportions
Webs w/o Longitudinal Stiffeners
Webs shall be proportioned such that
D / tw < 150
which D = depth of web = 54 in
tw = thickness of web = 0.5 in
D/ tw = 108 < 150 OK

Art.6.10.7.1 Compact Sections

Specified min. yield strength of Flg. & Webs <= 70 ksi
Web: 2(Dcp)/tw <= 3.76(Sqrt(E/Fyc))
PNA is located in the top flange
2(Dcp) / tw = 0.00
3.76(sqrt(E/Fyc) = 90.553
therefore 2(Dcp)/Tw <= 3.76(Sqrt(E/Fyc)) OK
Dcp = depth of the web in compression at the the plastic moment determined as specified in article D6.3.2

Art.6.10.2.2 flange proportions

0.10 <= Iyc / Iyt <= 10
Iyc: Moment of inertia of the compression flange fo the steel section about the vertical axis in the plane of the web (in^4)
Iyc= 486.000 inches^4
Iyt= Moment of inertia of the tension flange of the steel section about the vertical axis in the plane fo the web (in^4)
Iyt= 607.5 inches^4
0.10 <= 0.8 <= 10 OK
bf / 2(tf) <= 12.0 & bf >= D/6 & tf => 1.1(tw)
bf / 2(tf) = 9.0000 <= 12.0 OK
D / 5 = 10.8000 < 18.0000 inches OK , <===== see note below
(1.1)tw = 0.5500 < 1.0000 inches OK

Note: Commentary to Article 6.10.2.2 recommends that bf be taken greater than or equal to D / 5, instead of D / 6, as a more practical limit.
Since this in not necessarily a "hard and fact rule", it was decided to permit some leeway in the Specification for extreme cases with the D / 6 limit, but D / 5 is actually the prefered limit.

Article 6.10.7 Flexural Resistance - Composite Section in Positive Flexure

Compact sections shall satisfy the ductility requirement specified in Art. 6.10.7.3
The specified minimum yield strenghts of the flanges and web do not exceed 70.00 ksi
At the strength limit state, the section shall satisfy:
Mu + (1/3)(fl)(Sxt) < ϕf(Mn)
ϕf = resistance factor for flexure specified in Art. 6.5.4.2 = 1.00
fl = flange lateral bending stress determined as specified in Art. 6.10.1.6
(the compression flange is cont. supported by the slab, so fl = 0.0 ksi)
(The lateral bending stress from wind and from the pouring of the concrete deck induced on the tension flange is very small and therefore ignored)
Mn = nominal flexural resistance of the section determined as specified in Article 6.10.7.1.2 (kip-in)
Mu = vertical-bending moment determined as specified in Article 6.10.1.6 (Kip-in)
Myt = yeild moment with respect to the tension flange determined as specified in Article D6.2 (kip-in)
Myt = 83071 kip-in
Sxt = elastic section modulus about the major axis of the section to the tension flange taken as Myt / Fyt. (in^3), which Sxt = 1661.4 in^3
Which fl = 0.0 ksi due the compression flange being fully supported and ϕf = 1.0
therefore Mu < Mn

Calculation for Mn, Nominal Flexural Resistance:

The nominal flexural resistance of the section shall be taken as:
If Dp <= 0.1(Dt), then Mn = Mp
Otherwise, Mn = Mp(1.07 - 0.7(Dp/Dt))
Dp = distance from the top of the concrete slab to the neutral axis of the composite section at the plastic moment. (in)
Dp = 10.59 in
Dt = total depth of the composite section (in) = 66.8 inches
0.1(Dt) = 6.675 in
therefore : 10.59 > 6.675 therefore Mn = Mp(1.07-0.7(Dp/Dt))
Mp = 9975.3 foot-kip
which Mn = Mp(1.07-0.70(Dp/Dt)) = 9565.77 foot-kip

In a continous span, the nominal flexural resistance of the section shall not exceed:
Mn = (1.3)(Rh)(My) Rh = 1.0 for non-hybrid section
which Mn = 8999.4 kip foot <== controls

actual Strength 1 Moment = 7882.0 kip foot is less than Mn = 8999.4 kip foot OK

SECTION PROPERTIES FOR WELDED PLATE GIRDERS
Old Hickory Blvd. (SR-45) over I-65
Davidson County
Prepared by WHP
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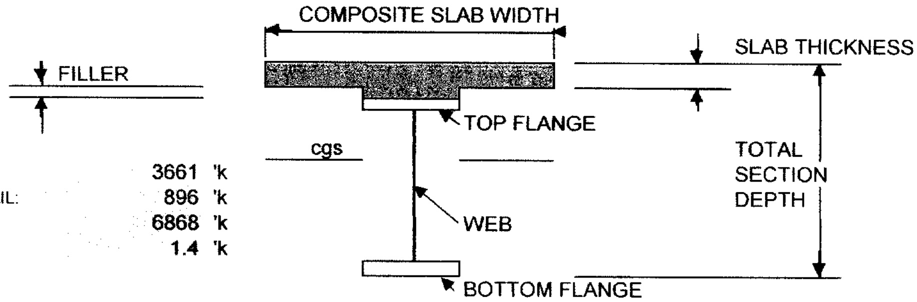


EXTERIOR BEAM
STRENGTH I LOADS

MOMENT DUE DL SLAB & BEAM:	3661	'k
MOMENT DUE DL WEARING AND RAIL:	896	'k
MOMENT DUE LIVE LOAD:	6868	'k
MOMENTS TAKEN AT SPAN POINT:	1.4	'k

REQUIRED INPUT: (inches)

COMPOSITE SLAB WIDTH:	117.00
SLAB THICKNESS:	9.00
FILLER:	1.50
TOP FLANGE WIDTH:	18.00
TOP FLANGE THICKNESS:	1.00
WEB DEPTH:	54.00
WEB THICKNESS:	0.50
BOTTOM FLANGE WIDTH:	18.00
BOTTOM FLG THICKNESS:	2.25
TOTAL DEPTH OF SECTION	67.75

NON-COMPOSITE PROPERTIES

(referance line in at bottom of bottom flange)

	Area	Y x Area	Y^2 x Area	Io	I ref
SLAB	0.00	0.0	0.0	0.0	0.0
TOP FLANGE	18.00	1021.5	57970.1	1.5	57971.6
WEB	27.00	789.8	23100.2	6561.0	29661.2
BOTTOM FLANGE	40.50	45.6	51.3	17.1	68.3
TOTALS	85.50	1856.8			87701.2

Ycgs= 21.72 in
I @ CGS= 47376.56 in^4

S slab= 0.0 in^3
S tf= 1333.3 in^3
S bf= 2181.5 in^3

COMPOSITE PROPERIES N=27

(reference line at the cgs of girder)

	AREA	Y X AREA	Y^2 x area	Io	I ref
SLAB	39	1619.78	67274.3	263.3	67537.5
GIRDER	85.5	0.00	0.00	47376.6	47376.6
TOTALS	124.5	1619.78			114914.1

Ycgs= 13.01 in
I @ cgs= 93840.22 in^4

S slab= 2841.7 in^3
S tf= 4166.5 in^3
S bf= 2702.2 in^3

COMPOSITE PROPERIES N=9

(reference line at the cgs of girder)

	AREA	Y X AREA	Y^2 x Area	Io	I ref
SLAB	117.0	4859.3	201822.8	789.8	202612.6
GIRDER	85.5	0.0	0	47376.6	47376.6
TOTALS	202.5	4859.3			249989.1

Ycgs= 24.00 in
I @ cgs= 133380.39 in^4

S slab= 6052.8 in^3
S tf= 11562.0 in^3
S bf= 2917.7 in^3

YIELD MOMENT CALULATIONS

Yield Moment at span pt 1.400
Applied Moment: (Strenght 1)
DLnc: 3661 'K
DLcomp 896 'K
LLcomp: 6868 'K
Total Actual Moment: 11425 'K

ADcomp IS THE ADDED MOMENT APPLIED
TO THE COMPOSITE SECTION SO THAT
THE STEEL WILL REACH ITS YIELD MOMENT.
Fy = YEILD STRESS = 50 KSI

ACTUAL STRESS:
Fslab= 17.40 KSI
Ftf= 42.66 KSI
Fbf= 52.36 KSI

Fy = DLnc / Snc + DLcomp / Scomp n=27 + ADcomp / Scomp N=9
SLOVE EQUATION FOR ADcomp
Top Flange: ADcomp = 13942 'K
Bottom Flange: ADcomp = 6293 'K CONTROLS

Modulus of Elasticity:
Concrete, Ec = 33,000((wc)^1.5)(f'c)^.5
wc = 0.150 kcf and f'c = 3 ksi
Ec = 3320.56 ksi
Steel, Es = 29,000 ksi

My = YEILD MOMENT = DLnc + DLcomp + ADcomp
My = 10850 'k > 11425 'K plastic section

If Moment is elastic, Dc = ((fdlnc + fdlcomp + fllcomp) / (fdlnc/Csteel + fdlcomp/C3n +fllcomp/Cn)) - tf

Csteel =	35.53 inches	fdlnc =	32.95 ksi
C3n =	22.52 inches	fdlcomp =	2.58 ksi
Cn =	11.54 inches	fllcomp =	7.13 ksi

Therefore, Dc = 30.40 inches

Article 6.10.1.10 Flange Reduction Factors

6.10.1.10 Hybrid Factor, Rh

All steel in the web, top and bottom flanges are Grade 50W. Therefore Rh = 1.0

6.10.1.10.2 Web Load-Shedding Factor, Rb

The section is composite and is in positive flexure and the web satisfies the requirement of Aritcle 6.10.2.1.1, D / tw <= 150

(**Note**: Ariticle 6.10.2.1.1 has been satisfiied which D / tw = 108 which is less than 150, then Rb = 1.0 and preceeding calc. for Rb need not apply)

Or If the web is longitudinally stiffened

Or If 2(Dc) / tw <= (lamda)rw

then , Rb, shall be taken as 1.0

Otherwise:

Rb = 1 - (awc / (1200 + 300(awc))(2Dc / tw - (lamda)rw) <= 1.00

in which:

(lamda)rw = limiting slenderness ratio for a noncompact web

5.7 (sqrt(E / Fyc)) = 137.27

awc = ratio of two times the web area in compression to the area of the compression flange

awc = 2(Dc)(tw) / (bfc)(tfc) = 1.69

Dc = depth of web in compression in the elastic range (in)

2(Dc) / tw = 121.617 (lamda)rw = 137.27

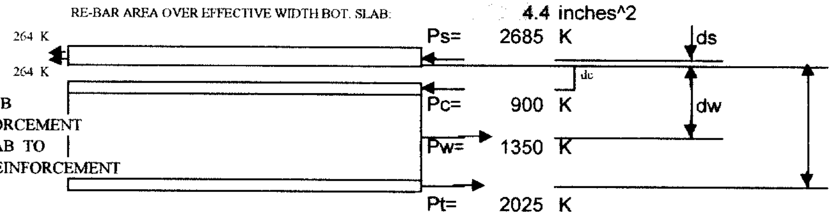
therfore: 2(Dc) / tw <= (lamda)rw then , Rb, shall be taken as 1.0

Calculation for Plastic Moment:

ASSUME 2 1/2" CLEAR TO TOP REINFORCEMENT
AND 1" CLEAR FOR BOTTOM REINFORCEMENT

RE-BAR AREA OVER EFFECTIVE WIDTH TOP SLAB 4.4 inches^2

RE-BAR AREA OVER EFFECTIVE WIDTH BOT. SLAB: 4.4 inches^2



Crt=DISTANCE FROM TOP OF SLAB
TO CENTER OF TOP SLAB REINFORCEMENT
Cbt=DISTANCE FROM TOP OF SLAB TO
TO CENTER OF BOTTOM SLAB REINFORCEMENT

Crt= 3.4375 IN
Cbt= 7.0625 IN

CALCULATION OF Y AND Mp FOR POSITIVE BENDING SECTIONS

APPENDIX D (D6.1 TABLE D6.1-1)

CASE NO.	PNA LOCATION	CONDITION	Y AND Mp FORMULAS
1	IN THE WEB	Pt+Pw>Pc+Ps+Prb+Prt	Y=(D/2)(((Pt-Pc-Ps-Prt-Prb)/Pw)+1) Mp=(Pw/2D)(Y^2+(D-Y)^2)+(Ps)(ds)+(Prt)(drt)+(Prb)(drb)+(Pc)(dc)+(Pt)(dt)
2	IN THE TOP FLANGE	Pt+Pw+Pc>Ps+Prb+Prt	Y=(tc/2)(((Pw+Pt-Ps-Prt-Prb)/Pc)+1) Mp=(Pc/2tc)(Y^2+(tc-Y)^2)+(Ps)(ds)+(Prt)(drt)+(Prb)(drb)+(Pw)(dw)+(Pt)(dt)
3	SLAB, BELOW Prb	Pt+Pw+Pc>(Crb/ts)Ps+Prb+Prt	Y=(ts)((Pc+Pw+Pt-Prt-Prb)/Ps) Mp=((Y^2)(Ps)/2ts)+(Prt)(drt)+(Prb)(drb)+(Pw)(dw)+(Pt)(dt)+(Pc)(dc)
4	SLAB, AT Prb	Pt+Pw+Pc+Prb>(Crb/ts)Ps+Prt	Y=Crb Mp=((Y^2)(Ps)/2ts)+(Prt)(drt)+(Pw)(dw)+(Pt)(dt)+(Pc)(dc)
5	SLAB, ABOVE Prb	Pt+Pw+Pc+Prb>(Crt/ts)Ps+Prt	Y=(ts)((Pc+Pw+Pt-Prt-Prb)/Ps) Mp=((Y^2)(Ps)/2ts)+(Prt)(drt)+(Prb)(drb)+(Pw)(dw)+(Pt)(dt)+(Pc)(dc)

DETERMINE THE LOCATION OF PNA:

CASE	Y	ds	d	dw	dt	drt	dbt	Mp	controling case
CASE NO 1, PNA IN THE WEB	0.00	7.00	0.50	27.00	55.13	8.06	4.44	14219	VOID
CASE NO 2,PNA IN THE TOP FLA	**0.59**	**6.59**	**-0.09**	**27.41**	**55.54**	**7.65**	**4.03**	**14206**	**controls**
CASE NO 3,PNA IN THE SLAB BEL(	12.56	8.06	1.56	25.94	54.07	9.12	5.50	14441	VOID
CASE NO 4,PNA AT Prb	7.06	2.56	3.94	31.44	59.56	3.63	0.00	14583	VOID
CASE NO 5,PNA IN THE SLAB ABO	14	9.83	3.33	24.17	52.30	10.89	7.27	14746	VOID

FOR CASE NO 1: Y IS MEASURED FROM TOP OF WED TO PNA
FOR CASE NO. 2: Y IS MEASURED FROM TOP OF TOP FLANGE TO PNA
FOR CASE NOS. 3, 4, & 5: Y IS MEASURED FROM TOP OF SLAB TO PNA

PLASTIC MOMENT, Mp= **14206** ' K

Y= **0.59** inches, PNA falls in the top flange

therefore Dcp = 0.000 inches, since PNA falls in the top flange

Check Ductiltity Requirement:

A) Art. 6.10.7.3
Dp <= 0.42 (Dt)
Where:
Dp = distance from the top of the concrete slab to the neutral axis of the composite section at the plastic moment (in)
Dp = 11.09 in
Dt= total depth of the composite section (in)
Dt = 67.75 in
0.42(Dt) = 28.455 in
therefore: 11.09 <= 28.46 OK

Art. 6.10.2.1.1 web proportions
Webs w/o Longitudinal Stiffeners
Webs shall be proportioned such that
D / tw < 150
which D = depth of web = 54 in
tw = thickness of web = 0.5 in
D/ tw = 108 < 150 OK

Art.6.10.7.1 Compact Sections

Specified min. yield strength of Flg. & Webs <= 70 ksi
Web: 2(Dcp)/tw <= 3.76(Sqrt(E/Fyc))
PNA is located in the top flange, Dcp = 0.0
2(Dcp) / tw = 0
3.76(sqrt(E/Fyc) = 90.553
therefore 2(Dcp)/Tw <= 3.76(Sqrt(E/Fyc)) OK
Dcp = depth of the web in compression at the plastic moment determined as specified in Article D6.3.2

Art.6.10.2.2 flange proportions

0.10 <= Iyc / Iyt <= 10
Iyc Moment of inertia of the compression flange fo the steel section about the vertical axis in the plane of the web (in^4)
Iyc= 486.0 inches^4
Iyt= Moment of inertia of the tension flange of the steel section about the vertical axis in the plane fo the web (in^4)
Iyt= 1093.5 inches^4
0.10 <= 0.444444 <= 10 OK
bf / 2(tf) <= 12.0 & bf >= D/6 & tf => 1.1(tw)
bf / 2(tf) = 9.0000 <= 12.0 OK
D / 5 = 10.8000 < 18.0000 inches OK <==== see note below
(1.1)tw = 0.5500 < 1.0000 inches OK

Note: Commentary to Article 6.10.2.2 recommends that bf be taken greater than or equal to D / 5, instead of D / 6, as a more practical limit.
Since this in not necessarily a "hard and fact rule", it was decided to permit some leeway in the Specification for extreme cases with the D / 6 limit, but D /5 is actually the prefered limit.

Article 6.10.7 Flexural Resistance - Composite Section in Positive Flexure

Compact sections shall satisfy the ductility requirement specified in Art. 6.10.7.3
The specified minimum yield strenghts of the flanges and web do not exceed 70.00 ksi
At the strength limit state, the section shall satisfy:
Mu + (1/3)(fl)(Sxt) < Øf(Mn)
Øf = resistance factor for flexure specified in Art. 6.5.4.2 = 1.00
fl = flange lateral bending stress determined as specified in Art. 6.10.1.6
(the compression flange is cont. supported by the slab, so fl = 0.0 ksi)
(The lateral bending stress from wind and from the pouring of the concrete deck induced on the tension flange is very small and therefore ignor
Mn = nominal flexural resistance of the section determined as specified in Article 6.10.7.1.2 (kip-in)
Mu = vertical-bending moment determined as specified in Article 6.10.1.6 (Kip-in)
Myt = yeild moment with respect to the tension flange determined as specified in Article D6.2 (kip-in)
Myt = 130203 kip-in
Sxt = elastic section modulus about the major axis of the section to the tension flange taken as Myt / Fyt. (in^3), which Sxt = 2604.1 in^3
Which fl = 0.0 ksi due the compression flange being fully supported and Øf = 1.0
therefore Mu < Mn

Calculation for Mn, Nominal Flexural Resistance:

The nominal flexural resistance of the section shall be taken as:
If Dp <= 0.1(Dt), then Mn = Mp
Otherwise, Mn = Mp(1.07 - 0.7(Dp/Dt))
Dp = distance from the top of the concrete slab to the neutral axis of the composite section at the plastic moment. (in)
Dp = 11.09 in
Dt = total depth of the composite section (in) = 67.8 inches
0.1(Dt) = 6.775 in
therefore : 11.09 > 6.775 therefore, Mn = Mp(1.07-0.7(Dp/Dt))
Mp = 14206 foot-kip
which Mn = Mp(1.07-0.70(Dp/Dt)) = 13572.73 kip foot <== controls

In a continous span, the nominal flexural resistance of the section shall not exceed:
Mn = (1.3)(Rh)(My) Rh = 1.0 for non-hybrid section
which Mn = 14105.3 kip foot

actual Strength 1 Moment = 11425.0 kip foot is less than Mn = 13572.7 kip foot OK

SECTION PROPERTIES FOR WELDED PLATE GIRDERS
Old Hickory Blvd. (SR-45) over I-65
Davidson County
Prepared by WHP
Date: July 30, 2003
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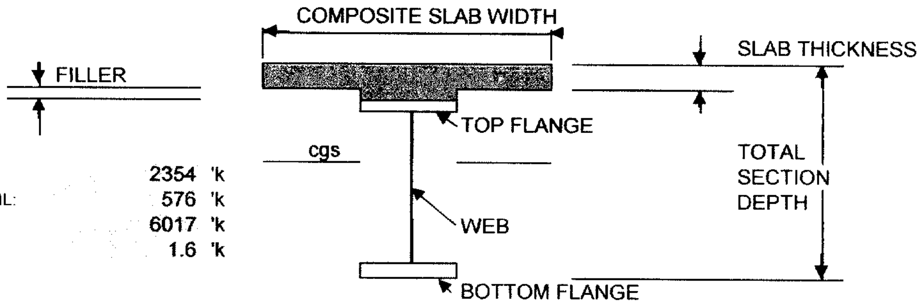

EXTERIOR BEAM
STRENGTH I LOADS

MOMENT DUE DL SLAB & BEAM:	2354 'k
MOMENT DUE DL WEARING AND RAIL:	576 'k
MOMENT DUE LIVE LOAD:	6017 'k
MOMENTS TAKEN AT SPAN POINT:	1.6 'k

REQUIRED INPUT: (inches)

COMPOSITE SLAB WIDTH:	117.00
SLAB THICKNESS:	9.00
FILLER:	1.50
TOP FLANGE WIDTH:	18.00
TOP FLANGE THICKNESS:	1.00
WEB DEPTH:	54.00
WEB THICKNESS:	0.50
BOTTOM FLANGE WIDTH:	18.00
BOTTOM FLG THICKNESS:	1.25
TOTAL DEPTH OF SECTION	66.75

NON-COMPOSITE PROPERTIES

(referance line in at bottom of bottom flange)

	Area	Y x Area	Y^2 x Area	Io	I ref
SLAB	0.00	0.0	0.0	0.0	0.0
TOP FLANGE	18.00	1003.5	55945.1	1.5	55946.6
WEB	27.00	762.8	21547.7	6561.0	28108.7
BOTTOM FLANGE	22.50	14.1	8.8	2.9	11.7
TOTALS	67.50	1780.3			84067.0

Ycgs= 26.38 in
I @ CGS= 37111.29 in^4

S slab= 0.0 in^3
S tf= 1242.2 in^3
S bf= 1407.1 in^3

note:
Termination point of 18"x 2¼" was moved from span pt 1.55 to span pt. 1.6 in order to meet allowable stresses.
59' plate was lengthen to 67.5'

COMPOSITE PROPERIES N=27

(reference line at the cgs of girder)

	AREA	Y X AREA	Y^2 x area	Io	I ref
SLAB	39	1399.1	50193.6	263.3	50456.9
GIRDER	67.5	0.0	0.0	37111.3	37111.3
TOTALS	106.5	1399.1			87568.1

Ycgs= 13.14 in
I @ cgs= 69187.39 in^4

S slab= 2540.1 in^3
S tf= 4133.6 in^3
S bf= 1751.0 in^3

COMPOSITE PROPERIES N=9

(reference line at the cgs of girder)

	AREA	Y X AREA	Y^2 x Area	Io	I ref
SLAB	117.0	4197.4	150580.8	789.8	151370.6
GIRDER	67.5	0.0	0.0	37111.3	37111.3
TOTALS	184.5	4197.4			188481.9

Ycgs= 22.75 in
I @ cgs= 92991.59 in^4

S slab= 5276.1 in^3
S tf= 13051.5 in^3
S bf= 1893.0 in^3

YIELD MOMENT CALULATIONS

Yield Moment at span pt 1.600
Applied Moment: (Strenght 1)
DLnc: 2354 'K
DLcomp 576 'K
LLcomp: 6017 'K
Total Actual Moment: 8947 'K

ADcomp IS THE ADDED MOMENT APPLIED TO THE COMPOSITE SECTION SO THAT THE STEEL WILL REACH ITS YIELD MOMENT.
Fy = YEILD STRESS = 50 KSI

ACTUAL STRESS:
Fslab= 16.41 KSI
Ftf= 29.94 KSI
Fbf= 62.17 KSI

Fy = DLnc / Snc + DLcomp / Scomp n=27 + ADcomp / Scomp N=9
SLOVE EQUATION FOR ADcomp
Top Flange: ADcomp = 27830 'K
Bottom Flange: ADcomp = 4098 'K CONTROLS

Modulus of Elasticity:
Concrete, Ec = 33,000((wc)^1.5)(f'c)^.5
wc = 0.150 kcf and f'c = 3 ksi
Ec = 3320.56 ksi
Steel, Es = 29,000 ksi

My = YEILD MOMENT = DLnc + DLcomp + ADcomp
My = 7028 'k > 8947 'K plastic section

If Moment is elastic, Dc = ((fdlnc + fdlcomp + fllcomp) / (fdlnc/Csteel + fdlcomp/C3n +fllcomp/Cn)) - tf

Csteel =	29.88 inches	fdlnc =	22.74 ksi
C3n =	16.74 inches	fdlcomp =	1.67 ksi
Cn =	7.13 inches	fllcomp =	5.53 ksi

Therefore, Dc = 24.13 inches

PLASTIC MOMENT CALCULATIONS
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Davidson County
Prepared by WHP
Date: September 15, 2003
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Article 6.10.1.10 Flange Reduction Factors

6.10.1.10 Hybrid Factor, Rh

All steel in the web, top and bottom flanges are Grade 50W. Therefore Rh = 1.0

6.10.1.10.2 Web Load-Shedding Factor, Rb

The section is composite and is in positive flexure and the web satisfies the requirement of Aritcle 6.10.2.1.1, D / tw < = 150

(Note: Ariticle 6.10.2.1.1 has been satisfiied which D / tw = 108 which is less than 150, then Rb = 1.0 and preceeding calculations for Rb need not apply)

Or If the web is longitudinally stiffened

Or If 2(Dc) / tw < = (lamda)rw

then , Rb, shall be taken as 1.0

Otherwise:

Rb = 1 - (awc / (1200 + 300(awc))(2Dc / tw - (lamda)rw) < = 1.00

in which:

(lamda)rw = limiting slenderness ratio for a noncompact web

5.7 (sqrt(E / Fyc)) = 137.27

awc = ratio of two times the web area in compression to the area of the compression flange

awc = 2(Dc)(tw) / (bfc)(tfc) = 1.34

Dc = depth of web in compression in the elastic range (in)

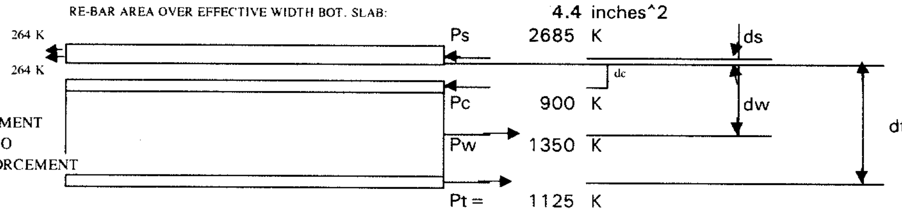
2(Dc) / tw = 96.518 (lamda)rw = 137.27

therfore: 2(Dc) / tw < = (lamda)rw then , Rb, shall be taken as 1.0

Calculation for Plastic Moment:

ASSUME 2 1/2" CLEAR TO TOP REINFORCEMENT
AND 1" CLEAR FOR BOTTOM REINFORCEMENT

RE-BAR AREA OVER EFFECTIVE WIDTH TOP SLAB: 4.4 inches^2
RE-BAR AREA OVER EFFECTIVE WIDTH BOT. SLAB: 4.4 inches^2



Crt=DISTANCE FROM TOP OF SLAB
TO CENTER OF TOP SLAB REINFORCEMENT
Cbt=DISTANCE FROM TOP OF SLAB TO
TO CENTER OF BOTTOM SLAB REINFORCEMENT

Crt = 3.4375 IN
Cbt = 7.0625 IN

CALCULATION OF Y AND Mp FOR POSITIVE BENDING SECTIONS
APPENDIX D (D6.1 TABLE D6.1-1)

CASE NO.	PNA LOCATION	CONDITION	Y AND Mp FORMULAS
1	IN THE WEB	Pt+Pw>Pc+Ps+Prb+Prt	Y=(D/2)(((Pt-Pc-Ps-Prt-Prb)/Pw)+1) Mp=(Pw/2D)(Y^2+(D-Y)^2)+(Ps)(ds)+(Prt)(drt)+(Prb)(drb)+(Pc)(dc)+(Pt)(dt)
2	IN THE TOP FLANGE	Pt+Pw+Pc>Ps+Prb+Prt	Y=(tc/2)(((Pw+Pt-Ps-Prt-Prb)/Pc)+1) Mp=(Pc/2tc)(Y^2+(tc-Y)^2)+(Ps)(ds)+(Prt)(drt)+(Prb)(drb)+(Pw)(dw)+(Pt)(dt)
3	SLAB, BELOW Prb	Pt+Pw+Pc>(Crb/ts)Ps+Prb+Prt	Y=(ts)((Pc+Pw+Pt-Prt-Prb)/Ps) Mp=((Y^2)(Ps)/2ts)+(Prt)(drt)+(Prb)(drb)+(Pw)(dw)+(Pt)(dt)+(Pc)(dc)
4	SLAB, AT Prb	Pt+Pw+Pc+Prb>(Crb/ts)Ps+Prt	Y=Crb Mp=((Y^2)(Ps)/2ts)+(Prt)(drt)+(Pw)(dw)+(Pt)(dt)+(Pc)(dc)
5	SLAB, ABOVE Prb	Pt+Pw+Pc+Prb>(Crt/ts)Ps+Prt	Y=(ts)((Pc+Pw+Pt-Prt-Prb)/Ps) Mp=((Y^2)(Ps)/2ts)+(Prt)(drt)+(Prb)(drb)+(Pw)(dw)+(Pt)(dt)+(Pc)(dc)

DETERMINE THE LOCATION OF PNA:

CASE	Y	ds		dw	dt	drt	dbt	Mp	controling case
CASE NO 1, PNA IN THE WEB	0.00	7.00	0.50	27.00	54.63	8.06	4.44	10037	VOID
CASE NO 2,PNA IN THE TOP FLAN	**0.09**	**6.09**	**0.41**	**27.91**	**55.54**	**7.15**	**3.53**	**9975**	**controls**
CASE NO 3,PNA IN THE SLAB BELO	9.54	5.04	-1.46	28.96	56.58	6.10	2.48	9774	VOID
CASE NO 4,PNA AT Prb	7.06	2.56	3.94	31.44	59.06	3.63	0.00	10069	VOID
CASE NO 5,PNA IN THE SLAB ABOV	11	6.81	0.31	27.19	54.81	7.87	4.25	10078	VOID

FOR CASE NO 1: Y IS MEASURED FROM TOP OF WED TO PNA
FOR CASE NO. 2: Y IS MEASURED FROM TOP OF TOP FLANGE TO PNA
FOR CASE NOS. 3, 4, & 5: Y IS MEASURED FROM TOP OF SLAB TO PNA

PLASTIC MOMENT, Mp = 9975 'K
Y = 0.09 inches, PNA falls in the top flange
therefore Dcp = 0.000 inches, since PNA falls in the top flange

Check Ductiltity Requirement:

A) Art. 6.10.7.3
Dp <= 0.42 (Dt)
Where:
Dp = distance from the top of the concrete slab to the neutral axis of the composite section at the plastic moment (in)
Dp = 10.59 in
Dt= total depth of the composite section (in)
Dt = 66.75 in
0.42(Dt) = 28.035 in
therefore: 10.59 <= 28.04 OK

Art. 6.10.2.1.1 web proportions
Webs w/o Longitudinal Stiffeners
Webs shall be proportioned such that
D / tw < 150
which D = depth of web = 54 in
tw = thickness of web = 0.5 in
D/ tw = 108 < 150 OK

Art.6.10.7.1 Compact Sections

Specified min. yield strength of Flg. & Webs <= 70 ksi
Web: 2(Dcp)/tw <= 3.76(Sqrt(E/Fyc))
PNA is located in the top flange, Dcp = 0.0
2(Dcp) / tw = 0
3.76(sqrt(E/Fyc) = 90.553
therefore 2(Dcp)/Tw <= 3.76(Sqrt(E/Fyc)) OK
Dcp = depth of the web in compression at the plastic moment detemined as specified in Article D6.3.2

Art.6.10.2.2 flange proportions

0.10 <= Iyc / Iyt <= 10
Iyc Moment of inertia of the compression flange fo the steel section about the vertical axis in the plane of the web (in^4)
Iyc= 486.0 inches^4
Iyt= Moment of inertia of the tension flange of the steel section about the vertical axis in the plane fo the web (in^4)
Iyt= 607.5 inches^4
0.10 <= 0.800 <= 10 OK
bf / 2(tf) <= 12.0 & bf >= D/6 & tf => 1.1(tw)
bf / 2(tf) = 9.0000 <= 12.0 OK
D / 5 = 10.8000 < 18.0000 inches OK <===== see special note
(1.1)tw = 0.5500 < 1.0000 inches OK

Note: Commentary to Article 6.10.2.2 recommends that bf be taken greater than or equal to D / 5, instead of D / 6, as a more practical limit.
Since this in not necessarily a "hard and fact rule", it was decided to permit some leeway in the Specification for extreme cases with the D / 6 limit, but D / 5 is actually the prefered limit.

Article 6.10.7 Flexural Resistance - Composite Section in Positive Flexure

Compact sections shall satisfy the ductility requirement specified in Art. 6.10.7.3
The specified minimum yield strenghts of the flanges and web do not exceed 70.00 ksi
At the strength limit state, the section shall satisfy:
Mu + (1/3)(fl)(Sxt) < Øf(Mn)
Øf = resistance factor for flexure specified in Art. 6.5.4.2 = 1.00
fl = flange lateral bending stress determined as specified in Art. 6.10.1.6
(the compression flange is cont. supported by the slab, so fl = 0.0 ksi)
(the lateral bending stress from wind and from the pouring of the concrete deck induced on the tension flange is very small and therefore ignored)
Mn = nominal flexural resistance of the section determined as specified in Article 6.10.7.1.2 (kip-in)
Mu = vertical-bending moment determined as specified in Article 6.10.1.6 (Kip-in)
Myt = yeild moment with respect to the tension flange determined as specified in Article D6.2 (kip-in)
Myt = 84333 kip-in
Sxt = elastic section modulus about the major axis of the section to the tension flange taken as Myt / Fyt. (in^3), which Sxt = 1686.7 in^3
Which fl = 0.0 ksi due the compression flange being fully supported and Øf = 1.0
therefore Mu < Mn

Calculation for Mn, Nominal Flexural Resistance:

The nominal flexural resistance of the section shall be taken as:
If Dp <= 0.1(Dt), then Mn = Mp
Otherwise, Mn = Mp(1.07 - 0.7(Dp/Dt))
Dp = distance from the top of the concrete slab to the neutral axis of the composite section at the plastic moment. (in)
Dp = 10.59 in
Dt = total depth of the composite section (in) = 66.8 inches
0.1(Dt) = 6.675 in
therefore : 10.59 > 6.675 therefore, Mn = Mp(1.07-0.7(Dp/Dt))
Mp = 9975.3 foot-kip
which Mn = Mp(1.07-0.70(Dp/Dt)) = 9565.77 foot-kip

In a continous span, the nominal flexural resistance of the section shall not exceed:
Mn = (1.3)(Rh)(My) Rh = 1.0 for non-hybrid section
which Mn = 9136.1 kip foot <== controls

actual Strength 1 Moment = 8947.0 kip foot is less than Mn = 9136.1 kip foot OK

SECTION PROPERTIES FOR WELDED PLATE GIRDERS
Old Hickory Blvd. (SR-45) over I-65
Davidson County
Prepared by WHP
Date: July 30, 2003
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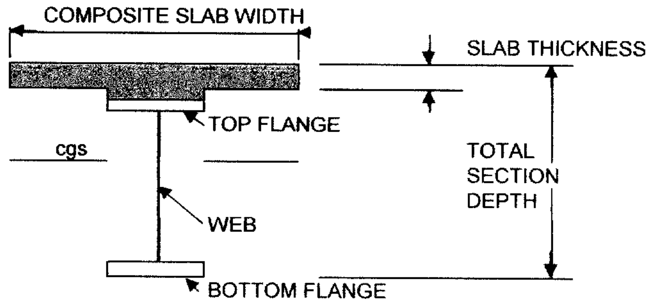


EXTERIOR BEAM
STRENGTH I LOADS

MOMENT DUE DL SLAB & BEAM:	915	'k
MOMENT DUE DL WEARING AND RAIL:	224	'k
MOMENT DUE LIVE LOAD:	4668	'k
MOMENTS TAKEN AT SPAN POINT:	1.7	'k

REQUIRED INPUT: (inches)

COMPOSITE SLAB WIDTH:	117.00
SLAB THICKNESS:	9.00
FILLER:	1.50
TOP FLANGE WIDTH:	18.00
TOP FLANGE THICKNESS:	1.00
WEB DEPTH:	54.00
WEB THICKNESS:	0.50
BOTTOM FLANGE WIDTH:	18.00
BOTTOM FLG THICKNESS:	1.25
TOTAL DEPTH OF SECTION	66.75

NON-COMPOSITE PROPERTIES

(referance line in at bottom of bottom flange)

	Area	Y x Area	Y^2 x Area	Io	I ref
SLAB	0.00	0.0	0.0	0.0	0.0
TOP FLANGE	18.00	1003.5	55945.1	1.5	55946.6
WEB	27.00	762.8	21547.7	6561.0	28108.7
BOTTOM FLANGE	22.50	14.1	8.8	2.9	11.7
TOTALS	67.50	1780.3			84067.0

Ycgs= 26.38 in
I @ CGS= 37111.29 in^4

S slab= 0.0 in^3
S tf= 1242.2 in^3
S bf= 1407.1 in^3

COMPOSITE PROPERIES N=27

(reference line at the cgs of girder)

	AREA	Y X AREA	Y^2 x area	Io	I ref
SLAB	39	1399.1	50193.6	263.3	50456.9
GIRDER	67.5	0.0	0.0	37111.3	37111.3
TOTALS	106.5	1399.1			87568.1

Ycgs= 13.14 in
I @ cgs= 69187.39 in^4

S slab= 2540.1 in^3
S tf= 4133.6 in^3
S bf= 1751.0 in^3

COMPOSITE PROPERIES N=9

(reference line at the cgs of girder)

	AREA	Y X AREA	Y^2 x Area	Io	I ref
SLAB	117.0	4197.4	150580.8	789.8	151370.6
GIRDER	67.5	0.0	0.0	37111.3	37111.3
TOTALS	184.5	4197.4			188481.9

Ycgs= 22.75 in
I @ cgs= 92991.59 in^4

S slab= 5276.1 in^3
S tf= 13051.5 in^3
S bf= 1893.0 in^3

YIELD MOMENT CALULATIONS

Yield Moment at span pt 1.700
Applied Moment: (Strenght 1)
DLnc: 915 'K
DLcomp 224 'K
LLcomp: 4668 'K
Total Actual Moment: 5807 'K

ADcomp IS THE ADDED MOMENT APPLIED
TO THE COMPOSITE SECTION SO THAT
THE STEEL WILL REACH ITS YIELD MOMENT.
Fy = YEILD STRESS = 50 KSI

ACTUAL STRESS:
Fslab= 11.68 KSI
Ftf= 13.78 KSI
Fbf= 38.93 KSI

Fy = DLnc / Snc + DLcomp / Scomp n=27 + ADcomp / Scomp N=9
SLOVE EQUATION FOR ADcomp
Top Flange: ADcomp = 44060 'K
Bottom Flange: ADcomp = 6414 'K CONTROLS

Modulus of Elasticity:
Concrete, Ec = 33,000((wc)^1.5)(f'c)^.5
wc = 0.150 kcf and f'c = 3 ksi
Ec = 3320.56 ksi
Steel, Es = 29,000 ksi

My = YEILD MOMENT = DLnc + DLcomp + ADcomp
My = 7553 'k > 5807 'K elastic section

If Moment is elastic, Dc = ((fdlnc + fdlcomp + fllcomp) / (fdlnc/Csteel + fdlcomp/C3n +fllcomp/Cn)) - tf

Csteel =	29.9 inches	fdlnc =	8.8 ksi
C3n =	16.7 inches	fdlcomp =	0.7 ksi
Cn =	7.1 inches	fllcomp =	4.3 ksi

Therefore, Dc = 22.3 inches

Article 6.10.1.10 Flange Reduction Factors

6.10.1.10 Hybrid Factor, Rh

All steel in the web, top and bottom flanges are Grade 50W. Therefore Rh = 1.0

6.10.1.10.2 Web Load-Shedding Factor, Rb

The section is composite and is in positive flexure and the web satisfies the requirement of Aritcle 6.10.2.1.1, D / tw <= 150

(Note: Ariticle 6.10.2.1.1 has been satisfiied which D / tw : 108 which is less than 150, then Rb = 1.0 and preceeding calculations for Rb need not apply)

Or If the web is longitudinally stiffened

Or If 2(Dc) / tw <= (lamda)rw

then , Rb, shall be taken as 1.0

Otherwise:

Rb = 1 - (awc / (1200 + 300(awc))(2Dc / tw - (lamda)rw) <= 1.00

in which:

(lamda)rw = limiting slenderness ratio for a noncompact web

5.7 (sqrt(E / Fyc)) = 137.27

awc = ratio of two times the web area in compression to the area of the compression flange

awc = 2(Dc)(tw) / (bfc)(tfc) = 1.34

Dc = depth of web in compression in the elastic range (in)

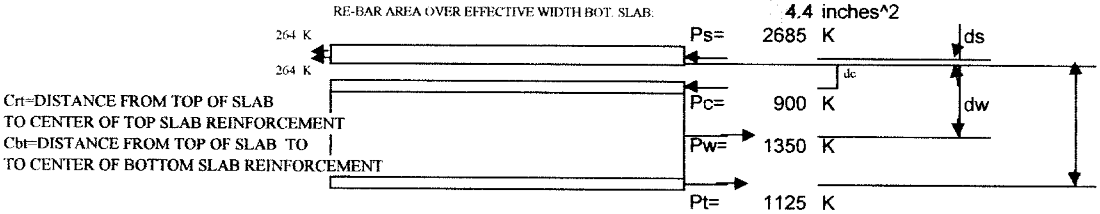
2(Dc) / tw = 96.518 (lamda)rw = 137.27

therfore: 2(Dc) / tw <= (lamda)rw then , Rb, shall be taken as 1.0

Calculation for Plastic Moment:

ASSUME 2 1/2" CLEAR TO TOP REINFORCEMENT
AND 1" CLEAR FOR BOTTOM REINFORCEMENT

RE-BAR AREA OVER EFFECTIVE WIDTH TOP SLAB: 4.4 inches^2
RE-BAR AREA OVER EFFECTIVE WIDTH BOT. SLAB: 4.4 inches^2



Crt= 3.4375 IN
Cbt= 7.0625 IN

CALCULATION OF Y AND Mp FOR POSITIVE BENDING SECTIONS

APPENDIX D (D6.1 TABLE D6.1-1)

CASE NO.	PNA LOCATION	CONDITION	Y AND Mp FORMULAS
1	IN THE WEB	Pt+Pw>Pc+Ps+Prb+Prt	Y=(D/2)(((Pt-Pc-Ps-Prt-Prb)/Pw)+1) Mp=(Pw/2D)(Y^2+(D-Y)^2)+(Ps)(ds)+(Prt)(drt)+(Prb)(drb)+(Pc)(dc)+(Pt)(dt)
2	IN THE TOP FLANGE	Pt+Pw+Pc>Ps+Prb+Prt	Y=(tc/2)(((Pw+Pt-Ps-Prt-Prb)/Pc)+1) Mp=(Pc/2tc)(Y^2+(tc-Y)^2)+(Ps)(ds)+(Prt)(drt)+(Prb)(drb)+(Pw)(dw)+(Pt)(dt)
3	SLAB, BELOW Prb	Pt+Pw+Pc>(Crb/ts)Ps+Prb+Prt	Y=(ts)((Pc+Pw+Pt-Prt-Prb)/Ps) Mp=((Y^2)(Ps)/2ts)+(Prt)(drt)+(Prb)(drb)+(Pw)(dw)+(Pt)(dt)+(Pc)(dc)
4	SLAB, AT Prb	Pt+Pw+Pc+Prb>(Crb/ts)Ps+Prt	Y=Crb Mp=((Y^2)(Ps)/2ts)+(Prt)(drt)+(Pw)(dw)+(Pt)(dt)+(Pc)(dc)
5	SLAB, ABOVE Prb	Pt+Pw+Pc+Prb>(Crt/ts)Ps+Prt	Y=(ts)((Pc+Pw+Pt-Prt-Prb)/Ps) Mp=((Y^2)(Ps)/2ts)+(Prt)(drt)+(Prb)(drb)+(Pw)(dw)+(Pt)(dt)+(Pc)(dc)

DETERMINE THE LOCATION OF PNA:

CASE	Y	ds	d	dw	dt	drt	dbt	Mp	controling case
CASE NO 1, PNA IN THE WEB	0.00	7.00	0.50	27.00	54.63	8.06	4.44	10037	VOID
CASE NO 2,PNA IN THE TOP FLA	**0.09**	**6.09**	**0.41**	**27.91**	**55.54**	**7.15**	**3.53**	**9975**	**controls**
CASE NO 3,PNA IN THE SLAB BEL	9.54	5.04	-1.46	28.96	56.58	6.10	2.48	9774	VOID
CASE NO 4,PNA AT Prb	7.06	2.56	3.94	31.44	59.06	3.63	0.00	10069	VOID
CASE NO 5,PNA IN THE SLAB ABC	11	6.81	0.31	27.19	54.81	7.87	4.25	10078	VOID

FOR CASE NO 1: Y IS MEASURED FROM TOP OF WED TO PNA
FOR CASE NO. 2: Y IS MEASURED FROM TOP OF TOP FLANGE TO PNA
FOR CASE NOS. 3, 4, & 5: Y IS MEASURED FROM TOP OF SLAB TO PNA

PLASTIC MOMENT, Mp= **9975** 'K
Y= 0.09 inches, PNA falls in the top flange
therefore Dcp = 0.000 inches, since PNA falls in the top flange

Check Ductiltity Requirement:

t. 6.10.7.3
Dp <= 0.42 (Dt)
Where:
Dp = distance from the top of the concrete slab to the neutral axi of the composite section at the plastic moment (in)
Dp = 10.59 in
Dt= total depth of the composite section (in)
Dt = 66.75 in
0.42(Dt) = 28.035 in
therefore: 10.59 <= 28.04 OK

Art. 6.10.2.1.1 web proportions
Webs w/o Longitudinal Stiffeners
Webs shall be proportioned such that
D / tw < 150
which D = depth of web = 54 in
tw = thickness of web = 0.5 in
D/ tw = 108 < 150 OK

Art.6.10.7.1 Compact Sections

Specified min. yield strength of Flg. & Webs <= 70 ksi
Web: 2(Dcp)/tw <= 3.76(Sqrt(E/Fyc))
PNA is located in the top flange, Dcp = 0.00
2(Dcp) / tw = 0
3.76(sqrt(E/Fyc) = 90.553
therefore 2(Dcp)/Tw <= 3.76(Sqrt(E/Fyc)) OK
Dcp = depth of the web in compression at the plastic moment determined as specified in Article D6.3.2

Art.6.10.2.2 flange proportions

0.10 <= Iyc / Iyt <= 10
Iyc Moment of inertia of the compression flange fo the steel section about the vertical axis in the plane of the web (in^4)
Iyc= 486.0 inches^4
Iyt= Moment of inertia of the tension flange of the steel section about the vertical axis in the plane fo the web (in^4)
Iyt= 607.5 inches^4
0.10 <= 0.8000 <= 10 OK
bf / 2(tf) <= 12.0 & bf >= D/6 & tf => 1.1(tw)
bf / 2(tf) = 9.0000 <= 12.0 OK
D / 6 = 9.0000 < 18.0000 inches OK
(1.1)tw = 0.5500 < 1.0000 inches OK

Article 6.10.7 Flexural Resistance - Composite Section in Positive Flexure

Compact sections shall satisfy the ductility requirement specified in Art. 6.10.7.3
The specified minimum yield strenghts of the flanges and web do not exceed 70.00 ksi
At the strength limit state, the section shall satisfy:
Mu + (1/3)(fl)(Sxt) < ϕf(Mn)
ϕf = resistance factor for flexure specified in Art. 6.5.4.2 = 1.00
fl = flange lateral bending stress determined as specified in Art. 6.10.1.6
(the compression flange is cont. supported by the slab, so fl = 0.0 ksi)
(the lateral bending stress from wind and from the pouring of the concrete deck induced on the tension flange is very small and therefore ignored)
Mn = nominal flexural resistance of the section determined as specified in Article 6.10.7.1.2 (kip-in)
Mu = vertical-bending moment determined as specified in Article 6.10.1.6 (Kip-in)
Myt = yeild moment with respect to the tension flange determined as specified in Article D6.2 (kip-in)
Myt = 90638 kip-in
Sxt = elastic section modulus about the major axis of the section to the tension flange taken as Myt / Fyt. (in^3), which Sxt = 1812.8 in^3
Which fl = 0.0 ksi due the compression flange being fully supported and ϕf = 1.0
therefore Mu < Mn

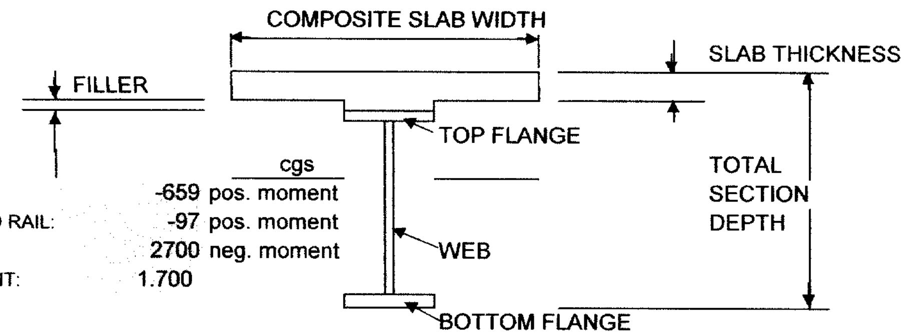
Calculation for Mn, Nominal Flexural Resistance:

The nominal flexural resistance of the section shall be taken as:
If Dp <= 0.1(Dt), then Mn = Mp
Otherwise, Mn = Mp(1.07 - 0.7(Dp/Dt))
Dp = distance from the top of the concrete slab to the neutral axis of the composite section at the plastic moment. (in)
Dp = 10.59 in
Dt = total depth of the composite section (in) = 66.8 inches
0.1(Dt) = 6.675 in
therefore : 10.59 > 6.675 therefore, Mn = Mp(1.07-0.7(Dp/Dt))
Mp = 9975.3 foot-kip
which Mn = Mp(1.07-0.70(Dp/Dt)) = 9565.77 foot-kip <= controls

In a continous span, the nominal flexural resistance of the section shall not exceed:
Mn = (1.3)(Rh)(My) Rh = 1.0 for non-hybrid section
which Mn = 9819.2 kip foot

actual Strength 1 Moment = 5807.0 kip foot is less than Mn = 9565.8 kip foot OK

NEGATIVE MOMENT SECTION DESIGN (NON-COMP) AND
SECTION PROPERTIES FOR WELDED PLATE GIRDERS
Old Hickory Blvd. over I-65
Davidson County
Prepared by WHP
Date: July 31, 2003
PAGE 1



EXTERIOR BEAM
STRENGTH I LOADS

MOMENT DUE DL SLAB & BEAM:	-659	pos. moment
MOMENT DUE DL WEARING AND RAIL:	-97	pos. moment
MOMENT DUE LIVE LOAD:	2700	neg. moment
MOMENTS TAKEN AT SPAN POINT:	1.700	

REQUIRED INPUT:

COMPOSITE SLAB WIDTH:	0.00
SLAB THICKNESS:	0.00
FILLER:	0.00
TOP FLANGE WIDTH:	18.00
TOP FLANGE THICKNESS:	1.00
WEB DEPTH:	54.00
WEB THICKNESS:	0.50
BOTTOM FLANGE WIDTH:	18.00
BOTTOM FLANGE THICKNES	1.25
TOTAL DEPTH OF SECTION	56.25

NON-COMPOSITE PROPERTIES (referance line in at bottom of bottom flange)

	area	Y X (area)	Y^2X(area)	Io	I ref
SLAB	0	0	0	0	0
TOP FLANGE	18	1003.5	55945.13	1.5	55946.6
WEB	27	762.75	21547.69	6561	28108.7
BOTTOM FLANGE	22.5	14.0625	8.789063	2.92969	11.7188
TOTALS	67.5	1780.31			84067

Ycgs= 26.38 in　　S slab= 0.00 in^3
I @ CGS= 37111.29 in^4　　S tf= 1242.22 in^3
S bf= 1407.06 in^3

COMPOSITE PROPERIES N=27

(reference line at the cgs of girder)

	area	Y X AREA	Y^2 X area	Io	I reference
SLAB	0	0	0	0	0
GIRDER	67.5	0	0	37111.3	37111.29
TOTALS	67.5	0			37111.29

Ycgs= 0.00 in　　S slab= 0.00 in^3
I @ cgs= 37111.29 in^4　　S tf= 1242.22 in^3
S bf= 1407.06 in^3

COMPOSITE PROPERIES N=9

(reference line at the cgs of girder)

	area	Y X Area	Y^2 X area	Io	I reference
SLAB	0.0	0	0	0	0
GIRDER	67.5	0	0	37111.3	37111.29
TOTALS	67.5	0			37111.29

Ycgs= 0.00 in　　S slab= 0.00 in^3
I @ cgs= 37111.29 in^4　　S tf= 1242.22 in^3
S bf= 1407.06 in^3

YIELD MOMENT CALULATIONS

Yeild Moment at Span Pt.. 1.70

Applied Moments: (STRENGHT I)
DLnc: -659 'K
DLcomp: -97 'K
LLcomp: 2700 'K
Total Actual Moment: 1944 'K

ADcomp is the added moment applied to the composite section so that the steel will reach its yield moment
Fy = Yeild Stress = 50 KSI

ACTUAL STRESS:
Ftf= 18.78 KSI
Fbf= 16.58 KSI

Fy = DLnc / Snc + DLcomp / Scomp n=27 + ADcomp / Scomp N=9

SLOVE EQUATION FOR ADcomp
Top flange: ADcomp = 5932 'K　　CONTROLS
Bottom flange: ADcomp = 6619 'K

My = YEILD MOMENT = DLnc + DLcomp + ADcomp
My = 5176 > 1944 'K　ELASTIC SECTION

ART.6.10.2 , Cross-section Proportion Limits

Webs without Longitudinal Stiffeners

Webs shall be proportion such that; D / tw <= 150

D / tw = 108 <= 150 OK

Flange proportion limits:

Compression and tension flanges shall be proportion such that

bf / 2(tf) <= 12	bf / 2(tf) =	9.00000 <= 12 OK		
bf => D / 5	bf =	18.0000 =>	10.8	Ok <=== see note below
tf => (1.1)tw	tf =	1.0000 =>	0.6 OK	

and 0.10 <= Iyc / Iyt <= 10

where

Iyt= moment of inertia of the tension flange of the steel section about the vertical axis in the plane of the web (in^4)

Iyc = moment of inertia of the compression flange of the steel section about the vertical axis in the plane of the web (in^4)

Iyt = 486 in^4

Iyc = 608 in^4 therefore 0.10 < 1.25 < 10.000

Iyc / Iyt = 1.250

Note: Commentary to Article 6.10.2.2 recommends that bf be taken greater than or equal to D / 5, instead of D / 6, as a more practical limit. Since this in not necessarily a "hard and fact rule", it was decided to permit some leeway in the Specification for extreme cases with the D / 6 limit, but D / 5 is actually the prefered limit.

NON - COMPOSITE SECTION: ART. 6.10.6.3

PROPOSED SECTION:			ACTUAL STRESS:		
TOP FLANGE WIDTH:	18	TOP FLANGE IN TENSION	Sb = St =	1407.06	IN^3
TOP FLANGE THICKNESS:	1		Ftf=	18.78	KSI
WEB DEPTH:	54		Fbf=	16.58	KSI
WEB THICKNESS:	0.5		EXTERIOR BEAM: STRENGTH I LOADS		
BOTTOM FLANGE WIDTH:	18	BOTTOM FLANGE IN COMPRESSION	MOMENT DUE DL SLAB & BEAM:	-659	'K
BOTTOM FLANGE THICKNESS:	1.25		MOMENT DUE DL WEARING AND RAIL:	-97	'K
TOTAL DEPTH OF SECTION	56.25		MOMENT DUE LIVE LOAD:	2700	'K
Cross-frame spacing, Lb =	325 inches		MOMENTS TAKEN AT SPAN POINT:	1.700	

Article 6.10.1.10 Flange Strength Reduction Factors

Article 6.10.1.10.1 Hybrid Factor, Rh

For homogenous built-up sections and built-up sections with a higher strength steel in the web than in both flanges, Rh shall be taken as 1.0

All flange and web steel used is grade 50W, therefore Rh = 1.0

Article 6.10.1.10.2 Web Load-Shedding Factor, Rb

Compact Slenderness limit requirement:

2(Dc)/tw <= ((LAMDA)rw) Then Rb = 1.00 Where:(Lamda)rw = limiting slenderness ratio for noncompact web = 5.7(E / Fyc)^.5

otherwise: (Lamda)rw = 137.2742

Rb = 1 - (awc/(1200+300(awc))(2(Dc)/tw - ((LAMDA)rw))) <= 1.0

Where: awc = ratio of two times the web area in compression to the area of the compression flange = (2(Dc)(tw) / (bfc)(tfc),

Dc = depth of the web in compression in the elastic range (in) which Dc = 28.88 inches

awc = 2(Dc)(tw) / (bfc)(tfc) = 1.283

2(Dc) / tw = 115.5

((LAMDA)rw)= 137.274 > 115.500 THEN: Rb = 1.00, Ignore Formula Below

IF 2(Dc)/tw => (Lamda)rw

Rb = 1 - (awc/(1200+300(awc))(2(Dc)/tw - ((LAMDA)rw))) = 1.02 <= 1.0, which 1.0 controls

Therefore: Rb = **1.00**

Art. 6.10.6 Strength Limit State

Article 6.10.6.2.3 Composite Sections in Negative Flexure and Non-Composite Sections:

the specified minimum yield strengths of the flanges do not exceed 70.0 ksi

the web satisfies the non-compact slenderness limit:

2(Dc) / tw <= 5.7(E / Fyc)^0.5, where Dc = depth of the web in comp. in the elastic range = 28.88 inches

5.7(E / Fyc)^.5 = 137.2742

2(Dc) / tw = 115.5

therefore : 116 <= 137.2742 OK

Note: This limit simply defines whether or not the optional provisions of Appendix A can be used to determine the nominal flexural resistance. Since the section meets the limit, the more elaborate procedures in Appendix A to determine the flexural resistance, obtaining the added capacity at the strenght limit state. This design did not elect to use Appendix A.

6.10.8 Flexural Resistance - Composite sections in Negative Flexure and Noncomposite Sections

Discretely Braced Flanges in Compression

fbu + (1/3)fl <= ∅f (Fnc)

Discretely Braced Flanges in Tension

fbu + (1/3)fl <= ∅f (Fnt)

where: ∅f = resistance factor for flexure specified in Article 6.5.4.2, ∅f = 1.0

fbu = flange stress calculated without consideration of flange lateral bending determined as specified in Article 6.10.1.6 (ksi)

fl = flange lateral bending stress determined as specified in Article 6.10.1.6 (ksi)

Fnc = Nominal flexural resistance of the flange determined as specified in Article 6.10.8.2 (ksi)

Fnt = Nominal flexural resistance of the flange determined as specified in Article 6.10.8.3 (ksi)

Article 6.10.8.2 Compression -flange flexural resistance

The nominal flexural resistance of the compression flange shall be taken as the smaller of the local buckling resistance determined, as specified in Art. 6.10.8.2.2 and the lateral torsional buckling resistance determined as specified in Art. 6.10.8.2.3.

Local Buckling Resistance:

The local buckling resistance of the compression flange shall be taken as:

If (lamda)f <= (lamda)pf, then:

Fnc = (Rb)(Rh)(Fyc)

otherwise:

Fnc = ((1-(1-(Fyr/(Rh)(Fyc))((lamda)f - (lamda)pf) / ((lamda)rf - (lamda)pf))(Rb)(Rh)(Fyc)

in which:

(lamda)f = slenderness ratio for the compression flange = bfc / (2(tfc)) = 7.20

(lamda)pf = limiting slenderness ratio for a compact flange = 0.38(E/Fyc)^.5 = 9.15

(lamda)rf = limiting slenderness ratio for a noncompact flg. =0.56(E/Fyr)^.5 = 14.35

Fyc = yeild strenght of the compression flange = 50 ksi

Fyr = smaller of the compression flange stress at the onset of nominal yielding, with consideration of residual stress effects but without consideration of flange lateral bending, or the specified minimum yeild strenght of the web (ksi)

0.70(Fyc) <= Fyw, which Fyw = 50.00 ksi and (0.70)(50)= 35.0000 ksi <= controls

Fyr = Rh(Fyt)(Sxt / Sxc) = 44.1 ksi

Rb = web load-shedding factor determined as specified in Article 6.10.1.10.2, which Rb = 1.000

Rh = hybrid factor determined as specified in Article 6.10.1.10.1, which Rh = 1.000

Therefore (lamda)f <= (lamda)pf 7.200 <= 9.152

Fnc = (Rb)(Rh)(Fyc) = 50.000 ksi

Art. 6.10.8.2.3 Lateral Torsional Buckling Resistance

For unbraced lenghts in which the member is prismatic, the lateral torsional buckling resistance of the compression flange shall be taken

If Lb <= Lp, then Fnc = (Rb)(Rh)(Fyc)

If Lp < Lb <= Lr, then Fnc = Cb(1-(1-(Fyr / (Rh)(Fyc)))((Lb - Lp) / (Lr - Lp)))(Rb)(Rh)(Fyc) <= (Rb)(Rh)(Fyc)

if Lb >= Lr, then Fnc = Fcr <= (Rb)(Rh)(Fyc)

which:

Lb = unbraced length (in),

Lb = 325.000 inches

Lp = limiting unbraced length to achieve the nominal flexural resistance of (Rb)(Rh)(Fyc) under uniform bending = (rt)(E / Fyc)^.5

Lp = 113.581 inches

Lr = limiting unbraced length to achieve the onset of nominal yeilding in either flange under uniform bending with consideration of compression flg. Residual stress effects (in)

Lr = (3.14)(rt)(E / Fyc)^.5 = 356.645 inches

Cb = moment gradient modifier

For unbraced cantilevers, or for members where (fmid) / f2 => 1.00 or f2 = 0.00, Cb = 1.00

Art. 6.10.8.2.3 Lateral Torsional Buckling Resistance (cont.)

For all other cases:

Cb = 1.75 - 1.05(f1 / f2) + 0.3(f1 / f2)^2 < = 2.3

(f1 / f2) shall be taken as negative when the moments cause reverse curvature

fmid = stress without consideration of lateral bending at the middle of the unbraced length of the flange under consideration, calculated from the moment envelope value that produces the largest compression at this point, or the smallest tension if this point is never in compression (ksi), fmid shall be due to factored loads and shall be taken as positive in compression and negative in tension.

Cross-frame location: 1.677

fmid = 24.00 ksi

fo = stress without consideration of lateral bending at the brace point opposite to the one corresponding to f2, calculated for the moment envelope value that produces the largest compression at this point, or the smallest tension if this point is never in compression (ksi),. fo shall be due to the factored loads and shall be taken as positive in compression and negative I tension.

f1 = (2)(fmid) - f2 => fo   fo = 2.14 ksi

f2 = except as noted below, largest compressive stress w/o consideration of lateral bending at either end of the unbraced length of the flg. under consideration, calculated from the critical moment envelope value (ksi), f2 shall be due to the factored loads and shall be taken as positve. If the stress is zero or tensile in the flange uner consideration at both ends of the unbraced length, f2, shall be taken as zero.

Cross-frame location: 1.833

f2 = 26.28 ksi ---> see Special Note below

f1 = 21.72 ksi

Cb = 1.09 < = 2.3

Special note: Since the flange transiton occurs at a distance less than 20% of the unbraced length the larger plate can be extended to the brace point. In determining the value for f2, the larger stress at the flange transition point can be ignored.

rt = effective radius of gyration for lateral torsional buckling, rt = (bfc) / (12(1+(1(Dc)(tw) / (3(bfc)(tfc)))^.5

rt = 4.716 inches

therefore: Lp < Lb < = Lr, 113.581 < 325.000 < = 356.645

Fnc = Cb(1-(1-(Fyr / (Rh)(Fyc)))((Lb-Lp) / (Lr-Lp)))(Rb)(Rh)(Fyc) < = (Rb)(Rh)(Fyc)

Fnc = 40.17 ksi, which (Rb)(Rh)(Fyc) = 50.000 ksi

Therefore, Fnc = 40.172 ksi

For unbraced lengths in which the member is non-prismatic, the lateral torional buckling resistance of the compression flange may be taken as the smallest resistance within the unbraced length under consideration determined from above equations as applicable, assuming the unbraced length is prismatic.

For unbraced lengths containing a transition to a smaller section at a distance less than or equal to 20 percent of the unbraced length from the brace point with the smaller moment, the lateral torsional buckling resistance may be determined assuming the transition to the smaller section does not exist.

Article 6.10.8.3 Tension Flange Flexural Resistance

The nominal flexural resistance of the tension flange shall be taken as:

Fnt = (Rh)(Fyt)

where: Rh = hybrid factor determined as specified in Article 6.10.1.10.1, which Rh = 1.0

Therefore: Fnt = (1.0)(50) = 50.00 ksi

Actual Strength 1 stresses:

Note:

Due to the magnitude of the lateral flange bending stresses, they have been ignored.
The lateral flange bending in the flanges would be induced by wind loads and slab placement construction loads.
These maximum lateral flange stresses were calculated from these loads and the max. stress was 0.82 ksi from the wind loads and 3.66 ksi for construction loads. The total induced lateral flange bending stress was approx. 4.48 ksi, which (1/3) fl = (4.48 / 3) = 1.5 ksi which on the most part can be ignored..

Discretely Braced Flanges in Compression

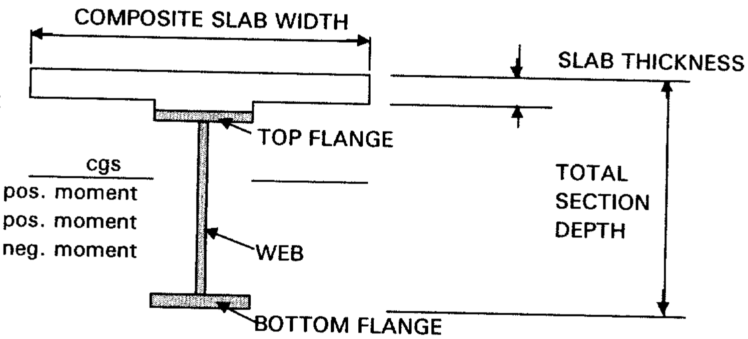
fbu + (1/3)fl < = ϕf (Fnc)

Discretely Braced Flanges in Tension

fbu + (1/3)fl < = ϕf (Fnt)

top flange in tension =	18.78	ksi	<	50.00 ksi OK
bottom flange in compression =	16.58	ksi	<	40.18 ksi OK

NEGATIVE MOMENT SECTION DESIGN (NON-COMP) AND
SECTION PROPERTIES FOR WELDED PLATE GIRDERS
Old Hickory Blvd. over I-65
Davidson County
Prepared by WHP
Date: July 30, 2003
'AGE 1



EXTERIOR BEAM
STRENGTH I LOADS

		cgs
MOMENT DUE DL SLAB & BEAM:	1419	pos. moment
MOMENT DUE DL WEARING AND RAIL:	347	pos. moment
MOMENT DUE LIVE LOAD:	3708	neg. moment
MOMENTS TAKEN AT SPAN POINT:	1.815	

REQUIRED INPUT:	
COMPOSITE SLAB WIDTH:	0.00
SLAB THICKNESS:	0.00
FILLER:	0.00
TOP FLANGE WIDTH:	24.00
TOP FLANGE THICKNESS:	1.06
WEB DEPTH:	54.00
WEB THICKNESS:	0.50
BOTTOM FLANGE WIDTH:	24.00
BOTTOM FLANGE THICKNESS:	1.06
TOTAL DEPTH OF SECTION	56.13

NON-COMPOSITE PROPERTIES (referance line in at bottom of bottom flange)

	area	Y X (area)	Y^2X(area)	Io	I ref
SLAB	0	0	0	0	0
TOP FLANGE	25.5	1417.64	78811.96	2.39893	78814.36
WEB	27	757.688	21262.61	6561	27823.61
BOTTOM FLANGE	25.5	13.5469	7.196777	2.39893	9.595703
TOTALS	78	2188.88			106647.6

Ycgs = 28.06 in
I @ CGS = 45222.25 in^4

S slab = 0.00 in^3
S tf = 1611.48 in^3
S bf = 1611.48 in^3

COMPOSITE PROPERIES N=27
(reference line at the cgs of girder)

	area	Y X AREA	Y^2 X area	Io	I reference
SLAB	0	0	0	0	0
GIRDER	78	0	0	45222.3	45222.25
TOTALS	78	0			45222.25

Ycgs = 0.00 in
I @ cgs = 45222.25 in^4

S slab = 0.00 in^3
S tf = 1611.48 in^3
S bf = 1611.48 in^3

COMPOSITE PROPERIES N=9
(reference line at the cgs of girder)

	area	Y X Area	Y^2 X area	Io	I reference
SLAB	0.0	0	0	0	0
GIRDER	78.0	0	0	45222.3	45222.25
TOTALS	78.0	0			45222.25

Ycgs = 0.00 in
I @ cgs = 45222.25 in^4

S slab = 0.00 in^3
S tf = 1611.48 in^3
S bf = 1611.48 in^3

YIELD MOMENT CALULATIONS

Yeild Moment at Span Pt..: 1.82
Applied Moments: (STRENGHT I)
DLnc: 1419 'K
DLcomp: 347 'K
LLcomp: 3708 'K
Total Actual Moment: 5474 'K

ADcomp is the added moment applied
to the composite section so that
the steel will reach its yield moment
Fy = Yeild Stress = 50 KSI

ACTUAL STRESS:
Ftf = 40.76 KSI
Fbf = 40.76 KSI

Fy = DLnc / Snc + DLcomp / Scomp n=27 + ADcomp / Scomp N=9
SLOVE EQUATION FOR ADcomp
Top flange: ADcomp = 4949 'K
Bottom flange: ADcomp = 4949 'K
My = YEILD MOMENT = DLnc + DLcomp + ADcomp
My = 6715 > 5474 'K ELASTIC SECTION

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NEGATIVE MOMENT SECTION DESIGN (NON-COMP)
Old HIckory Blvd. over I-65
Davidson County
Prepared by WHP
Date: September 15, 2003
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ART.6.10.2 , Cross-section Proportion Limits

Webs without Longitudinal Stiffeners

Webs shall be proportion such that; D / tw <= 150

D / tw = 108 <= 150 OK

Flange proportion limits:

Compression and tension flanges shall be proportion such that

bf / 2(tf) <= 12	bf / 2(tf) =	12.75000 <= 12 OK	
bf => D / 5	bf =	24.0000 => 10.8	OK <=== see note below
tf => (1.1)tw	tf =	1.0625 => 0.6 OK	
and 0.10 <= Iyc / Iyt <= 10			

where

Iyt = moment of inertia of the tension flange of the steel section about the vertical axis in the plane of the web (in^4)

Iyc = moment of inertia of the compression flange of the steel section about the vertical axis in the plane of the web (in^4)

Iyt = 1224 in^4

Iyc = 1224 in^4 therefore 0.10 < 1.000 <= 10.0 OK

Iyc / Iyt = 1.000

Note: Commentary to Article 6.10.2.2 recommends that bf be taken greater than or equal to D / 5, instead of D / 6, as a more practical limit. Since this in not necessarily a "hard and fact rule", it was decided to permit some leeway in the Specification for extreme cases with the D / 6 limit, but D / 5 is actually the prefered limit.

NON - COMPOSITE SECTION: ART. 6.10.6.3

PROPOSED SECTION:

TOP FLANGE WIDTH:	24	TOP FLANGE IN TENSION
TOP FLANGE THICKNESS:	1.0625	
WEB DEPTH:	54	
WEB THICKNESS:	0.5	
BOTTOM FLANGE WIDTH:	24	BOTTOM FLANGE IN COMPRESSION
BOTTOM FLANGE THICKNESS:	1.0625	
TOTAL DEPTH OF SECTION	56.125	
Cross-frame spacing, Lb =	325 inches	

ACTUAL STRESS:

Sb = St = 1611.48 IN^3

Ftf= 40.76 KSI

Fbf= 40.76 KSI

EXTERIOR BEAM: STRENGTH I LOADS

MOMENT DUE DL SLAB & BEAM:	1419	'K
MOMENT DUE DL WEARING AND RAIL:	347	'K
MOMENT DUE LIVE LOAD:	3708	'K
MOMENTS TAKEN AT SPAN POINT:	1.815	

Article 6.10.1.10 Flange Strength Reduction Factors

Article 6.10.1.10.1 Hybrid Factor, Rh

For homogenous built-up sections and built-up sections with a higher strength steel in the web than in both flgs, Rh shall be taken as 1.0

All flange and web steel used is grade 50W, therefore Rh = 1.0

Article 6.10.1.10.2 Web Load-Shedding Factor, Rb

Compact Slenderness limit requirement:

2(Dc)/tw <= ((LAMDA)rw) Then Rb = 1.00 Where:(Lamda)rw = limiting slenderness ratio for noncompact web = 5.7(E / Fyc)^.5

otherwise: (Lamda)rw = 137.2742

Rb = 1 - (awc/(1200+300(awc))(2(Dc)/tw - ((LAMDA)rw))) <= 1.0

Where: awc = ratio of two times the web area in compression to the area of the compression flange = (2(Dc)(tw) / (bfc)(tfc),

Dc = depth of the web in compression in the elastic range (in) which Dc = 27.00 inches

awc = 2(Dc)(tw) / (bfc)(tfc) = 1.059

2(Dc) / tw = 108

((LAMDA)rw)= 137.274 > 108.000 THEN: Rb = 1.00, Ignore Formula Below

IF 2(Dc)/tw => (Lamda)rw

Rb = 1 - (awc/(1200+300(awc))(2(Dc)/tw - ((LAMDA)rw))) = 1.02 <= 1.0, which 1.0 controls

Therefore: Rb = **1.00**

Art. 6.10.6 Strength Limit State

Article 6.10.6.2.3 Composite Sections in Negative Flexure and Non-Composite Sections:

the specified minimum yield strengths of the flanges do not exceed 70.0 ksi

the web satisfies the non-compact slenderness limit:

2(Dc) / tw <= 5.7(E / Fyc)^0.5, where Dc = depth of the web in comp. in the elastic range = 27.00 inches

5.7(E / Fyc)^.5 = 137.2742

2(Dc) / tw = 108

therefore : 108 <= 137.2742 OK

Note: This limit simply defines whether or not the optional provisions of Appendix A can be used to determine the nominal flexural resistance. Since the section meets the limit, the more elaborate procedures in Appendix A to determine the flexural resistance, obtaining the added capacity at the strenght limit state. This design did not elect to use Appendix A.

Art. 6.10.8 Flexural Resistance - Composite sections in Negative Flexure and Noncomposite Sections

Discretely Braced Flanges in Compression

fbu + (1/3)fl < = Of (Fnc)

Discretely Braced Flanges in Tension

fbu + (1/3)fl < = Of (Fnt)

where: ϕf = resistance factor for flexure specified in Article 6.5.4.2, ϕf = 1.0

fbu = flange stress calculated without consideration of flange lateral bending determined as specified in Article 6.10.1.6 (ksi)

fl = flange lateral bending stress determined as specified in Article 6.10.1.6 (ksi)

Fnc = Nominal flexural resistance of the flange determined as specified in Article 6.10.8.2 (ksi)

Fnt = Nominal flexural resistance of the flange determined as specified in Article 6.10.8.3 (ksi)

Article 6.10.8.2 Compression -flange flexural resistance

The nominal flexural resistance of the compression flange shall be taken as the smaller of the local buckling resistance determined, as specified in Art. 6.10.8.2.2 and the lateral torsional buckling resistance determined as specified in Art. 6.10.8.2.3.

Local Buckling Resistance:

The local buckling resistance of the compression flange shall be taken as:

If (lamda)f < = (lamda)pf, then:

Fnc = (Rb)(Rh)(Fyc)

otherwise:

Fnc = ((1-(1-(Fyr/(Rh)(Fyc))((lamda)f - (lamda)pf) / ((lamda)rf - (lamda)pf))(Rb)(Rh)(Fyc)

in which:

(lamda)f = slenderness ratio for the compression flange = bfc / (2(tfc)) = 11.29

(lamda)pf = limiting slenderness ratio for a compact flange = 0.38(E/Fyc)^.5 = 9.15

(lamda)rf = limiting slenderness ratio for a noncompact flg. =0.56(E/Fyr)^.5 = 13.49

Fyc = yeild strenght of the compression flange = 50 ksi

Fyr = smaller of the compression flange stress at the onset of nominal yielding, with consideration of residual stress effects but without consideration of flange lateral bending, or the specified minimum yeild strenght of the web (ksi)

0.70(Fyc) < = Fyw, which Fyw = 50.00 ksi and (0.70)(50) = 35.0000 ksi < = controls

Fyr = Rh(Fyt)(Sxt / Sxc) = 50.0 ksi

Rb = web load-shedding factor determined as specified in Article 6.10.1.10.2, which Rb = 1.000

Rh = hybrid factor determined as specified in Article 6.10.1.10.1, which Rh = 1.000

Therefore (lamda)f = > (lamda)pf, 11.294 < 9.152

so that : Fnc = ((1-(1-(Fyr/(Rh)(Fyc))((lamda)f - (lamda)pf) / ((lamda)rf - (lamda)pf))(Rb)(Rh)(Fyc)

reduction factor: ((1-(1-(Fyr/(Rh)(Fyc))((lamda)f - (lamda)pf) / ((lamda)rf - (lamda)pf)) = 0.852

Fnc = 42.59 ksi

Art. 6.10.8.2.3 Lateral Torsional Buckling Resistance

For unbraced lenghts in which the member is prismatic, the lateral torsional buckling resistance of the compression flange shall be taken as :

If Lb < = Lp, then Fnc = (Rb)(Rh)(Fyc)

If Lp < Lb < = Lr, then Fnc = Cb(1-(1-(Fyr / (Rh)(Fyc)))((Lb - Lp) / (Lr - Lp)))(Rb)(Rh)(Fyc) < = (Rb)(Rh)(Fyc)

if Lb > = Lr, then Fnc = Fcr < = (Rb)(Rh)(Fyc)

which:

Lb = unbraced length (in),

Lb = 325.000 inches

Lp = limiting unbraced length to achieve the nominal flexural resistance of (Rb)(Rh)(Fyc) under uniform bending = (rt)(E / Fyc)^.5

Lp = 153.831 inches

Lr = limiting unbraced length to achieve the onset of nominal yeilding in either flange under uniform bending with consideration of compression flg. Residual stress effects (in)

Lr = (3.14)(rt)(E / Fyc)^.5 = 483.030 inches

Cb = moment gradient modifier

For unbraced cantilevers, or for members where (fmid) / f2 = > 1.0 or f2 = 0.00, Cb = 1.00

Art. 6.10.8.2.3 Lateral Torsional Buckling Resistance (cont.)

For all other cases:

Cb = 1.75 - 1.05(f1 / f2) + 0.3(f1 / f2)^2 <= 2.3

(f1 / f2) shall be taken as negative when the moments cause reverse curvature

fmid = stress without consideration of lateral bending at the middle of the unbraced length of the flange under consideration, calculated from the moment envelope value that produces the largest compression at this point, or the smallest tension if this point is never in compression (ksi), fmid shall be due to factored loads and shall be taken as positive in compression and negative in tension.

Cross-frame location: 1.677

fmid = 24.00 ksi

fo = stress without consideration of lateral bending at the brace point opposite to the one corresponding to f2, calculated for the moment envelope value that produces the largest compression at this point, or the smallest tension if this point is never in compression (ksi),. of shall be due to the factored loads and shall be taken as positive in compression and negative tension.

f1 = (2)(fmid) - f2 => fo   fo = 2.14 ksi

f2 = except as noted below, largest compressive stress w/o consideration of lateral bending at either end of the unbraced length of the flg. under consideration, calculated from the critical moment envelope value (ksi), f2 shall be due to the factored loads and shall be taken as positve. If the stress is zero or tensile in the flange uner consideration at both ends of the unbraced length, f2, shall be taken as zero.

Cross-frame location: 1.833

f2 = 26.28 ksi ----> see special note below

f1 = 21.72 ksi

Cb = 1.09 <= 2.3

Special note: Since the flange transiton occurs at a distance less than 20% of the unbraced length the larger plate can be extended to the brace point. In determining the value for f2, the larger stress at the flange transition point can be ignored.

rt = effective radius of gyration for lateral torsional buckling, rt = (bfc) / (12(1+(1(Dc)(tw) / (3(bfc)(tfc)))^.5

rt = 6.387 inches

therefore: Lp < Lb <= Lr,   153.831 < 325.000 <= 483.030

Fnc = Cb(1-(1-(Fyr / (Rh)(Fyc)))((Lb-Lp) / (Lr-Lp)))(Rb)(Rh)(Fyc) <= (Rb)(Rh)(Fyc)

Fnc = 45.88 ksi,   which (Rb)(Rh)(Fyc) = 50.000 ksi

Therefore, Fnc = 45.877 ksi

Note: This unbraced section is sudjected to reverse curvature bending. The Specifications states that "For unbraced lengths of noncomposite monosymmetric sections subjected to reverse curvature bending, the lateral torsional buckling resistance shall be checked for both flanges." This was ignored since the majority of the un-braced length is non-composite.

For unbraced lengths in which the member is non-prismatic, the lateral torional buckling resistance of the compression flange may be taken as the smallest resistance within the unbraced length under consideration determined from above equations as applicable, assuming the unbraced length is prismatic.

For unbraced lengths containing a transition to a smaller section at a distance less than or equal to 20 percent of the unbraced length from the brace point with the smaller moment, the lateral torsional buckling resistance may be determined assuming the transition to the smaller section does not exist.

Article 6.10.8.3 Tension Flange Flexural Resistance

The nominal flexural resistance of the tension flange shall be taken as:

Fnt = (Rh)(Fyt)

where: Rh = hybrid factor determined as specified in Article 6.10.1.10.1, which Rh = 1.0

Therefore: Fnt = (1.0)(50) = 50.00 ksi

Actual Strength 1 stresses:

Note:

Due to the magnitude of the lateral flange bending stresses, they have been ignored.
The lateral flange bending in the flanges would be induced by wind loads and slab placement construction loads.
These maximum lateral flange stresses were calculated from these loads and the max. stress was 0.82 ksi from the wind loads and 3.66 ksi for construction loads. The total induced lateral flange bending stress was approx. 4.48 ksi, which (1/3) fl = (4.48 / 3) = 1.5 ksi which on the most part can be ignored..

Discretely Braced Flanges in Compression

fbu + (1/3)fl <= Of (Fnc)

Discretely Braced Flanges in Tension

fbu + (1/3)fl <= Of (Fnt)

from Article 6.10.8.2 Compression -flange flexural resistance

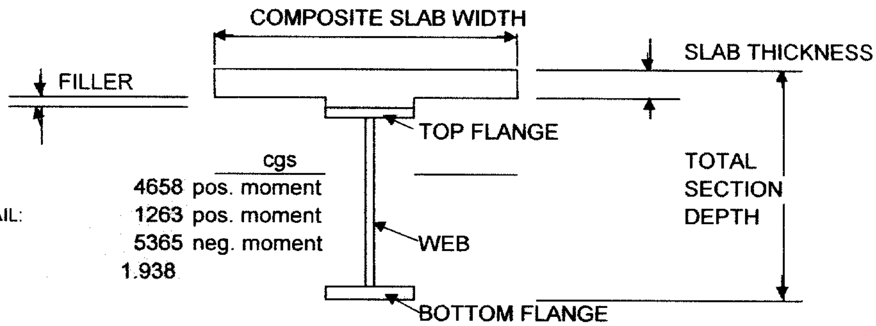
allowable Fnc = 42.59 ksi <= controls

from Art. 6.10.8.2.3 Lateral Torsional Buckling Resistance

allowable Fnc = 45.88 ksi

top flange in tension =	40.76	ksi	<	50.00 ksi OK
bottom flange in compression =	40.76	ksi	<	42.59 ksi OK

NEGATIVE MOMENT SECTION DESIGN (NON-COMP) AND
SECTION PROPERTIES FOR WELDED PLATE GIRDERS
Old Hickory Blvd. over I-65
Davidson County
Prepared by WHP
Date: March 25, 2003



EXTERIOR BEAM
STRENGTH I LOADS

MOMENT DUE DL SLAB & BEAM:	4658	pos. moment
MOMENT DUE DL WEARING AND RAIL:	1263	pos. moment
MOMENT DUE LIVE LOAD:	5365	neg. moment
MOMENTS TAKEN AT SPAN POINT:	1.938	

REQUIRED INPUT:

COMPOSITE SLAB WIDTH:	0.00
SLAB THICKNESS:	0.00
FILLER:	0.00
TOP FLANGE WIDTH:	24.00
TOP FLANGE THICKNESS:	2.00
WEB DEPTH:	54.00
WEB THICKNESS:	0.50
BOTTOM FLANGE WIDTH:	24.00
BOTTOM FLANGE THICKNE!	2.00
TOTAL DEPTH OF SECTION	58

NON-COMPOSITE PROPERTIES (referance line in at bottom of bottom flange)

	area	Y X (area)	Y^2X(area)	Io	I ref
SLAB	0	0	0	0	0
TOP FLANGE	48	2736	155952	16	155968
WEB	27	783	22707	6561	29268
BOTTOM FLANGE	48	48	48	16	64
TOTALS	123	3567			185300

Ycgs= 29.00 in
I @ CGS= 81857.00 in^4

S slab= 0.00 in^3
S tf= 2822.66 in^3
S bf= 2822.66 in^3

COMPOSITE PROPERIES N=27
(reference line at the cgs of girder)

	area	Y X AREA	Y^2 X area	Io	I reference
SLAB	0	0	0	0	0
GIRDER	123	0	0	81857	81857
TOTALS	123	0			81857

Ycgs= 0.00 in
I @ cgs= 81857.00 in^4

S slab= 0.00 in^3
S tf= 2822.66 in^3
S bf= 2822.66 in^3

COMPOSITE PROPERIES N=9
(reference line at the cgs of girder)

	area	Y X Area	Y^2 X area	Io	I reference
SLAB	0.0	0	0	0	0
GIRDER	123	0	0	81857	81857
TOTALS	123	0			81857

Ycgs= 0.00 in
I @ cgs= 81857.00 in^4

S slab= 0.00 in^3
S tf= 2822.66 in^3
S bf= 2822.66 in^3

YIELD MOMENT CALULATIONS

Yeild Moment at Span Pt.. 1.94
Applied Moments: (STRENGHT I)
DLnc: 4658 'K
DLcomp: 1263 'K
LLcomp: 5365 'K
Total Actual Moment: 11286 'K

ADcomp is the added moment applied to the composite section so that the steel will reach its yield moment
Fy = Yeild Stress = 50 KSI

ACTUAL STRESS:
Ftf= 47.98 KSI
Fbf= 47.98 KSI

Fy = DLnc / Snc + DLcomp / Scomp n=27 + ADcomp / Scomp N=9
SLOVE EQUATION FOR ADcomp
Top flange: ADcomp = 5840 'K
Bottom flange: ADcomp = 5840 'K
My = YEILD MOMENT = DLnc + DLcomp + ADcomp
My = 11761 > 11286 'K ELASTIC SECTION

ART.6.10.2 , Cross-section Proportion Limits

Webs without Longitudinal Stiffeners

Webs shall be proportion such that; D / tw <= 150

D / tw = 108 <= 150 OK

Flange proportion limits:

Compression and tension flanges shall be proportion such that

bf / 2(tf) <= 12	bf / 2(tf) =	6.000 <= 12 OK			
bf => D / 5	bf =	24.000 =>	10.8	OK <==== See note below	
tf => (1.1)tw	tf =	2.000 =>	0.6 OK		
and 0.10 <= Iyc / Iyt <= 10					

where

Iyt = moment of inertia of the tension flange of the steel section about the vertical axis in the plane of the web (in^4)

Iyc = moment of inertia of the compression flange of the steel section about the vertical axis in the plane of the web (in^4)

Iyt =	2304	in^4				
Iyc =	2304	in^4	therefore	0.10 <	1.0000 < 10.0	OK
Iyc / Iyt =	1.000					

Note: Commentary to Article 6.10.2.2 recommends that bf be taken greater than or equal to D / 5, instead of D / 6, as a more practical limit. Since this in not necessarily a "hard and fact rule", it was decided to permit some leeway in the Specification for extreme cases with the D / 6 limit, but D / 5 is actually the prefered limit.

NON - COMPOSITE SECTION: ART. 6.10.6.3

PROPOSED SECTION:		
TOP FLANGE WIDTH:	24	TOP FLANGE IN TENSION
TOP FLANGE THICKNESS:	2	
WEB DEPTH:	54	
WEB THICKNESS:	0.5	
BOTTOM FLANGE WIDTH:	24	BOTTOM FLANGE IN COMPRESSION
BOTTOM FLANGE THICKNESS:	2	
TOTAL DEPTH OF SECTION	58	
Cross-frame spacing, Lb =	325 inches	

ACTUAL STRESS:		
Sb = St =	2822.66	IN^3
Ftf=	47.98	KSI
Fbf=	47.98	KSI
EXTERIOR BEAM: STRENGTH I LOADS		
MOMENT DUE DL SLAB & BEAM:	4658	'K
MOMENT DUE DL WEARING AND RAIL:	1263	'K
MOMENT DUE LIVE LOAD:	5365	'K
MOMENTS TAKEN AT SPAN POINT:	1.938	

Article 6.10.1.10 Flange Strength Reduction Factors

Article 6.10.1.10.1 Hybrid Factor, Rh

For homogenous built-up sections and built-up sections with a higher strength steel in the web than in both flgs, Rh shall be taken as 1.0

All flange and web steel used is grade 50W, therefore Rh = 1.0

Article 6.10.1.10.2 Web Load-Shedding Factor, Rb

Compact Slenderness limit requirement:

2(Dc)/tw <= ((LAMDA)rw) Then Rb = 1.00 Where:(Lamda)rw = limiting slenderness ratio for noncompact web = 5.7(E / Fyc)^.5

otherwise: (Lamda)rw = 137.2742

Rb = 1 - (awc/(1200+300(awc))(2(Dc)/tw - ((LAMDA)rw))) <= 1.0

Where: awc = ratio of two times the web area in compression to the area of the compression flange = (2(Dc)(tw) / (bfc)(tfc),

Dc = depth of the web in compression in the elastic range (in) which Dc = 27.00 inches

awc = 2(Dc)(tw) / (bfc)(tfc) = 0.563

2(Dc) / tw = 108

((LAMDA)rw)= 137.274 > 108.000 THEN: Rb = 1.00, Ignore Formula Below

IF 2(Dc)/tw => (Lamda)rw

Rb = 1 - (awc/(1200+300(awc))(2(Dc)/tw - ((LAMDA)rw))) = 1.01 <= 1.0, which 1.0 controls

Therefore: Rb = **1.00**

Art. 6.10.6 Strength Limit State

Article 6.10.6.2.3 Composite Sections in Negative Flexure and Non-Composite Sections:

the specified minimum yield strengths of the flanges do not exceed 70.0 ksi

the web satisfies the non-compact slenderness limit:

2(Dc) / tw <= 5.7(E / Fyc)^0.5, where Dc = depth of the web in comp. in the elastic range = 27.00 inches

5.7(E / Fyc)^.5 = 137.2742

2(Dc) / tw = 108

therefore : 108 <= 137.2742 OK

Note: This limit simply defines whether or not the optional provisions of Appendix A can be used to determine the nominal flexural resistance. Since the section meets the limit, the more elaborate procedures in Appendix A to determine the flexural resistance, obtaining the added capacity at the strenght limit state. This design did not elect to use Appendix A.

Art. 6.10.8 Flexural Resistance - Composite sections in Negative Flexure and Noncomposite Sections

Discretely Braced Flanges in Compression

fbu + (1/3)fl <= Øf (Fnc)

Discretely Braced Flanges in Tension

fbu + (1/3)fl <= Øf (Fnt)

where: Øf = resistance factor for flexure specified in Article 6.5.4.2, Øf = 1.0

fbu = flg. stress calculated without consideration of flange lateral bending determined as specified in Article 6.10.1.6 (ksi)

fl = flange lateral bending stress determined as specified in Article 6.10.1.6 (ksi)

Fnc = Nominal flexural resistance of the flange determined as specified in Article 6.10.8.2 (ksi)

Fnt = Nominal flexural resistance of the flange determined as specified in Article 6.10.8.3 (ksi)

Article 6.10.8.2 Compression -flange flexural resistance

The nominal flexural resistance of the compression flange shall be taken as the smaller of the local buckling resistance determined, as specified in Art. 6.10.8.2.2 and the lateral torsional buckling resistance determined as specified in Art. 6.10.8.2.3.

Local Buckling Resistance:

The local buckling resistance of the compression flange shall be taken as:

If (lamda)f <= (lamda)pf, then:

Fnc = (Rb)(Rh)(Fyc)

otherwise:

Fnc = ((1-(1-(Fyr/(Rh)(Fyc))((lamda)f - (lamda)pf) / ((lamda)rf - (lamda)pf))(Rb)(Rh)(Fyc)

in which:

(lamda)f = slenderness ratio for the compression flange = bfc / (2(tfc)) = 6.00

(lamda)pf = limiting slenderness ratio for a compact flange = 0.38(E/Fyc)^.5 = 9.15

(lamda)rf = limiting slenderness ratio for a noncompact flg. =0.56(E/Fyr)^.5 = 13.49

Fyc = yeild strenght of the compression flange = 50 ksi

Fyr = smaller of the compression flange stress at the onset of nominal yielding, with consideration of residual stress effects but without consideration of flange lateral bending, or the specified minimum yeild strenght of the web (ksi)

0.70(Fyc) <= Fyw, which Fyw = 50.00 ksi and (0.70)(50)= 35.0000 ksi <= controls

Fyr = Rh(Fyt)(Sxt / Sxc) = 50.0 ksi

Rb = web load-shedding factor determined as specified in Article 6.10.1.10.2, which Rb = 1.000

Rh = hybrid factor determined as specified in Article 6.10.1.10.1, which Rh = 1.000

Therefore (lamda)f <= (lamda)p 6.000 <= 9.152

Fnc = (Rb)(Rh)(Fyc) = 50.000 ksi

Art. 6.10.8.2.3 Lateral Torsional Buckling Resistance

For unbraced lenghts in which the member is prismatic, the lateral torsional buckling resistance of the compression flange shall be taken a

If Lb <= Lp, then Fnc = (Rb)(Rh)(Fyc)

If Lp < Lb <= Lr, then Fnc = Cb(1-(1-(Fyr / (Rh)(Fyc)))((Lb - Lp) / (Lr - Lp)))(Rb)(Rh)(Fyc) <= (Rb)(Rh)(Fyc)

if Lb >= Lr, then Fnc = Fcr <= (Rb)(Rh)(Fyc)

which:

Lb = unbraced length (in),

Lb = 325.000 inches

Lp = limiting unbraced length to achieve the nominal flexural resistance of (Rb)(Rh)(Fyc) under uniform bending = (rt)(E / Fyc)^.5

Lp = 159.542 inches

Lr = limiting unbraced length to achieve the onset of nominal yeilding in either flange under uniform bending with consideration of compression flg. Residual stress effects (in)

Lr = (3.14)(rt)(E / Fyc)^.5 = 500.963 inches

Cb = moment gradient modifier

For unbraced cantilevers, or for members where (fmid) / f2 => 1.00 or f2 = 0.00, Cb = 1.00

6.10.8.2.3 Lateral Torsional Buckling Resistance (cont.)

For all other cases:

Cb = 1.75 - 1.05(f1 / f2) + 0.3(f1 / f2)^2 <= 2.3

(f1 / f2) shall be taken as negative when the moments cause reverse curvature

fmid = stress without consideration of lateral bending at the middle of the unbraced length of the flange under consideration, calculated from the moment envelope value that produces the largest compression at this point, or the smallest tension if this point is never in compression (ksi), fmid shall be due to factored loads and shall be taken as positive in compression and negative in tension.

Cross-frame location: 1.833

fmid = 41.92 ksi

fo = stress without consideration of lateral bending at the brace point opposite to the one corresponding to f2, calculated for the moment envelope value that produces the largest compression at this point, or the smallest tension if this point is never in compression (ksi),. fo shall be due to the factored loads and shall be taken as positive in compression and negative tension.

f1 = (2)(fmid) - f2 => fo   fo = 26.30 ksi

f2 = except as noted below, largest compressive stress w/o consideration of lateral bending at either end of the unbraced length of the flg. under consideration, calculated from the critical moment envelope value (ksi), f2 shall be due to the factored loads and shall be taken as positve. If the stress is zero or tensile in the flange uner consideration at both ends of the unbraced length, f2, shall be taken as zero.

Cross-frame location: 2.0000

f2 = 47.98 ksi --> max. stress in the section occurs at the flange trans. point with in the un-braced ler

f1 = 35.86 ksi

Cb = 1.13 <= 2.3

rt = effective radius of gyration for lateral torsional buckling, rt = (bfc) / (12(1+(1(Dc)(tw) / (3(bfc)(tfc)))^.5

therefore: Lp < Lb <= Lr,

Fnc = Cb(rt = 6.625 inches

159.542 < 325.000 <= 500.963

Fnc = 48.15 ksi, which (Rb)(Rh)(Fyc) = 50.000 ksi

Therefore, Fnc = 48.151 ksi

For unbraced lengths in which the member is non-prismatic, the lateral torional buckling resistance of the compression flange may be taken as the smallest resistance within the unbraced length under consideration determined from above equations as applicable, assuming the unbraced length is prismatic.

For unbraced lengths containing a transition to a smaller section at a distance less than or equal to 20 percent of the unbraced length from the brace point with the smaller moment, the lateral torsional buckling resistance may be determined assuming the transition to the smaller section does not exist.

Article 6.10.8.3 Tension Flange Flexural Resistance

The nominal flexural resistance of the tension flange shall be taken as:

Fnt = (Rh)(Fyt)

where: Rh = hybrid factor determined as specified in Article 6.10.1.10.1, which Rh = 1.0

Therefore: Fnt = (1.0)(50) = 50.00 ksi

Actual Strength 1 stresses:

Note:

Due to the magnitude of the lateral flange bending stresses, they have been ignored.
The lateral flange bending in the flanges would be induced by wind loads and slab placement construction loads.
These maximum lateral flange stresses were calculated from these loads and the max. stress was 0.82 ksi from the wind loads and 3.66 ksi for construction loads. The total induced lateral flange bending stress was approx. 4.48 ksi, which (1/3) fl = (4.48 / 3) = 1.5 ksi which on the most part can be ignored..

Discretely Braced Flanges in Compression

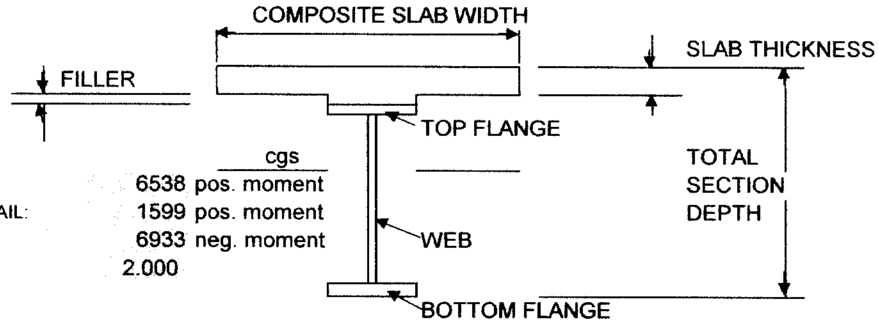
fbu + (1/3)fl <= Of (Fnc)

Discretely Braced Flanges in Tension

fbu + (1/3)fl <= Of (Fnt)

top flange in tension =	47.98	ksi	< 50.00 ksi OK
bottom flange in compression =	47.98	ksi	< 48.15 ksi OK

NEGATIVE MOMENT SECTION DESIGN (NON-COMP) AND
SECTION PROPERTIES FOR WELDED PLATE GIRDERS
Old Hickory Blvd. over I-65
Davidson County
Prepared by WHP
Date: March 25, 2003
PAGE 1



EXTERIOR BEAM
STRENGTH I LOADS

MOMENT DUE DL SLAB & BEAM:	6538	pos. moment
MOMENT DUE DL WEARING AND RAIL:	1599	pos. moment
MOMENT DUE LIVE LOAD:	6933	neg. moment
MOMENTS TAKEN AT SPAN POINT:	2.000	

REQUIRED INPUT:

COMPOSITE SLAB WIDTH:	0.00
SLAB THICKNESS:	0.00
FILLER:	0.00
TOP FLANGE WIDTH:	24.00
TOP FLANGE THICKNESS:	2.75
WEB DEPTH:	54.00
WEB THICKNESS:	0.50
BOTTOM FLANGE WIDTH:	24.00
BOTTOM FLANGE THICKNE:	2.75
TOTAL DEPTH OF SECTION	59.5

NON-COMPOSITE PROPERTIES (referance line in at bottom of bottom flange)

	area	Y X (area)	Y^2X(area)	Io	I ref
SLAB	0	0	0	0	0
TOP FLANGE	66	3836.25	222982	41.5938	223024
WEB	27	803.25	23896.69	6561	30457.7
BOTTOM FLANGE	66	90.75	124.7813	41.5938	166.375
TOTALS	159	4730.25			253648

Ycgs= 29.75 in
I @ CGS= 112922.8 in^4

S slab= 0.00 in^3
S tf= 3795.7 in^3
S bf= 3795.7 in^3

COMPOSITE PROPERIES N=27
(reference line at the cgs of girder)

	area	Y X AREA	Y^2 X area	Io	I reference
SLAB	0	0	0	0	0
GIRDER	159	0	0	112923	112922.8
TOTALS	159	0			112922.8

Ycgs= 0.00 in
I @ cgs= 112922.8 in^4

S slab= 0.00 in^3
S tf= 3795.72 in^3
S bf= 3795.72 in^3

COMPOSITE PROPERIES N=9
(reference line at the cgs of girder)

	area	Y X Area	Y^2 X area	Io	I reference
SLAB	0.0	0	0	0	0
GIRDER	159	0	0	112923	112922.8
TOTALS	159	0			112922.8

Ycgs= 0.00 in
I @ cgs= 112922.8 in^4

S slab= 0.00 in^3
S tf= 3795.72 in^3
S bf= 3795.72 in^3

YIELD MOMENT CALULATIONS

Yeild Moment at Span Pt.. 2.00
Applied Moments: (STRENGHT I)
DLnc: 6538 'K
DLcomp: 1599 'K
LLcomp: 6933 'K
Total Actual Moment: 15070 'K

ADcomp is the added moment applied to the composite section so that the steel will reach its yield moment
Fy = Yeild Stress = 50 KSI

ACTUAL STRESS:
Ftf= 47.64 KSI
Fbf= 47.64 KSI

Fy = DLnc / Snc + DLcomp / Scomp n=27 + ADcomp / Scomp N=9
SLOVE EQUATION FOR ADcomp
Top flange: ADcomp = 7679 'K
Bottom flange: ADcomp = 7679 'K
My = YEILD MOMENT = DLnc + DLcomp + ADcomp
My = 15816 > 15070 'K ELASTIC SECTION

ART.6.10.2 , Cross-section Proportion Limits

Webs without Longitudinal Stiffeners

Webs shall be proportion such that; D / tw <= 150

D / tw = 108 <= 150 OK

Flange proportion limits:

Compression and tension flanges shall be proportion such that

bf / 2(tf) <= 12	bf / 2(tf) =	4.36 <= 12 OK		
bf => D / 5	bf =	24.00 =>	10.8	OK <===== see note below
tf => (1.1)tw	tf =	2.7500 =>	0.6 OK	

and 0.10 <= Iyc / Iyt <= 10

where

Iyt = moment of inertia of the tension flange of the steel section about the vertical axis in the plane of the web (in^4)

Iyc = moment of inertia of the compression flange of the steel section about the vertical axis in the plane of the web (in^4)

Iyt = 3168 in^4

Iyc = 3168 in^4 therefore 0.10 < 1.0000 < 10 OK

Iyc / Iyt = 1.000

Note: Commentary to Article 6.10.2.2 recommends that bf be taken greater than or equal to D / 5, instead of D / 6, as a more practical limit. Since this in not necessarily a "hard and fact rule", it was decided to permit some leeway in the Specification for extreme cases with the D / 6 limit, but D / 5 is actually the prefered limit.

NON - COMPOSITE SECTION: ART. 6.10.6.3

PROPOSED SECTION:

TOP FLANGE WIDTH:	24	TOP FLANGE IN TENSION
TOP FLANGE THICKNESS:	2.75	
WEB DEPTH:	54	
WEB THICKNESS:	0.5	
BOTTOM FLANGE WIDTH:	24	BOTTOM FLANGE IN COMPRESSION
BOTTOM FLANGE THICKNESS:	2.75	
TOTAL DEPTH OF SECTION	59.5	
Cross-frame spacing, Lb =	325 inches	

ACTUAL STRESS:

Sb = St =	3795.72	IN^3
Ftf=	47.64	KSI
Fbf=	47.64	KSI

EXTERIOR BEAM: STRENGTH I LOADS

MOMENT DUE DL SLAB & BEAM:	6538	'K
MOMENT DUE DL WEARING AND RAIL:	1599	'K
MOMENT DUE LIVE LOAD:	6933	'K
MOMENTS TAKEN AT SPAN POINT:	2.000	

Article 6.10.1.10 Flange Strength Reduction Factors

Article 6.10.1.10.1 Hybrid Factor, Rh

For homogenous built-up sections and built-up sections with a higher strength steel in the web than in both flgs, Rh shall be taken as 1.0

All flange and web steel used is grade 50W, therefore Rh = 1.0

Article 6.10.1.10.2 Web Load-Shedding Factor, Rb

Compact Slenderness limit requirement:

2(Dc)/tw <= ((LAMDA)rw) Then Rb = 1.00 Where:(Lamda)rw = limiting slenderness ratio for noncompact web = 5.7(E / Fyc)^.5

otherwise: (Lamda)rw = 137.2742

Rb = 1 - (awc/(1200+300(awc))(2(Dc)/tw - ((LAMDA)rw))) <= 1.0

Where: awc = ratio of two times the web area in compression to the area of the compression flange = (2(Dc)(tw) / (bfc)(tfc),

Dc = depth of the web in compression in the elastic range (in) which Dc = 27.00 inches

awc = 2(Dc)(tw) / (bfc)(tfc) = 0.409

2(Dc) / tw = 108

((LAMDA)rw)= 137.274 > 108.000 THEN: Rb = 1.00, Ignore Formula Below

IF 2(Dc)/tw => (Lamda)rw

Rb = 1 - (awc/(1200+300(awc))(2(Dc)/tw - ((LAMDA)rw))) = 1.01 <= 1.0, which 1.0 controls

Therefore: Rb = **1.00**

Art. 6.10.6 Strength Limit State

Article 6.10.6.2.3 Composite Sections in Negative Flexure and Non-Composite Sections:

the specified minimum yield strengths of the flanges do not exceed 70.0 ksi

the web satisfies the non-compact slenderness limit:

2(Dc) / tw <= 5.7(E / Fyc)^0.5, where Dc = depth of the web in comp. in the elastic range = 27.00 inches

5.7(E / Fyc)^.5 = 137.2742

2(Dc) / tw = 108

therefore : 108 <= 137.2742 OK

Note: This limit simply defines whether or not the optional provisions of Appendix A can be used to determine the nominal flexural resistance. Since the section meets the limit, the more elaborate procedures in Appendix A to determine the flexural resistance, obtaining the added capacity at the strenght limit state. This design did not elect to use Appendix A.

Art. 6.10.8 Flexural Resistance - Composite sections in Negative Flexure and Noncomposite Sections

Discretely Braced Flanges in Compression

fbu + (1/3)fl <= ϕf (Fnc)

Discretely Braced Flanges in Tension

fbu + (1/3)fl <= ϕf (Fnt)

where: ϕf = resistance factor for flexure specified in Article 6.5.4.2, ϕf = 1.0

fbu = flg. stress calculated without consideration of flange lateral bending determined as specified in Article 6.10.1.6 (ksi)

fl = flange lateral bending stress determined as specified in Article 6.10.1.6 (ksi)

Fnc = Nominal flexural resistance of the flange determined as specified in Article 6.10.8.2 (ksi)

Fnt = Nominal flexural resistance of the flange determined as specified in Article 6.10.8.3 (ksi)

Article 6.10.8.2 Compression -flange flexural resistance

The nominal flexural resistance of the compression flange shall be taken as the smaller of the local buckling resistance determined, as specified in Art. 6.10.8.2.2 and the lateral torsional buckling resistance determined as specified in Art. 6.10.8.2.3.

Local Buckling Resistance:

The local buckling resistance of the compression flange shall be taken as:

If (lamda)f <= (lamda)pf, then:

Fnc = (Rb)(Rh)(Fyc)

otherwise:

Fnc = ((1-(1-(Fyr/(Rh)(Fyc))((lamda)f - (lamda)pf) / ((lamda)rf - (lamda)pf))(Rb)(Rh)(Fyc)

in which:

(lamda)f = slenderness ratio for the compression flange = bfc / (2(tfc)) = 4.36

(lamda)pf = limiting slenderness ratio for a compact flange = 0.38(E/Fyc)^.5 = 9.15

(lamda)rf = limiting slenderness ratio for a noncompact flg. =0.56(E/Fyr)^.5 = 13.49

Fyc = yeild strenght of the compression flange = 50 ksi

Fyr = smaller of the compression flange stress at the onset of nominal yielding, with consideration of residual stress effects but without consideration of flange lateral bending, or the specified minimum yeild strenght of the web (ksi)

0.70(Fyc) <= Fyw, which Fyw = 50.00 ksi and (0.70)(50)= 35.0000 ksi <= controls

Fyr = Rh(Fyt)(Sxt / Sxc) = 50.0 ksi

Rb = web load-shedding factor determined as specified in Article 6.10.1.10.2, which Rb = 1.000

Rh = hybrid factor determined as specified in Article 6.10.1.10.1, which Rh = 1.000

Therefore (lamda)f <= (lamda)p 4.364 <= 9.152

Fnc = (Rb)(Rh)(Fyc) = 50.000 ksi

Art. 6.10.8.2.3 Lateral Torsional Buckling Resistance

For unbraced lenghts in which the member is prismatic, the lateral torsional buckling resistance of the compression flange shall be taken a

If Lb <= Lp, then Fnc = (Rb)(Rh)(Fyc)

If Lp < Lb <= Lr, then Fnc = Cb(1-(1-(Fyr / (Rh)(Fyc)))((Lb - Lp) / (Lr - Lp)))(Rb)(Rh)(Fyc) <= (Rb)(Rh)(Fyc)

if Lb >= Lr, then Fnc = Fcr <= (Rb)(Rh)(Fyc)

which:

Lb = unbraced length (in),

Lb = 325.000 inches

Lp = limiting unbraced length to achieve the nominal flexural resistance of (Rb)(Rh)(Fyc) under uniform bending = (rt)(E / Fyc)^.5

Lp = 161.440 inches

Lr = limiting unbraced length to achieve the onset of nominal yeilding in either flange under uniform bending with consideration of compression flg. Residual stress effects (in)

Lr = (3.14)(rt)(E / Fyc)^.5 = 506.923 inches

Cb = moment gradient modifier

For unbraced cantilevers, or for members where (fmid) / f2 => 1.0 or f2 = 0.00

6.10.8.2.3 Lateral Torsional Buckling Resistance (cont.)

For all other cases:

Cb = 1.75 - 1.05(f1 / f2) + 0.3(f1 / f2)^2 <= 2.3

f1 / f2 shall be taken as negative when the moments cause reverse curvature

fmid = stress without consideration of lateral bending at the middle of the unbraced length of the flange under consideration, calculated from the moment envelope value that produces the largest compression at this point, or the smallest tension if this point is never in compression (ksi), fmid shall be due to factored loads and shall be taken as positive in compression and negative in tension.

Cross-frame location: 1.833

fmid = 41.92 ksi

fo = stress without consideration of lateral bending at the brace point opposite to the one corresponding to f2, calculated for the moment envelope value that produces the largest compression at this point, or the smallest tension if this point is never in compression (ksi),. fo shall be due to the factored loads and shall be taken as positive in compression and negative tension.

f1 = (2)(fmid) - f2 => fo   fo = 23.3 ksi

f2 = except as noted below, largest compressive stress w/o consideration of lateral bending at either end of the unbraced length of the flg. under consideration, calculated from the critical moment envelope value (ksi), f2 shall be due to the factored loads and shall be taken as positve. If the stress is zero or tensile in the flange uner consideration at both ends of the unbraced length, f2, shall be taken as zero.

Cross-frame location: 2.000

f2 = 47.98 ksi --> max. stress in the section occurs at the flange trans. point with in the un-braced length

f1 = 35.86 ksi

Cb = 1.13 <= 2.3

rt = effective radius of gyration for lateral torsional buckling, rt = (bfc) / (12(1+(1(Dc)(tw) / (3(bfc)(tfc)))^.5

rt = 6.703 inches

therefore: Lp < Lb <= Lr,   161.440 < 325.000 <= 506.923

Fnc = Cb(1-(1-(Fyr / (Rh)(Fyc)))((Lb-Lp) / (Lr-Lp)))(Rb)(Rh)(Fyc) <= (Rb)(Rh)(Fyc)

Fnc = 48.34 ksi, which (Rb)(Rh)(Fyc) = 50.000 ksi

Therefore, Fnc = 48.340 ksi

, unbraced lengths in which the member is non-prismatic, the lateral torional buckling resistance of the compression flange may be taken as the smallest resistance within the unbraced length under consideration determined from above equations as applicable, assuming the unbraced length is prismatic.

For unbraced lengths containing a transition to a smaller section at a distance less than or equal to 20 percent of the unbraced length from the brace point with the smaller moment, the lateral torsional buckling resistance may be determined assuming the transition to the smaller section does not exist.

Article 6.10.8.3 Tension Flange Flexural Resistance

The nominal flexural resistance of the tension flange shall be taken as:

Fnt = (Rh)(Fyt)

where: Rh = hybrid factor determined as specified in Article 6.10.1.10.1, which Rh = 1.0

Therefore: Fnt = (1.0)(50) = 50.00 ksi

Actual Strength 1 stresses:

Note:

Due to the magnitude of the lateral flange bending stresses, they have been ignored.
The lateral flange bending in the flanges would be induced by wind loads and slab placement construction loads.
These maximum lateral flange stresses were calculated from these loads and the max. stress was 0.82 ksi from the wind loads and 3.66 ksi for construction loads. The total induced lateral flange bending stress was approx. 4.48 ksi, which (1/3) fl = (4.48 / 3) = 1.5 ksi which on the most part can be ignored..

Discretely Braced Flanges in Compression

fbu + (1/3)fl <= Of (Fnc)

Discretely Braced Flanges in Tension

fbu + (1/3)fl <= Of (Fnt)

top flange in tension =	47.64	ksi	< 50.00 ksi OK
,ottom flange in compression =	47.64	ksi	< 48.34 ksi OK

Negative Moment Design (Non-Composite)
State Route 45 (Olc ory Blvd.) over I-65
Davidson County
Date: September 30, 2003

Check for the need for A mplifying first-order v alues of fl (Article 6.10.1.6) in final condition.

note: This only applies to the bottom f lange in the negativ e moment region.

All discretely braced shall satisf y: fl <= 0.60(Fyt)
Flange lateral bending stresses may be determined directly from first-order elastic analy sis in discretely braced compression flangess for which:

Lb <= 1.2 (Lp)(((Cb)(Rb)) / (f bm / Fyc)))^0.5

Cb = moment gradient modif ier specified in Article 6.10.8.2.3 or Article A6.3.3, as applicalble
fbm = largest v alue of the compression stress in the f lange under consideration, calculated without consideration of flange lateral bending, throughout the unbraced length (ksi), f bm shall be determined by factored loads.
Lb = unbraced length (in)
Lp = limiting unbraced length specif ied in Article 6.10.8.2.3 (in)
Fyc = yeild strenght of the compression f lange
Rb = web load-shedding f actor determined as specif ied in Article 6.10.1.10.2

If the abov e equations are not satisf ied, second order elastic compression f lange lateral bending stresses shall be determined
Second order compression f lange lateral bending stresses may be determined by amplifying first order v alues as follows:

fl = (0.85 / (1 - fbm / Fcr)(fl1) => fl1

where :
fl1 = first order compression f lange lateral bending stress at the section under consideration, or the max imum first order lateral bending stress in the compression f lange under consideration throughout the unbraced length, as applicable (ksi)
Fcr = elastic lateral torsional buckling stress f or the flange under consideration determined f rom Article 6.10.8.2.3 or A6.3.3 may only be applied for unbraced lenghts in which the web is compact or noncompact.

Fcr = (Cb)(Rb)((pi)^ 2)(E) / (Lb / rt)^2
rt = (btc) / (12(1 + .33(Dc)(tw)/((bf c)(tfc)))

* **Note**: In final condition, only lateral loads that are applied are f rom the wind loads.
Lateral loads from cantilev er slab supports during construction are made in a earlier construction checks.

Span Pt.	top flange		bottom flange		Comp. flg. slenderness ratios			rt	un-braced lengths			Cb	Fcr	fbm	limiting	does factor	amplifying	*fl, lateral	amplified
	width	thickness	width	thickness	(lamda)f	(lamda)pf	(lamda)rf		Lb	Lp	Lr			max.	ratio **	apply	factor	flg stress	fl
1.7000	24.000	1.063	24.000	1.063	11.294	9.152	13.487	6.39	325.0	153.8	483.1	1.09	120.4	43.24	207.3	yes	1.33	0.46	0.61
1.7500	24.000	1.063	24.000	1.063	11.294	9.152	13.487	6.39	325.0	153.8	483.1	1.09	120.4	27.18	261.4	yes	1.10	0.46	0.50
1.8000	24.000	1.063	24.000	1.063	11.294	9.152	13.487	6.39	325.0	153.8	483.1	1.09	120.4	36.36	226.0	yes	1.22	0.46	0.56
1.8150	24.000	1.063	24.000	1.063	11.294	9.152	13.487	6.39	325.0	153.8	483.1	1.09	120.4	40.67	213.7	yes	1.28	0.46	0.59
1.8150	24.000	2.000	24.000	2.000	6.000	9.152	13.487	6.62	325.0	159.5	501.0	1.09	129.5	23.27	293.0	yes	1.04	0.23	0.24
1.8333	24.000	2.000	24.000	2.000	6.000	9.152	13.487	6.62	325.0	159.5	501.0	1.13	134.3	26.30	280.6	yes	1.06	0.23	0.24
1.9000	24.000	2.000	24.000	2.000	6.000	9.152	13.487	6.62	325.0	159.5	501.0	1.13	134.3	37.49	235.0	yes	1.18	0.23	0.27
1.9167	24.000	2.000	24.000	2.000	6.000	9.152	13.487	6.62	325.0	159.5	501.0	1.13	134.3	41.92	222.3	yes	1.24	0.23	0.28
1.9375	24.000	2.000	24.000	2.000	6.000	9.152	13.487	6.62	325.0	159.5	501.0	1.13	134.3	47.98	207.8	yes	1.32	0.23	0.30
1.9375	24.000	2.750	24.000	2.750	4.364	9.152	13.487	6.70	325.0	161.4	506.9	1.13	137.5	35.68	243.8	yes	1.15	0.17	0.20
2.000	24.000	2.750	24.000	2.750	4.364	9.152	13.487	6.70	325.0	161.4	506.9	1.13	137.5	47.64	211.0	yes	1.30	0.17	0.22

Span Pt.	top flange		bottom flange		Fnc	Fyc	*fl, lateral	fbu, act.comp.	total	total	check
	width	thickness	width	thickness	lateral	Local	flg.stress	flg. Stress	fl+fbu<=Fyc	fbu+(1/3)fl<=Fnc	
1.7000	24.000	1.063	24.000	1.063	45.87	42.59	0.61	11.62	12.23	11.82	OK
1.7500	24.000	1.063	24.000	1.063	45.87	42.59	0.50	24.00	24.50	24.17	OK
1.8000	24.000	1.063	24.000	1.063	45.87	42.59	0.56	36.36	36.92	36.55	OK
1.8150	24.000	1.063	24.000	1.063	45.87	42.59	0.59	40.67	41.26	40.87	OK
1.8150	24.000	2.000	24.000	2.000	45.87	50.00	0.24	23.27	23.51	23.35	OK
1.8333	24.000	2.000	24.000	2.000	48.15	50.00	0.24	26.30	26.54	26.38	OK
1.9000	24.000	2.000	24.000	2.000	48.15	50.00	0.27	37.49	37.76	37.58	OK
1.9167	24.000	2.000	24.000	2.000	48.15	50.00	0.28	41.92	42.20	42.01	OK
1.9375	24.000	2.000	24.000	2.000	48.15	50.00	0.30	47.98	48.28	48.08	OK
1.9375	24.000	2.750	24.000	2.750	48.34	50.00	0.20	35.68	35.88	35.74	OK
2.000	24.000	2.750	24.000	2.750	48.34	50.00	0.22	47.64	47.86	47.71	OK

Compression flange resistance
Old Hickory Blvd [illegible] I-65
Davidson County
Date: September 15, 2003

Compression Flange Flexural resistance

allowables for compression flange for within unbraced sections

Applied load Modifier = 0.95
Strength-I Limit State: (0.95)((Yp)(Dc) + (1.75)(LL))

Load Factor for permanent loads, Yp.

type of load	load factor, Yp Maximum	Minimum
Dc	1.25	0.90
Dw	1.50	0.65

Cb = moment gradient modifer.
Cb = 1.75 - 1.05 (f1 / f2) + .3(f1 / f2)^2
f1 = 2 (fmid) - f2
f2 = largest compressive stress in the unbraced section

web thickness = 0.5 inches, Web depth = 54 inches, Live Load Distribution Factor for Moment = 0.914

Point Description	Span point	total girder depth	top flange width	top flange thickness	bottom flange width	bottom flange thickness	non-factored moments DLnoncomp.	DL comp	LL max pos	LL max neg	Inoncomp	I comp n=27	Icomp n=9	Ycgs non-comp	Ycgs comp n=9	Ycgs comp n=27	Max positive stress TF	Max positive stress BF	Max. Negative stress TF	Max. Negative stress BF	DL stress TF	DL stress BF	Span point	Point Description
cross-frame	1.00	56.25	18.00	1.00	18.00	1.25	0	0	0	0	37111	68120	91883	26.4	39.1	48.7	0.0	0.0	0.0	0.0	0.00	0.00	1.00	cross-frame
fmid	1.08	56.25	18.00	1.00	18.00	1.25	1145	2334	1540	-203	37111	68120	91883	26.4	39.1	48.7	25.5	-49.4	22.9	-32.5	13.13	-11.60	1.08	fmid
cross-frame	1.16	56.25	18.00	1.00	18.00	1.25	1597	413	2749	-406	37111	68120	91883	26.4	39.1	48.7	24.2	-46.8	19.5	-16.3	18.32	-16.17	1.16	cross-frame
fmid	1.25	57.25	18.00	1.00	18.00	2.25	2698	550	3721	-635	47376	92173	131438	21.7	34.3	45.2	37.4	-44.4	30.1	-17.1	25.06	-17.62	1.25	fmid
cross-frame	1.33	57.25	18.00	1.00	18.00	2.25	3010	614	4269	-846	47376	92173	131438	21.7	34.3	45.2	41.9	-50.3	33.4	-18.3	32.17	-19.66	1.33	cross-frame
fmid	1.42	57.25	18.00	1.00	18.00	2.25	3028	618	4507	-1057	47376	92173	131438	21.7	34.3	45.2	42.5	-52.0	33.2	-17.1	32.36	-19.78	1.42	fmid
cross-frame	1.50	57.25	18.00	1.00	18.00	2.25	2752	561	4444	-1269	47376	92173	131438	21.7	34.3	45.2	39.2	-49.4	29.7	-13.6	29.41	-17.98	1.50	cross-frame
fmid	1.58	57.25	18.00	1.00	18.00	2.25	2111	430	4041	-1481	47376	92173	131438	21.7	34.3	45.2	31.2	-41.8	21.9	-7.2	22.56	-13.79	1.58	fmid
cross-frame	1.67	56.25	18.00	1.00	18.00	1.25	1171	239	3368	-1692	37111	68120	91883	26.4	39.1	48.7	19.5	-46.7	11.9	2.1	15.53	-11.86	1.67	cross-frame
fmid	1.75	56.13	24.00	1.06	24.00	1.06	-55	-12	2456	-2067	45222	45222	45222	28.1	28.1	28.1	27.2	-27.2	-24.0	24.0	-0.49	0.49	1.75	fmid
cross-frame	1.83	58.00	24.00	2.00	24.00	2.00	-1571	-322	1436	-2541	81857	81857	81857	29.0	29.0	29.0	-0.6	0.6	-26.3	26.3	-7.93	7.93	1.83	cross-frame
fmid	1.92	58.00	24.00	2.00	24.00	2.00	-3396	-692	516	-3187	81857	81857	81857	29.0	29.0	29.0	-18.0	18.0	-41.9	41.9	-17.14	17.14	1.92	fmid
cross-frame	2.00	59.50	24.00	2.75	24.00	2.75	-5505	-1122	0	-4563	112923	112923	112923	29.8	29.8	29.8	-25.7	25.7	-47.6	47.6	-20.67	20.67	2.00	cross-frame

Note: Referance point for Ycgs is the bottom of the bottom flange
Positive stresses are in compression   Negative stresses are in tension.

Slab depth + filler = 10.50 inches
Lb = unbraced length = 325.00 inches
Lp = (rt)(E / Fyc)^.5
Lr = (rt)(3.14)(E / Fyc)^.5
rt = (bfc) / (12(1+(1/3)(Dc)(tw)/((bfc)(tfc))^.5

top flange width	top flange thickness	bottom flange width	bottom flange thickness	Dc	rt top flg	rt bot.flg
18.00	1.00	18.00	1.25	24.950	4.683	4.774
18.00	1.00	18.00	2.25	30.400	4.590	4.899
18.00	1.00	18.00	1.25	24.130	4.698	4.786
24.00	1.06	24.00	1.06	27.000	6.388	6.388
24.00	2.00	24.00	2.00	27.000	6.625	6.625
24.00	2.75	24.00	2.75	27.000	6.704	6.704

*** Maximum Compressive Stress (LL + DL) in the section between cross-frames

Point Description	DL f1	DL f2	DL + LL f1	DL+LL f2	Cb DL	Cb DL+LL	rt	Lp	Lr	reduc. factor	allow. stress	Fractored Comp. (+) DL stress TF	Max Neg stress BF	span point	Factored Stresses DL+LLcomp stress ***	Dlcomp stress ***
cross-frame	7.95	18.32	25.51	25.51	1.35	1.00	4.683	112.79	354.15	0.74	36.81	0.00	0.00	1.000		
fmid	7.95	18.32	25.51	25.51	1.35	1.00	4.683	112.79	354.15	0.74	36.81	13.13	-32.53	1.080	25.51	18.32
cross-frame	7.95	18.32	25.51	25.51	1.35	1.00	4.683	112.79	354.15	0.74	36.81	18.32	-16.30	1.160		
cross-frame	17.941	32.17	32.89	41.94	1.25	1.10	4.590	110.55	347.11	0.73	40.22	18.32	-16.30	1.160		
fmid	17.941	32.17	32.89	41.94	1.25	1.10	4.590	110.55	347.11	0.73	40.22	25.06	-17.14	1.250	41.94	32.17
cross-frame	17.941	32.17	32.89	41.94	1.25	1.10	4.590	110.55	347.11	0.73	40.22	32.17	-18.26	1.333		
cross-frame	32.555	32.17	42.55	42.55	0.99	1.00	4.590	110.55	347.11	0.73	36.40	32.17	-18.26	1.333		
fmid	32.555	32.17	42.55	42.55	0.99	1.00	4.590	110.55	347.11	0.73	36.40	32.36	-17.08	1.417	43.20	32.95
cross-frame	32.555	32.17	42.55	42.55	0.99	1.00	4.590	110.55	347.11	0.73	36.40	29.41	-13.59	1.500		
cross-frame	15.7111	29.41	23.08	39.25	1.27	1.23	4.590	110.55	347.11	0.73	44.83	29.41	-13.59	1.500		
fmid	15.7111	29.41	23.08	39.25	1.27	1.23	4.590	110.55	347.11	0.73	44.83	22.56	-7.24	1.583	39.25	29.41
cross-frame	15.7111	29.41	23.08	39.25	1.27	1.23	4.698	113.14	355.25	0.74	45.41	15.53	2.14	1.667		
cross-frame	7.93	15.53	21.70	26.30	1.29	1.08	4.698	113.14	355.25	0.74	39.87	15.53	2.14	1.667		
fmid	7.93	15.53	21.70	26.30	1.29	1.08	6.388	153.84	483.07	0.84	45.63	-0.49	24.00	1.750	45.52	13.43
cross-frame	7.93	15.53	21.70	26.30	1.29	1.08	6.388	153.84	483.07	0.84	45.63	-7.93	26.30	1.833		
cross-frame	13.622	20.67	35.87	47.98	1.18	1.13	6.625	159.55	501.0	0.85	48.15	-7.93	26.30	1.833		
fmid	13.622	20.67	35.87	47.98	1.18	1.13	6.625	159.55	501.0	0.85	48.15	-17.14	41.92	1.917	47.98	20.67
cross-frame	13.622	20.67	35.87	47.98	1.18	1.13	6.704	161.45	506.9	0.86	48.34	-20.67	47.64	2.000		

<= top flange fully supported by the slab under final loading does not apply
<= top flange fully supported by the slab under final loading does not apply

Factored Strenght I Stresses
Old Hickory Blvd. (I-65
Davidson County
Date: April 24, 2003

Summary of Strenght I Stresses:

Applied load Modifier = 0.95
Strength-I Limit State: (0.95)((Yp)(Dc) + (1.75)(LL))

Load Factor for permanent loads, Yp.

type of load	load factor, Yp Maximum	Minimum
Dc	1.25	0.90
Dw	1.50	0.65

web thickness = 0.5 inches,Web depth = 54 inches,Live Load Distribution Factor for Moment = 0.914

Point Description	Span point	total girder depth	top flange width	top flange thickness	bottom flange width	bottom flange thickness	non-factored moments DL noncomp.	DL comp	LL max pos	LL max neg	Inoncomp	I comp n=27	Icomp n=9	Ycgs non-comp	Ycgs comp n=9	Ycgs comp n=27	Max positive stress TF	Max positive stress BF	Max. Negative stress TF	Max. Negative stress BF	Span point
cross-frame	1.000	56.250	18.000	1.000	18.000	1.2500	0	0	0	0	37111	68120	91883	26.38	39.08	48.68	0.00	0.00	0.00	0.00	1.000
fmid	1.080	56.250	18.000	1.000	18.000	1.25	1145	2334	1540	-203	37111	68120	91883	26.38	39.08	48.68	25.51	-49.37	22.89	-32.53	1.080
tenth point	1.100	56.250	18.000	1.000	18.000	1.25	1431	292	1925	-254	37111	68120	91883	26.38	39.08	48.68	20.56	-35.96	17.29	-14.90	1.100
cross-frame	1.160	56.250	18.000	1.000	18.000	1.25	1597	413	2749	-406	37111	68120	91883	26.38	39.08	48.68	24.23	-46.78	19.49	-16.30	1.160
flange trans.	1.180	56.250	18.000	1.000	18.000	1.25	2224	454	3023	-457	37111	68120	91883	26.38	39.08	48.68	32.01	-56.18	26.78	-22.56	1.180
flange trans.	1.180	57.250	18.000	1.000	18.000	2.25	2224	454	3023	-457	47376	92173	131438	21.72	34.26	45.18	30.77	-36.36	24.94	-14.55	1.180
tenth point	1.200	57.250	18.000	1.000	18.000	2.25	2422	494	3298	-508	47376	92173	131438	21.72	34.26	45.18	33.52	-39.63	27.14	-15.78	1.200
fmid	1.250	57.250	18.000	1.000	18.000	2.25	2698	550	3721	-635	47376	92173	131438	21.72	34.26	45.18	37.41	-44.44	30.12	-17.14	1.250
tenth point	1.300	57.250	18.000	1.000	18.000	2.25	2973	606	4144	-762	47376	92173	131438	21.72	34.26	45.18	41.30	-49.24	33.08	-18.50	1.300
cross-frame	1.333	57.250	18.000	1.000	18.000	2.25	3010	614	4269	-846	47376	92173	131438	21.72	34.26	45.18	41.94	-50.32	33.37	-18.26	1.333
tenth point	1.400	57.250	18.000	1.000	18.000	2.25	3083	629	4520	-1015	47376	92173	131438	21.72	34.26	45.18	43.20	-52.47	33.93	-17.78	1.400
fmid	1.417	57.250	18.000	1.000	18.000	2.25	3028	618	4507	-1057	47376	92173	131438	21.72	34.26	45.18	42.55	-51.96	33.23	-17.08	1.417
cross-frame	1.500	57.250	18.000	1.000	18.000	2.25	2752	561	4444	-1269	47376	92173	131438	21.72	34.26	45.18	39.25	-49.40	29.68	-13.59	1.500
fmid	1.583	57.250	18.000	1.000	18.000	2.25	2111	430	4041	-1481	47376	92173	131438	21.72	34.26	45.18	31.16	-41.85	21.92	-7.24	1.583
flange trans.	1.600	57.250	18.000	1.000	18.000	2.25	1982	404	3960	-1523	47376	92173	131438	21.72	34.26	45.18	29.54	-40.33	20.36	-5.97	1.600
flange trans.	1.600	56.250	18.000	1.000	18.000	1.25	1982	404	3960	-1523	37111	68120	91883	26.38	39.08	48.68	30.42	-62.29	22.19	-9.32	1.600
cross-frame	1.667	56.250	18.000	1.000	18.000	1.25	1171	239	3368	-1692	37111	68120	91883	26.38	39.08	48.68	19.52	-46.74	11.92	2.14	1.667
flange trans.	1.700	56.250	18.000	1.000	18.000	1.25	771	157	3072	-1777	37111	68120	91883	26.38	39.08	48.68	14.13	-39.03	6.85	7.82	1.700
flange trans.	1.700	56.125	24.000	1.063	24.000	1.06	771	157	3072	-1777	45222	45222	45222	28.06	28.06	28.06	43.24	-43.24	-11.62	11.62	1.700
fmid	1.750	56.125	24.000	1.063	24.000	1.06	-55	-12	2456	-2067	45222	45222	45222	28.06	28.06	28.06	27.18	-27.18	-24.00	24.00	1.750
tenth point	1.800	56.125	24.000	1.063	24.000	1.06	-880	-180	1839	-2357	45222	45222	45222	28.06	28.06	28.06	11.12	-11.12	-36.36	36.36	1.800
flange trans.	1.815	56.063	24.000	1.063	24.000	1.00	-1194	-244	1656	-2440	45222	45222	45222	28.06	28.06	28.06	5.58	-5.59	-40.67	40.76	1.815
flange trans.	1.815	58.000	24.000	2.000	24.000	2.00	-1194	-244	1656	-2440	81857	81857	81857	29.00	29.00	29.00	3.19	-3.19	-23.27	23.27	1.815
cross-frame	1.833	58.000	24.000	2.000	24.000	2.00	-1571	-322	1436	-2541	81857	81857	81857	29.00	29.00	29.00	-0.61	0.61	-26.30	26.30	1.833
tenth point	1.900	58.000	24.000	2.000	24.000	2.00	-2973	-606	619	-2911	81857	81857	81857	29.00	29.00	29.00	-14.68	14.68	-37.49	37.49	1.900
fmid	1.917	58.000	24.000	2.000	24.000	2.00	-3396	-692	516	-3187	81857	81857	81857	29.00	29.00	29.00	-18.00	18.00	-41.92	41.92	1.917
flange trans.	1.938	58.000	24.000	2.000	24.000	2.00	-3923	-886	387	-3531	81857	81857	81857	29.00	29.00	29.00	-22.67	22.67	-47.98	47.98	1.938
flange trans.	1.938	59.500	24.000	2.750	24.000	2.75	-3923	-886	387	-3531	112923	112923	112923	29.75	29.75	29.75	-16.86	16.86	-35.68	35.68	1.938
cross-frame	2.000	59.500	24.000	2.750	24.000	2.7500	-5505	-1122	0	-4563	112923	112923	112923	29.75	29.75	29.75	-25.72	25.72	-47.64	47.64	2.000

Note: Referance point for Ycgs is the bottom of the bottom flange

Positive stresses are in compression

Negative stresses are in tension.

Wind Effects on Girder Flanges
Old Hickory Blvd. Over I-65
Davidson County
Designer: whp
Date: February 3, 2003

Article 6.10.1.6 Flange lateral Bending Stressess

The Stress fl, (lateral flange bending stress) shall be determined as the largest value of the flange stress due to lateral bending throughout the unbraced length under consideration. For a straight girder the only major factors that produce lateral flange bending is wind loads and during construction the movement of the screed machine while the deck slab is being poured. Narrow top flanges can lead to prolbems during construction, such as out-of-plane distortions of the girder compression flanges and webs. By satisfying the following guideline reduces potential problems: bfc => L / 85

where, L = length of the girder shipping piece = **1368.00** inches
therefore the mim.bfc = 16.09 inches, use 18 inch wide flange

all discretely braced flanges shall satisfy: fl <= 0.6(Fyf)
flange lateral bending stessess may be determined directly from first-order elastic analysis in discretely braced compression flanges for which: Lb <= 1.2(Lp)((Cb(Rb) / (fbm / Fyc)))^0.5
where: fbm = largest value of the compression-flange stress within the unbraced length under consideration calculated without consideration of flange lateral bending (ksi). fbm shall be determined based on factored loads.
Lb = unbraced length (in) = **325.00** inches
Lp = limiting unbraced length specified in Article 6.10.8.2.3 (in) = **125.14** inches, for 18" x 1" flange
Rb = web load-shedding factor determined as specified in Art.6.10.1.10.2 = **1.00**
Cb= moment gradient modifier specified in Article 6.10.8.2.3 = **1.35**

Article 4.6.2.7 Lateral wind load distribution in Multi-beam Bridges
I-Sections:
In bridges with composite decks, non-composite decks with concrete haunches, and other decks that can provide horizontal diaphragm action, wind load on the upper half of the outside beam, the deck, vehicles, barriers, and appurtenances shall be assumed to be directly transmitted to the deck, acting as a lateral diaphragm carring this load to the supports. Wind load on the lower half of the outside beam shall be assumed to be applied laterally to the lower flange
The wind force , W, may be applied to the flanges of the exterior members. For composite and non-composite members with cast-in-place concrete slab, W, need not be applied to top flange.

W = (n)(Y)(Pd)(d) / 2
where: n = load modifier specified in Article 1.3.2 (3.4.1) = 0.950
Y = load factor specified in Table 3.4.1-1 for the pariclular group loading combination for Strength III loading for wind exceeding 55 MPH, Y = 1.40
Pd = design horizontal wind pressure specified in Article 3.8.1 (ksf)
W = factored wind force per unit length applied to the flange (k/ft)
d = depth of member (ft)

Article 3.8.1 Horizonal Wind Pressures
Wind load is assumed to be uniformly distriduted on the area exposed to the wind.
article 3.8.12.1, Wind Pressure on structures, Pd = (Pb)(Vdz / Vb)^2 = (Pb)(Vdz^2) / 10,000
where: Pb = base pressure specified in table 1 (ksf), for beams Pb = 0.050
Vdz = design wind velocity at design elevation Z, MPH
Vb = base wind velocity of 100 mph at 30.0 ft height, yeilding design pressure specified in 3.8.1.2 & 3.8.2
Vdz = 2.5(Vo)(V30 / Vb)(ln(Z /Zo))
Vo = friction velocity, as specified in table 1 for various upwind surface characteristics (mph) =10.9 for suburban
Z = hieght of structure at which wind loads are being calculated, measured from ground surface
Zo = friction length of upstream fetch, as specified in table 1, = 3.28 for suburban
V30 = wind velocity at 30.0 ft. above low ground (mph) = Vb =100 mph in absence of better criterion
therefore, Vdz = (2.5)(10.9)(100 / 100)(ln(20.0 / 3.28))= 49.26 mph
Pd = (0.050)(Vdz / Vb)^2 = (0.050)(Vdz^2) / 10,000 = 0.0121 ksf

Factored wind force (W) = (0.95)(Y)(Pd)(d) / 2
d = parapet depth + slab depth + filler + web depth + thichness of bottom flange

parapet :	**2.667** feet	which d =	11.292 feet
slab depth:	**0.750** feet	W =	0.0911 kip / ft
filler depth:	**1.125** feet		
web depth:	**4.500** feet		
bot.flg th.	**2.250** feet		

Wind Effects on Girder Flanges (cont.)
Old Hickory Blvd. Over I-65
Davidson County
Designer: whp
Date: September 30, 2003

Maximum lateral moment in a flange: (C4.6.2.7.1)

First and second load path: Mw = maximum lateral moment in the flange due to factored wind loading (kip-ft)
Mw = (W)(Lb)^2 / 10 where Lb = spacing of brace points (ft)
Lb = cross-frame spacing = 27.0833 feet
W = 0.0911 kip/ft
Mw = 6.6841 kip-ft
Third load path: (assumes that cross-frames act as struts in distrubuting the wind force on the exterior flange to adjacent flanges.)
The maximum wind moment on the loaded flange may be computed as:.
Mw = (W)(Lb^2) / 10 + (W)(L^2) / (8(Nb))
where: Nb = number of longitudinal members = 8
L = span length (ft) = 162.5 feet
therefore: (Mw) = (W)(Lb^2) / 10 + (W)(L^2) / (8(Nb)) = 44.282 kip-ft
use the third load path, which (Mw) = 44.282 kip-ft
Which: bfb = 18.000 inches Fyb = 50.00 ksi
tfb = 1.000 inches
Stress from the wind load: (Fw) = (Mw) / section modulus
I = tfb(bfb)^3 / 12 so the Section Modulus = I / (.5)(bfb)
for which: Fw = (6)(Mw) / (tfb)(bfb)^2
use strenght III and Strenght V load combinations:
Strength III: 1.25(DC) + 1.50(DW) + 1.4(WS), Y = 1.40 for this loading
Strength V: 1.25(DC) + 1.50(DW) + 1.35(LL+IM) + 0.4(WS), Y = 0.40 for this loading
where Fw = 0.820 ksi for strenght III loadings, Y = 1.40
Fw = 0.234 ksi for strenght V loadings, Y = 0.40

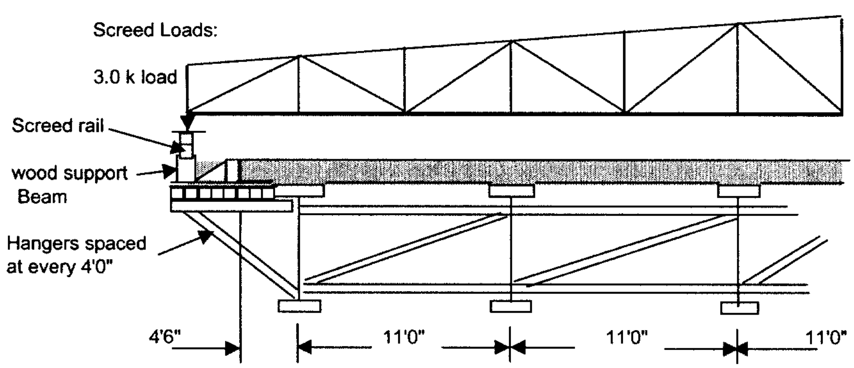
Lateral flange stresses due to wind loads

Span Pt.	top flange		bottom flange		web		top flg, Fw	Bot.flg,Fw
	width	thickness	width	thickness	depth	thickness	ksi	ksi
1.0000	18.0000	1.0000	18.0000	1.2500	54.0000	0.5000	0.8200	0.6560
1.1000	18.0000	1.0000	18.0000	1.2500	54.0000	0.5000	0.8200	0.6560
1.1800	18.0000	1.0000	18.0000	1.2500	54.0000	0.5000	0.8200	0.6560
1.1800	18.0000	1.0000	18.0000	2.2500	54.0000	0.5000	0.8200	0.3645
1.2000	18.0000	1.0000	18.0000	2.2500	54.0000	0.5000	0.8200	0.3645
1.3000	18.0000	1.0000	18.0000	2.2500	54.0000	0.5000	0.8200	0.3645
1.4000	18.0000	1.0000	18.0000	2.2500	54.0000	0.5000	0.8200	0.3645
1.5000	18.0000	1.0000	18.0000	2.2500	54.0000	0.5000	0.8200	0.3645
1.6000	18.0000	1.0000	18.0000	2.2500	54.0000	0.5000	0.8200	0.3645
1.6000	18.0000	1.0000	18.0000	1.2500	54.0000	0.5000	0.8200	0.6560
1.7000	18.0000	1.0000	18.0000	1.2500	54.0000	0.5000	0.8200	0.6560
1.7000	24.0000	1.0000	24.0000	1.0000	54.0000	0.5000	0.4613	0.4613
1.8000	24.0000	1.0000	24.0000	1.0000	54.0000	0.5000	0.4613	0.4613
1.8125	24.0000	1.0000	24.0000	1.0000	54.0000	0.5000	0.4613	0.4613
1.8150	24.0000	2.0000	24.0000	2.0000	54.0000	0.5000	0.2306	0.2306
1.9000	24.0000	2.0000	24.0000	2.0000	54.0000	0.5000	0.2306	0.2306
1.9380	24.0000	2.0000	24.0000	2.0000	54.0000	0.5000	0.2306	0.2306
1.9380	24.0000	2.7500	24.0000	2.7500	54.0000	0.5000	0.1677	0.1677
2.0000	24.0000	2.7500	24.0000	2.7500	54.0000	0.5000	0.1677	0.1677

note: shaded area is a compact section in the final conditions, and non-compact before the slab is poured and cured,
Strength V is not applied to this section

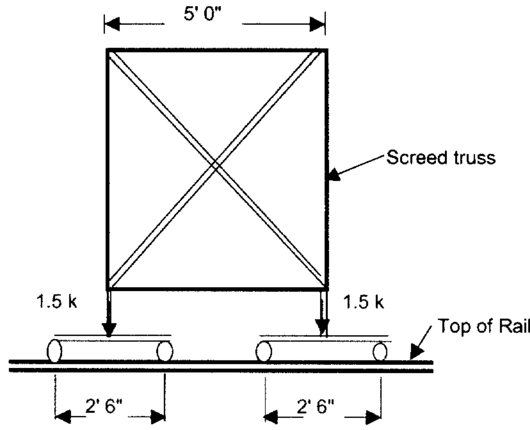
Lateral Flange Bending From slab pouring operations:

These lateral bending stresses will be produced from the jacks for supporting the cantilever forms, wet concrete slab,
the screed machine and other construction loads.



Conservative, assume all the loads from the screed will be resisted by the ext. girder
Typical Elevation View

Wind Effects on Girder Flanges (cont.)
Old Hickory Blvd. Over I-65
Davidson County
Designer: whp
Date: September 30, 2003



End Screed Elevation

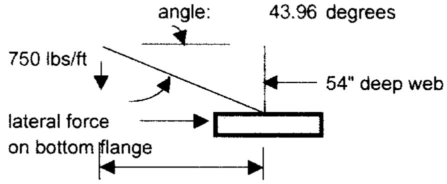
Note: Forms between Girders will be either SIP stee or concrete and will not add any loads to the girders. All forms will be completely in place before the slab is poured.

The cantilever loads will be spread over 16' which is four bracket supports. The wood beam supporting the screed rail and along with the screed rail will distribute the loads.

Calculation of loads:

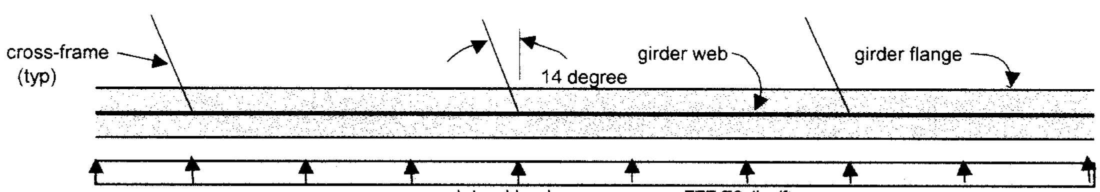
Screed truss:	3000 lbs / 16 feet =	187.50	lbs/ft
wet concrete slab:	(150 lbs/ft^3)(4.67x.75) =	525.38	lbs/ft
cantilever forms:			
wood support beam:	(60 lbs/ft^3)(.3x.3)(2) =	13.32	lbs/ft
2 x 4 forming mat.:	(60lbs/ft^3)(.17x.3)(8) =	7.34	lbs/ft
3/4" plywood:	(60lbs/ft^3)(4.5x.06) =	16.20	lbs/ft
total form and screed loads =		749.73	lbs/ft

angle: 43.96 degrees



4.500 ft cantilever

Lateral force = (750 lbs/ft) / (tan 43.96)
Lateral force = 777.7 lbs/ft



lateral loads = w = 777.70 lbs/ft

max. moment = (w)(l^2)(0.80) / 8, Note: 0.80 is a continuity factor for continous beams
l = cross-frame spacing = 27.083 feet
max.mom.at cross-frame=(w)(l^2)(0.80)/8= 57.04 kip-feet
max. mom. at mid. point = 0.04(w)(l^2) = 22.82 kip-feet
Min. flange dimensions: Moment of Inertia = Io = (b)(h^3) / 12
flange width: 18.0000 inches Io = 607.500 inches^4
flange thickness: 1.2500 inches
Stress = (moment)(distance to point) / (moment of inertia)
Stress = 10.14 ksi

For investigating the constructibility of flexural members, all loads shall be factored as specified in Article 3.4.2.
Load factors for the weight of the structure and appurtenances shall be taken less than 1.25. Unless otherwise specified by the Owner, the load factor for construction loads, for equipment and for dynamic effects shall not be less than 1.5. The load factor for wind shall not be less than 1.25. All other load factors shall be taken as 1.0. Since the weight of the screed is known, a load factor of 1.25 will be used.

top flange		bottom flange		web		Io	at cross-frame		at mid.span	
width	thickness	width	thickness	depth	thickness		lateral flg. stress	factored lat. flg stress	lateral flg. stress	factored lat. flg stress
18.0000	1.0000	18.0000	1.0000	54.0000	0.5000	486.0	12.68	15.85	5.07	6.34
18.0000	1.0000	18.0000	1.2500	54.0000	0.5000	607.5	10.14	12.68	4.06	5.07
18.0000	1.0000	18.0000	2.2500	54.0000	0.5000	1093.5	5.63	7.04	2.25	2.82
24.0000	1.0630	24.0000	1.0630	54.0000	0.5000	1224.6	6.71	8.38	2.68	3.35
24.0000	2.0000	24.0000	2.0000	54.0000	0.5000	2304.0	3.57	4.46	1.43	1.78
24.0000	2.7500	24.0000	2.7500	54.0000	0.5000	3168.0	2.59	3.24	1.04	1.30

Constructibility Check
State Route 45 (Old Hickory Blvd.) over I-65
Davidson County
Date: September 30, 2003

Article 6.10.3 Constructibility

The provisions of Article 2.5.3 shall apply. In addition to providing adequate strength, nominal yielding or reliance on post-buckling resistance shall not be permitted for main load carrying members during critical stages of construction, except for yielding of the web in hybrid members. This shall be accomplished by satisfying the requirements of Article 6.10.3.2 and 6.10.3.3 at each critical construction stage. For sections condition, but are noncomposite during construction, the provisions of Article 6.10.3.4 shall apply.
For investigating the constructibility of flexural members, all loads shall be factored as specified in Article 3.4.2.
Load factors for the weight of the structure and appurtenances shall be taken less than 1.25. Unless otherwise specified by the Owner, the load factor for construction loads, for equipment and for dynamic effects shall not be less than 1.5. The load factor for wind shall not be less than 1.25. All other load factors shall be taken as 1.0.

Article 6.10.3.2.1 Discretely Braced Flanges in Compression

For critical stages of construction, each of the following requirements shall be satisfied. For sections with slender webs, the first equation shall not be checked when fl is equal to zero. For sections with compact or noncompact webs, the third equation shall not be checked.

(1) fbu + fl <= ϕf (Rh)(Fyc)
(2) fbu + (1/3)fl <= ϕf(Fnc)
(3) fbu <= ϕf(Fcrw)

Where:
ϕf = resistiance factor for flexure specified in Article 6.5.4.2, ϕf = 1.00
fbu = flange stress calculated without consideration of flange lateral bending determined as specified in Art. 6.10.1.6 (ksi)
fl = flange lateral bending stress determined as specified in Article 6.10.1.6 (ksi)
Fcrw = nominal elastic bending-buckling resistance for webs specified in Article 6.10.1.9 (ksi),
which Fcrw = 0.9(E)(k) / (D / tw)^2 <= Fyc
k = bend-buckling coefficient = 9 / (Dc / D)^2
Fnc = nominal flexural resistance of the flange (ksi)
note: Fnc shall be determined as specified in Article 6.10.8.2. For sections with compact or non-compact webs, the lateral torsional buckling resistance may be taken as Mnc determined as specified in Article A6.3.3 divided by Sxc. In computing Fnc for constructibility, for web load-shedding factor, Rb, shall be taken as 1.0
My = yield moment with respect to the compression flange determined as specified in Article D6.2 (kip-inch)
Rh = hybrid factor specified Article 6.10.1.10.1
Sxc = elastic section modulus about the major axis of the section to the compression flange taken as Myc / Fyc (in^3)

Article 6.10.3.2.2 Discretely Braced Flanges in Tension

For critical stages of construction, the following requirement shall be satisfied:
fbu + fl <= ϕf (Rh)(Fyf)

Article 6.10.3.2.3 Continuously Braced Flanges in Tension or Compression

For critical stages of construction, the following requirement shall be satisfied:
fbu <= ϕf (Rh)(Fyf)
For sections with slender webs, flanges in compression shall also satisfy:
fbu <= ϕf(Fcrw) < Fyc

Applied Construction Dead Loads:

applied load factor: 1.25 for dead loads and Applied load modifier: 0.95
web thickness = 0.5000 inches, Web depth 54.000 inches

Span Pt.	top flange		bottom flange		non-factored dead load		Moment	location	factored dead loads			Web-bend buckling k =	Fcrw
	width	thickness	width	thickness	Moment	Shear	of inertia	of CGS	stress TF	stress BF	shear	9/(Dc/D)^2	
1.000 cf	18.000	1.000	18.000	1.250	0	101.6	37111	26.38	0.00	0.00	121	37.73	50.00
1.1000	18.000	1.000	18.000	1.250	1431	77.9	37111	26.38	16.42	14.49	93	37.73	50.00
1.167 cf	18.000	1.000	18.000	1.250	1597	62	37111	26.38	18.32	16.17	74	37.73	50.00
1.1800	18.000	1.000	18.000	1.250	2224	58.9	37111	26.38	25.51	22.52	70	37.73	50.00
1.1800	18.000	1.000	18.000	2.250	2224	58.9	47376	21.72	23.77	14.53	70	55.63	50.00
1.2000	18.000	1.000	18.000	2.250	2422	54.2	47376	21.72	25.88	15.82	64	55.63	50.00
1.3000	18.000	1.000	18.000	2.250	2973	30.5	47376	21.72	31.77	19.42	36	55.63	50.00
1.333 cf	18.000	1.000	18.000	2.250	3010	22.7	47376	21.72	32.17	19.66	27	55.63	50.00
1.4000	18.000	1.000	18.000	2.250	3083	6.8	47376	21.72	32.95	20.14	8	55.63	50.00
1.500 cf	18.000	1.000	18.000	2.250	2752	-16.9	47376	21.72	29.41	17.98	-20	55.63	50.00
1.6000	18.000	1.000	18.000	2.250	1982	-40.7	47376	21.72	21.18	12.95	-48	55.63	50.00
1.6000	18.000	1.000	18.000	1.250	1982	-40.7	37111	26.38	22.74	20.07	-48	37.73	50.00
1.667 cf	18.000	1.000	18.000	1.250	1171	-56.6	37111	26.38	13.43	11.86	-67	37.73	50.00
1.7000	18.000	1.000	18.000	1.250	771	-64.4	37111	26.38	8.84	7.81	-76	37.71	50.00
1.7000	24.000	1.063	24.000	1.063	771	-64.4	42865	28.00	7.21	7.18	-76	33.47	50.00
1.8000	24.000	1.063	24.000	1.063	-880	-88.1	42865	28.00	-8.23	-8.19	-105	33.47	50.00
1.8150	24.000	1.063	24.000	1.063	-1194	-91.7	42865	28.00	-11.16	-11.11	-109	33.47	50.00
1.8150	24.000	2.000	24.000	2.000	-1194	-91.7	81857	29.00	-6.03	-6.03	-109	31.21	50.00
1.833 cf	24.000	2.000	24.000	2.000	-1571	-95.9	81857	29.00	-7.93	-7.93	-114	31.21	50.00
1.9000	24.000	2.000	24.000	2.000	-2973	-111.8	81857	29.00	-15.01	-15.01	-133	31.21	50.00
1.9375	24.000	2.000	24.000	2.000	-3923	120.7	81857	29.00	-19.81	-19.81	143	31.21	50.00
1.9375	24.000	2.750	24.000	2.750	-3923	120.7	112923	29.75	-14.73	-14.73	143	29.65	50.00
2.000 cf	24.000	2.750	24.000	2.750	-5505	-135.5	112923	29.75	-20.67	-20.67	-161	29.65	50.00

Article 6.10.8.2 Compression -flange flexural resistance

The nominal flexural resistance of the compression flange shall be taken as the smaller of the local buckling resistance determined, as specified in Art. 6.10.8.2.2 and the lateral torsional buckling resistance determined as specified in Art. 6.10.8.2.3.

Local Buckling Resistance

The local buckling resistance of the compression flange shall be taken as:

If (lamda)f <= (lamda)pf, then: Fnc = (Rb)(Rh)(Fyc)

otherwise: Fnc = (1-(1-(1-(Fyr/(Rh)(Fyc))((lamda)f - (lamda)pf) / ((lamda)rf - (lamda)pf))(Rb)(Rh)(Fyc)

in which: (lamda)f = slenderness ratio for the compression flange = bfc / (2(tfc))

(lamda)pf = limiting slenderness ratio for a compact flange = 0.38(E/Fyc)^.5

(lamda)rf = limiting slenderness ratio for a noncompact flg. =0.56(E/Fyr)^.5

Fyc = yeild strenght of the compression flange = 50 ksi

Fyr = smaller of the compression flange stress at the onset of nominal yielding, with consideration of residual stress effects but without consideration of flange lateral bending, or the specified minimum yeild strenght of the web (ksi)

Fyr = 0.70(Fyc) <= Fyw, no smaller than Fyr = (0.70)(50) = 35 ksi

Yeild strength of web and both flanges is 50.00 ksi, therefore Fyr = 50.00 ksi

Rb = web load-shedding factor determined as specified in Article 6.10.1.10.2, which Rb = 1.00

Rh = hybrid factor determined as specified in Article 6.10.1.10.1, which Rh = 1.00

Art. 6.10.8.2.3 Lateral Torsional Buckling Resistance

For unbraced lenghts in which the member is prismatic, the lateral torsional buckling resistance of the compression flange shall be taken as :

If Lb <= Lp, then Fnc = (Rb)(Rh)(Fyc)

If Lp < Lb <= Lr, then Fnc = Cb(1-(1-(Fyr / (Rh)(Fyc))((Lb-Lp) / (Lr-Lp))(Rb)(Rh)(Fyc) <= (Rb)(Rh)(Fyc)

if Lb >= Lr, then Fnc = Fcr <= (Rb)(Rh)(Fyc)

which: Lb = unbraced length (in),

Lp = limiting unbraced length to achieve the nominal flexural resistance of (Rb)(Rh)(Fyc) under uniform bending = rt(E / Fyc)^.5

Lr = limiting unbraced length to achieve the onset of nominal yeilding in either flange under uniform bending with consideration of compression flg. Residual stress effects (in) which: Lr = (3.14)(rt)(E / Fyc)^.5

Cb = moment gradient modifier. In leiu of an altenative rational analysis, Cb may by calculated as follows:

For unbraced cantilevers and for members where fmid / f2 => 1 or f2 = 0.0 therefore: Cb = 1.0

For all other cases: Cb = 1.75 - 1.05(f1 / f2) + 0.3(f1 / f2)^2 <= 2.3

Fcr = elastic lateral torsional buckling stress = Cb(Rb)(pi^2)(E) / (Lb /rt)^2

rt = effective radius of gyration for lateral torsional buckling, rt = (bfc) / (12(1+(1(Dc)(tw) / (3(bfc)(tfc)))^.5

Dc = depth of the web in compression in the elastic range (in). For composite section, Dc, shall be determined as specified in Art. D6.3.1.

fmid = stress without consideration of lateral bending at the middle of the unbraced length of the flange under consideration, calculated from the moment envelope value that produces the largest compression at this point, or the smallest tension if this point is never in compression (ksi). (fmid) shall be due to the factored loads and shall be taken as positive in compression and negative in tension.

fo = stress without consideration of lateral bending at the brace point opposite to the one corresponding to f2, calculated from the moment envelope value that produces the largest compression at this point, or the smallest tension if this point is never in compression (ksi). (fo) shall be taken as positive in compression and negative n tension.

f1 = 2(fmid) - f2 => fo

f2 = except as noted below, largest compressive stress without consideration of lateral bending at either end of the unbraced length of the flange under consideration, calculated from the critical moment envelope (ksi). (f2) shall be due to the factored loads and shall be taken as positive. If the stress is zero or tensile in the flange under consideration at both ends of the unbraced length, f2 shall be taken as zero.

Compression-Flange Flexural Resistance and Lateral Torsional Buckling Resistance Check:

(assuming the entire slab is poured at one time)

Span Pt.	top flange		bottom flange		Comp. flg. slenderness ratios			rt	un-braced lengths			f1	f2	fo	Cb	Fnc	Fnc	*fl lateral	act.comp	total	total	check
	width	thickness	width	thickness	(lamda)f	(lamda)pf	(lamda)rf	top flg.	Lb	Lp	Lr					Lateral	Local	flg stress	flg Stress	fl+fbu<Fyc	fbu+(1/3)fl<Fnc	
1.0000	18.000	1.000	18.000	1.250	9.000	9.152	13.487	4.65	325.0	112.1	352.0	8.4	19.28	0.0	1.35	49.57	50.00	15.85	0.00	15.85	5.28	OK
1.0800	18.000	1.000	18.000	1.250	9.000	9.152	13.487	4.65	325.0	112.1	352.0	8.4	19.28	0.0	1.35	49.57	50.00	15.85	13.82	29.67	19.10	OK
1.1000	18.000	1.000	18.000	1.250	9.000	9.152	13.487	4.65	325.0	112.1	352.0	8.4	19.28	0.0	1.35	49.57	50.00	15.85	16.42	32.27	21.69	OK
1.1600	18.000	1.000	18.000	1.250	9.000	9.152	13.487	4.65	325.0	112.1	352.0	8.4	19.28	0.0	1.35	49.57	50.00	15.85	18.32	34.17	23.60	OK
1.1800	18.000	1.000	18.000	1.250	9.000	9.152	13.487	4.65	325.0	112.1	352.0	26.8	33.86	19.3	1.11	40.58	50.00	15.85	25.51	41.36	30.79	OK
1.1800	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	26.8	33.86	19.3	1.11	40.20	50.00	15.85	23.77	39.62	29.05	OK
1.2000	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	26.8	33.86	19.3	1.11	40.20	50.00	15.85	25.88	41.73	31.16	OK
1.2500	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	26.8	33.86	19.3	1.11	40.20	50.00	15.85	30.35	46.20	35.63	OK
1.3000	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	26.8	33.86	19.3	1.11	40.20	50.00	15.85	31.77	47.62	37.05	OK
1.3333	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	30.8	33.90	31.0	1.04	37.90	50.00	15.85	32.17	48.02	37.45	OK
1.4000	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	30.8	33.90	31.0	1.04	37.90	50.00	15.85	32.95	48.80	38.23	NG
1.4165	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	30.8	33.90	31.0	1.04	37.90	50.00	15.85	32.37	48.22	37.65	OK
1.5000	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	30.8	33.90	31.0	1.04	37.90	50.00	15.85	29.41	45.26	34.69	OK
1.5833	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	19.2	30.96	23.9	1.21	44.09	50.00	15.85	25.10	40.95	30.38	OK
1.6000	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	19.2	30.96	23.9	1.21	44.09	50.00	15.85	22.74	38.59	28.01	OK
1.6000	18.000	1.000	18.000	1.250	9.000	9.152	13.487	4.65	325.0	112.1	352.0	19.2	30.96	23.9	1.21	44.51	50.00	15.85	21.18	37.03	26.46	OK
1.6667	18.000	1.000	18.000	1.250	9.000	9.152	13.487	4.65	325.0	112.1	352.0	19.2	30.96	8.4	1.21	44.51	50.00	15.85	22.74	38.59	28.01	OK
1.7000	18.000	1.000	18.000	1.250	9.000	9.152	13.487	4.65	325.0	112.1	352.0	-7.7	23.90	8.4	1.42	50.00	50.00	12.68	7.81	20.48	12.03	OK
1.7000	24.000	1.063	24.000	1.063	11.294	9.152	13.487	6.39	325.0	153.8	483.1	-7.7	23.90	8.4	1.42	50.00	42.59	3.57	7.18	10.75	8.37	OK
1.7500	24.000	1.063	24.000	1.063	11.294	9.152	13.487	6.39	325.0	153.8	483.1	-7.7	23.90	8.4	1.42	50.00	42.59	3.57	8.09	11.66	9.28	OK
1.8000	24.000	1.063	24.000	1.063	11.294	9.152	13.487	6.39	325.0	153.8	483.1	-7.7	23.90	8.4	1.42	50.00	42.59	3.57	8.19	11.76	9.38	OK
1.8150	24.000	1.063	24.000	1.063	11.294	9.152	13.487	6.39	325.0	153.8	483.1	-7.7	23.90	8.4	1.42	50.00	42.59	3.57	11.11	14.68	12.30	OK
1.8150	24.000	2.000	24.000	2.000	6.000	9.152	13.487	6.62	325.0	159.5	501.0	-7.7	23.90	8.4	1.42	50.00	50.00	1.75	6.03	7.78	6.61	OK
1.8333	24.000	2.000	24.000	2.000	6.000	9.152	13.487	6.62	325.0	159.5	501.0	11.5	21.80	8.4	1.28	50.00	50.00	4.46	7.93	12.39	9.42	OK
1.9000	24.000	2.000	24.000	2.000	6.000	9.152	13.487	6.62	325.0	159.5	501.0	11.5	21.75	8.4	1.28	50.00	50.00	1.78	15.01	16.79	15.60	OK
1.9167	24.000	2.000	24.000	2.000	6.000	9.152	13.487	6.62	325.0	159.5	501.0	11.5	21.75	8.4	1.28	50.00	50.00	1.78	16.64	18.42	17.23	OK
1.9375	24.000	2.000	24.000	2.000	6.000	9.152	13.487	6.62	325.0	159.5	501.0	11.5	21.75	8.4	1.28	50.00	50.00	1.78	19.81	21.59	20.40	OK
1.9375	24.000	2.750	24.000	2.750	4.364	9.152	13.487	6.70	325.0	161.4	506.9	11.5	21.75	8.4	1.28	50.00	50.00	1.30	14.73	16.03	15.16	OK
2.000	24.000	2.750	24.000	2.750	4.364	9.152	13.487	6.70	325.0	161.4	506.9	11.5	21.75	8.4	1.28	50.00	50.00	3.24	20.67	23.90	21.75	OK

The dark grey shaded area denotes the flange that is in compression

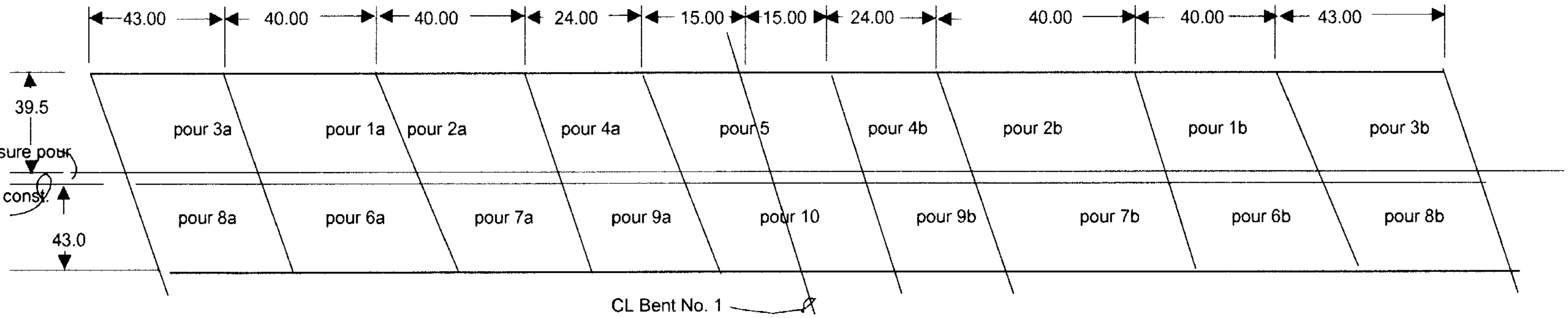
* The lateral flange bending stress for the forming braces for the cantilevers is only applied to the bottom flange when it is in compression and the top flange when it is in compression.

Art. 6.10.8.2.3 Lateral Torsional Buckling Resistance

*** NG, The stresses shown are the total stress from pouring the whole slab at once. The slab will be poured in sections according to a slab pouring sequence. Slab pouring sequence requires pouring the middle section first which would reduce the stresses below the allowable.

Constructibility Check (cont.)
State Route 45 Hickory Blvd.) over I-65
Davidson County
Date: September 30, 2003

Slab Pouring Sequence:



Span Pt.	top flange width	top flange thickness	bottom flange width	bottom flange thickness	dl beam TF stress	tf stress pour 1a	total stress	tf stress pour 1b	total stress	tf stress pour 2a	total stress	tf stress pour 2b	total stress	tf stress pour 3a	total stress	tf stress pour 3b	total stress	tf stress pour 4a	total stress	tf stress pour 4b	total stress	tf stress pour 5	total stress
1.000	18.000	1.000	18.000	1.250	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.100	18.000	1.000	18.000	1.250	2.01	4.05	7.19	-0.82	6.22	2.02	8.61	-0.97	7.46	5.35	13.82	-0.09	13.71	0.14	13.87	-0.10	13.75	0.00	13.75
1.180	18.000	1.000	18.000	1.250	3.20	7.34	12.52	-1.49	10.75	3.66	15.10	-1.76	13.01	7.38	21.78	-0.17	21.58	0.26	21.88	-0.19	21.66	0.00	21.67
1.180	18.000	1.000	18.000	2.250	2.98	6.84	11.66	-1.39	10.02	3.41	14.07	-1.64	12.12	6.88	20.29	-0.17	20.09	0.26	20.39	-0.19	20.17	0.00	20.18
1.200	18.000	1.000	18.000	2.250	3.19	7.55	12.75	-1.53	10.94	3.76	15.40	-1.81	13.25	7.02	21.59	-0.19	21.37	0.28	21.71	-0.21	21.47	0.00	21.47
1.300	18.000	1.000	18.000	2.250	3.88	11.11	17.79	-0.74	16.91	1.82	19.08	-0.88	18.04	2.05	20.47	-0.28	20.14	0.42	20.64	-0.31	20.28	0.01	20.29
1.400	18.000	1.000	18.000	2.250	3.87	12.30	19.21	-0.99	18.03	2.43	20.92	-1.17	19.53	1.63	21.47	-0.37	21.03	0.57	21.70	-0.41	21.22	0.01	21.23
1.500	18.000	1.000	18.000	2.250	3.18	10.55	16.30	-1.24	14.83	3.04	18.44	-1.46	16.71	1.22	18.16	-0.46	17.61	0.71	18.45	-0.51	17.84	0.01	17.85
1.545	18.000	1.000	18.000	2.250	2.64	9.05	13.88	-4.16	8.94	10.01	20.82	-1.59	18.93	1.04	20.16	-0.50	19.56	0.72	20.42	-0.56	19.76	0.01	19.77
1.545	18.000	1.000	18.000	1.250	2.84	9.71	14.90	-4.47	9.59	10.74	22.35	-1.60	20.45	1.04	21.69	-0.51	21.09	0.72	21.95	-0.56	21.28	0.01	21.29
1.600	18.000	1.000	18.000	1.250	1.95	7.50	11.22	-4.92	5.38	10.89	18.30	-1.76	16.21	0.81	17.17	-0.56	16.51	0.85	17.52	-0.62	16.79	0.01	16.80
1.700	18.000	1.000	18.000	1.250	-0.08	3.71	4.31	-5.73	-2.50	8.64	7.76	-2.05	5.33	0.40	5.81	-0.65	5.04	0.98	6.20	-0.72	5.35	0.02	5.37
1.700	24.000	1.063	24.000	1.063	-0.06	2.86	3.32	-4.42	-1.92	6.66	5.99	-1.87	3.76	0.37	4.20	-0.59	3.49	0.90	4.56	-0.66	3.78	0.02	3.79
1.800	24.000	1.063	24.000	1.063	-2.16	-0.16	-2.76	-5.06	-8.77	2.53	-5.76	-5.98	-12.85	-0.05	-12.91	-1.89	-15.16	2.71	-11.94	-0.66	-12.72	0.02	-12.70
1.815	24.000	1.063	24.000	1.063	-2.51	-0.62	-3.71	-5.16	-9.83	1.80	-7.70	-6.09	-14.93	-0.21	-15.18	-1.93	-17.47	2.51	-14.49	-0.77	-15.41	0.06	-15.33
1.815	24.000	2.000	24.000	2.000	-1.43	-0.36	-2.12	-2.94	-5.62	1.03	-4.39	-3.48	-8.53	-0.12	-8.67	-1.10	-9.98	1.43	-8.28	-0.62	-9.02	0.05	-8.96
1.900	24.000	2.000	24.000	2.000	-3.59	-2.59	-7.35	-3.25	-11.21	-1.14	-12.56	-3.84	-17.12	-0.63	-17.88	-1.22	-19.32	0.54	-18.68	-0.69	-19.50	0.02	-19.48
1.938	24.000	2.000	24.000	2.000	-3.51	-2.44	-7.07	-3.39	-11.09	-2.14	-13.64	-4.00	-18.39	-0.86	-19.42	-1.27	-20.92	-0.08	-21.01	-1.41	-22.69	-0.11	-22.82
1.938	24.000	2.750	24.000	2.750	-2.61	-1.82	-5.26	-2.52	-8.25	-1.59	-10.14	-2.98	-13.68	-0.64	-14.44	-0.94	-15.56	-0.06	-15.63	-1.05	-16.87	-0.08	-16.97
2.000	24.000	2.750	24.000	2.750	-3.59	-2.59	-7.35	-2.69	-10.54	-2.78	-13.84	-3.17	-17.60	-0.92	-18.69	-1.00	-19.89	-0.99	-21.06	-1.12	-22.38	-0.37	-22.83

Shaded area is previous poured slab in which the compression flange will become fully supported.

The over-stresses occur in section between cross-frames at section 1.333 to Section 1.500 in pour 1a
Revised compressive stresses in un-supported compression flange accounding for pouring slab in sections

Span Pt.	top flange width	top flange thickness	bottom flange width	bottom flange thickness	Comp. flg. slenderness ratios (lamda)f	(lamda)pf	(lamda)rf	rt	un-braced lengths Lb	Lp	Lr	f1	f2	fo	Cb	Fnc lateral	Fnc Local	*fl, lateral flg stress	fbu,act.comp. flg. Stress	total fbu+fl<=Fyc	total fbu+(1/3)fl<=Fnc	check	
1.3330	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	30.8	33.90	31.0	1.04	37.93	50.00	15.85	18.26	34.11	23.49	OK	braced point
1.4000	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	30.8	33.90	31.0	1.04	37.93	50.00	15.85	19.21	35.06	24.44	OK	
1.5000	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	30.8	33.90	31.0	1.04	37.93	50.00	15.85	16.30	32.15	21.53	OK	braced point

Constructibility Check (cont.)
State Rout' (Old Hickory Blvd.) over I-65
Davidson (y
Date: September 30, 2003

Check for the need for Amplifying first-order values of fl (Article 6.10.1.6)

Note: This only applies to the constructability check. Where the amplication will occur will be in the smaller top flanges that are in compression and unbraced during pouring of the slab. The wider cross-frame spacing will have an effect on the amount of the amplication of the calculated lateral flange bending stress.
The maximum lateral stress usually occurs at the cross-frame locations.

All discretely braced shall satisfy: fl <= 0.60(Fyt)
Flange lateral bending stresses may be determined directly from first-order elastic analysis in discretely braced compression flangess for which:

Lb <= 1.2 (Lp)((Cb)(Rb)) / (fbm / Fyc))^0.5

Cb = moment gradient modifier specified in Article 6.10.8.2.3 or Article A6.3.3, as applicalble
fbm = largest value of the compression stress in the flange under consideration, calculated without consideration of flange lateral bending, throughout the unbraced length (ksi), fbm shall be determined by factored loads.
Lb = unbraced length (in)
Lp = limiting unbraced length specified in Article 6.10.8.2.3 (in)
Fyc = yeild strenght of the compression flange
Rb = web load-shedding factor determined as specified in Article 6.10.1.10.2

If the above equations are not satisfied, second order elastic compression flange lateral bending stresses shall be determined
Second order compression flange lateral bending stresses may be determined by amplifying first order values as follows:

fl = (0.85 / (1 - fbm / Fcr)(fl1) => fl1

where :
fl1 = first order compression flange lateral bending stress at the section under consideration, or the maximum first order lateral bending stress in the compression flange under consideration throughout the unbraced length, as applicable (ksi)
Fcr = elastic lateral torsional buckling stress for the flange under consideration determined from Article 6.10.8.2.3 or A6.3.3 may only be applied for unbraced lenghts in which the web is compact or noncompact.

Fcr = (Cb)(Rb)((pi)^2)(E) / (Lb / rt)^2
rt = (btc) / (12(1 + .33(Dc)(tw)/((bfc)(tfc)))

Note: (fbm)max used for amplification factor calculation is based off the entire slab being pouring not accounting for the pouring sequence of the slab (Which gives a higher amplification factor). Using the lower stresses accounting for the sequenced pours, will lower amplification factor.

Span Pt.	top flange		bottom flange		Comp. flg. slenderness ratios			rt	un-braced lengths			Cb	Fcr	fbm	limiting	does factor	amplifying	*fl, lateral	amplified
	width	thickness	width	thickness	(lamda)f	(lamda)pf	(lamda)rf		Lb	Lp	Lr			max.	ratio **	apply	factor	flg stress	fl
1.0000	18.000	1.000	18.000	1.250	9.000	9.152	13.487	4.65	325.0	112.1	352.0	1.35	79.2	18.32	0.0	yes	1.11	15.85	17.52
1.0800	18.000	1.000	18.000	1.250	9.000	9.152	13.487	4.65	325.0	112.1	352.0	1.35	79.2	18.32	258.3	yes	1.11	15.85	17.52
1.1000	18.000	1.000	18.000	1.250	9.000	9.152	13.487	4.65	325.0	112.1	352.0	1.35	79.2	18.32	258.3	yes	1.11	15.85	17.52
1.1600	18.000	1.000	18.000	1.250	9.000	9.152	13.487	4.65	325.0	112.1	352.0	1.35	79.2	18.32	258.3	yes	1.11	15.85	17.52
1.1800	18.000	1.000	18.000	1.250	9.000	9.152	13.487	4.65	325.0	112.1	352.0	1.11	64.9	32.17	176.4	yes	1.69	15.85	26.72
1.1800	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	1.11	62.7	32.17	173.4	yes	1.75	15.85	27.66
1.2000	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	1.11	62.7	32.17	173.4	yes	1.75	15.85	27.66
1.2500	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	1.11	62.7	32.17	173.4	yes	1.75	15.85	27.66
1.3000	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	1.11	62.7	32.17	173.4	yes	1.75	15.85	27.66
1.3333	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	1.04	59.1	32.17	168.4	yes	1.86	15.85	29.55
1.4000	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	1.04	59.1	32.95	166.4	yes	1.92	15.85	30.43
1.4165	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	1.04	59.1	32.95	166.4	yes	1.92	15.85	30.43
1.5000	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	1.04	59.1	32.37	167.9	yes	1.88	15.85	29.77
1.5833	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	1.21	68.8	32.37	181.0	yes	1.61	15.85	25.45
1.6000	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	1.21	68.8	32.37	181.0	yes	1.61	15.85	25.45
1.6000	18.000	1.000	18.000	1.250	9.000	9.152	13.487	4.65	325.0	112.1	352.0	1.21	71.2	32.37	184.2	yes	1.56	15.85	24.72
1.6667	18.000	1.000	18.000	1.250	9.000	9.152	13.487	4.65	325.0	112.1	352.0	1.21	71.2	22.74	219.7	yes	1.25	15.85	19.80
1.7000	18.000	1.000	18.000	1.250	9.000	9.152	13.487	4.65	325.0	112.1	352.0	1.42	83.3	22.74	237.7	yes	1.17	12.68	14.83
1.7000	24.000	1.063	24.000	1.063	11.294	9.152	13.487	6.39	325.0	153.8	483.1	1.42	156.8	22.74	326.2	no	1.00	8.91	8.91
1.7500	24.000	1.063	24.000	1.063	11.294	9.152	13.487	6.39	325.0	153.8	483.1	1.42	156.8	22.74	326.2	no	1.00	8.91	8.91
1.8000	24.000	1.063	24.000	1.063	11.294	9.152	13.487	6.39	325.0	153.8	483.1	1.42	156.8	22.74	326.2	no	1.00	8.91	8.91
1.8150	24.000	1.063	24.000	1.063	11.294	9.152	13.487	6.39	325.0	153.8	483.1	1.42	156.8	22.74	326.2	no	1.00	8.91	8.91
1.8150	24.000	2.000	24.000	2.000	6.000	9.152	13.487	6.62	325.0	159.5	501.0	1.42	168.7	22.74	338.3	no	1.00	4.46	4.46
1.8333	24.000	2.000	24.000	2.000	6.000	9.152	13.487	6.62	325.0	159.5	501.0	1.28	152.1	20.67	337.0	no	1.00	4.46	4.46
1.9000	24.000	2.000	24.000	2.000	6.000	9.152	13.487	6.62	325.0	159.5	501.0	1.28	151.8	20.67	336.6	no	1.00	4.46	4.46
1.9167	24.000	2.000	24.000	2.000	6.000	9.152	13.487	6.62	325.0	159.5	501.0	1.28	151.8	20.67	336.6	no	1.00	4.46	4.46
1.9375	24.000	2.000	24.000	2.000	6.000	9.152	13.487	6.62	325.0	159.5	501.0	1.28	151.8	20.67	336.6	no	1.00	4.46	4.46
1.9375	24.000	2.750	24.000	2.750	4.364	9.152	13.487	6.70	325.0	161.4	506.9	1.28	155.4	20.67	340.6	no	1.00	3.24	3.24
2.000	24.000	2.750	24.000	2.750	4.364	9.152	13.487	6.70	325.0	161.4	506.9	1.28	155.4	20.67	340.6	no	1.00	3.24	3.24

From the Pouring Sequence: The over-stresses occur in section between cross-frames at section 1.333 to Section 1.500 in pour 1a
Revised compressive stresses in un-supported compression flange accounding for pouring slab in sections

Span Pt.	top fange		bottom flange		Comp. flg. slenderness ratios			rt	un-braced lengths			f1	f2	fo	Cb	Fnc	Fnc	*fl, lateral	fbu, act.comp	total	total	check
	width	thickness	width	thickness	(lamda)f	(lamda)pf	(lamda)rf		Lb	Lp	Lr					lateral	Local	flg stress	flg. Stress	fl+fbu<=Fyc	fbu+(1/3)fl<=Fnc	
1.3333	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	30.84	33.90	30.96	1.04	37.90	50.00	29.55	18.26	47.81	28.01	OK
1.4000	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	30.84	33.90	30.96	1.04	37.90	50.00	30.43	19.21	49.64	29.34	OK
1.5000	18.000	1.000	18.000	2.250	9.000	9.152	13.487	4.58	325.0	110.2	346.1	30.84	33.90	30.96	1.04	37.90	50.00	29.77	16.30	46.07	26.12	OK

All discretely braced shall satisfy: fl <= 0.60(Fyt)
0.60(Fyt) = 30.00 ksi

maximum fl = 30.43 ksi at span point 1.40, which is less than
which : 30.43 ksi, is close enough to 30.00 ksi, say OK see above note.

84A

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$$EGAD422100000000000 Old Hickory Blvd./ I-65      HENRY PATE
*B010001   SPANS 162.5-162.5,  pour no 5
1STEEL I-BEAM2HS20000 0000 0000 000000002900111700 24350
2000010000100001 927    500000 50000 650000000000000
31 162500 1 4 6 5      5 5  1
400 162500
421  00000
422  00000
423  54000
424    500
425   0000
431 113500 132500 152500 162500
432  18000  24000  24000  24000
433   1000   1063   2000   2750
441  29500  88500 113500 132500 152500 162500
442  18000  18000  18000  24000  24000  24000
443   1250   2250   1250   1063   2000   2750
451  43000  83000 123000 147000 162500
452 111000 111000 111000 111000  00000
453   9000   9000   9000   9000  00000
454   3250   3250   3250   3250  00000
455   0000   0000   0000   0000  00000
481  43000  83000 123000 147000 162500
482   0000   0000   0000   0000  00000
491  43000  83000 123000 147000 162500
492   0000   0000   0000   0000   1240
32 162500 1 4 6 5      5 5  1
400 162500
421  00000
422  00000
423  54000
424    500
425   0000
431  10000  30000  49000 162500
432  24000  24000  24000  18000
433   2750   2000   1063   1000
441  10000  30000  49000  74000 133000 162500
442  24000  24000  24000  18000  18000  18000
443   2750   2000   1063   1250   2250   1250
451  15000  39000  79000 119000 162500
452 000000 000000 000000 000000  00000
453   0000 111000 111000 111000 111000
454   0000   9000   9000   9000   9000
455   0000   3250   3250   3250   3250
481  15000  39000  79000 119000 162500
482   0000   0000   0000   0000  00000
491  15000  39000  79000 119000 162500
492   1240   0000   0000   0000   0000
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STEEL I-BEAM **Dead Load Beam** STRESSES AT SPAN TENTH AND MISCELLANEOUS POINTS

* ACTUAL STRESSES(PSI) * ACTUAL RANGE IN STRESSES (PSI)

	TOTAL DEAD LOAD			D.L. + L.L. POS.			D.L. + L.L. NEG.			*	
S.PT.	TOP CONC.	TOP STEEL	BOT. STEEL	TOP CONC.	TOP STEEL	BOT. STEEL	TOP CONC.	TOP STEEL	BOT. STEEL	TOP STEEL	BOT. STEEL
1.100	0	2006	-1771	0	2007	-1771	0	2006	-1771	1	0
1.200	0	3191	-1950	0	3192	-1951	0	3191	-1950	1	1
1.300	0	3878	-2370	0	3879	-2370	0	3877	-2370	2	0
1.400	0	3873	-2367	0	3874	-2368	0	3873	-2367	1	1
1.500	0	3177	-1941	0	3178	-1942	0	3176	-1941	2	1
1.600	0	1951	-1723	0	1952	-1724	0	1951	-1722	1	2
1.700	0	-80	80	0	-80	80	0	-81	81	1	1
1.800	0	-2160	2160	0	-2159	2159	0	-2160	2160	1	1
1.900	0	-2785	2785	0	-2785	2785	0	-2786	2786	1	1
2.000	0	-3593	3593	0	-3593	3593	0	-3594	3594	1	1
2.100	0	-2785	2785	0	-2785	2785	0	-2786	2786	1	1
2.200	0	-2160	2160	0	-2159	2159	0	-2160	2160	1	1
2.300	0	-80	80	0	-80	80	0	-81	81	1	1
2.400	0	1951	-1723	0	1952	-1724	0	1951	-1722	1	2
2.500	0	3177	-1941	0	3178	-1942	0	3176	-1941	2	1
2.600	0	3873	-2367	0	3874	-2368	0	3873	-2367	1	1
2.700	0	3878	-2370	0	3879	-2370	0	3877	-2370	2	0
2.800	0	3191	-1950	0	3192	-1951	0	3191	-1950	1	1
2.900	0	2006	-1771	0	2007	-1771	0	2006	-1771	1	0
1.182	0	3202	-2827	0	3203	-2827	0	3202	-2827	1	0
1.182	0	2983	-1823	0	2984	-1824	0	2983	-1823	1	1
1.265	0	3708	-2266	0	3709	-2267	0	3708	-2266	1	1
1.265	0	3708	-2266	0	3709	-2267	0	3708	-2266	1	1
1.511	0	3047	-1862	0	3049	-1863	0	3047	-1862	2	1
1.511	0	3047	-1862	0	3049	-1863	0	3047	-1862	2	1
1.545	0	2644	-1616	0	2645	-1617	0	2644	-1616	1	1
1.545	0	2838	-2506	0	2839	-2507	0	2838	-2505	1	2
1.698	0	-74	65	0	-73	65	0	-75	66	2	1
1.698	0	-57	57	0	-56	56	0	-58	58	2	2
1.757	0	-1197	1197	0	-1197	1197	0	-1198	1198	1	1
1.757	0	-1197	1197	0	-1197	1197	0	-1198	1198	1	1
1.815	0	-2505	2505	0	-2505	2505	0	-2506	2506	1	1
1.815	0	-1430	1430	0	-1430	1430	0	-1431	1431	1	1
1.905	0	-2869	2869	0	-2869	2869	0	-2869	2869	0	0
1.905	0	-2869	2869	0	-2869	2869	0	-2869	2869	0	0
1.938	0	-3514	3514	0	-3514	3514	0	-3515	3515	1	1
1.938	0	-2613	2613	0	-2613	2613	0	-2613	2613	0	0
2.062	0	-2613	2613	0	-2613	2613	0	-2613	2613	0	0
2.062	0	-3514	3514	0	-3514	3514	0	-3515	3515	1	1
2.092	0	-2925	2925	0	-2925	2925	0	-2926	2926	1	1
2.092	0	-2925	2925	0	-2925	2925	0	-2926	2926	1	1
2.185	0	-1430	1430	0	-1430	1430	0	-1431	1431	1	1
2.185	0	-2505	2505	0	-2505	2505	0	-2506	2506	1	1
2.240	0	-1264	1264	0	-1264	1264	0	-1265	1265	1	1
2.240	0	-1264	1264	0	-1264	1264	0	-1265	1265	1	1
2.302	0	-57	57	0	-56	56	0	-58	58	2	2
2.302	0	-74	65	0	-73	65	0	-75	66	2	1
2.455	0	2838	-2506	0	2839	-2507	0	2838	-2505	1	2
2.455	0	2644	-1616	0	2645	-1617	0	2644	-1616	1	1
2.486	0	3014	-1842	0	3015	-1842	0	3013	-1841	2	1
2.486	0	3014	-1842	0	3015	-1842	0	3013	-1841	2	1
2.732	0	3725	-2277	0	3726	-2277	0	3725	-2277	1	0
2.732	0	3725	-2277	0	3726	-2277	0	3725	-2277	1	0
2.818	0	2983	-1823	0	2984	-1824	0	2983	-1823	1	1
2.818	0	3202	-2827	0	3203	-2827	0	3202	-2827	1	0

STEEL I-BEAM **Pour No. 1** STRESSES AT SPAN TENTH AND MISCELLANEOUS POINTS

* ACTUAL STRESSES(PSI) * ACTUAL RANGE IN STRESSES (PSI)

	TOTAL DEAD LOAD			D.L. + L.L. POS.			D.L. + L.L. NEG. *				
S.PT.	TOP CONC.	TOP STEEL	BOT. STEEL	TOP CONC.	TOP STEEL	BOT. STEEL	TOP CONC.	TOP STEEL	BOT. STEEL	TOP STEEL	BOT. STEEL
1.100	0	4050	-3576	0	4051	-3576	0	4050	-3576	1	0
1.200	0	7547	-4613	0	7548	-4613	0	7547	-4613	1	0
1.300	0	11106	-6788	0	11107	-6788	0	11106	-6788	1	0
1.400	0	12302	-7518	0	12303	-7519	0	12301	-7518	2	1
1.500	0	10550	-6448	0	10551	-6449	0	10550	-6448	1	1
1.600	0	7500	-6621	0	7501	-6622	0	7499	-6621	2	1
1.700	0	2807	-2807	0	2807	-2807	0	2806	-2806	1	1
1.800	0	-164	164	0	-164	164	0	-165	165	1	1
1.900	0	-1791	1791	0	-1791	1791	0	-1792	1792	1	1
2.000	0	-2594	2594	0	-2594	2594	0	-2595	2595	1	1
2.100	0	-3140	3140	0	-3139	3139	0	-3140	3140	1	1
2.200	0	-4887	4887	0	-4886	4886	0	-4887	4887	1	1
2.300	0	-4276	4276	0	-4275	4275	0	-4276	4276	1	1
2.400	0	-4756	4199	0	-4755	4198	0	-4757	4199	2	1
2.500	0	-3693	2257	0	-3692	2256	0	-3693	2257	1	1
2.600	0	-2954	1805	0	-2953	1805	0	-2954	1805	1	0
2.700	0	-2215	1354	0	-2214	1353	0	-2216	1354	2	1
2.800	0	-1477	902	0	-1476	902	0	-1477	902	1	0
2.900	0	-792	699	0	-792	699	0	-792	699	0	0
1.182	0	7340	-6480	0	7341	-6481	0	7340	-6480	1	1
1.182	0	6838	-4179	0	6839	-4180	0	6838	-4179	1	1
1.265	0	9900	-6051	0	9901	-6051	0	9900	-6051	1	0
1.265	0	9900	-6051	0	9901	-6051	0	9900	-6051	1	0
1.511	0	10001	-6112	0	10002	-6113	0	10001	-6112	1	1
1.511	0	10001	-6112	0	10002	-6113	0	10001	-6112	1	1
1.545	0	9046	-5528	0	9047	-5529	0	9045	-5528	2	1
1.545	0	9709	-8571	0	9710	-8572	0	9708	-8571	2	1
1.698	0	3707	-3273	0	3708	-3273	0	3706	-3272	2	1
1.698	0	2856	-2856	0	2857	-2857	0	2856	-2856	1	1
1.757	0	1115	-1115	0	1115	-1115	0	1114	-1114	1	1
1.757	0	1115	-1115	0	1115	-1115	0	1114	-1114	1	1
1.815	0	-621	621	0	-621	621	0	-622	622	1	1
1.815	0	-355	355	0	-355	355	0	-355	355	0	0
1.905	0	-1869	1869	0	-1869	1869	0	-1870	1870	1	1
1.905	0	-1869	1869	0	-1869	1869	0	-1870	1870	1	1
1.938	0	-2444	2444	0	-2444	2444	0	-2445	2445	1	1
1.938	0	-1817	1817	0	-1817	1817	0	-1818	1818	1	1
2.062	0	-2434	2434	0	-2434	2434	0	-2435	2435	1	1
2.062	0	-3274	3274	0	-3274	3274	0	-3274	3274	0	0
2.092	0	-3166	3166	0	-3166	3166	0	-3167	3167	1	1
2.092	0	-3166	3166	0	-3166	3166	0	-3167	3167	1	1
2.185	0	-2844	2844	0	-2844	2844	0	-2845	2845	1	1
2.185	0	-4981	4981	0	-4980	4980	0	-4981	4981	1	1
2.240	0	-4642	4642	0	-4642	4642	0	-4643	4643	1	1
2.240	0	-4642	4642	0	-4642	4642	0	-4643	4643	1	1
2.302	0	-4266	4266	0	-4266	4266	0	-4267	4267	1	1
2.302	0	-5537	4888	0	-5536	4887	0	-5537	4889	1	2
2.455	0	-4317	3811	0	-4316	3810	0	-4318	3812	2	2
2.455	0	-4022	2458	0	-4021	2457	0	-4023	2458	2	1
2.486	0	-3795	2319	0	-3794	2319	0	-3795	2319	1	0
2.486	0	-3795	2319	0	-3794	2319	0	-3795	2319	1	0
2.732	0	-1977	1208	0	-1976	1207	0	-1977	1208	1	1
2.732	0	-1977	1208	0	-1976	1207	0	-1977	1208	1	1
2.818	0	-1340	819	0	-1340	819	0	-1341	819	1	0

STEEL I-BEAM **Pour No. 1a** STRESSES AT SPAN TENTH AND MISCELLANEOUS POINTS

	*	ACTUAL STRESSES(PSI)								*	ACTUAL RANGE IN STRESSES (PSI)	
	TOTAL DEAD LOAD			D.L. + L.L. POS.			D.L. + L.L. NEG.			*		
S.PT.	TOP CONC.	TOP STEEL	BOT. STEEL	TOP CONC.	TOP STEEL	BOT. STEEL	TOP CONC.	TOP STEEL	BOT. STEEL		TOP STEEL	BOT. STEEL
1.100	0	-820	724	0	-820	723	0	-820	724		0	1
1.200	0	-1529	934	0	-1528	933	0	-1529	934		1	1
1.300	-41	-742	1124	-41	-741	1123	-41	-742	1124		1	1
1.400	-54	-989	1499	-54	-989	1498	-54	-989	1499		0	1
1.500	-68	-1236	1873	-68	-1236	1873	-68	-1236	1874		0	1
1.600	0	-4923	4346	0	-4922	4345	0	-4923	4347		1	2
1.700	0	-4425	4425	0	-4425	4425	0	-4426	4426		1	1
1.800	0	-5058	5058	0	-5057	5057	0	-5058	5058		1	1
1.900	0	-3250	3250	0	-3250	3250	0	-3250	3250		0	0
2.000	0	-2685	2685	0	-2685	2685	0	-2686	2686		1	1
2.100	0	-1901	1901	0	-1901	1901	0	-1902	1902		1	1
2.200	0	-335	335	0	-335	335	0	-336	336		1	1
2.300	0	2657	-2657	0	2657	-2657	0	2656	-2656		1	1
2.400	0	7333	-6474	0	7334	-6475	0	7333	-6473		1	2
2.500	0	10421	-6369	0	10422	-6369	0	10420	-6369		2	0
2.600	0	12198	-7455	0	12199	-7456	0	12198	-7455		1	1
2.700	0	11029	-6740	0	11030	-6741	0	11029	-6740		1	1
2.800	0	7496	-4581	0	7496	-4581	0	7495	-4581		1	0
2.900	0	4022	-3551	0	4023	-3552	0	4022	-3551		1	1
1.182	0	-1489	1315	0	-1488	1314	0	-1489	1315		1	1
1.182	0	-1387	848	0	-1387	847	0	-1388	848		1	1
1.265	0	-2022	1236	0	-2021	1235	0	-2023	1236		2	1
1.265	-36	-654	991	-36	-654	991	-36	-654	.991		0	0
1.511	-69	-1263	1914	-69	-1263	1913	-69	-1263	1914		0	1
1.511	0	-3904	2386	0	-3903	2385	0	-3905	2386		2	1
1.545	0	-4163	2544	0	-4162	2544	0	-4164	2545		2	1
1.545	0	-4468	3945	0	-4467	3944	0	-4469	3945		2	1
1.698	0	-5731	5059	0	-5730	5059	0	-5732	5060		2	1
1.698	0	-4416	4416	0	-4415	4415	0	-4416	4416		1	1
1.757	0	-4785	4785	0	-4785	4785	0	-4786	4786		1	1
1.757	0	-4785	4785	0	-4785	4785	0	-4786	4786		1	1
1.815	0	-5155	5155	0	-5155	5155	0	-5156	5156		1	1
1.815	0	-2944	2944	0	-2944	2944	0	-2944	2944		0	0
1.905	0	-3266	3266	0	-3266	3266	0	-3267	3267		1	1
1.905	0	-3266	3266	0	-3266	3266	0	-3267	3267		1	1
1.938	0	-3388	3388	0	-3388	3388	0	-3389	3389		1	1
1.938	0	-2520	2520	0	-2520	2520	0	-2520	2520		0	0
2.062	0	-1903	1903	0	-1903	1903	0	-1903	1903		0	0
2.062	0	-2559	2559	0	-2559	2559	0	-2559	2559		0	0
2.092	0	-2033	2033	0	-2032	2032	0	-2033	2033		1	1
2.092	0	-2033	2033	0	-2032	2032	0	-2033	2033		1	1
2.185	0	-454	454	0	-454	454	0	-455	455		1	1
2.185	0	-796	796	0	-796	796	0	-797	797		1	1
2.240	0	861	-861	0	861	-861	0	860	-860		1	1
2.240	0	861	-861	0	861	-861	0	860	-860		1	1
2.302	0	2707	-2707	0	2707	-2707	0	2706	-2706		1	1
2.302	0	3513	-3101	0	3514	-3102	0	3512	-3101		2	1
2.455	0	9558	-8438	0	9559	-8439	0	9557	-8437		2	2
2.455	0	8905	-5442	0	8906	-5443	0	8904	-5442		2	1
2.486	0	9786	-5981	0	9787	-5981	0	9785	-5980		2	1
2.486	0	9786	-5981	0	9787	-5981	0	9785	-5980		2	1
2.732	0	9914	-6059	0	9915	-6060	0	9914	-6059		1	1
2.732	0	9914	-6059	0	9915	-6060	0	9914	-6059		1	1
2.818	0	6791	-4151	0	6792	-4151	0	6791	-4151		1	0
2.818	0	7289	-6435	0	7290	-6436	0	7289	-6435		1	1

STEEL I-BEAM **Pour No. 2** STRESSES AT SPAN TENTH AND MISCELLANEOUS POINTS

	* ACTUAL STRESSES(PSI)								*	ACTUAL RANGE IN STRESSES (PSI)	
	TOTAL DEAD LOAD			D.L. + L.L. POS.			D.L. + L.L. NEG. *				
S.PT.	TOP CONC.	TOP STEEL	BOT. STEEL	TOP CONC.	TOP STEEL	BOT. STEEL	TOP CONC.	TOP STEEL	BOT. STEEL	TOP STEEL	BOT. STEEL
1.100	0	2017	-1781	0	2017	-1781	0	2017	-1780	0	1
1.200	0	3759	-2297	0	3759	-2297	0	3758	-2297	1	0
1.300	100	1824	-2764	100	1824	-2764	100	1824	-2764	0	0
1.400	134	2432	-3685	134	2432	-3686	134	2432	-3685	0	1
1.500	168	3040	-4607	168	3040	-4607	168	3040	-4607	0	0
1.600	0	10885	-9609	0	10886	-9610	0	10884	-9609	2	1
1.700	0	6590	-6590	0	6591	-6591	0	6590	-6590	1	1
1.800	0	2534	-2534	0	2534	-2534	0	2533	-2533	1	1
1.900	0	-1144	1144	0	-1144	1144	0	-1145	1145	1	1
2.000	0	-2779	2779	0	-2779	2779	0	-2779	2779	0	0
2.100	0	-3363	3363	0	-3363	3363	0	-3363	3363	0	0
2.200	0	-5234	5234	0	-5234	5234	0	-5235	5235	1	1
2.300	0	-4580	4580	0	-4579	4579	0	-4580	4580	1	1
2.400	0	-5095	4498	0	-5094	4497	0	-5095	4498	1	1
2.500	-377	-2155	1711	-377	-2154	1711	-377	-2155	1712	1	1
2.600	-302	-1724	1369	-301	-1723	1369	-302	-1724	1369	1	0
2.700	-226	-1293	1027	-226	-1292	1026	-226	-1293	1027	1	1
2.800	0	-1582	967	0	-1581	966	0	-1582	967	1	1
2.900	0	-849	749	0	-848	749	0	-849	749	1	0
1.182	0	3662	-3233	0	3663	-3233	0	3662	-3233	1	0
1.182	0	3412	-2085	0	3412	-2085	0	3411	-2085	1	0
1.265	0	4973	-3039	0	4974	-3040	0	4973	-3039	1	1
1.265	89	1609	-2438	89	1609	-2438	89	1609	-2438	0	0
1.511	175	3165	-4796	175	3166	-4797	175	3165	-4796	1	1
1.511	0	9784	-5980	0	9785	-5980	0	9784	-5979	1	1
1.545	0	10008	-6117	0	10009	-6117	0	10008	-6116	1	1
1.545	0	10742	-9483	0	10743	-9485	0	10742	-9483	1	2
1.698	0	8640	-7628	0	8641	-7629	0	8640	-7627	1	2
1.698	0	6658	-6658	0	6658	-6658	0	6657	-6657	1	1
1.757	0	4486	-4486	0	4487	-4487	0	4486	-4486	1	1
1.757	0	4486	-4486	0	4487	-4487	0	4486	-4486	1	1
1.815	0	1801	-1801	0	1801	-1801	0	1801	-1801	0	0
1.815	0	1028	-1028	0	1029	-1029	0	1028	-1028	1	1
1.905	0	-1264	1264	0	-1264	1264	0	-1264	1264	0	0
1.905	0	-1264	1264	0	-1264	1264	0	-1264	1264	0	0
1.938	0	-2141	2141	0	-2141	2141	0	-2142	2142	1	1
1.938	0	-1592	1592	0	-1592	1592	0	-1593	1593	1	1
2.062	0	-2608	2608	0	-2608	2608	0	-2608	2608	0	0
2.062	0	-3507	3507	0	-3507	3507	0	-3507	3507	0	0
2.092	0	-3392	3392	0	-3392	3392	0	-3392	3392	0	0
2.092	0	-3392	3392	0	-3392	3392	0	-3392	3392	0	0
2.185	0	-3047	3047	0	-3047	3047	0	-3047	3047	0	0
2.185	0	-5335	5335	0	-5334	5334	0	-5335	5335	1	1
2.240	0	-4972	4972	0	-4972	4972	0	-4973	4973	1	1
2.240	0	-4972	4972	0	-4972	4972	0	-4973	4973	1	1
2.302	0	-4570	4570	0	-4569	4569	0	-4570	4570	1	1
2.302	0	-5931	5236	0	-5930	5235	0	-5931	5236	1	1
2.455	0	-4624	4082	0	-4623	4081	0	-4625	4083	2	2
2.455	0	-4308	2633	0	-4307	2632	0	-4309	2633	2	1
2.486	0	-4065	2484	0	-4064	2484	0	-4065	2484	1	0
2.486	-387	-2214	1759	-387	-2214	1758	-388	-2215	1759	1	1
2.732	-202	-1153	916	-201	-1153	916	-202	-1153	916	0	0
2.732	0	-2117	1294	0	-2116	1293	0	-2118	1294	2	1
2.818	0	-1436	877	0	-1435	877	0	-1436	877	1	0
2.818	0	-1541	1360	0	-1540	1360	0	-1541	1361	1	1

STEEL I-BEAM **Pour No. 2a** STRESSES AT SPAN TENTH AND MISCELLANEOUS POINTS

* ACTUAL STRESSES(PSI) * ACTUAL RANGE IN STRESSES (PSI)

	TOTAL DEAD LOAD			D.L. + L.L. POS.			D.L. + L.L. NEG. *			ACTUAL RANGE IN STRESSES (PSI)	
S.PT.	TOP CONC.	TOP STEEL	BOT. STEEL	TOP CONC.	TOP STEEL	BOT. STEEL	TOP CONC.	TOP STEEL	BOT. STEEL	TOP STEEL	BOT. STEEL
1.100	0	-969	855	0	-969	855	0	-969	856	0	1
1.200	0	-1806	1104	0	-1805	1103	0	-1806	1104	1	1
1.300	-48	-876	1328	-48	-876	1328	-48	-876	1328	0	0
1.400	-64	-1169	1771	-64	-1168	1770	-64	-1169	1771	1	1
1.500	-80	-1461	2214	-80	-1461	2213	-80	-1461	2214	0	1
1.600	-108	-1760	4095	-108	-1759	4094	-108	-1760	4096	1	2
1.700	-115	-1878	4364	-115	-1878	4364	-115	-1878	4365	0	1
1.800	0	-5976	5976	0	-5976	5976	0	-5977	5977	1	1
1.900	0	-3840	3840	0	-3840	3840	0	-3840	3840	0	0
2.000	0	-3173	3173	0	-3173	3173	0	-3173	3173	0	0
2.100	0	-1621	1621	0	-1621	1621	0	-1622	1622	1	1
2.200	0	1792	-1792	0	1792	-1792	0	1791	-1791	1	1
2.300	0	5941	-5941	0	5941	-5941	0	5940	-5940	1	1
2.400	0	10162	-8972	0	10163	-8972	0	10162	-8971	1	1
2.500	843	4814	-3824	843	4814	-3824	843	4814	-3824	0	0
2.600	674	3851	-3059	674	3852	-3059	674	3851	-3059	1	0
2.700	506	2888	-2294	506	2889	-2294	505	2888	-2294	1	0
2.800	0	3534	-2160	0	3535	-2160	0	3534	-2160	1	0
2.900	0	1896	-1674	0	1897	-1675	0	1896	-1674	1	1
1.182	0	-1760	1553	0	-1759	1553	0	-1760	1554	1	1
1.182	0	-1639	1002	0	-1639	1001	0	-1640	1002	1	1
1.265	0	-2390	1460	0	-2389	1460	0	-2390	1461	1	1
1.265	-42	-773	1171	-42	-773	1171	-42	-773	1171	0	0
1.511	-82	-1492	2262	-82	-1492	2261	-82	-1492	2262	0	1
1.511	-82	-1492	2262	-82	-1492	2261	-82	-1492	2262	0	1
1.545	-88	-1591	2411	-88	-1591	2411	-88	-1591	2412	0	1
1.545	-98	-1597	3717	-98	-1597	3716	-98	-1597	3717	0	1
1.698	-126	-2048	4767	-126	-2048	4767	-126	-2048	4768	0	1
1.698	-115	-1873	4355	-115	-1873	4354	-115	-1874	4355	1	1
1.757	-125	-2030	4719	-125	-2030	4719	-125	-2030	4720	0	1
1.757	0	-5655	5655	0	-5654	5654	0	-5655	5655	1	1
1.815	0	-6091	6091	0	-6091	6091	0	-6092	6092	1	1
1.815	0	-3479	3479	0	-3479	3479	0	-3479	3479	0	0
1.905	0	-3860	3860	0	-3859	3859	0	-3860	3860	1	1
1.905	0	-3860	3860	0	-3859	3859	0	-3860	3860	1	1
1.938	0	-4004	4004	0	-4004	4004	0	-4004	4004	0	0
1.938	0	-2977	2977	0	-2977	2977	0	-2978	2978	1	1
2.062	0	-1962	1962	0	-1962	1962	0	-1963	1963	1	1
2.062	0	-2639	2639	0	-2639	2639	0	-2639	2639	0	0
2.092	0	-1825	1825	0	-1825	1825	0	-1825	1825	0	0
2.092	0	-1825	1825	0	-1825	1825	0	-1825	1825	0	0
2.185	0	596	-596	0	597	-597	0	596	-596	1	1
2.185	0	1045	-1045	0	1045	-1045	0	1044	-1044	1	1
2.240	0	3669	-3669	0	3669	-3669	0	3669	-3669	0	0
2.240	0	3669	-3669	0	3669	-3669	0	3669	-3669	0	0
2.302	0	6010	-6010	0	6010	-6010	0	6009	-6009	1	1
2.302	0	7799	-6886	0	7800	-6886	0	7799	-6885	1	1
2.455	0	10086	-8905	0	10087	-8905	0	10086	-8904	1	1
2.455	0	9397	-5743	0	9398	-5744	0	9397	-5743	1	1
2.486	0	9238	-5646	0	9239	-5646	0	9237	-5646	2	0
2.486	881	5033	-3997	881	5033	-3998	881	5032	-3997	1	1
2.732	451	2577	-2047	451	2578	-2047	451	2577	-2047	1	0
2.732	0	4731	-2891	0	4732	-2892	0	4730	-2891	2	1
2.818	0	3208	-1960	0	3209	-1961	0	3208	-1960	1	1
2.818	0	3443	-3040	0	3444	-3041	0	3443	-3040	1	1

STEEL I-BEAM **Pour No. 3** STRESSES AT SPAN TENTH AND MISCELLANEOUS POINTS

* ACTUAL STRESSES(PSI) * ACTUAL RANGE IN STRESSES (PSI)

	TOTAL DEAD LOAD			D.L. + L.L. POS.			D.L. + L.L. NEG.			*	
S.PT.	TOP CONC.	TOP STEEL	BOT. STEEL	TOP CONC.	TOP STEEL	BOT. STEEL	TOP CONC.	TOP STEEL	BOT. STEEL	TOP STEEL	BOT. STEEL
1.100	0	5350	-4723	0	5350	-4723	0	5349	-4723	1	0
1.200	0	7022	-4291	0	7022	-4292	0	7021	-4291	1	1
1.300	113	2047	-3102	113	2047	-3102	113	2047	-3101	0	1
1.400	90	1633	-2475	90	1634	-2476	90	1633	-2475	1	1
1.500	67	1220	-1849	67	1220	-1849	67	1220	-1849	0	0
1.600	49	809	-1884	49	810	-1885	49	809	-1884	1	1
1.700	22	361	-839	22	361	-840	22	361	-839	0	1
1.800	0	-50	50	0	-50	50	0	-51	51	1	1
1.900	0	-632	632	0	-632	632	0	-633	633	1	1
2.000	0	-919	919	0	-919	919	0	-919	919	0	0
2.100	0	-1112	1112	0	-1112	1112	0	-1112	1112	0	0
2.200	0	-1731	1731	0	-1731	1731	0	-1731	1731	0	0
2.300	-181	-835	1085	-181	-834	1085	-181	-835	1086	1	1
2.400	-174	-835	1000	-174	-834	1000	-174	-835	1001	1	1
2.500	-124	-712	566	-124	-712	565	-124	-713	566	1	1
2.600	-99	-570	453	-99	-569	452	-99	-570	453	1	1
2.700	-74	-427	339	-74	-427	339	-74	-427	339	0	0
2.800	0	-523	319	0	-522	319	0	-523	319	1	0
2.900	0	-280	247	0	-280	247	0	-280	248	0	1
1.182	0	7384	-6519	0	7385	-6520	0	7384	-6519	1	1
1.182	0	6879	-4204	0	6880	-4205	0	6879	-4204	1	1
1.265	0	6857	-4190	0	6858	-4191	0	6856	-4190	2	1
1.265	122	2218	-3361	122	2218	-3362	122	2218	-3361	0	1
1.511	65	1175	-1781	65	1176	-1782	65	1175	-1781	1	1
1.511	65	1175	-1781	65	1176	-1782	65	1175	-1781	1	1
1.545	57	1035	-1569	57	1036	-1570	57	1035	-1569	1	1
1.545	64	1039	-2419	64	1039	-2420	64	1039	-2419	0	1
1.698	24	401	-933	24	401	-934	24	401	-933	0	1
1.698	22	367	-853	22	367	-853	22	367	-852	0	1
1.757	8	145	-337	8	145	-337	8	145	-337	0	0
1.757	0	404	-404	0	404	-404	0	403	-403	1	1
1.815	0	-213	213	0	-213	213	0	-213	213	0	0
1.815	0	-121	121	0	-121	121	0	-122	122	1	1
1.905	0	-660	660	0	-660	660	0	-660	660	0	0
1.905	0	-660	660	0	-660	660	0	-660	660	0	0
1.938	0	-864	864	0	-864	864	0	-865	865	1	1
1.938	0	-643	643	0	-643	643	0	-643	643	0	0
2.062	0	-862	862	0	-862	862	0	-863	863	1	1
2.062	0	-1160	1160	0	-1160	1160	0	-1160	1160	0	0
2.092	0	-1122	1122	0	-1122	1122	0	-1122	1122	0	0
2.092	0	-1122	1122	0	-1122	1122	0	-1122	1122	0	0
2.185	0	-1007	1007	0	-1007	1007	0	-1008	1008	1	1
2.185	0	-1764	1764	0	-1764	1764	0	-1765	1765	1	1
2.240	0	-1644	1644	0	-1644	1644	0	-1645	1645	1	1
2.240	-197	-906	1178	-197	-906	1178	-197	-907	1179	1	1
2.302	-181	-833	1083	-181	-833	1082	-181	-833	1083	0	1
2.302	-203	-972	1165	-203	-971	1164	-203	-972	1165	1	1
2.455	-158	-757	908	-158	-757	907	-158	-758	908	1	1
2.455	-136	-776	616	-135	-775	616	-136	-776	616	1	0
2.486	-128	-732	581	-128	-732	581	-128	-732	582	0	1
2.486	-128	-732	581	-128	-732	581	-128	-732	582	0	1
2.732	-66	-381	303	-66	-381	302	-66	-381	303	0	1
2.732	0	-700	428	0	-699	427	0	-700	428	1	1
2.818	0	-475	290	0	-474	289	0	-475	290	1	1
2.818	0	-509	450	0	-509	449	0	-510	450	1	1

STEEL I-BEAM **Pour No. 3a** STRESSES AT SPAN TENTH AND MISCELLANEOUS POINTS

* ACTUAL STRESSES(PSI) * ACTUAL RANGE IN STRESSES (PSI)

	TOTAL DEAD LOAD			D.L. + L.L. POS.			D.L. + L.L. NEG.			ACTUAL RANGE IN STRESSES (PSI)	
S.PT.	TOP CONC.	TOP STEEL	BOT. STEEL	TOP CONC.	TOP STEEL	BOT. STEEL	TOP CONC.	TOP STEEL	BOT. STEEL	TOP STEEL	BOT. STEEL
1.100	-5	-92	216	-5	-92	215	-5	-92	216	0	1
1.200	-10	-185	280	-10	-184	280	-10	-185	280	1	0
1.300	-15	-277	420	-15	-277	420	-15	-277	420	0	0
1.400	-20	-370	560	-20	-370	560	-20	-370	561	0	1
1.500	-25	-462	701	-25	-462	700	-25	-462	701	0	1
1.600	-34	-557	1296	-34	-557	1296	-34	-557	1297	0	1
1.700	-36	-594	1382	-36	-594	1381	-36	-594	1382	0	1
1.800	0	-1892	1892	0	-1892	1892	0	-1893	1893	1	1
1.900	0	-1216	1216	0	-1215	1215	0	-1216	1216	1	1
2.000	0	-1004	1004	0	-1004	1004	0	-1005	1005	1	1
2.100	0	-736	736	0	-735	735	0	-736	736	1	1
2.200	0	-211	211	0	-211	211	0	-212	212	1	1
2.300	103	476	-619	103	477	-620	103	476	-619	1	1
2.400	261	1248	-1496	261	1249	-1497	261	1248	-1496	1	1
2.500	348	1988	-1579	348	1989	-1580	348	1988	-1579	1	1
2.600	472	2698	-2143	472	2698	-2143	472	2698	-2143	0	0
2.700	596	3407	-2706	597	3408	-2707	596	3407	-2706	1	1
2.800	0	6973	-4262	0	6974	-4262	0	6973	-4261	1	1
2.900	0	5323	-4700	0	5324	-4700	0	5323	-4700	1	0
1.182	-10	-168	392	-10	-168	391	-10	-168	392	0	1
1.182	-9	-168	254	-9	-167	254	-9	-168	254	1	0
1.265	-13	-244	371	-13	-244	370	-13	-244	371	0	1
1.265	-13	-244	371	-13	-244	370	-13	-244	371	0	1
1.511	-26	-472	716	-26	-472	715	-26	-472	716	0	1
1.511	-26	-472	716	-26	-472	715	-26	-472	716	0	1
1.545	-27	-504	763	-27	-503	763	-27	-504	763	1	0
1.545	-31	-505	1177	-31	-505	1176	-31	-505	1177	0	1
1.698	-40	-648	1509	-39	-648	1509	-40	-648	1510	0	1
1.698	-36	-593	1379	-36	-593	1378	-36	-593	1379	0	1
1.757	-39	-643	1494	-39	-642	1494	-39	-643	1494	1	0
1.757	0	-1790	1790	0	-1790	1790	0	-1791	1791	1	1
1.815	0	-1928	1928	0	-1928	1928	0	-1929	1929	1	1
1.815	0	-1101	1101	0	-1101	1101	0	-1101	1101	0	0
1.905	0	-1222	1222	0	-1222	1222	0	-1222	1222	0	0
1.905	0	-1222	1222	0	-1222	1222	0	-1222	1222	0	0
1.938	0	-1267	1267	0	-1267	1267	0	-1268	1268	1	1
1.938	0	-942	942	0	-942	942	0	-943	943	1	1
2.062	0	-723	723	0	-723	723	0	-723	723	0	0
2.062	0	-972	972	0	-972	972	0	-973	973	1	1
2.092	0	-783	783	0	-783	783	0	-783	783	0	0
2.092	0	-783	783	0	-783	783	0	-783	783	0	0
2.185	0	-215	215	0	-215	215	0	-215	215	0	0
2.185	0	-377	377	0	-377	377	0	-377	377	0	0
2.240	0	218	-218	0	219	-219	0	218	-218	1	1
2.240	26	120	-156	26	120	-157	26	120	-156	0	1
2.302	105	486	-631	105	486	-632	105	485	-631	1	1
2.302	118	566	-679	118	567	-679	118	566	-679	1	0
2.455	341	1632	-1956	341	1632	-1956	341	1632	-1956	0	0
2.455	292	1672	-1328	293	1672	-1328	292	1672	-1328	0	0
2.486	331	1890	-1501	331	1891	-1502	331	1890	-1501	1	1
2.486	331	1890	-1501	331	1891	-1502	331	1890	-1501	1	1
2.732	647	3695	-2934	647	3695	-2935	647	3694	-2934	1	1
2.732	0	6782	-4145	0	6783	-4145	0	6781	-4145	2	0
2.818	0	6835	-4177	0	6836	-4178	0	6835	-4177	1	1
2.818	0	7337	-6477	0	7337	-6478	0	7336	-6477	1	1

STEEL I-BEAM **Pour No. 4** STRESSES AT SPAN TENTH AND MISCELLANEOUS POINTS

* ACTUAL STRESSES(PSI) * ACTUAL RANGE IN STRESSES (PSI)

	TOTAL DEAD LOAD			D.L. + L.L. POS.			D.L. + L.L. NEG. *				
S.PT.	TOP CONC.	TOP STEEL	BOT. STEEL	TOP CONC.	TOP STEEL	BOT. STEEL	TOP CONC.	TOP STEEL	BOT. STEEL	TOP STEEL	BOT. STEEL
1.100	8	141	-330	8	142	-330	8	141	-330	1	0
1.200	15	282	-428	15	282	-429	15	282	-428	0	1
1.300	23	424	-642	23	424	-643	23	424	-642	0	1
1.400	31	565	-857	31	565	-857	31	565	-857	0	0
1.500	39	707	-1071	39	707	-1072	39	707	-1071	0	1
1.600	52	851	-1981	52	851	-1982	52	851	-1981	0	1
1.700	55	908	-2112	55	908	-2112	55	908	-2111	0	1
1.800	0	2714	-2714	0	2715	-2715	0	2714	-2714	1	1
1.900	0	538	-538	0	539	-539	0	538	-538	1	1
2.000	0	-987	987	0	-987	987	0	-988	988	1	1
2.100	0	-1195	1195	0	-1195	1195	0	-1195	1195	0	0
2.200	0	-1860	1860	0	-1859	1859	0	-1860	1860	1	1
2.300	-195	-897	1166	-195	-897	1166	-195	-897	1166	0	0
2.400	-187	-897	1075	-187	-896	1074	-187	-897	1075	1	1
2.500	-134	-765	608	-133	-765	607	-134	-766	608	1	1
2.600	-107	-612	486	-107	-612	486	-107	-612	486	0	0
2.700	-80	-459	365	-80	-458	364	-80	-459	365	1	1
2.800	-53	-306	243	-53	-305	242	-53	-306	243	1	1
2.900	-31	-149	179	-31	-149	178	-31	-149	179	0	1
1.182	15	257	-599	15	257	-600	15	257	-599	0	1
1.182	14	256	-389	14	256	-389	14	256	-388	0	1
1.265	20	374	-567	20	374	-567	20	374	-567	0	0
1.265	20	374	-567	20	374	-567	20	374	-567	0	0
1.511	39	722	-1094	40	722	-1095	39	722	-1094	0	1
1.511	39	722	-1094	40	722	-1095	39	722	-1094	0	1
1.545	42	770	-1167	42	770	-1167	42	770	-1166	0	1
1.545	47	773	-1799	47	773	-1799	47	773	-1798	0	1
1.698	60	981	-2284	60	981	-2284	60	981	-2283	0	1
1.698	55	897	-2086	55	897	-2087	55	897	-2086	0	1
1.757	60	980	-2277	60	980	-2278	60	980	-2277	0	1
1.757	0	2729	-2729	0	2729	-2729	0	2728	-2728	1	1
1.815	0	2507	-2507	0	2507	-2507	0	2506	-2506	1	1
1.815	0	1431	-1431	0	1432	-1432	0	1431	-1431	1	1
1.905	0	471	-471	0	471	-471	0	471	-471	0	0
1.905	0	471	-471	0	471	-471	0	471	-471	0	0
1.938	0	-77	77	0	-77	77	0	-78	78	1	1
1.938	0	-57	57	0	-57	57	0	-58	58	1	1
2.062	0	-926	926	0	-926	926	0	-927	927	1	1
2.062	0	-1246	1246	0	-1246	1246	0	-1246	1246	0	0
2.092	0	-1205	1205	0	-1205	1205	0	-1205	1205	0	0
2.092	0	-1205	1205	0	-1205	1205	0	-1205	1205	0	0
2.185	0	-1082	1082	0	-1082	1082	0	-1083	1083	1	1
2.185	0	-1896	1896	0	-1895	1895	0	-1896	1896	1	1
2.240	0	-1767	1767	0	-1766	1766	0	-1767	1767	1	1
2.240	-211	-974	1266	-211	-974	1266	-212	-974	1266	0	0
2.302	-194	-895	1163	-194	-895	1163	-194	-895	1164	0	1
2.302	-218	-1044	1251	-218	-1044	1251	-218	-1044	1251	0	0
2.455	-170	-814	976	-170	-813	975	-170	-814	976	1	1
2.455	-146	-834	662	-145	-833	662	-146	-834	662	1	0
2.486	-137	-787	625	-137	-786	624	-137	-787	625	1	1
2.486	-137	-787	625	-137	-786	624	-137	-787	625	1	1
2.732	-71	-410	325	-71	-409	325	-71	-410	325	1	0
2.732	-71	-410	325	-71	-409	325	-71	-410	325	1	0
2.818	-48	-278	220	-48	-277	220	-48	-278	220	1	0
2.818	-56	-271	325	-56	-271	324	-56	-271	325	0	1

STEEL I-BEAM **Pour No. 4a** STRESSES AT SPAN TENTH AND MISCELLANEOUS POINTS

* ACTUAL STRESSES(PSI) * ACTUAL RANGE IN STRESSES (PSI)

S.PT.	TOTAL DEAD LOAD TOP CONC.	TOTAL DEAD LOAD TOP STEEL	TOTAL DEAD LOAD BOT. STEEL	D.L. + L.L. POS. TOP CONC.	D.L. + L.L. POS. TOP STEEL	D.L. + L.L. POS. BOT. STEEL	D.L. + L.L. NEG. TOP CONC.	D.L. + L.L. NEG. TOP STEEL	D.L. + L.L. NEG. BOT. STEEL *	ACTUAL RANGE TOP STEEL	ACTUAL RANGE BOT. STEEL
1.100	-6	-103	240	-6	-103	239	-6	-103	240	0	1
1.200	-11	-205	311	-11	-205	311	-11	-205	311	0	0
1.300	-17	-308	467	-17	-308	466	-17	-308	467	0	1
1.400	-22	-411	623	-22	-411	622	-22	-411	623	0	1
1.500	-28	-514	779	-28	-514	778	-28	-514	779	0	1
1.600	-38	-619	1441	-38	-619	1440	-38	-619	1441	0	1
1.700	-40	-660	1535	-40	-660	1535	-40	-660	1536	0	1
1.800	-46	-755	1755	-46	-755	1754	-46	-755	1755	0	1
1.900	-37	-686	1213	-37	-686	1212	-37	-686	1213	0	1
2.000	0	-1116	1116	0	-1116	1116	0	-1117	1117	1	1
2.100	0	382	-382	0	383	-383	0	382	-382	1	1
2.200	0	2471	-2471	0	2472	-2472	0	2471	-2471	1	1
2.300	278	1278	-1661	278	1278	-1661	277	1278	-1661	0	0
2.400	267	1277	-1531	267	1278	-1532	267	1277	-1531	1	1
2.500	191	1090	-866	191	1091	-866	191	1090	-866	1	0
2.600	152	872	-693	153	873	-693	152	872	-693	1	0
2.700	114	654	-519	114	655	-520	114	654	-519	1	1
2.800	76	436	-346	76	436	-346	76	436	-346	0	0
2.900	44	212	-255	44	213	-255	44	212	-255	1	0
1.182	-11	-187	436	-11	-187	435	-11	-187	436	0	1
1.182	-10	-186	282	-10	-186	282	-10	-186	282	0	0
1.265	-15	-272	412	-15	-272	411	-15	-272	412	0	1
1.265	-15	-272	412	-15	-272	411	-15	-272	412	0	1
1.511	-29	-525	795	-29	-525	795	-29	-525	796	0	1
1.511	-29	-525	795	-29	-525	795	-29	-525	796	0	1
1.545	-30	-560	848	-30	-559	848	-31	-560	848	1	0
1.545	-34	-562	1308	-34	-562	1307	-34	-562	1308	0	1
1.698	-44	-720	1677	-44	-720	1676	-44	-720	1678	0	2
1.698	-40	-659	1532	-40	-659	1531	-40	-659	1532	0	1
1.757	-44	-714	1660	-43	-714	1660	-44	-714	1661	0	1
1.757	-44	-714	1660	-43	-714	1660	-44	-714	1661	0	1
1.815	-47	-769	1788	-47	-769	1788	-47	-769	1789	0	1
1.815	-34	-622	1098	-34	-622	1098	-34	-622	1099	0	1
1.905	-38	-690	1219	-38	-690	1219	-38	-690	1219	0	0
1.905	0	-1358	1358	0	-1358	1358	0	-1358	1358	0	0
1.938	0	-1409	1409	0	-1408	1408	0	-1409	1409	1	1
1.938	0	-1047	1047	0	-1047	1047	0	-1048	1048	1	1
2.062	0	-178	178	0	-178	178	0	-179	179	1	1
2.062	0	-240	240	0	-240	240	0	-241	241	1	1
2.092	0	268	-268	0	268	-268	0	267	-267	1	1
2.092	0	268	-268	0	268	-268	0	267	-267	1	1
2.185	0	1290	-1290	0	1290	-1290	0	1290	-1290	0	0
2.185	0	2259	-2259	0	2260	-2260	0	2259	-2259	1	1
2.240	0	2485	-2485	0	2485	-2485	0	2484	-2484	1	1
2.240	298	1370	-1780	298	1370	-1781	297	1369	-1780	1	1
2.302	274	1261	-1639	274	1261	-1640	274	1261	-1639	0	1
2.302	307	1471	-1763	308	1471	-1763	307	1471	-1763	0	0
2.455	242	1159	-1390	243	1160	-1390	242	1159	-1389	1	1
2.455	208	1188	-943	208	1188	-944	208	1188	-943	0	1
2.486	196	1121	-890	196	1121	-890	196	1120	-890	1	0
2.486	196	1121	-890	196	1121	-890	196	1120	-890	1	0
2.732	102	584	-463	102	584	-464	102	583	-463	1	1
2.732	102	584	-463	102	584	-464	102	583	-463	1	1
2.818	69	396	-314	69	396	-314	69	396	-314	0	0
2.818	80	386	-463	81	387	-463	80	386	-463	1	0

STEEL I-BEAM **Pour No. 5** STRESSES AT SPAN TENTH AND MISCELLANEOUS POINTS

	*			ACTUAL STRESSES(PSI)						*	ACTUAL RANGE IN
	TOTAL DEAD LOAD			D.L. + L.L. POS.			D.L. + L.L. NEG. *				
S.PT.	TOP CONC.	TOP STEEL	BOT. STEEL	TOP CONC.	TOP STEEL	BOT. STEEL	TOP CONC.	TOP STEEL	BOT. STEEL	TOP STEEL	BOT. STEEL
1.100	0	2	-5	0	2	-6	0	2	-5	0	1
1.200	0	4	-7	0	4	-7	0	4	-7	0	0
1.300	0	7	-11	0	7	-11	0	7	-11	0	0
1.400	0	9	-14	0	9	-15	0	9	-14	0	1
1.500	0	12	-18	0	12	-19	0	12	-18	0	1
1.600	0	14	-34	0	14	-35	0	14	-33	0	2
1.700	0	15	-36	0	15	-37	0	15	-36	0	1
1.800	1	17	-41	1	18	-42	1	17	-41	1	1
1.900	0	16	-28	0	16	-28	0	16	-28	0	0
2.000	0	-372	372	0	-372	372	0	-372	372	0	0
2.100	4	22	-26	4	22	-26	4	22	-25	0	1
2.200	6	27	-35	6	27	-36	5	27	-35	0	1
2.300	5	24	-31	5	24	-31	5	23	-31	1	0
2.400	5	24	-28	5	24	-29	4	23	-28	1	1
2.500	3	20	-16	3	21	-16	3	20	-16	1	0
2.600	2	16	-13	3	17	-13	2	16	-12	1	1
2.700	2	12	-9	2	12	-10	2	12	-9	0	1
2.800	1	8	-6	1	8	-6	1	8	-6	0	0
2.900	0	4	-4	1	4	-5	0	3	-4	1	1
1.182	0	4	-10	0	4	-10	0	4	-10	0	0
1.182	0	4	-6	0	4	-7	0	4	-6	0	1
1.265	0	6	-9	0	6	-10	0	6	-9	0	1
1.265	0	6	-9	0	6	-10	0	6	-9	0	1
1.511	0	12	-18	0	12	-19	0	12	-18	0	1
1.511	0	12	-18	0	12	-19	0	12	-18	0	1
1.545	0	13	-20	0	13	-20	0	13	-19	0	1
1.545	0	13	-31	0	13	-31	0	13	-30	0	1
1.698	1	17	-39	1	17	-40	1	17	-39	0	1
1.698	0	15	-36	0	15	-37	0	15	-36	0	1
1.757	1	17	-39	1	17	-39	1	16	-39	1	0
1.757	1	17	-39	1	17	-39	1	16	-39	1	0
1.815	3	64	-149	3	64	-149	3	64	-149	0	0
1.815	2	51	-91	2	52	-92	2	51	-91	1	1
1.905	0	9	-17	0	9	-17	0	9	-17	0	0
1.905	0	19	-19	0	19	-19	0	18	-18	1	1
1.938	0	-109	109	0	-109	109	0	-109	109	0	0
1.938	0	-81	81	0	-81	81	0	-81	81	0	0
2.062	0	-81	81	0	-81	81	0	-81	81	0	0
2.062	0	-109	109	0	-109	109	0	-109	109	0	0
2.092	0	10	-10	0	10	-10	0	9	-9	1	1
2.092	1	7	-8	1	7	-8	1	6	-7	1	1
2.185	14	71	-83	14	71	-83	14	70	-83	1	0
2.185	21	98	-128	21	98	-128	21	98	-128	0	0
2.240	5	26	-34	5	26	-34	5	26	-33	0	1
2.240	5	26	-34	5	26	-34	5	26	-33	0	1
2.302	5	24	-31	5	24	-31	5	23	-31	1	0
2.302	5	28	-33	6	28	-34	5	27	-33	1	1
2.455	4	21	-26	4	22	-26	4	21	-26	1	0
2.455	3	22	-17	4	23	-18	3	22	-17	1	1
2.486	3	21	-16	4	21	-17	3	21	-16	0	1
2.486	3	21	-16	4	21	-17	3	21	-16	0	1
2.732	1	11	-8	2	11	-9	1	10	-8	1	1
2.732	1	11	-8	2	11	-9	1	10	-8	1	1
2.818	1	7	-5	1	7	-6	1	7	-5	0	1
2.818	1	7	-8	1	7	-9	1	7	-8	0	1

Article 6.10.3.3 Shear

The use of tension field action is not permitted until after the deck has harden or is made composite.
Design this section under the un-stiffened web requirements for shear
Webs with transverse stiffeners, with or without longitudinal stiffeners, shall satisfy the following requirement during critical stages of construction:

V <= Øv (Vcr)

where: Øv = resistance factor for shear specified in Article 6.5.4.2, Øv = 1.00
V = shear in the web at the section under consideration due to the total permanent load. (kip)
Vcr = Shear buckling resistance determined from the following equation:
Vcr = C (Vp)
Vp = plastic shear force = 0.58 (Fyw) (D) (tw) = 783.00 kips
C = ratio of the shear buckling resistance to the shear yield strength determined as specified in Article 6.10.9.3.2, with the shear buckling coefficient, **k, taken equal to 5.0**
Note: The shear buckling coefficient, k, to be used in calculating the constant C is defined as 5.0 for un-stiffened web panels.

Article 6.10.9.3.2, the ratio, C, shall be determined as specified below:
If D / tw <= 1.12 ((E)(K) / (Fyw))^.5 then C = 1.0
If 1.12 ((E)(K) / (Fyw))^.5 <= D / tw <= 1.4((E)(K) / (Fyw))^.5
then, C = (1.12 / (D / tw)) (E(K) / Fyw)^.5
If D / tw => 1.40 ((E)(K) / (Fyw))^.5
then, C = (1.57 / (D / tw)^2)(E (K) / (Fyw)) <= controling equation for C

Shear Checks:

	Span Point	factored DL Shear	do	K	D/ tw	(EK/Fyw)^.5	1.12 X (EK/Fyw)^.	1.40 X (EK/Fyw)^.5	C	allowable shear	total shear	check
cross-frame =>	1.0000	121.0	82.0	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	121.0	OK
stifferner =>	1.0420	109.0	121.5	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	109.0	OK
	1.1000	93.0	121.5	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	93.0	OK
stifferner =>	1.1050	92.0	121.5	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	92.0	OK
cross-frame =>	1.1670	74.0	162.5	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	74.0	OK
	1.1850	69.0	162.5	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	69.0	OK
	1.2000	64.0	162.5	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	64.0	OK
stifferner =>	1.2500	50.0	162.5	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	50.0	OK
	1.3000	36.0	162.5	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	36.0	OK
cross-frame =>	1.3300	27.0	162.5	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	27.0	OK
	1.4000	8.0	162.5	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	8.0	OK
cross-frame =>	1.5000	20.1	162.5	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	20.1	OK
	1.6000	48.0	162.5	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	48.0	OK
cross-frame =>	1.6770	70.0	162.5	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	70.0	OK
	1.7000	76.0	162.5	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	76.0	OK
	1.8000	105.0	162.5	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	105.0	OK
	1.8150	109.0	162.5	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	109.0	OK
cross-frame =>	1.8330	114.0	82.0	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	114.0	OK
stifferner =>	1.8730	125.0	82.0	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	125.0	OK
	1.9000	133.0	82.0	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	133.0	OK
stifferner =>	1.9161	138.0	82.0	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	138.0	OK
	1.9380	144.0	82.0	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	144.0	OK
stifferner =>	1.9580	149.0	82.0	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	149.0	OK
stifferner =>	1.9790	155.0	61.5	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	155.0	OK
CL Pier =>	2.0000	161.0	41.0	5.000	108.00	53.85165	60.314	75.392	0.390	305.6412	161.0	OK

Article 6.10.4 Service Limit State Check

Permanent Deformations

For the purposes of this article, the Service II load combination specified in Table 3.4.1-1 shall apply.
For members with shear connectors provided throughout their entire length that also satisfy the provisions of Article 6.10.1.7, flexural stresses caused by Service II loads applied to the composite section may be computed computed using the short-term or long-term composite section, as approiate, assuming the concrete slab is effective for both positive and negative flexure.

Flexure:

Flanges shall satisfy the following requirements:

For the top steel flange of composite sections:

ff <= 0.95 (Rh) (Fyt)

For the bottom steel flange of composite sections:

ff + (fl)(1 / 2) <= (0.95)(Rh)(Fyf)

For both steel flanges of non-composite sections:

ff + (fl)(1 / 2) <= (0.80)(Rh)(Fyf)

where:

ff = flange stress at the section under consideration due to the Service II loads calculated without consideration of flange lateral bending. (ksi)

fl = flange lateral bending stress at the section under consideration due to Service II loads determined as specified in Article 6.10.1.6 (ksi)

Note: Since the girders are striaght, the effects of lateral flange bend is minimal. The lateral loads would be from the wind, slab pouring operations, effects of the skew, differential deflection and etc..
The size and magnitude of the construction and wind loads were calculated and determined to be relatively small and have little effect. In the service limit state is assumed that fl = 0.0 ksi

Rh = hybrid factor determined as specified in Article 6.10.1.6 (ksi), Rh = 1.0

Slender webs sections without longitudinal stiffeners subject to negative flexure shall also satisfy the following requirement:

fc <= Fcrw <= Fyc

where: fc = compression flange stress at the section under consideration due to the Service II loads calculated without consideration flange lateral bending. (KSI)

Fcrw = nominal elastic bend-buckling resistance for webs without longitudinal stiffeners determined as specified in Article 6.10.1.9 (ksi)

Fcrw = (0.90)(E)(K) / (D / tw)^2, where K = 9 / (Dc / D)^2

Fyc = yeild strenght of the compression flange

Service II Loads and checks

Section Properties:

web thickness = 0.5 inches, Web depth = 54 inches

Span Pt.	top flange		bottom flange		non-comp properties		comp. N=27 prop.		comp. N=9 properties	
	width	thickness	width	thickness	sb	st	sb	st	sb	st
1.000 cf	18.000	1.000	18.000	1.2500	1407.1	1242.2	1743.2	3966.7	1887.3	12143.9
1.100	18.000	1.000	18.000	1.2500	1407.1	1242.2	1743.2	3966.7	1887.3	12143.9
1.167 cf	18.000	1.000	18.000	1.2500	1407.1	1242.2	1743.2	3966.7	1887.3	12143.9
1.1800	18.000	1.000	18.000	1.2500	1407.1	1242.2	1743.2	3966.7	1887.3	12143.9
1.1800	18.000	1.000	18.000	2.2500	2181.5	1333.3	2690.3	4009.6	2909.3	10888.3
1.2000	18.000	1.000	18.000	2.2500	2181.5	1333.3	2690.3	4009.6	2909.3	10888.3
1.3000	18.000	1.000	18.000	2.2500	2181.5	1333.3	2690.3	4009.6	2909.3	10888.3
1.333 cf	18.000	1.000	18.000	2.2500	2181.5	1333.3	2690.3	4009.6	2909.3	10888.3
1.4000	18.000	1.000	18.000	2.2500	2181.5	1333.3	2690.3	4009.6	2909.3	10888.3
1.500 cf	18.000	1.000	18.000	2.2500	2181.5	1333.3	2690.3	4009.6	2909.3	10888.3
1.6000	18.000	1.000	18.000	2.2500	2181.5	1333.3	2690.3	4009.6	2909.3	10888.3
1.6000	18.000	1.000	18.000	1.2500	1407.1	1242.2	1743.2	3966.7	1887.3	12143.9
1.667 cf	18.000	1.000	18.000	1.2500	1407.1	1242.2	1743.2	3966.7	1887.3	12143.9
1.7000	18.000	1.000	18.000	1.2500	1407.1	1242.2	1743.2	3966.7	1887.3	12143.9
1.7000	24.000	1.063	24.000	1.0625	1611.48	1611.48	0	0	0	0
1.8000	24.000	1.063	24.000	1.0625	1611.48	1611.48	0	0	0	0
1.8150	24.000	1.063	24.000	1.0625	1611.48	1611.48	0	0	0	0
1.8150	24.000	2.000	24.000	2.0000	2822.7	2822.7	0	0	0	0
1.833 cf	24.000	2.000	24.000	2.0000	2822.7	2822.7	0	0	0	0
1.9000	24.000	2.000	24.000	2.0000	2822.7	2822.7	0	0	0	0
1.9375	24.000	2.000	24.000	2.0000	2822.7	2822.7	0	0	0	0
1.9375	24.000	2.750	24.000	2.7500	3795.7	3795.7	0	0	0	0
2.000 cf	24.000	2.750	24.000	2.7500	3795.7	3795.7	0	0	0	0

Service Limit State Check
State Route 45 (OHB) over I-65
Davidson County
Date: August 8, 2003

Service II Moments and Stresses:

	Service II Moments				top flange Service II Stresses				bottom Flange Service II Stresses			
Span Pt.	non-comp. DL mom.	comp. DL mom.	max LL moment	mim LL moment	non-comp. DL mom.	comp. DL mom.	max LL moment	mim LL moment	non-comp. DL mom.	comp. DL mom.	max LL moment	mim LL moment
1.000 cf	0	0	0	0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1.100	1431	292	2236	-492	13.8	0.9	2.2	-0.5	12.2	2.0	14.2	-3.1
1.167 cf	2095	427	3305	-558	20.2	1.3	3.3	-0.6	17.9	2.9	21.0	-3.5
1.1800	2224	454	3513	-570	21.5	1.4	3.5	-0.6	19.0	3.1	22.3	-3.6
1.1800	2224	454	3513	-570	20.0	1.4	3.9	-0.6	12.2	2.0	14.5	-2.4
1.2000	2422	494	3832	-590	21.8	1.5	4.2	-0.6	13.3	2.2	15.8	-2.4
1.3000	2973	606	4815	-885	26.8	1.8	5.3	-1.0	16.4	2.7	19.9	-3.6
1.333 cf	3009	614	4959	-982	27.1	1.8	5.5	-1.1	16.6	2.7	20.5	-4.1
1.4000	3083	629	5252	-1180	27.7	1.9	5.8	-1.3	17.0	2.8	21.7	-4.9
1.500 cf	2753	561	5165	-1474	24.8	1.7	5.7	-1.6	15.1	2.5	21.3	-6.1
1.6000	1982	404	4601	-1770	17.8	1.2	5.1	-2.0	10.9	1.8	19.0	-7.3
1.6000	1982	404	4601	-1770	19.1	1.2	4.5	-1.7	16.9	2.8	29.3	-11.3
1.667 cf	1171	239	3913	-1968	11.3	0.7	3.9	-1.9	10.0	1.6	24.9	-12.5
1.7000	771	157	3570	-2065	7.4	0.5	3.5	-2.0	6.6	1.1	22.7	-13.1
1.7000	771	157	3570	-2065	5.7	1.2	26.6	-15.4	5.7	1.2	26.6	-15.4
1.8000	-881	-180	2137	-2739	-6.6	-1.3	15.9	-20.4	-6.6	-1.3	15.9	-20.4
1.8150	-1195	-244	1924	-2836	-8.9	-1.8	14.3	-21.1	-8.9	-1.8	14.3	-21.1
1.8150	-1195	-244	1924	-2836	-5.1	-1.0	8.2	-12.1	-5.1	-1.0	8.2	-12.1
1.833 cf	-1571	-321	1456	-2952	-6.7	-1.4	6.2	-12.5	-6.7	-1.4	6.2	-12.5
1.9000	-2973	-606	719	-3383	-12.6	-2.6	3.1	-14.4	-12.6	-2.6	3.1	-14.4
1.9375	-3923	-800	449	-4103	-16.7	-3.4	1.9	-17.4	-16.7	-3.4	1.9	-17.4
1.9375	-3923	-800	449	-4103	-12.4	-2.5	1.4	-13.0	-12.4	-2.5	1.4	-13.0
2.000 cf	-5506	-1122	0	-5302	-17.4	-3.5	0.0	-16.8	-17.4	-3.5	0.0	-16.8

Total Service II Stress:

	top flange		Bottom Fange		Allowable stress		top flange Check	Bottom flange Check	Slender Web Check:		
Span Pt.	max. positive	max. negative	max. positive	max. negative	top flange	bottom flange			Dc	K	Fcrw
1.000 cf	0.0	0.0	0.0	0.0	47.5	47.5	OK	OK	0	0.00	0.0
1.100	16.9	14.2	28.4	11.1	47.5	47.5	OK	OK	19.64	68.01	50.0
1.167 cf	24.8	21.0	41.8	17.3	47.5	47.5	OK	OK	19.60	68.32	50.0
1.1800	26.3	22.3	44.4	18.5	47.5	47.5	OK	OK	19.59	68.36	50.0
1.1800	25.2	20.7	28.7	11.9	47.5	47.5	OK	OK	24.75	42.85	50.0
1.2000	27.5	22.6	31.3	13.1	47.5	47.5	OK	OK	24.74	42.87	50.0
1.3000	33.9	27.6	38.9	15.4	47.5	47.5	OK	OK	24.63	43.26	50.0
1.333 cf	34.4	27.8	39.7	15.2	47.5	47.5	OK	OK	24.55	43.55	50.0
1.4000	35.4	28.3	41.4	14.9	47.5	47.5	OK	OK	24.39	44.12	50.0
1.500 cf	32.1	24.8	38.9	11.6	47.5	47.5	OK	OK	23.92	45.88	50.0
1.6000	24.1	17.1	31.7	5.4	47.5	47.5	OK	OK	22.84	50.31	50.0
1.6000	24.9	18.6	48.9	8.4	47.5	47.5	OK	NG	17.72	83.61	50.0
1.667 cf	15.9	10.1	36.5	-0.9	47.5	47.5	OK	OK	15.88	104.03	50.0
1.7000	11.5	5.9	30.4	-5.5	47.5	47.5	OK	OK	14.29	128.50	50.0
1.7000	33.5	-8.5	33.5	-8.5	40.0	40.0	OK	OK	27.00	36.00	50.0
1.8000	8.0	-28.3	8.0	-28.3	40.0	40.0	OK	OK	27.00	36.00	50.0
1.8150	3.6	-31.8	3.6	-31.8	40.0	40.0	OK	OK	27.00	36.00	50.0
1.8150	2.1	-18.2	2.1	-18.2	40.0	40.0	OK	OK	27.00	36.00	50.0
1.833 cf	-1.9	-20.6	-1.9	-20.6	40.0	40.0	OK	OK	27.00	36.00	50.0
1.9000	-12.2	-29.6	-12.2	-29.6	40.0	40.0	OK	OK	27.00	36.00	50.0
1.9375	-18.2	-37.5	-18.2	-37.5	40.0	40.0	OK	OK	27.00	36.00	50.0
1.9375	-13.5	-27.9	-13.5	-27.9	40.0	40.0	OK	OK	27.00	36.00	50.0
2.000 cf	-21.0	-37.7	-21.0	-37.7	40.0	40.0	OK	OK	27.00	36.00	50.0

Note: shaded area is composite section for LL and comp. DL

Fatique Design
Old Hickory Bvld. Over I-65
Davidson County
Date: July 30, 2003

Article 6.10.5 Fatique and Fracture Limit State

Details shall be investigated for fatique as specified in Article 6.6.1. The Fatique load combination specifed in Article 6.6.1.
The Fatique load combination specified in table 3.4.1-1 and the fatique live load specified in Article 3.6.1.4 shall apply
The provisions for fatique in shear connectors specified in Artilces6.10.10.2 and 10.10.3 shall apply.

Fatique Live Load Ranges (from fatique truck w/o lane loading):

Span Pt.	Live Load Range	
	shear	moment
1.0000	59.2	0.0
1.1000	51.6	837.9
1.2000	46.4	1435.3
1.3000	45.9	1846.4
1.4000	46.6	2047.3
1.5000	47.6	2098.6
1.6000	48.7	2036.2
1.7000	50.1	1813.8
1.8000	52.0	1467.5
1.9000	54.8	1068.0
2.0000	57.6	921.1

Art. 3.6.1.4 Fatique load:
The fatique load shall be one design truck or axles thereof specified in Art. 3.6.1.2.2 but with a constant spacing of 30.0 ft. between the 32.0 kip axles. The dynamic load allowance specified in art. 3.6.2 shall be applied to the fatique loading.
applied load modifier not applied for fatique loading.

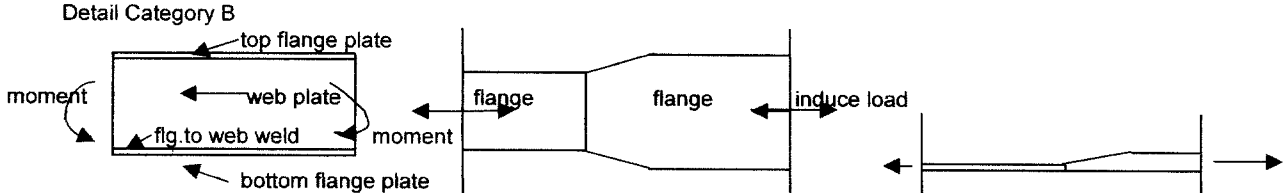
Art. 6.5.3 Fatique and Fracture limit State
Components and details shall be investigated for fatique as specified in Art. 6.6.
The fatique load combination, specified in table 3.4.1-1 and the fatique live load specified in Art. 3.6.1.4 shall be apply. Webs of plate girders shall satisfy the provisions of Art. 6.10.4

Art. 3.6.1.4.2 Frequency
Single lane average daily truck traffic, ADTTsl = (P) (ADTT). The ADTT in the number of trucks per day in one direction averaged. over the design life of the structure. (P) is specified in table1, which (P) = 0.80 for three or more lanes loaded.
In lieu of site specific fraction of truck data, an approximate fraction of the ADTT can be used.

Average daily traffic (ADT) for 2021 = ADT(2021) = 27480 use 10% of ADT for ADTT
Average daily truck traffic (ADTT) for 2021 = 2748
which therefore: ADTTsl = (0.80)(ADTT) = 2198

Art. 6.6 Fatique and Fracture Considerations
Load induced fatique considerations
(Y)(delta(f)) <= (delta(F)n)
where: Y = load factor specified in table 3.4.1-1 for fatique load combination = 0.75
delta(f) = the force effect, live load stress range due to the passage of the fatique load as specified in art. 3.6.1.4 (ksi)
delta(F)n = the nominal fatique resistance as specified in art. 6.6.1.2.5 (ksi)

Art. 6.6.1.2.5 Fatigue Resistance
Detail Category B



web to flange welds (fillet or beveled welds)

Flange transitions in Width (slopes no steeper than 1.0 to 2.5)

Flange transitions in thickness (slopes no steeper than 1.0 to 2.5)

Nominal Fatique Resistance:
delta(F)n = (A / N)^.33 => 1 / 2 (delta(F)th) N = 365(75)(n)((ADTT)sl)
where: A = constant taken from table 1
n = number of stress range cycles per truck passage (table 2) = 1.0
delta(F)th = constant amplitude fatique threshold (table3)
N = (365 days/year)(75 year design life)(1.0)(ADTT) = 60181200 cycles
A = 1.2E+10 and delta(F)th = 16.00 ksi
delta(F)n = (A / N)^0.33 = 5.83 ksi => 0.5(delta(F)th) = 8.0000 ksi
delta(F)n is less than .5(delta(F)th) use the .5(delta(F)th) = 8.0000 ksi
if a section is near a support, n = 1.5 then N = 365(75)(1.5)(ADTT) = 90271800
delta(F)n = (A / N)^0.33 = 5.10 ksi => 0.5(delta(F)th) = 8.0000 ksi
delta(F)n is less than .5(delta(F)th) use the .5(delta(F)th) = 8.0000 ksi

Detail Category C', (welding near or to the tension flange, welds near the end of transverse stiffeners)
A = 4.4E+09 and delta(F)th = 12.000 ksi
delta(F)n = (A / N)^0.33 = 4.12 ksi => 0.5(delta(F)th) = 6.000 ksi
if a section is near a support, n = 1.5 then N = 365(75)(1.5)(ADTT) = 90271800
delta(F)n = (A / N)^0.33 = 3.65 ksi => 0.5(delta(F)th) = 6.0000 ksi
delta(F)n is less than .5(delta(F)th) use the .5(delta(F)th) = 6.0000 ksi

Fatique / Fracture Considerations (cont)
Old Hicko., Blvd. over I-65
Davidson County
Date: September 15, 2003

Fatique LL stress range caculations: (using BT Beam Program)

Note: The Specifications requires the stress range to be compute from the difference between the maximum and minimum stresses calculated from the appropriate section (non-comp or comp). The BT beam program only gives a moment range instead of the max. and min. values. The stress ranges were calculated from the composite section in positive moment region and from the non-composite section in the negative moment region. This does not follow the Specifications entirely but was done since the BT Beam program was used thoughout the design.

Positive fatique -load moments are assumed to be applied to the composite section (N = 9)
Neqative fatique-load moments are assumed to be applied to the steel section only.
Y = 0.75
Live Load Distribution Factor for moment = 0.914 lanes per girder for one lane loaded
Live Load Distribution Factor for shear = 1.080 lanes per girder for two or more lanes loaded
For Fatique the multi-presense factor of 1.2 for a singlely loaded lane does not apply.
Adjusted Load Distribution Factor for moment = 0.762 lanes per girder

	span point	(.75)(LL moment) fat. range	(LL w/ LLDF) fat. range	Sb	St	tf stress Y(Delta f)	bf stress Y(delta f)	control. Y(Delta f)	allowable catogory B	allowable catogory C'	mim allowable catogory	span point
	1.000	0	0	1892	13051	0.00	0.00	0.00	8.00	6.00	C'	1.000
	1.100	838	638	1892	13051	0.59	4.05	4.05	8.00	6.00	C'	1.100
POS. BENDING	1.185	1346	1025	1892	13051	0.94	6.50	6.50	8.00	6.00	B	1.185
COMPOSITE	1.185	1346	1025	2918	11562	1.06	4.22	4.22	8.00	6.00	C'	1.185
PROPERTIES	1.200	1435	1093	2918	11562	1.13	4.50	4.50	8.00	6.00	C'	1.200
(N = 9)	1.300	1846	1406	2918	11562	1.46	5.78	5.78	8.00	6.00	C'	1.300
FOR STRESSES	1.400	2047	1559	2918	11562	1.62	6.41	6.41	8.00	6.00	B	1.400
	1.500	2099	1598	2918	11562	1.66	6.57	6.57	8.00	6.00	B	1.500
	1.600	2036	1551	2918	11562	1.61	6.38	6.38	8.00	6.00	B	1.600
	1.600	2036	1551	1892	13051	1.43	9.84	9.84	8.00	6.00	fails **	1.600
FIELD SPLICE	1.700	1814	1382	1892	13051	1.27	8.76	8.76	8.00	6.00	fails **	1.700
FIELD SPLICE	1.700	1814	1382	1611	1611	10.29	10.29	10.29	8.00	6.00	fails **	1.700
	1.800	1467	1118	1611	1611	8.33	8.33	8.33	8.00	6.00	fails **	1.800
NEG. BENDING	1.815	1408	1072	1611	1611	7.99	7.99	7.99	8.00	6.00	B	1.815
STEEL	1.815	1408	1072	2823	2823	4.56	4.56	4.56	8.00	6.00	C'	1.815
SECTION ONLY	1.900	1068	813	2823	2823	3.46	3.46	3.46	8.00	6.00	C'	1.900
FOR STRESSES	1.938	1012	771	2823	2823	3.28	3.28	3.28	8.00	6.00	C'	1.938
	1.938	1012	771	3796	3796	2.44	2.44	2.44	8.00	6.00	C'	1.938
	2.000	921	702	3796	3796	2.22	2.22	2.22	8.00	6.00	C'	2.000

** May have to increase the tension plate thickenss to lower stress range to satisfy this requirement
Span point 1.7 is the location of the field splice and fatique will be checked in the splice design.
The field splice is located at the dead load contra-flexure.

Fatique and Fracture Considerations (cont)
Old Hickory Blv ver I-65
Davidson County
Date: September 15, 2003

Fatique LL stress range caculations: (using Qcon Program)

Positive fatique -load moments are assumed to be applied to the composite section (N = 9)
Negative fatique-load moments are assumed to be applied to the steel section only.
Y = 0.75
Live Load Distribution Factor for moment = 0.914 lanes per girder for one lane loaded
Live Load Distribution Factor for shear = 1.080 lanes per girder for two or more lanes loaded
For Fatique the multi-presense factor of 1.2 for a singlely loaded lane does not apply
Adjusted Load Distribution Factor for moment = 0.762 lanes per girder

	span point	(.75)(LLDF)(LL max.m fat. range	.75(LLDF)(LL min.m fat. range	Sb non comp	St non comp	Sb comp n=9	St comp n=9	tf stress Y(delta f)	bf stress Y(delta f)	controling Y(Delta f)	allowable catogory B	allowable catogory C'	mim allowable catogory
	1.000	0	0	1892	13051	1893	13051.5	0.00	0.00	0.00	8.00	6.00	C'
	1.100	578	-71	1892	13051	1893	13051.5	0.60	3.73	3.73	8.00	6.00	C'
POS. BENDING	1.185	913	-132	1892	13051	1893	13051.5	0.96	5.91	5.91	8.00	6.00	C'
COMPOSITE	1.185	913	-132	2918	11562	2917.7	11562	1.08	3.89	3.89	8.00	6.00	C'
PROPERTIES	1.200	972	-143	2918	11562	2917.7	11562	1.16	4.15	4.15	8.00	6.00	C'
(N =9)	1.300	1217	-214	2918	11562	2917.7	11562	1.49	5.23	5.23	8.00	6.00	C'
FOR STRESSES	1.400	1302	-285	2918	11562	2917.7	11562	1.65	5.65	5.65	8.00	6.00	C'
	1.500	1270	-357	2918	11562	2917.7	11562	1.69	5.59	5.59	8.00	6.00	C'
	1.600	1150	-428	2918	11562	2917.7	11562	1.64	5.17	5.17	8.00	6.00	C'
	1.600	1150	-428	1892	13051	2343	12179	1.53	6.31	6.31	8.00	6.00	B
FIELD SPLICE	1.700	906	-499	1892	13051	2343	12179	1.35	-3.16	1.35	8.00	6.00	C'
FIELD SPLICE	1.700	906	-499	1693	1693	1693	1693	9.96	-3.54	9.96	8.00	6.00	fails
	1.800	568	-570	1693	1693	1693	1693	8.07	-4.04	8.07	8.00	6.00	fails
NEG. BENDING	1.815	512	-581	1693	1693	1693	1693	7.75	-4.12	7.75	8.00	6.00	B
STEEL	1.815	512	-581	2823	2823	2823	2823	4.65	-2.47	4.65	8.00	6.00	C'
SECTION ONLY	1.900	196	-641	2823	2823	2823	2823	3.56	-2.72	3.56	8.00	6.00	C'
FOR STRESSES	1.938	122	-668	2823	2823	2823	2823	3.36	-2.84	3.36	8.00	6.00	C'
	1.938	122	-668	3796	3796	3796	3796	2.50	-2.11	2.50	8.00	6.00	C'
	2.000	0	-713	3796	3796	3796	3796	2.25	-2.25	2.25	8.00	6.00	C'

** May have to increase the tension plate thickenss to lower stress range to satisfy this requirement
Span point 1.7 is the location of the field splice and fatique will be checked in the splice design.
The field splice is located at the dead load contra-flexure.

Due to Fatique :
increase 18" x 1 1/4" bottom plate to 18" x 1 11/16"
increase 24" X 1 1/16" top & Bottom plate to 24" X 1 1/4"

Note:
The 25' long 18" x 1.25" bottom plate had to be increased to 18" x 1 11/16" to meet fatique requirements of Cat. B for Weathering Steel infinite life. Also the adjoining 19' long plates on the other side of the field splice had to be increased from 24" x 1 1/16" to 24" x 11/4" to meet the fatique requirement. The effect of this increase will increase the number of bolts and size of splice plates in the field splice.

Fatigue and Fracture Considerations (cont)
Old Hickory Blvd. over I-65
Davidson County
Date: September 15, 2003

Article 6.10.5.2 Fracture

Fracture toughness requirements specificed in the contract documents shall be conformance with the provisions of Article 6.6.2

Article 6.10.5.3 Special Fatique Requirement for the Webs

For the purposes of this article, the factored fatique load shall be taken as twice that calculated using the Fatique load combinations specified in Table 3.4.1-1, with the fatique live load taken as specified in Article 3.6.1.4

Article 6.10.5.3.2 Shear

Webs with transverse stiffeners, with or without longitudinal stiffeners, shall satisfy the following requirement: V < = Vcr

where:

V = shear in the web at the section under consideration due to the unfactored permanent loads plus the factored fatique load (Kip)

Vcr = shear buckling resistance determined from equation 6.10.9.2-1 (kip)

Vn = Vcr = (C)(Vp) in which: Vp = 0.58(Fyw)(D)(tw)

C = ratio of the shear buckling resistance to the shear yield strength as specified in Article 6.10.9.3.2, with the shear buckling coefficient, k, equal to 5.0

The ration, C, shall be determined as specified below:

If D / tw <= 1.12 (E(k)/Fyw)^.5 then C = 1.0

If 1.12(E(k)/Fyw)^.5 <= D / tw <= 1.4(E(k)/Fyw)^.5, then C = (1.12/(D/tw))(E(k)/Fyw)^.5

If 1.4(E(k)/Fyw)^.5 <= D/tw, then C = (1.57/(D/tw)^2)(E(k)/Fyw)

D / tw = 108.00

1.12(E(k) / Fyw)^.5 = 60.31

1.40(E(k) / Fyw)^.5 = 75.39

therefore: 1.4(E(k)/Fyw)^.5 <= D/tw, then C = (1.57/(D/tw)^2)(E(k)/Fyw)

C = (1.57/(D/tw)^2)(E(k)/Fyw) = 0.39

Vn = Vcr = (C)(Vp) in which: Vp = 0.58(Fyw)(D)(tw)

Vcr = 526.97 kips

Live Load Distribution Factor for shear = 1.080 lanes per girder for two or more lanes loaded

Span Pt.	Live Load		Dead load	Dead load	2(Live Load	Shear	Vcr	check
	shear ***	mom.range	nc shear	comp shear	shear w LLDF)	V		
1.000	59.20	0.0	101.60	20.70	127.87	250.17	526.97	OK
1.100	51.60	837.9	77.90	15.90	111.46	205.26	526.97	OK
1.200	46.40	1435.3	54.20	11.10	100.22	165.52	526.97	OK
1.300	45.90	1846.4	30.50	6.20	99.14	135.84	526.97	OK
1.400	46.60	2047.3	6.80	1.40	100.66	108.86	526.97	OK
1.500	-47.60	2098.6	-16.90	-3.50	-102.82	-123.22	-526.97	OK
1.600	-48.70	2036.2	-40.70	-8.30	-105.19	-154.19	-526.97	OK
1.700	-50.10	1813.8	-64.40	-13.10	-108.22	-185.72	-526.97	OK
1.800	-52.00	1467.5	-88.10	-18.00	-112.32	-218.42	-526.97	OK
1.900	-54.80	1068.0	-111.80	-22.80	-118.37	-252.97	-526.97	OK
2.000	-57.60	921.1	-135.50	-27.60	-124.42	-287.52	-526.97	OK

*** Note: The Specifications only require to check the shear not the shear range in this particular check. The BT Beam program only gave the LL shear range, which was used in this check.

Fatique and Fracture Considerations (cont)
Old Hickory Blvd over I-65
Davidson County
Date: August 20, 2003

Fatique LL stress range caculations:

Positive fatique -load moments are assumed to be applied to the composite section (N = 9)
Neqative fatique-load moments are assumed to be applied to the steel section only.
Y = 0.75
Live Load Distribution Factor for moment = 0.914 lanes per girder for one lane loaded
Live Load Distribution Factor for shear = 1.080 lanes per girder for two or more lanes loaded
For Fatique the multi-presense factor of 1.2 for a singlely loaded lane does not apply
Adjusted Load Distribution Factor for moment = 0.762 lanes per girder

	span point	(.75)(LL moment) fat. range	(LL w/ LLDF) fat. range	Sb	St	tf stress Y(Delta f)	bf stress Y(delta f)	controlin Y(Delta f)	allowable catogory B	allowable catogory C'	mim allowable catogory	span point
	1.000	0	0	1892	13051	0.00	0.00	0.00	8.00	6.00	C'	1.000
	1.100	838	638	1892	13051	0.59	4.05	4.05	8.00	6.00	C'	1.100
POS. BENDING	1.185	1346	1025	1892	13051	0.94	6.50	6.50	8.00	6.00	B	1.185
COMPOSITE	1.185	1346	1025	2918	11562	1.06	4.22	4.22	8.00	6.00	C'	1.185
PROPERTIES	1.200	1435	1093	2918	11562	1.13	4.50	4.50	8.00	6.00	C'	1.200
(N =9)	1.300	1846	1406	2918	11562	1.46	5.78	5.78	8.00	6.00	C'	1.300
FOR STRESSES	1.400	2047	1559	2918	11562	1.62	6.41	6.41	8.00	6.00	B	1.400
	1.500	2099	1598	2918	11562	1.66	6.57	6.57	8.00	6.00	B	1.500
	1.600	2036	1551	2918	11562	1.61	6.38	6.38	8.00	6.00	B	1.600
	1.600	2036	1551	2343	12179	1.53	7.94	7.94	8.00	6.00	B	1.600
FIELD SPLICE	1.700	1814	1382	2343	12179	1.36	7.08	7.08	8.00	6.00	B	1.700
FIELD SPLICE	1.700	1814	1382	1693	1693	9.79	9.79	9.79	8.00	6.00	fails **	1.700
	1.800	1467	1118	1693	1693	7.92	7.92	7.92	8.00	6.00	B	1.800
NEG. BENDING	1.815	1408	1072	1693	1693	7.60	7.60	7.60	8.00	6.00	B	1.815
STEEL	1.815	1408	1072	2823	2823	4.56	4.56	4.56	8.00	6.00	C'	1.815
SECTION ONLY	1.900	1068	813	2823	2823	3.46	3.46	3.46	8.00	6.00	C'	1.900
FOR STRESSES	1.938	1012	771	2823	2823	3.28	3.28	3.28	8.00	6.00	C'	1.938
	1.938	1012	771	3796	3796	2.44	2.44	2.44	8.00	6.00	C'	1.938
	2.000	921	702	3796	3796	2.22	2.22	2.22	8.00	6.00	C'	2.000

** May have to increase the tension plate thickenss to lower stress range to satisfy this requirement
Span point 1.7 is the location of the field splice and fatique will be checked in the splice design.
The field splice is located at the dead load contra-flexure.

Due to Fatique :
increase 18" x 1 1/4" bottom plate to 18" x 1 11/16"
increase 24" X 1 1/16" top & Bottom plate to 24" X 1 1/4"

Note:
The 25' long 18" x 1.25" bottom plate had to be increased to 18" x 1 11/16" to meet fatique requirements of Cat. B for Weathering Steel infinite life. Also the adjoining 19' long plates on the other side of the field splice had to be increased from 24" x 1 1/16" to 24" x 11/4" to meet the fatique reguirement. The effect of this increase will increase the number of bolts and size of splice plates in the field splice.

Fatique an⁄ ‾racture Considerations (cont)
Old Hickory d. over I-65
Davidson County
Date: August 20, 2003

Fatique LL stress range caculations: (using Qcon Program)

Positive fatique -load moments are assumed to be applied to the composite section (N = 9)
Neqative fatique-load moments are assumed to be applied to the steel section only.
Y = 0.75
Live Load Distribution Factor for moment = 0.914 lanes per girder for one lane loaded
Live Load Distribution Factor for shear = 1.080 lanes per girder for two or more lanes loaded
For Fatique the multi-presense factor of 1.2 for a singlely loaded lane does not apply
Adjusted Load Distribution Factor for moment= 0.762 lanes per girder

	span point	(.75)(LLDF)(LL max fat. range	.75(LLDF)(LL min. fat. range	Sb non comp	St non comp	Sb comp n=9	St comp n=9	tf stress Y(delta f)	bf stress Y(delta f)	controling Y(Delta f)	allowable catogory B	allowable catogory C	mim allowable catogory	span point
	1.000	0	0	1892	13051	1893	13051.5	0.00	0.00	0.00	8.00	6.00	C'	1.000
	1.100	578	-71	1892	13051	1893	13051.5	0.60	3.73	3.73	8.00	6.00	C'	1.100
POS. BENDING	1.185	913	-132	1892	13051	1893	13051.5	0.96	5.91	5.91	8.00	6.00	C'	1.185
COMPOSITE	1.185	913	-132	2918	11562	2917.7	11562	1.08	3.89	3.89	8.00	6.00	C'	1.185
PROPERTIES	1.200	972	-143	2918	11562	2917.7	11562	1.16	4.15	4.15	8.00	6.00	C'	1.200
(N =9)	1.300	1217	-214	2918	11562	2917.7	11562	1.49	5.23	5.23	8.00	6.00	C'	1.300
FOR STRESSES	1.400	1302	-285	2918	11562	2917.7	11562	1.65	5.65	5.65	8.00	6.00	C'	1.400
	1.500	1270	-357	2918	11562	2917.7	11562	1.69	5.59	5.59	8.00	6.00	C'	1.500
	1.600	1150	-428	2918	11562	2917.7	11562	1.64	5.17	5.17	8.00	6.00	C'	1.600
	1.600	1150	-428	1892	13051	2343	12179	1.53	6.31	6.31	8.00	6.00	B	1.600
FIELD SPLICE	1.700	906	-499	1892	13051	2343	12179	1.35	-3.16	1.35	8.00	6.00	C'	1.700
FIELD SPLICE	1.700	906	-499	1693	1693	1693	1693	9.96	-3.54	9.96	8.00	6.00	fails *	1.700
	1.800	568	-570	1693	1693	1693	1693	8.07	-4.04	8.07	8.00	6.00	fails *	1.800
NEG. BENDING	1.815	512	-581	1693	1693	1693	1693	7.75	-4.12	7.75	8.00	6.00	B	1.815
STEEL	1.815	512	-581	2823	2823	2823	2823	4.65	-2.47	4.65	8.00	6.00	C'	1.815
SECTION ONLY	1.900	196	-641	2823	2823	2823	2823	3.56	-2.72	3.56	8.00	6.00	C'	1.900
FOR STRESSES	1.938	122	-668	2823	2823	2823	2823	3.36	-2.84	3.36	8.00	6.00	C'	1.938
	1.938	122	-668	3796	3796	3796	3796	2.50	-2.11	2.50	8.00	6.00	C'	1.938
	2.000	0	-713	3796	3796	3796	3796	2.25	-2.25	2.25	8.00	6.00	C'	2.000

** May have to increase the tension plate thickenss to lower stress range to satisfy this requirement
Span point 1.7 is the location of the field splice and fatique will be checked in the splice design.
The field splice is located at the dead load contra-flexure.

Due to Fatique :
increase 18" x 1 1/4" bottom plate to 18" x 1 11/16"
increase 24" X 1 1/16" top & Bottom plate to 24" X 1 1/4"

Note:
The 25' long 18" x 1.25" bottom plate had to be increased to 18" x 1 11/16" to meet fatique requirements of Cat. B for Weathering Steel infinite life. Also the adjoining 19' long plates on the other side of the field splice had to be increased from 24" x 1 1/16" to 24" x 11/4" to meet the fatique reguirement. The effect of this increase will increase the number of bolts and size of splice plates in the field splice.

Shear Design
Old Hickory Blvd. (SR-45) over I-65
Davidson County
Date: April 14, 2003

Article 6.10.9 Shear Resistance

The factored shear resistance of a web panel, Vn, shall be taken as:

$$Vr = (\phi v)(Vn)$$

where: ϕv = resistance factor for shear specified in Article 6.5.4.2, ϕv = 1.00

Vn = nominal shear resistance determined as specified in Article 6.10.9.3 for unstiffened and stiffened webs, respectively

Transverse intermediate stiffeners shall be designed as specified in Art. 6.10.11.3

Interior web panels of non-hybrid and hybrid I-shaped members:

Without a longitudinal stiffeners and with a transverse stiffener spacing not exceeding 3(D) shall be considered stiffened, and the provisions of Article 6.10.9.3 shall apply. Otherwise the panel shall be considered unstiffened, and the provisions of Article 6.10.9.2 shall apply

For stiffened webs, provisions for the end panels shall be as specified in Article 6.10.9.3.3

Article 6.10.9.3 Nominal Resistance of Unstiffened Webs

The nominal shear resistance of unstiffened webs shall be taken as:

Vn = Vcr = (C)(Vp) tw = 0.500 inches

in which: Vp = 0.58(Fyw)(D)(tw) D = 54.000 inches

Vp = 783.00 kips

which: C = ratio of the shear buckling resistance to the shear yield strength determined as specified in Article 6.10.9.3.2 with the shear buckling coefficient, k, taken equal to 5.0.

Vcr = shear buckling resistance (kip), Vp = plastic shear force (kip)

Vn = nominal shear resistance (kip)

Article 6.10.9.3 Nominal Resistance of Unstiffened Webs

The nominal shear resistance of transversely stiffened interior web panels shall be as specified in Articles 6.10.9.3.2.

The nominal shear resistance of transversely stiffened end web panels shall be as specified in Articles 6.10.9.3.3.

The total web depth, D, shall be used in determining the nominal shear resistance of web panels. The required transverse stiffener spacing shall be calculated using the maximum shear in the panel.

Stiffeners shall satisfy the requirements specified in Article 6.10.11

Article 6.10.9.3.2 Interior Panels

The nominal shear resistance of an interior web panel complying with the provisions of Article 6.10.9.1 and with the section along the entire panel proportioned such that:

2(D)(tw) / ((bfc)(tfc) + (bft)(tft)) <= 2.5

shall be taken as:

Vn = Vp (C + (0.87(1-C)) / (1 + (do / D)^2)

where: do = stiffener spacing (in)

C = ratio of the shear buckling resistance to the shear yield strength

The ratio, C, shall be determined as specified below:

If D / tw <= 1.12 (E(k) / Fyw)^.5, then C = 1.0

If 1.12 (E(k) / Fyw)^.5 <= D / tw <=1.40 (E(k) / Fyw)^.5, then C = (1.12 / (D/tw)) (E(k) / Fyw)^.5

If D / tw => 1.40 (E(k) / Fyw)^.5, then C = (1.57 / (D/tw)^2) (E(k) / Fyw)

k = shear buckling coefficient = 5 + 5 / (do / D)^2

Otherwise, the nominal shear resistance shall be taken as the shear buckling resistance determined from

2(D)(tw) / ((bfc)(tfc) + (bft)(tft)) <= 2.5

Article 6.10.9.3.3 End panels

The nominal shear resistance of a web end panels shall be taken as: Vn = Vcr = (C)(Vp)

The transverse stiffener spacing for end panels shall not exceed 1.5(D)

Shear Design
Old Hickory Blvd. (SR-45) over I-65
Davidson County
Date: November 21, 2003

Strength I Shears design shear check (Stiffened web):

See Un-stiffen web check for the area of the web that can be consider to be un-stiffened.

D / tw = 108.000

	Span Point	DL Shear	LL shear	Strength I shears	do	k stiffen web	1.12(Ek/Fyw)^.5	1.40(Ek/Fyw)^.5	C	Vn	Strength I shears
cross-frame =>	1.0000	150.2	242	392	82.0	7.17	72.22	90.27	0.56	603.18	392
stifferner =>	1.0420	133.4	226	359	121.5	5.99	66.00	82.50	0.47	513.35	359
	1.1000	110.2	203	313	121.5	5.99	66.00	82.50	0.47	513.35	313
stifferner =>	1.1050	108.2	201	309	121.5	5.99	66.00	82.50	0.47	513.35	309
cross-frame =>	1.1670	83.3	179	262	325.0	5.14	61.14	76.43	0.40	380.95	262
	1.1850	76.1	172	248	325.0	5.14	61.14	76.43	0.40	380.95	248
	1.2000	70.1	167	237	325.0	5.14	61.14	76.43	0.40	380.95	237
stifferner =>	1.2500	50.1	151	201	325.0	5.14	61.14	76.43	0.40	380.95	201
	1.3000	30	134	164	325.0	5.14	61.14	76.43	0.40	380.95	164
cross-frame =>	1.3300	23.3	125	148	325.0	5.14	61.14	76.43	0.40	380.95	148
	1.4000	10	104	114	325.0	5.14	61.14	76.43	0.40	380.95	114
cross-frame =>	1.5000	50	131	181	325.0	5.14	61.14	76.43	0.40	380.95	181
	1.6000	90.1	162	252	325.0	5.14	61.14	76.43	0.40	380.95	252
cross-frame =>	1.6770	121.3	186	307	325.0	5.14	61.14	76.43	0.40	380.95	307
	1.7000	130.2	193	323	325.0	5.14	61.14	76.43	0.40	380.95	323
	1.8000	170.3	224	394	325.0	5.14	61.14	76.43	0.40	380.95	394
	1.8150	176.3	229	405	325.0	5.14	61.14	76.43	0.40	380.95	405
cross-frame =>	1.8330	183.5	234	418	82.0	7.17	72.22	90.27	0.56	603.18	418
stifferner =>	1.8730	199.6	246	446	82.0	7.17	72.22	90.27	0.56	603.18	446
	1.9000	210.3	254	464	82.0	7.17	72.22	90.27	0.56	603.18	464
stifferner =>	1.9161	216.8	259	476	82.0	7.17	72.22	90.27	0.56	603.18	476
	1.9380	225.5	265	491	82.0	7.17	72.22	90.27	0.56	603.18	491
stifferner =>	1.9580	233.6	271	505	82.0	7.17	72.22	90.27	0.56	603.18	505
stifferner =>	1.9790	241.9	278	520	61.5	8.85	80.26	100.33	0.69	680.03	520
CL Pier =>	2.0000	250.5	283	534	41.0	13.67	99.74	124.68	0.92	764.61	534

The entire panel proportioned such that:

ratio check = 2(D)(tw) / ((bfc)(tfc) + (bft)(tft)) <= 2.5

Span Pt.	top flange		bottom flange		web		ratio	limiting	check
	width	thickness	width	thickness	depth	thickness	check	value	
1.0000	18.0000	1.0000	18.0000	1.2500	54.0000	0.5000	1.3333	2.5000	OK
1.1000	18.0000	1.0000	18.0000	1.2500	54.0000	0.5000	1.3333	2.5000	OK
1.1800	18.0000	1.0000	18.0000	1.2500	54.0000	0.5000	1.3333	2.5000	OK
1.1800	18.0000	1.0000	18.0000	2.2500	54.0000	0.5000	0.9231	2.5000	OK
1.2000	18.0000	1.0000	18.0000	2.2500	54.0000	0.5000	0.9231	2.5000	OK
1.3000	18.0000	1.0000	18.0000	2.2500	54.0000	0.5000	0.9231	2.5000	OK
1.4000	18.0000	1.0000	18.0000	2.2500	54.0000	0.5000	0.9231	2.5000	OK
1.5000	18.0000	1.0000	18.0000	2.2500	54.0000	0.5000	0.9231	2.5000	OK
1.6000	18.0000	1.0000	18.0000	2.2500	54.0000	0.5000	0.9231	2.5000	OK
1.6000	18.0000	1.0000	18.0000	1.2500	54.0000	0.5000	1.3333	2.5000	OK
1.7000	18.0000	1.0000	18.0000	1.2500	54.0000	0.5000	1.3333	2.5000	OK
1.7000	24.0000	1.0625	24.0000	1.0625	54.0000	0.5000	1.0588	2.5000	OK
1.8000	24.0000	1.0625	24.0000	1.0625	54.0000	0.5000	1.0588	2.5000	OK
1.8125	24.0000	1.0625	24.0000	1.0265	54.0000	0.5000	1.0771	2.5000	OK
1.8150	24.0000	2.0000	24.0000	2.0000	54.0000	0.5000	0.5625	2.5000	OK
1.9000	24.0000	2.0000	24.0000	2.0000	54.0000	0.5000	0.5625	2.5000	OK
1.9380	24.0000	2.0000	24.0000	2.0000	54.0000	0.5000	0.5625	2.5000	OK
1.9380	24.0000	2.7500	24.0000	2.7500	54.0000	0.5000	0.4091	2.5000	OK
2.0000	24.0000	2.7500	24.0000	2.7500	54.0000	0.5000	0.4091	2.5000	OK

Shear Design
Old Hickory Blvd. (SR-45) over I-65
Davidson County
Date: November 21, 2003

Location of Web that can be consider to be un-stiffened:

For un-stiffened webs, Vn = (C)(Vp)
When spacing exceeds 3(D), the section is considered to be un-stiffened, K = 5.0
Shaded area is area of web that is considered to be un-stiffened

Strength I Shears design shear check (Un-Stiffened web, K= 5.0):

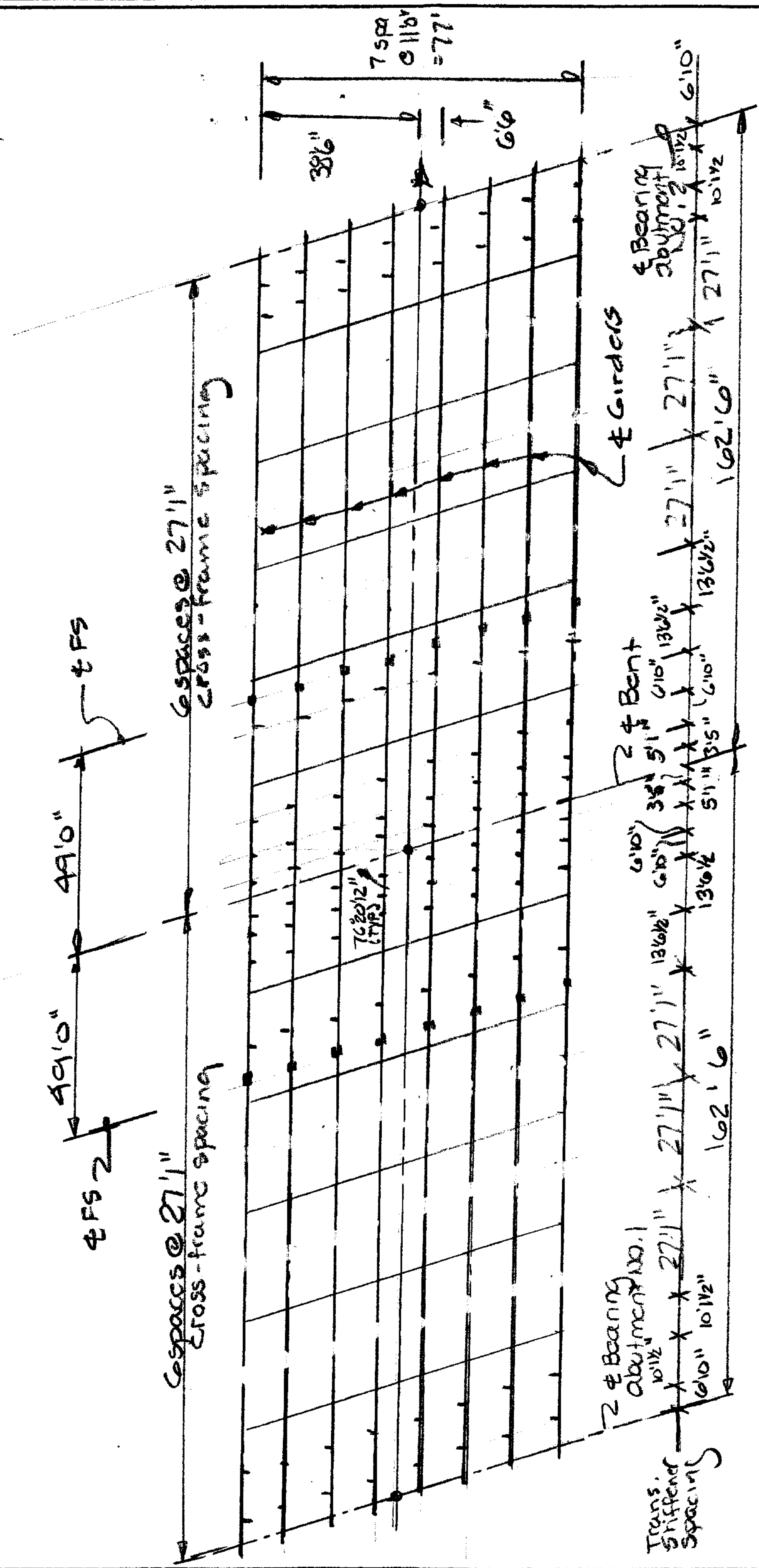
D / tw = 108.000

	Span Point	DL Shear	LL shear	Strength I shears	k un-stiffen web	1.12(Ek/Fyw)^.5	1.40(Ek/Fyw)^.5	C	Vn	Strength I shears	Stiffen/ unstiffen
cross-frame =>	1.0000	150.2	242.0	392	5.00	60.31	75.39	0.39	305.64	392	stiffen
	1.1000	110.2	203.0	313	5.00	60.31	75.39	0.39	305.64	313	stiffen
cross-frame =>	1.1670	83.3	179.0	262	5.00	60.31	75.39	0.39	305.64	262	unstiffen
	1.1850	76.1	172.0	248	5.00	60.31	75.39	0.39	305.64	248	unstiffen
	1.2000	70.1	167.0	237	5.00	60.31	75.39	0.39	305.64	237	unstiffen
	1.3000	30.0	134.0	164	5.00	60.31	75.39	0.39	305.64	164	unstiffen
cross-frame =>	1.3300	23.3	125.0	148	5.00	60.31	75.39	0.39	305.64	148	unstiffen
	1.4000	10.0	104.0	114	5.00	60.31	75.39	0.39	305.64	114	unstiffen
cross-frame =>	1.5000	50.0	131.0	181	5.00	60.31	75.39	0.39	305.64	181	unstiffen
	1.6000	90.1	162.0	252	5.00	60.31	75.39	0.39	305.64	252	unstiffen
cross-frame =>	1.6770	121.3	186.0	307	5.00	60.31	75.39	0.39	305.64	307	stiffen
	1.7000	130.2	193.0	323	5.00	60.31	75.39	0.39	305.64	323	stiffen
	1.8000	170.3	224.0	394	5.00	60.31	75.39	0.39	305.64	394	stiffen
	1.8150	176.3	229.0	405	5.00	60.31	75.39	0.39	305.64	405	stiffen
cross-frame =>	1.8330	183.5	234.0	418	5.00	60.31	75.39	0.39	305.64	418	stiffen
	1.9000	210.3	254.0	464	5.00	60.31	75.39	0.39	305.64	464	stiffen
	1.9380	225.5	265.0	491	5.00	60.31	75.39	0.39	305.64	491	stiffen
CL Pier =>	2.0000	250.5	283.0	534	5.00	60.31	75.39	0.39	305.64	534	stiffen

DESIGNED BY	DATE	PROJECT SR 45 / I-65	PAGE OF
CHECKED BY	DATE	SUBJECT Girder layout	COUNTY Davidson

TENNESSEE DEPARTMENT OF TRANSPORTATION DIVISION OF STRUCTURES 1100 J. K. POLK OFFICE B[illegible] NASHVILLE, TENNESSEE 3721[illegible]

Girder layout



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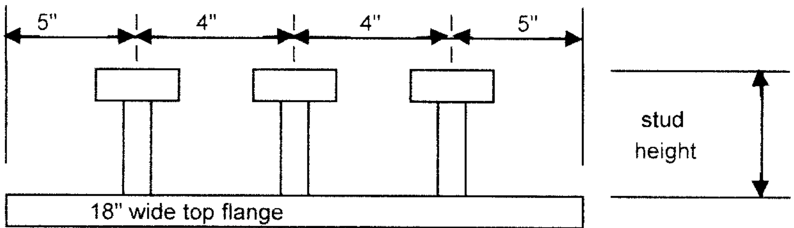
DT-1412

Shear Connector Design
Old Hickory Blvd. over I-65
Davidson County
Date: July 30, 2003

Art. 6.10.10 Shear Connectors

The ratio of the heigth to the deameter of a stud shear connector shall not be less than 4.0.
The shear stud connectors shall not be closer than 4.0 stud diameters center to center transverse to longitudinal axis of the support member.
Use a 7/8" circular shear stud. Three (3) studs per row
The center to center pitch of the shear connectors shall not exceed 24.0 inches and shall not be less than six stud diameters.
The clear depth of concrete cover over the tops of the shear connectors should not be less than 2.0 inches and should penetrate at least 2.0 inches into the deck slab.
The clear distance between the edge of the top flange to the edge of the nearest shear connector shall not be less than 1.0 inch



max top flange thickness = 2.75 "
min top flange thickness = 1.00 "
slab thickness = 9.00 "
top of slab to top of web = 1 ' 1.25"

min trans. stud spacing
4 x stud dia. = 3.50 "
4 > 3.5 ok

stud hieght range:
2" clear requirement = 13.25 - 2.00 - 2.75 = 8.50 "
2" penetration requirement = 13.25 - 9.00 - 1.00 = 3.25 "
max. height for 7/8 " stud has to be greater than 4/.875 = 4.57"
use a stud height of 5.5" which 3.25 < 5.50 < 8.50 ok

The pitch of the studs are based off fatique limit state art. 6.10.10.2 and 6.10.10.3.
The number of shear connectors shall not be less than the number required to satisfy the strength limit state as specified in art. 6.10.7.4.4.

The pitch of the connectors shall not be less than:

p <= (n)(Zr) / (Vsr)
in which:
Vsr = Vr (Q) / I

where:
n = number of studs in cross-section = 3
I = moment of inertia of the short-term composite section (in^4)
p = pitch of shear connectors along the longitudinal axis (in)
Q = first moment of the transformed area of the concrete deck about the neutral axis of the short-term composite section (in^3)
Vsr = horizontal fatique shear range per unit length (kip/in)
Zr = shear fatique resistance of an individual shear connector determined as specified in art. 6.10.10.3 (kip)
Vf = vertical shear force range under the Fatique load combination specified in Table 3.4 1-1 with the fatique live load taken as specified in article 6.10.10.2 (kip)

Article 6.10.10.2 Fatique Resistance

The fatique resistance of an individual shear connector, Zr , shall be taken as:

Zr = (alpha)(d^2) >= (5.5)(d^2) / 2
(5.5)(d^2) / 2 = 2.11 kip per stud
art. 6.6.1.2.5 fatique resistance (for calculation of N)
N = (365)(75)(n)(ADTT)sl

N = (365)(75)(1.00)(2198)
N = 6,017,025

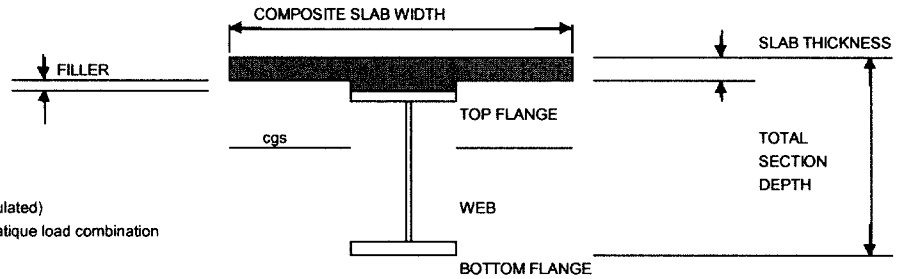
alpha = 34.5 - (4.28)(log N)
N = number of cycles specified in art. 6.6.1.2.5
d = diameter of the stud (in) = .875 "

n = number of stress range cycles per truck passage taken from table 2 = 1.00
(ADTT)sl = single lane ADTT as specified in art. 3.6.1.4
(ADTT)sl= (p)(ADTT), p = 0.85 for two lanes loaded
(ADTT)sl = (0.80)(0.10)(27480) = 2198, ADT(2021) = 27480 for this s

alpha = 34.5 - (4.28)(log N)
alpha = 5.484
Therefore: Zr for a single stud = (5.484)(.875^2) =4.20 kip per stud => 2.11 use 4.18 kip per stud

SHEAR STUD SPACING CALCULATIONS (CONT)

Old Hickory Blvd. over I-65
Davidson County
Prepared by: WHP
Date: August 21, 2003



Stud Spacing = p <= (n)(Zr) / (Vsr)
Vsr = Vf (Q) / I
where: n = 3 studs per row
Zr = 4.18 kips per stud (as periously calculated)
Vf = verticle shear force range under the fatique load combination

Calculation for I and Q at each section change

Section no 1

COMPOSITE SLAB WIDTH:	111.0
SLAB THICKNESS:	9.000
FILLER:	3.250
TOP FLANGE WIDTH:	18.000
TOP FLANGE THICKNESS:	1.000
WEB DEPTH:	54.000
WEB THICKNESS:	0.500
BOTTOM FLANGE WIDTH:	18.000
BOTTOM FLANGE THICKNESS:	1.250
TOTAL DEPTH OF SECTION	68.500

COMPOSITE PROPERIES N = 27 (referance line in at bottom of bottom flange)

	AREA	Y X (AREA)	Y^2X(AREA)	Io	I ref
SLAB	37.00	2368.00	151552.00	249.75	151801.75
TOP FLANGE	18.00	1003.50	55945.13	1.50	55946.63
WEB	27.00	762.75	21547.69	6561.00	28108.69
BOTTOM FLANGE	22.50	14.06	8.79	2.93	11.72
TOTALS	104.50	4148.31			235868.78

Ycgs=	39.70	S slab=	2471.7
I @ CGS=	71194.2	S tf=	4300.9
		S bf=	1793.4

Q = first moment of the transformed slab area about the neutral axis
Q = 899.22 IN^3 Q / I = 0.0126

Section no 2

COMPOSITE SLAB WIDTH:	111.0
SLAB THICKNESS:	9.000
FILLER:	3.250
TOP FLANGE WIDTH:	18.000
TOP FLANGE THICKNESS:	1.000
WEB DEPTH:	54.000
WEB THICKNESS:	0.500
BOTTOM FLANGE WIDTH:	18.000
BOTTOM FLANGE THICKNESS:	2.250
TOTAL DEPTH OF SECTION	69.500

COMPOSITE PROPERIES N = 27 (referance line in at bottom of bottom flange)

	AREA	Y X (AREA)	Y^2X(AREA)	Io	I ref
SLAB	37.00	2405.00	156325.00	249.75	156574.75
TOP FLANGE	18.00	1021.50	57970.13	1.50	57971.63
WEB	27.00	789.75	23100.19	6561.00	29661.19
BOTTOM FLANGE	40.50	45.56	51.26	17.09	68.34
TOTALS	122.50	4261.81			244275.91

Ycgs=	34.79	S slab=	2766.0
I @ CGS=	96006.1	S tf=	4274.6
		S bf=	2759.6

Q = first moment of the transformed slab area about the neutral axis
Q = 1117.8 IN^3 Q / I = 0.0116

Section no 3

COMPOSITE SLAB WIDTH:	111.0
SLAB THICKNESS:	9.000
FILLER:	3.250
TOP FLANGE WIDTH:	18.000
TOP FLANGE THICKNESS:	1.000
WEB DEPTH:	54.000
WEB THICKNESS:	0.500
BOTTOM FLANGE WIDTH:	18.000
BOTTOM FLANGE THICKNESS:	1.250
TOTAL DEPTH OF SECTION	68.500

COMPOSITE PROPERIES N = 27 (referance line in at bottom of bottom flange)

	AREA	Y X (AREA)	Y^2X(AREA)	Io	I ref
SLAB	37.00	2368.00	151552.00	249.75	151801.75
TOP FLANGE	18.00	1003.50	55945.13	1.50	55946.63
WEB	27.00	762.75	21547.69	6561.00	28108.69
BOTTOM FLANGE	22.50	14.06	8.79	2.93	11.72
TOTALS	104.50	4148.31			235868.78

Ycgs=	39.70	S slab=	2471.7
I @ CGS=	71194.2	S tf=	4300.9
		S bf=	1793.4

Q = first moment of the transformed slab area about the neutral axis
Q = 899.2 IN^3 Q / I = 0.0126

Section no 4

stud termination point

AREA OF RE-BAR IN SECT.(21-#5'S)	8.800
DIST. FROM TOP OF WEB TO CGS RE-BAR	7.750
TOP FLANGE WIDTH:	24.000
TOP FLANGE THICKNESS:	1.063
WEB DEPTH:	54.000
WEB THICKNESS:	0.500
BOTTOM FLANGE WIDTH:	24.000
BOTTOM FLANGE THICKNESS:	1.063
TOTAL DEPTH OF SECTION	56.125

COMPOSITE PROPERIES (w/ re-bar) (referance line in at bottom of bottom flange)

	AREA	Y X (AREA)	Y^2X(AREA)	Io	I ref
Re-bar (21-#5'S)	8.80	552.75	34719.61	0.00	34719.61
TOP FLANGE	25.50	1417.64	78811.96	2.40	78814.36
WEB	27.00	757.69	21262.61	6561.00	27823.61
BOTTOM FLANGE	25.50	13.55	7.20	2.40	9.60
TOTALS	86.80	2741.63			141367.17

Ycgs=	31.59	S re-bar=	1696.3
I @ CGS=	54771.5	S tf=	2232.0
		S bf=	1734.1

Q = first moment of the transformed slab area about the neutral axis
Q = 177.2 IN^3 Q / I = 0.0032

LLDR for shear = 1.08

Note: Vf = shear force range under
LL + I determined for fatique limit state
which Vf = (0.75)(faq. VLL non-factored)(LLDF)
(table 3.4.1-1 load combinations)
The stud spacing is based on fatique,
the total number of studs required in
the composite reqion is based on
the strength limit state as check on
the following page.

span point	Vf	n	Zr	Q/I	cal. pitch	pitch
1.00	47.95	3	4.20	0.012631	20.80	use 18"
1.10	41.80	3	4.20	0.012631	23.87	use 18"
1.20	37.58	3	4.20	0.011643	28.80	use 18"
1.30	37.18	3	4.20	0.011643	29.11	use 18"
1.40	37.26	3	4.20	0.011643	29.05	24 " controls
1.50	38.56	3	4.20	0.011643	28.07	use 18"
1.60	39.45	3	4.20	0.012631	25.29	use 18"
1.70	40.58	3	4.20	0.003236	N/A	N/A

Article 6.10.10.4 Strength Limit State

The factored shear resistance of a single shear connector, Qr, at the strength limit state shall be taken as: Qr = (Osc)(Qn)
where: Qn = nominal shear resistance of a single shear connector determined as specified in Article 6.10.10.4.3 (kip)
Osc = resistance factor for shear connectors specified in Article 6.5.4.2, Osc = 0.85

At the strength limit state, the minimum number of shear connectors, n, over the region under consideration shall be taken as:
n = P / Qr
where: P = total nominal shear force determined as specified in Article 6.10.10.4.2 (kip)
Qr = factored shear resistance of one shear connector determined from Qr = (Osc)(Qn)

Article 6.10.10.4.2 Nominal Shear Force

For simple spans and for continuous spans that are noncomposite for negative shear force, P, between the point of maximum positive design live load plus impact moment and each adjacent point of zero moment shall be taken as: P = Pp
in which: Pp = total longitudinal shear force in the concrete deck at the point of maximum positive live load plus impact moment (kip) taken as the lesser of either:
P1p = 0.85(f'c)(bs)(ts) or P2p = Fyw(D)(tw) + (Fyt)(bft)(tft) + (Fyc)(bfc)(tfc)
where: bs = effective width of the concrete deck (in) = 111.00 in
ts = thickness of the concrete deck (in) = 9.00 in
D = depth of web (in) = 54.00 in
tw = thickness of the web (in) = 0.5000 in
f'c = compressive strength of slab (ksi) = 3.000 ksi

For 18.000" x 1.000" top flange and 18.000" x 1.25" bottom flange

bft =	18.0000 in			
tft =	1.0000 in	P1p =	2547 kips	<= controls
bfc =	18.0000 in	P2p =	3375 kips	
tfc =	1.2500 in			

For 18.000" x 1.000" top flange and 18.000" x 2.25" bottom flange

bft =	18.0000 in			
tft =	1.0000 in	P1p =	2547 kips	<= controls
bfc =	18.0000 in	P2p =	4275 kips	
tfc =	2.2500 in			

For 24.000" x 1.000" top flange and 24.000" x 1.00" bottom flange

bft =	24.0000 in			
tft =	1.0000 in	P1p =	2547 kips	<= controls
bfc =	24.0000 in	P2p =	3750 kips	
tfc =	1.0000 in			

Minimum number of shear connnectors, n, over the region: n = P / Qr, which Qr = (Osc)(Qn)
Qr = (0.85)(30.0) = 25.5 kip per stud
controling P = 2547 kips
n = P / Qr = 100 connectors
number of rows = 33 rows of 3 studs
connector spacing = 18 inches
Region 1, between Abut and Max. Postive moment
length of girder between abutment and maximum positive moment = 65 feet
number of rows of connectors in the region
number of rows = 43 rows of 3 studs
which 43 > 33 ok
Region 2, Max.Postive moment and DL inflection
length of girder between maximum positive moment and DL inflection = 48.5 feet
number of rows of connectors in the region
number of rows = 32 rows of 3 studs
which 33 is approx. equal to 32, OK

Article 6.10.10.4.3 Nominal Shear Resistance

The nominal shear resistance of one stud shear connector embedded in a concrete deck shall be taken as:
Qn = 0.5(Asc)(f'c(Ec))^.5 <= Asc(Fu)
where: Asc = cross-sectional area of a stud shear connector (in^2)
Asc = (pi)(.875 / 2)^2 = (3.14)(.875/2)^2 = 0.601 in^2
Ec = modulus of elasticity of the deck concrete determined as specified in Article 5.4.2.4 (ksi)
which Ec = 33,000 (.150)^1.5 (f'c)^.5 = 3320.56 ksi
Fu = specified minimum tensile strength of a stud shear connector determined as specified in Article 6.4.4 (ksi)
Fu = 60.00 ksi
Qn = 0.5(Asc)(f'c(Ec))^.5 = 30.0 kip per stud <= controls
but no greater than Asc(Fu) = 36.1 kip per stud

Article 6.10.10.3 Special Requirements for Points of Permanent Load Contra-flexure

For members that are noncomposite of negative flexure in the final condition, additional shear connectors shall be a provided in the region of points of permanent load contra-flexure. The number of additional connectors, nac, shall be taken as:
nac = As (fsr) / Zr
where: As = total area of longitudinal reinforcement over the interior support within the effective concrete deck width (in^2)
As = (111.0 inches)(0.31in^2 per bar)/(12.00 inch spacing) = 2.88 in^2
fsr = stress range in the longitudinal reinforcement over the interior support under the Fatique load combination specified in Table 3.4.1-1 with the fatique live load taken as specified in Article 3.6.1.4 (ksi)
max. stress range at point of contra-flexure is 13.1 ksi
Zr = fatique shear resistance of an individual shear connector determined as specified in Art. 6.10.10.2 (ki)
Zr = 4.20 kip per stud or 12.6 kip per row
therefore: nac = As(fsr) / Zr = 3.02 extra rows of studs
The additional shear connectors shall be placed within a distance equal to one-third of the effective concrete deck width on each side of the point of permanent load contra-flexure. Field splices should be placed so as not to interfere with the shear connectors.

Transverse Stiffener Design
Old Hickory Blvd. Over I-65
Davidson County
Designer: whp
Date: April 16, 2003

Article 6.10.11 Stiffeners

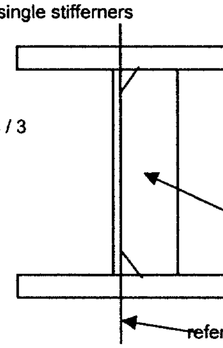
Article 6.10.11.1.1 Transverse Intermediate Stiffener Design:

Transverse stiffeners shall consist of plates or angles welded or bolted to either one or both sides of the web .
Stiffeners not used as connection plates shall be a tight fit at the compression flange, but need not be in bearing with the tension flange.
Stiffeners used as connection plates for diaphragms or cross-frames shall be connected by welding or bolting to both flanges.
The distance between the end of the web-to-stiffner weld and the near edge of the web to flange fillet weld shall not be less than 4(tw) or more than 6(tw)

Assume transverse stiffener sized: width: 6.000 inches thickness: 0.500 inches
Height: 54.000 inches

Article 6.10.11.1.2 Projecting Width:
The width, bt, of each projecting stiffener element shall satisfy: bt => 2.0 + d / 30 and 16.0(tp) => bt => 0.25(bf)
where: d = steel section depth (inches) = 59.500 inches
tp = thickness of the projecting element (inches) = 0.500 inches
Fys = specified minimum yeild strenght of the stiffener (ksi) = 50.000 ksi
bf = full width of the wider steel flange at a section (inches) = 24.000 inches
bt => 2.0 + d / 30 = 3.983 inches <= 6.000 inches OK
and 16.0(tp) = 8.000 inches
0.25(bf) = 6.000 inches
which, 8.000 inches => 6.000 inches => 6.000 inches OK

Article 6.10.11.1.3 Moment of Inertia:
The moment of inertia of any transverse stiffener shall satisfy: It => do(J)(tw)^3
where J = 2.5(D/do)^2 - 2.0 => 0.50
It = moment of interia of the transverse stiffener taken about the edge in contact with the web for single stiffeners
and about the midthickness of the web for stiffener pairs (inches^4)
tw = web thichness (inches) = 0.500 inches
do = transverse stiffener spacing (inches) = 41.000 inches
D = web depth (inches) = 54.000 inches
Which the moment interia taken at the contact surface between the web and stiffener, It = (tp)(bt)^3 / 3
It = (tt)(bt)^3 / 3 = 36.000 inches^4
therefore J = 2.5(D / do)^2 - 2.0 = 2.337 => 0.50, OK
do(J)(tw)^3 = 11.976 inches^4, It > do(J)(tw)^3, OK



Article 6.10.11.1.4 Area of stiffener:
Transverse intermediate stiffeners required to carry forces imposed by tension field action of the web as
specified in Article 6.10.9.3 shall satisfy:
As => (0.15(B)(D /tw)(1 - C)(Vu / Vr) -18)(Fyw / Fcrs)(tw^2)
which: Fcrs = 0.311(E) / (bt / tp)^2 <= Fys
Vr = factored shear resistance as specified in Article 6.10.9.1 (kip)
Vu = shear due to factored loads at the strength limit state (kip)
As = stiffener area: total area of both stiffeners for pairs (inches^2) = 3.000 inches^2
B = 1.0 for stiffener pairs, B = 1.8 for single angles, B = 2.4 for single plate stiffners,
C = ratio of the shear buckling stress to the shear yield strength as specified in Art. 6.10.9.3.2,
Fyw = spec. minmum yield strenght of the web (ksi) = 50.00 ksi
Fys = specified min. yield strenght of the stiffener (ksi) = 50.00 ksi

do	C	Vu	max.Vr	B	Asmin	As	check
41.00	0.91	484	514	2.4	-3.676	3.000	OK
61.50	0.67	470	544	2.4	-1.729	3.000	OK
82.00	0.54	457	562	2.4	-0.864	3.000	OK
121.50	0.45	389	506	2.4	-0.390	3.000	OK
162.00	0.42	375	364	2.4	1.308	3.000	OK

Therefore use : 6.000 inches X 0.500 inches X 54.000 inches for Transverse stiffeners

Cross-Frame Stiffeners:

Stiffeners used as connecting plates for diaphragms or cross-frames shall be connected by welding or bolting to both flanges.
For Webs in which: D / tw <= 2.5 (E / Fyw)^.5, stiffeners used as connection plates are only requried to satisfy the provision of Art. 6.10.11.1.2
D/tw = 108.00 => 2.5(E / Fyw)^.5 = 60.21 must meet the provisions of Art. 6.10.11.1.2, Art. 6.10.11.1.3, and Art. 6.10.11.1.4
Article 6.10.11.1.2 Projecting Width
Minmum width required for frame attachment: bt = 1" clear + 1.5 " + 3.0" + 1.5" = 7.0" & tw = 0.500 inches
Projecting width: The width, bt, of each projecting stiffener element shall satisfy: bt => 2.0 + d / 30 and 16.0(tp) => bt => 0.25(bf)
bt => 2.0 + d / 30 = 3.983 inches <= 7.000 inches OK
and 16.0(tp) = 8.000 inches
0.25(bf) = 1.750 inches
8.000 inches => 7.000 inches => 1.750 inches OK
Article 6.10.11.1.3 Moment of Inertia
Moment of Inertia: The moment of inertia of any transverse stiffener shall satisfy: It => do(J)(tw)^3
where J = 2.5(D/do)^2 - 2.0 => 0.50
Which the moment interia taken at the contact surface between the web and stiffener, It = (tp)(bt)^3 / 3
It = (tt)(bt)^3 / 3 = 57.167 inches^4
therefore J = 2.5(D / do)^2 - 2.0 = 2.337 => 0.50, OK
do(J)(tw)^3 = 11.976 inches^4, It > do(J)(tw)^3, OK
Article 6.10.11.1.4 Area of Stiffners
Area of Stiffener: As => (0.15(B)(D /tw)(1 - C)(Vu / Vr) -18)(Fyw / Fcrs)(tw^2)
which: Fcrs = 0.311(E) / (bt / tp)^2 <= Fys
worest case would be max. stiffener spacing do = 162 inches
and C = 0.420 and B = 1.0000 for stiffener pairs
Vu / Vr = 1.000 which Fcr = 46.015 ksi
mim As = -2.151 inches^2, which < 3.5 inches^2 OK

Therefore use : 7.000 inches X 0.500 inches X 54.000 inches for cross-frame stiffeners

Bearing Stiffener Design
Old Hickory Blvd. Over I-65
Davidson County
Designer: whp
Date: November 21, 2003

<u>Article 6.10.11.2 Bearing Stifferners</u>
Bearing Stiffener Design:
General requirements:
Bearing reactions and other concentrated loads, either in the final state or during construction, shall be resisted by bearing stiffeners.
Bearing stiffeners shall be placed on the webs of plate girders at all bearing locations and at all locations supporting concentrated loads.
Bearing stiffeners shall consist of one or more plates or angles welded or bolted ot both sides of the web. The connection to the web shall be designed to transmit the full bearing force due to the factored loads.
The stiffeners shall extend to the full depth of the web and, as closely as practical, to the outer edges of the flanges.
Each stiffener shall be either milled to fit against the flange through which it receives its reaction or attached to that flange by a full penetration groove weld.
Pier:
Bearing Stiffener located at the CL of Pier: assume the following: **11.750** inches X **0.750** inches
use Grade 50W steel, Fys = **50.00** ksi Bottom flange width = **24.000** inches

Article 6.10.11.2.2 Projecting Width
Projecting Width: The width, bt, of each projecting stiffener element shall satisfy: bt <= 0.48(tp)(E / Fys)^0.5
where: tp = thickness of projecting element (in) and Fys = specified minimum yield strength of the stiffener (ksi)
mim bt = 0.48(tp)(E / Fys)^0.5 = 8.670 inches < 11.750 inches OK

Article 6.10.11.2.3 Bearing Resistance
Bearing Resistance: The factored bearing resistance for fitted ends of bearing stiffeners shall be taken as:
(Rsb)r = (Ob)(Rsb)n
in which: (Rsb)n = nominal bearing resistance for the fitted ends of bearing stiffeners (kips) = 1.4(Apn)(Fys)
where: Ob = resistance factor for bearing specified in Art. 6.5.4.2, Ob = 1.00
Apn = area of the projecting elements of the stiffener outside of the web to flange fillet weld but not beyound the edge of the flange (in^2)
use a 1 inch clip on the bearing stiffener on the bottom flange. Apn = 8.0625 inches^2
allowable (rRsb) = 1.4 (Apn) (Fys) (Ob) = 564.375 ksi
Strenght I reaction at the centerline of the Pier = **958.40** kips
actual bearing stress = 118.871318 ksi, < 564.375 ksi, OK

Article 6.10.11.2.4 Axial Resistance of Bearing Stiffeners:
The factored axial resistance, Pr, shall be determined as specified in Art. 6.9.2.1 using the specified minimum yield strenght of the stiffener plates, Fys. The radius of gyration shall be computed about the mid-thickness of the web and the effective length shall be 0.75(D), where D is the web depth.
Article 6.10.11.2.4b Effective Section
Effective Section: For stiffeners consisiting of two plates welded to the web, the effective column section shall consist of the two stiffener elements, plus a centrally located strip of web extending not more than 9(tw) on each side of the stiffeners. If more than one pair of stiffeners is used, the effective column section shall consist of all stiffener elements, plus a centrally located strip of web extending not more than 9(tw) on each side of the outer projecting elements of the group. The strip of web shall not be included in the effective section at interior supports of continuous span hybrid members if: Fyw / Fyf < 0.70

Art. 6.9.2.1 Axial Compression: Pr = (Yc)(Pn)
where: Yc = resistance factor for compression (art. 6.5.4.2) = 0.90
Pn = nominal compressive resistance as specified in art. 6.9.4 and art. 6.9.5 (kip)

Art. 6.9.4 Non-composite Nominal Compressive resistance:
For members that saifisy the width/thickness requriements specified in art. 6.9.4.2, the nominal compressive resistance,Pn, shall be taken as: if (lamda) <= 2.25, then Pn =(0.66^(lamda))(Fy)(As)
if (lamda) > 2.25, then Pn =(0.88(Fy)(As)) / (lamda)
which lamda = ((K(l) / (rs)(3.1428))^2)(Fy / E)
As = gross cross-sectional area (in^2) tw = thickness of web = **0.500** inches
Fy = yield strength of the material (ksi) D = depth of web = **54** inches
K = effective length factor specified in art. 4.6.2.5 = 0.75 for bolted and welded connections
l = unbraced length = (0.75)(depth of web) = 40.5 inches
rs = radius of gyration about the mid-thickness of the web
area of section = (2)(bearing stiffener width)(bearing stiffener thichness) + (2)(9)(Thickness of the web) = 26.625 inches^2
radius of gyration = (moment of Inertia of section / area of section)^0.50
Moment of inertia about the mid-thickness of web = (2.0)((ts)(bt)^3) /12 + (2.0)(9)(tw)(tw)^3 /12 + (2.0)(ts)(bt)(dist. Cl web to CL stiffener)^2
Moment of inertia about the mid-thickness of web = 864.09 inches^4 lamba = 0.0050
radius of gyration = 5.70 inches
also must satifisy Art. 6.9.2, bt / ts <= (k)(E / FY)^0.5 which k = 0.45 for plates supported on one side
bt / ts = 15.667
(k)(E / Fy)^0.5 = 18.062 greater than 15.667 OK
since (lamda) <= 2.25, then Pn =(0.66^(lamda))(Fy)(As) = 1328.482 kips
Pr = (Yc)(Pn) = 1195.634 kips
Strenght I reaction at the centerline of the Pier = 958.40 kips, < 1195.634 kips, OK

Therefore use: 11.750 inch X 0.750 inch X 54.00 inch, bearing stifferner at Pier

111

Bearing Stiffener Design (cont:)
Abutment:
Bearing Stiffener located at the Abutment: assume the following: 8.750 inches X 0.750 inches
use Grade 50W steel, Fys = 50.00 ksi Bottom flange width = 18.000 inches

Projecting Width: The width, bt, of each projecting stiffener element shall satisfy: bt <= 0.48(tp)(E / Fys)^0.5
where: tp = thickness of projecting element (in) and Fys = specified minimum yield strength of the stiffener (ksi)
mim bt = 0.48(tp)(E / Fys)^0.5 = 8.670 inches < 8.750 inches OK

Bearing Resistance: The factored bearing resistance, Br, shall be taken as Br = (Yb)(Apn)(Fys)
where: Apn = area of the projecting elements of the stiffener outside of the web to flange fillet welds but not beyound the edge of the flange. (inches^2)
Yb = resistance factor for bearing specified in Article 6.5.4.2 = 1.00 for bearing on milled surfaces
use a 1 inch clip on the bearing stiffener on the bottom flange. Apn = 5.8125 inches^2
allowable Br = (Yb)(Apn)(Fys) = 290.625 ksi
Strenght I reaction at the centerline of the Abut. = 392.30 kips
actual bearing stress = 67.4924731 ksi, < 290.625 ksi, OK

Axial Resistance of Bearing Stiffeners:
The factored axial resistance, Pr, shall be determined as specified in Art. 6.9.2.1. The radius of gyration shall be computed about the mid-thickness of the web and the effective length shall be 0.75(D), where D is the web depth.

Effective Section: For stiffeners consisiting of two plates welded to the web, the effective column section shall consist of the two stiffener elements, plus a centrally located strip of web extending not more than 9(tw) on each side of the stiffeners. If more than one pair of stiffeners is used, the effective column section shall consist of all stiffener elements, plus a centrally located strip of web extending not more than 9(tw) on each side of the outer projecting elements of the group. The strip of web shall not be included in the effective section at interior supports of continuous span hybrid members if: Fyw / Fyf < 0.70

Art. 6.9.2.1 Axial Compression: Pr = (Yc)(Pn)
where: Yc = resistance factor for compression (art. 6.5.4.2) = 0.90
Pn = nominal compressive resistance as specified in art. 6.9.4 and art. 6.9.5 (kip)

Art. 6.9.4 Non-composite Nominal Compressive resistance:
For members that saifisy the width/thickness requriements specified in art. 6.9.4.2, the nominal compressive resistance,Pn, shall be taken as: if (lamda) <= 2.25, then Pn =(0.66^(lamda))(Fy)(As)
if (lamda) > 2.25, then Pn =(0.88(Fy)(As)) / (lamda)
which lamda = ((K(l) / (rs)(3.1428))^2)(Fy / E)
As = gross cross-sectional area (in^2) tw = thickness of web = 0.500 inches
Fy = yield strength of the material (ksi) D = depth of web = 54 inches
K = effective length factor specified in art. 4.6.2.5 = 0.75 for bolted and welded connections
l = unbraced length = (0.75)(depth of web) = 40.5 inches
rs = radius of gyration about the mid-thickness of the web
area of section = (2)(bearing stiffener width)(bearing stiffener thichness) + (2)(9)(Thickness of the web) = 22.125 inches^2
radius of gyration = (moment of Inertia of section / area of section)^0.50
Moment of inertia about the mid-thickness of web = (2.0)((ts)(bt)^3) /12 + (2.0)(9)(tw)(tw)^3 /12 + (2.0)(ts)(bt)(dist. Cl web to CL stiffener)^2
Moment of inertia about the mid-thickness of web = 364.59 inches^4 lamba = 0.0099
radius of gyration = 4.06 inches

also must satifisy Art. 6.9.2, bt / ts <= (k)(E / FY)^0.5 which k = 0.45 for plates supported on one side
bt / ts = 11.667
(k)(E / Fy)^0.5 = 18.062 greater than 11.667 OK

Therefore since (lamda) <= 2.25, then Pn =(0.66^(lamda))(Fy)(As) = 1101.724 kips
Pr = (Yc)(Pn) = 991.552 kips
Strenght I reaction at the centerline of the Pier = 392.30 kips, < 991.552 kips, OK

Therefore use:	8.750 inch	X	0.750 inch	X	54.00 inch, bearing stifferner at Abutments

AISIsplice, Analysis and Design Software
for Bolted Splices of Steel Bridges
Version 3.0
Dr. Firas I. Sheikh-Ibrahim, PE
HDR Engineering, Inc.
3 Gateway Center
Pittsburgh, PA 15222-1074
http://www.hdrinc.com/

PROJECT NAME: SR 45 (Old Hickory Blvd.) over I-65
INPUT FILE NAME: C:\AISCSplice\Splice\sr42OLB.dat
RUN DATE & TIME: Thursday, Mar 27, 2003, 10:00 AM

I. SPLICE ANALYSIS SUMMARY:

1. Top Flange Splice meets ALL Design Requirements.

2. Bottom Flange Splice meets ALL Design Requirements.

3. Web Splice meets ALL Design Requirements.

1. PROBLEM DEFINITION:

Analysis Type:	Splice Capacity Check
Unit Preference:	Customary (US)
AASHTO Analysis/Design Method:	1999 AASHTO-LRFD
Span Type:	Continuous Span

Load Factors (AASHTO 3.4.1&2):	
Strength DC Maximum/Minimum:	1.25, 0.90
Strength DW Maximum/Minimum:	1.50, 0.65
Strength LL+IM:	1.75
Strength IV DC:	1.50
Service DC and DW:	1.00
Service LL+IM:	1.30
Service DW Minimum:	0.00
Fatigue LL+IM:	0.75
Fatigue Life :	75
Construction:	1.25

Resistance Factors (AASHTO 6.5.4):	
Flexure, Øf:	1.00
Shear, Øv:	1.00
Compression, Øc:	0.90
Tension Net Fracture, Øu:	0.80
Tension Gross Yield, Øy:	0.95
Bolt Bearing, Øbb:	0.80
Bolt Shear, Øs:	0.80
Block Shear, Øbs:	0.80
Minimum Longitudinal Bolt Rows:	2 (web), 2 (flange)
Inner/Outer Splice Area Variance:	10.0 % (maximum)

Load Modifier (AASHTO 1.3.2.1):	1.00
Ductility Factor :	1.00
Redundancy Factor :	1.00
Operational Importance Factor:	1.00

Applied Minimums (AASHTO 6.13.6.1.4): 1999 AASHTO LRFD (Flexure & Shear)

Unfactored Loads at the Splice Centerline:

Component:	Moment (K·ft)	Shear (Kip)
DC 1	707.0	59.0

DECK	0.0	0.0
DC 2	64.0	25.0
DW	157.1	13.1
+(LL+I)	2,808.0	20.3
-(LL+I)	-1,624.0	116.0
FATIGUE+(LL+I)	1,370.0	37.6
FATIGUE-(LL+I)	0.0	0.0

Left Plate Girder Properties:
- Flange Steel/Web Steel: M270 Gr50W
- Top Flange Dimension: 1.0 in × 18.0 in
- Bottom Flange Dimension: 1.25 in × 18.0 in
- Web Dimension: 0.5 in × 54.0 in
- Web Shear Nominal Strength: 454.0 K

Right Plate Girder Properties:
- Flange Steel/Web Steel: M270 Gr50W
- Top Flange Dimension: 1.0 in × 24.0 in
- Bottom Flange Dimension: 1.0 in × 24.0 in
- Web Dimension: 0.5 in × 54.0 in
- Web Nominal Shear Strength: 454.0 K

Number of Flange Splice Plates: 3
Girder Clear Gap: 0.5 in
Girder Alignment: Web Center
ADTT/Truck Lane Availability: 178, 3-Lanes (or more)

Connection Bolt Properties:
- Bolt Designation: 7/8 in AASHTO M164 (A325)
- Design Bolt Hole Size: 1 in
- Minimum Pitch/Gage Spacing: 3 in
- Minimum Edge/End Distance: 1.5 in

- Faying Surface Class: B
- Bolt Slip Resistance: 39.0 K/bolt/2planes
- Web Bolt Threads: Excluded
- Web Bolt Shear Resistance: 55.4 K/bolt/2planes
- Flange Bolt Threads: Excluded
- Flange Bolt Shear Resistance: 55.4 K/bolt/2planes

Concrete Slab Design Type: Normal Weight Concrete
Concrete Slab Properties:
- Reinforcing Steel Area: 4.4 in^2
- Reinforcing Steel Centroid: 1.5 in (from slab bottom)
- Reinforcing Steel Fy: 60.0 ksi
- Compressive Strength, fc': 3.0 ksi
- Slab Thickness: 9.0 in
- Effective Width: 111.0 in
- Haunch Depth: 1.5 in (above left girder)

Span Length: 162.0 ft
Splice Located Near Int. Support: YES

Top Flange Splice Properties:
- Plate Steel: M270 Gr50W
- Outer Plate Dimension: 1 - .5 × 18 × 27 in
- Inner Plate Dimension: 2 - .5625 × 8.25 × 27 in
- Longitudinal Bolt Pattern: 4 rows of 4 Bolts @ 3 in Pitch
- Dimensions X1, X2, X3, X4: 6 in, 1.5 in, 6 in, 2 in

Bottom Flange Splice Properties:
- Plate Steel: M270 Gr50W
- Outer Plate Dimension: 1 - .625 × 18 × 31 in
- Inner Plate Dimension: 2 - .6875 × 8.25 × 31 in
- Longitudinal Bolt Pattern: 5 rows of 4 Bolts @ 3 in Pitch
- Dimensions X5, X6, X7, X8: 4 in, 1.5 in, 6 in, 2 in

Web Splice Properties:
- Plate Steel: M270 Gr50W

Plate Dimension: 2 - .375 × 13 × 51.75 in
Longitudinal Bolt Pattern: 2 rows of 16 Bolts @ 3.25 in Spacing
Dimensions X9, X10, X11: 4 in, 1.5 in, 3 in

Calculated Section Properties:

Section Type:		Noncomposite	-Composite*	nComposite	3nComposite
I steel	(in4)	37,111.3	41,575.6	91,882.7	68,120.1
S top of Steel	(in^3)	1,242.2	1,492.1	12,143.9	3,966.7
S top of web	(in^3)	1,285.2	1,547.7	13,993.3	4,212.0
S bottom of web	(in^3)	1,477.1	1,532.1	1,937.1	1,800.8
S bottom of steel	(in^3)	1,407.1	1,464.6	1,887.3	1,743.2

Calculated Section Properties used for Positive Strength:

Effective Tension Flange Area (Bottom Flange): (AASHTO 6.10.3.6)
ß = (An/Ag)/[(Øu×Futf/Øy×Fytf)-1] (>=0) = 0.139
Ae = An + ß×Ag (<=Ag) = 20.6 in^2
Ae/Ag = 0.917

Section Type:		Noncomposite	-Composite*	nComposite	3nComposite
I steel	(in4)	35,836.9	NA	87,521.5	65,307.0
S top of Steel	(in^3)	1,229.7	NA	12,400.7	3,964.5
S top of web	(in^3)	1,273.4	NA	14,447.7	4,220.8
S bottom of web	(in^3)	1,385.9	NA	1,825.6	1,695.1
S bottom of steel	(in^3)	1,322.0	NA	1,779.2	1,641.8

Calculated Section Properties used for Negative Strength:

Effective Tension Flange Area (Top Flange): (AASHTO 6.10.3.6)
ß = (An/Ag)/[(Øu×Futf/Øy×Fytf)-1] (>=0) = 0.14
Ae = An + ß×Ag = 16.5 in^2
Ae/Ag = 0.917

Section Type:		Noncomposite	-Composite*	nComposite	3nComposite
I steel	(in4)	35,792.2	40,432.6	NA	NA
S top of Steel	(in^3)	1,172.0	1,421.5	NA	NA
S top of web	(in^3)	1,211.6	1,473.3	NA	NA
S bottom of web	(in^3)	1,463.3	1,522.5	NA	NA
S bottom of steel	(in^3)	1,392.2	1,454.1	NA	NA

*includes the concrete reinforcing steel

2. DESIGN LIMIT STATES:

2.1 During Construction (AASHTO 3.4.2):

Mconstruction: 883.8 K·ft
Vconstruction: 73.8 K

STRESS DISTRIBUTION

STRESS COMPONENT	TOP OF STEEL (K/in^2)	TOP OF WEB (K/in^2)	BOT OF WEB (K/in^2)	BOT OF STEEL (K/in^2)
f(CON):	-8.54	-8.25	7.18	7.54

=> Average: (fTF): -8.39 (fBF): 7.36

Pcon (TF) = -151.1 K
Pcon (BF) = 165.6 K
Vcon (Web) = 73.8 K
Mcon (Web) = 156.2 K·ft
Hcon (Web) = -14.5 K

2.2 Fatigue Limit State (AASHTO 3.4.1):

Threshhold: 12.71 ksi (AASHTO 6.6.1.2.5)

+Mfatigue: 1,027.5 K·ft
-Mfatigue: 0.0 K·ft
+Vfatigue: 28.2 K
-Vfatigue: 0.0 K

STRESS DISTRIBUTION

STRESS COMPONENT	TOP OF STEEL (K/in²)	TOP OF WEB (K/in²)	BOT OF WEB (K/in²)	BOT OF STEEL (K/in²)
f(+FAT):	-1.02	-0.88	6.37	6.53
f(-FAT):	0.0	0.0	0.0	0.0

=> +Average: +(fTF): -0.95 +(fBF): 6.45
-Average: -(fTF): 0.00 -(fBF): 0.00
Range: FfatTF: NA (6.6.1.2.1) FfatBF: 6.45

Pfat (TF) = 0.0 K·ft
Pfat (BF) = 145.1 K·ft
Vfat (Web) = 28.2 K
Mfat (Web) = 73.4 K·ft
Hfat (Web) = 74.0 K

2.3 Strength Limit State (AASHTO 3.4.1):

2.3.1a For POSITIVE moment:

POSITIVE STRESS DISTRIBUTION

STRESS COMPONENT	TOP OF STEEL (K/in²)	TOP OF WEB (K/in²)	BOT OF WEB (K/in²)	BOT OF STEEL (K/in²)
f(DC1):	-6.9	-6.7	6.1	6.4
f(DC2):	-0.2	-0.2	0.5	0.5
f(DW):	-0.5	-0.4	1.1	1.1
f(+LL+I):	-2.7	-2.3	18.5	18.9
+ffactored:	-14.3	-13.3	42.2	43.5

=> +Average: +fTF : -13.8 +fBF : 42.8

Apply Flexural Strength Minimums:
(Noncompact Girder)
+Fcf = ½(+fcf/Rh + ALPHA·Øf·Fyf) >= 0.75·ALPHA·Øf·Fyf
+Rcf = |Fcf/fcf|
+Fncf = Rcf |fncf/Rh| >= 0.75·ALPHA·Øf·Fyf

+F(TF)= [-31.91 >= -37.50] = -37.50 (+F(TF)/f(TF)=2.71)
+F(BF)= [46.415 >= 37.50] = 46.415 (+F(BF)/f(BF)=1.08)
=> The bottom flange is the controlling flange
=> +Fcf = +F(BF)
=> +Rcf = 1.08

+Muw = (1×54²/12)×|42.2-(-13.3)|×1.08/12
+Muw = 561.9 × 1.08 = 608.9 K·ft

+Huw = (1×54/2)×[42.2+(-13.3)]×1.08/1
+Huw = 389.89 × 1.08 = 422.5 K

2.3.1b For NEGATIVE moment:

NEGATIVE STRESS DISTRIBUTION

STRESS COMPONENT	TOP OF STEEL (K/in²)	TOP OF WEB (K/in²)	BOT OF WEB (K/in²)	BOT OF STEEL (K/in²)
f(DC1):	-7.2	-7.0	5.8	6.1
f(DC2):	-0.5	-0.5	0.5	0.5
f(DW):	-1.3	-1.3	1.2	1.3
f(-LL+I):	13.7	13.2	-12.8	-13.4
-ffactored:	16.1	15.5	-15.9	-16.7

=> -Average: -fTF : 15.8 -fBF : -16.3

```
 Apply Flexural Strength Minimums:
     (Noncompact Girder)
-Fcf   = ½(-fcf/Rh + ALPHA·Øf·Fyf) >= 0.75·ALPHA·Øf·Fyf
-Rcf   = |Fcf/fcf|
-Fncf = Rcf |fncf/Rh| >= 0.75·ALPHA·Øf·Fyf

-F(TF)= [32.919 >= 37.50] = 37.50                   (-F(TF)/f(TF)=2.37)
-F(BF)= [-33.143 >= -37.50] = -37.50                (-F(BF)/f(BF)=2.30)
=> The bottom flange is the controlling flange
=> -Fcf   = -F(BF)
=> -Rcf   = 2.30

-Muw      = (-1×54²/12)×|-15.9 -(15.5)|×2.30/12
-Muw      = -318.6 × 2.30 = -733.6 K·ft

-Huw      = (1×54/2)×[-15.9+(15.5)]×2.30/1
-Huw      = -5.09 × 2.30 = -11.7 K

 2.3.2  For Shear:
 -----------------
 +Vstrength =       160.2 K
 -Vstrength =         0.0 K
  Vr =Øv·Vn = ±     454.0 K

 Apply Shear Strength Minimums:
 For Vu < 0.5Vr: Vuw = 1.5Vu
 Else:               Vuw = ½(Vu + Vr)

+Vuw ( Vu <  0.5Vr ) = 1.5Vu          = 240.3 K
-Vuw ( Vu <  0.5Vr ) = 1.5Vu          = 0.0 K

 2.3.3  Design Forces:
 --------------------
   Pu    (TF)   =-37.5 × 18.
   Pu    (TF)   =       -675.0 K (compression)

   Pu    (TF)   =37.5 × 16.5
   Pu    (TF)   =        618.9 K (tension)

   Pu    (BF)   =46.4 × 20.6
   Pu    (BF)   =        957.6 K (tension)

   Pu    (BF)   =-37.5 × 22.5
   Pu    (BF)   =       -843.8 K (compression)

   Vuw          =        240.3 K
   Muw          =        608.9 K·ft, -733.6 K·ft
   Huw          =        422.5 K, -11.7 K

2.4  Service Limit State (AASHTO 3.4.1):
_______________________________________

 2.4.1a  For POSITIVE Moment:
 ---------------------------:
```

POSITIVE STRESS DISTRIBUTION

STRESS COMPONENT	TOP OF STEEL (K/in^2)	TOP OF WEB (K/in^2)	BOT OF WEB (K/in^2)	BOT OF STEEL (K/in^2)
f(DC1):	-6.8	-6.6	5.7	6.0
f(DC2):	-0.2	-0.2	0.4	0.4
f(DW):	-0.5	-0.4	1.0	1.1
f(+LL+I):	-2.8	-2.4	17.4	17.9
+Ffactored:	-11.1	-10.4	29.8	30.8

```
 => +Average:      +fTF  : -10.7                    +fBF  : 30.3


 +Mser (web) = (1×54²/12)×|-10.4-(29.8)|/12
```

+Mser (web) = 406.9 K·ft

+Hser (web) = (1×54/2)×[-10.4+(29.8)]/1
+Hser (web) = 262.8 K

2.4.1b For NEGATIVE Moment:

NEGATIVE STRESS DISTRIBUTION

STRESS COMPONENT	TOP OF STEEL (K/in^2)	TOP OF WEB (K/in^2)	BOT OF WEB (K/in^2)	BOT OF STEEL (K/in^2)
f(DC1):	-6.8	-6.6	5.7	6.0
f(DC2):	-0.5	-0.5	0.5	0.5
f(DW):	-1.3	-1.2	1.2	1.3
f(-LL+I):	13.1	12.6	-12.7	-13.3
-Ffactored:	9.6	9.3	-10.3	-10.7

=> -Average: -fTF : 9.5 -fBF : -10.5

-Mser (web) = $(-1\times54^2/12)\times|9.3-(-10.3)|/12$
-Mser (web) = -198.1 K·ft

-Hser (web) = (1×54/2)×[9.3+(-10.3)]/1
-Hser (web) = -13.8 K

2.4.2 For Shear:

+Vservice = 123.5 K
-Vservice = 0.0 K

2.4.3 Design Forces:

Pser (TF) = -193.2 K (compression), 170.2 K (tension)
Pser (BF) = 681.7 K (tension), -236.6 K (compression)
Vser (Web) = 123.5 K
Mser (Web) = 406.9 K·ft, -198.1 K·ft
Hser (Web) = 262.8 K, -13.8 K

3. SUMMARY OF AASHTO SPLICE DESIGN FORCES:

DESIGN LOAD/LIMIT STATE:	CONSTRUCTION	FATIGUE	SERVICE	STRENGTH
Top Splice Force (K)	-151.1	0.0	-193.2	-675.0
			170.2	618.9
Bottom Splice Force (K)	165.6	145.1	681.7	957.6
			-236.6	-843.8
Web Splice Shear (K)	73.8	28.2	123.5	240.3
Web Splice Moment (K·ft)	156.2	73.4	406.9	608.9
			-198.1	-733.6
Web Splice Horizontal(K)	-14.5	74.0	262.8	422.5
			-13.8	-11.7

4. SPLICE ANALYSIS:

4.1 Top Flange Splice Analysis:

4.1.1 Plate Analysis:

Maximum Inner/Outer Plate Area Variation:
Maximum Req'd Area Percentage = 10.0% >= 3.1% *OK*
=> Total forces are thus divided EQUALLY between the Inner & Outer Splice.

The Strength Limit State / During Construction:

Yielding of the Gross Section:

Compression Agreq(Inner Splice)=|-337.5|/(50×0.90)= 7.5 in^2
Compression Agreq(Outer Splice)=|-337.5|/(50×0.90)= 7.5 in^2

Tension Agreq (Inner Splice) = 309.5 / (50×0.95) = 6.52 in^2
Tension Agreq (Outer Splice) = 309.5 / (50×0.95) = 6.52 in^2

Fracture of the Net Section:
Tension Anreq (Inner Splice) = 309.5 / (70×0.80) = 5.53 in^2
Tension Anreq (Outer Splice) = 309.5 / (70×0.80) = 5.53 in^2

The Fatigue Limit State (Inner and Outer Splices):
Agreq (Inner Splice) = 0.0 / 13 = 0.0 in^2
Agreq (Outer Splice) = 0.0 / 13 = 0.0 in^2

The Service Limit State:
Prevention of Permanent Deflection (Inner Splice):
Agreq (+M) = |-96.6| / (50×0.95) = 2.03 in^2
Agreq (-M) = |85.1| / (50×0.80) = 2.13 in^2

Prevention of Permanent Deflection (Outer Splice):
Agreq (+M) = |-96.6| / (50×0.95) = 2.03 in^2
Agreq (-M) = |85.1| / (50×0.80) = 2.13 in^2

Summary:

Criterion	Req'd Gross Area(in^2)	Req'd Net Area(in^2)
	INNER SPLICE PLATES	
Gross Yielding	7.5	NA
Net Fracture	NA	5.53
Fatigue	0.0	NA
Permanent Deflection	2.13	NA
½ Flange Area (min)	9.0	NA
Controlling Values =	9.0	5.53
Actual Inner Splice Area =	9.28 *OK*	7.03 *OK*
	OUTER SPLICE PLATE	
Gross Yielding	7.5	NA
Net Fracture	NA	5.53
Fatigue	0.0	NA
Permanent Deflection	2.13	NA
½ Flange Area (min)	9.0	NA
Controlling Values =	9.0	5.53
Actual Outer Splice Area =	9.0 *OK*	7.0 *OK*

4.1.2 Bolt Analysis:

Check 4 rows of 4 bolts @ 3 in pitch (Total Bolts = 16)

Plate Slip Resistance:
Single Slip Plane = 312.0 => 0.50Max(|Pser|,|Pcon|) = |-96.6| K *OK*

Bolt Shear Requirement:
Single Shear Plane = 443.3 => 0.50Pu = |-337.5| K *OK*

Plate Bearing Requirement:
Inner Splice (T) = 945.0 => 0.50Pu = 309.5 K *OK*
Outer Splice (T) = 840.0 => 0.50Pu = 309.5 K *OK*
Girder Flange(T) = 1,881.6 => 1.00Pu = 618.9 K *OK*

Inner Splice (C) = 1,058.4 => 0.50Pu = |-337.5| K *OK*
Outer Splice (C) = 940.8 => 0.50Pu = |-337.5| K *OK*
Girder Flange(C) = 1,881.6 => 1.00Pu = |-675.0| K *OK*

Minimum Bolt Rows Requirement:
Default Minimum Value Specified = 2 rows <= 4 rows *OK*

4.1.3 Spacing Limit Analysis:

All Pitch, Gage, Edge and End Distance values are in accordance with AASHTO Art. 6.13.2.6.

4.1.4 Block Shear Rupture Analysis:

Outer Splice Factored Design Load (0.50Pu(T)) = 309.5 K *OK*
Outer Splice Plate Resistance (Path 1) = 471.8 K

Inner Splice Factored Design Load (0.50Pu(T)) = 309.5 K *OK*
Inner Splice Plate Resistance (Path 4) = 565.4 K

Girder Flange Factored Design Load (1.0Pu(T)) = 618.9 K *OK*
Left Girder Flange Resistance (Path 6) = 1,015.9 K

4.2 Bottom Flange Splice Analysis:

4.2.1 Plate Analysis:

Maximum Inner/Outer Plate Area Variation:
Maximum Req'd Area Percentage = 10.0% >= 0.8% *OK*
=> Total forces are thus divided EQUALLY between the Inner & Outer Splice.

The Strength Limit State / During Construction:

Yielding of the Gross Section:

Compression Agreq(Inner Splice)=|-421.9|/(50×0.90)= 9.38 in^2
Compression Agreq(Outer Splice)=|-421.9|/(50×0.90)= 9.38 in^2

Tension Agreq (Inner Splice) = 478.8 / (50×0.95) = 10.08 in^2
Tension Agreq (Outer Splice) = 478.8 / (50×0.95) = 10.08 in^2

Fracture of the Net Section:
Tension Anreq (Inner Splice) = 478.8 / (70×0.80) = 8.55 in^2
Tension Anreq (Outer Splice) = 478.8 / (70×0.80) = 8.55 in^2

The Fatigue Limit State (Inner and Outer Splices):
Agreq (Inner Splice) = 72.6 / 13 = 5.71 in^2
Agreq (Outer Splice) = 72.6 / 13 = 5.71 in^2

The Service Limit State:
Prevention of Permanent Deflection (Inner Splice):
Agreq (+M) = |340.8| / (50×0.95) = 7.18 in^2
Agreq (-M) = |-118.3| / (50×0.80) = 2.96 in^2

Prevention of Permanent Deflection (Outer Splice):
Agreq (+M) = |340.8| / (50×0.95) = 7.18 in^2
Agreq (-M) = |-118.3| / (50×0.80) = 2.96 in^2

Summary:

Criterion	Req'd Gross Area(in^2)	Req'd Net Area(in^2)
	INNER SPLICE PLATES	
Gross Yielding	10.08	NA
Net Fracture	NA	8.55
Fatigue	5.71	NA
Permanent Deflection	7.18	NA

½ Flange Area (min)	11.25	NA
Controlling Values =	11.25	8.55
Actual Inner Splice Area =	11.34 *OK*	8.59 *OK*

OUTER SPLICE PLATE

Gross Yielding	10.08	NA
Net Fracture	NA	8.55
Fatigue	5.71	NA
Permanent Deflection	7.18	NA
½ Flange Area (min)	11.25	NA
Controlling Values =	11.25	8.55
Actual Outer Splice Area =	11.25 *OK*	8.75 *OK*

4.2.2 Bolt Analysis:

Check 5 rows of 4 bolts @ 3 in pitch (Total Bolts = 20)

Plate Slip Resistance:
Single Slip Plane = 390.0 => 0.50Max(|Pser|,|Pcon|) = 340.8 K *OK*

Bolt Shear Requirement:
Bolt Strength Reduction Factor = 0.86 (due to thick fillers, AASTHO 6.13.6.1.5)
Reduction Factor (Left) = 1.00 (Filler Thickness = 0 in)
Reduction Factor (Right)= 0.86 (Filler Thickness = 0.25 in)
Single Shear Plane = 478.6 => 0.50Pu = 478.8 K *OK*

Plate Bearing Requirement:
Inner Splice (T) = 1,478.4 => 0.50Pu = 478.8 K *OK*
Outer Splice (T) = 1,344.0 => 0.50Pu = 478.8 K *OK*
Girder Flange(T) = 2,217.6 => 1.00Pu = 957.6 K *OK*

Inner Splice (C) = 1,617.0 => 0.50Pu = |-421.9| K *OK*
Outer Splice (C) = 1,470.0 => 0.50Pu = |-421.9| K *OK*
Girder Flange(C) = 2,352.0 => 1.00Pu = |-843.8| K *OK*

Minimum Bolt Rows Requirement:
Default Minimum Value Specified = 2 rows <= 5 rows *OK*

4.2.3 Spacing Limit Analysis:

All Pitch, Gage, Edge and End Distance values are in accordance with AASHTO Art. 6.13.2.6.

4.2.4 Block Shear Rupture Analysis:

Outer Splice Factored Design Load (0.50Pu(B)) = 478.8 K *OK*
Outer Splice Plate Resistance (Path 1) = 633.3 K

Inner Splice Factored Design Load (0.50Pu(B)) = 478.8 K *OK*
Inner Splice Plate Resistance (Path 4) = 738.8 K

Girder Flange Factored Design Load (1.0Pu(B)) = 957.6 K *OK*
Left Girder Flange Resistance (Path 6) = 1,351.1 K

4.3 Web Splice Analysis:

4.3.1 Bolt Analysis:

Minimum Bolt Rows Requirement:
Default Minimum Value Specified = 2 rows <= 2 rows *OK*

Minimum Bolts/Row Requirement:
Minimum Bolts @ Maximum Spacing = 10 bolts <= 16 *OK*

Plate Slip Requirement:

Slip Resistance/Bolt = 39.0 K/bolt

During Construction: Crequired = 1.89
e = 28.9222 in
Theta = 11.1°
Cprovided = 9.11 *OK*

Service Limit State: Crequired = 3.17
e = 43.0448 in
Theta = 64.8°
Cprovided = 4.65 *OK*

Bolt Shear Requirement:

Shear Resistance/Bolt = 55.4 K/bolt

Strength Limit State: Crequired = 4.34
e = 33.9124 in
Theta = 60.4°
Cprovided = 5.77 *OK*

Plate Bearing Requirement (Left Girder Web):

Strength Limit State: Crequired = 5.72
e = 33.9124 in
Theta = 60.4°
Cprovided = 5.77 *OK*

Above resistances calculated w/r/to the given splice dimensions of:
Number of Bolts/Row = 16 bolts
Vertical Bolt Spacing = 3.25 in
Vertical End Distance = 1.5 in
Number of Bolt Rows = 2 rows
Longitudinal Bolt Spacing = 3 in
Longitudinal End Distance = 1.5 in
Geometric Eccentricity = 3.5 in
Splice Plate Thickness = 0.375 in

4.3.2 Plate Analysis:

Shear Analysis:
The Strength Limit State / During Construction:

Yielding of the Gross Section:
Agreq = |240.3| / (1.00×0.58×50) = 8.28 in²

Fracture of the Net Section:
Anreq = |240.3| / (0.80×0.58×70) = 7.4 in²

Summary:

Criterion	Req'd Gross Area(in²)	Req'd Net Area(in²)
Gross Yielding	8.28	NA
Net Fracture	NA	7.4
Controlling Values =	8.28	7.4
Actual Web Splice Area =	38.81 *OK*	26.81 *OK*

Flexural Analysis:
Splice Plates' Elastic Section Modulus, Sxx = 334.8in³
Web Splice Bolt Group Eccentricity, e = 3.5 in

During Construction:
fm = 6.37 ksi
fh = 0.37 ksi
fcon = 6.74 ksi <= 50 ksi *OK*

The Fatigue Limit State:

+fm = 2.92 ksi
+fh = 1.91 ksi
-fm = 0.00 ksi
-fh = 0.00 ksi
ffat = 4.83 ksi <= 13 ksi *OK*

The Service Limit State:
fm = 15.88 ksi
fh = 6.77 ksi
fser = 22.65 ksi <= 48 ksi *OK*

The Strength Limit State:
fm = 24.34 ksi
fh = 10.89 ksi
fstr = 35.23 ksi <= 50 ksi *OK*

4.3.3 Spacing Limit Analysis:

All Spacing, Edge, and End Distance values are in accordance with AASHTO Art. 6.13.2.6.

4.3.4 Block Shear Rupture Analysis:

Web Splice Factored Design Load (Vuw) = 240.3 K *OK*
Web Splice Resistance (Path 1) = 981.5 K

5. SUMMARY:

PERFORMANCE RATIOS:

Performance Ratios are the ratio of the applied load to the respective omponent resistance (i.e., LOAD / RESISTANCE).

TOP FLANGE SPLICE (1.00):

Splice Plates (1.00):	
Gross Area (Outer Splice)	1.00
Gross Area (Inner Splice)	0.97
Net Area (Outer Splice)	0.79
Net Area (Inner Splice)	0.79
Connection Bolts (0.76):	
Bolt Slip	0.31
Bolt Shear	0.76
Plate Bearing	0.37
Block Shear	0.66

BOTTOM FLANGE SPLICE (1.00):

Splice Plates (1.00):	
Gross Area (Outer Splice)	1.00
Gross Area (Inner Splice)	0.99
Net Area (Outer Splice)	0.98
Net Area (Inner Splice)	0.99
Connection Bolts (1.00):	
Bolt Slip	0.87
Bolt Shear	1.00
Plate Bearing	0.43
Block Shear	0.76

WEB SPLICE (0.99):

Connection Bolts (0.99):	
Bolt Slip - Construction	0.21

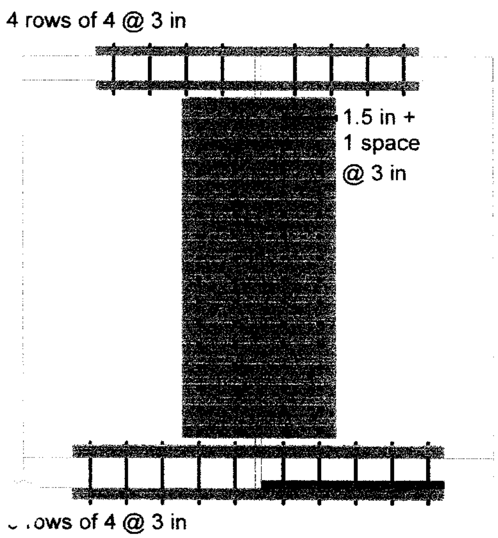
Bolt Slip - Service	0.68
Bolt Shear - Strength	0.75
Plate Bearing - Strength	0.99
Splice Plates (0.70):	
Gross Area - Strength	0.21
Net Area - Strength	0.28
Flexure - Construction	0.13
Flexure - Fatigue	0.38
Flexure - Service	0.48
Flexure - Strength	0.70
Block Shear	0.24

REQUIRED FILLERS:
(1) 0.25 × 18 × 15.25 in

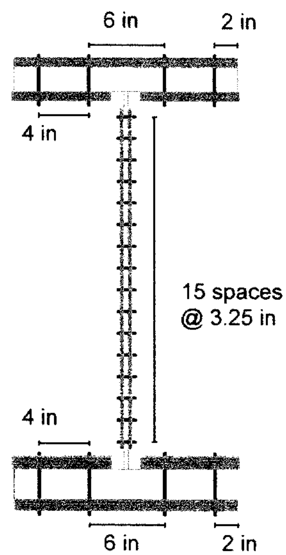
AISIsplice, Analysis and Design Software
for Bolted Splices of Steel Bridges
Version 3.0

Dr. Firas I. Sheikh-Ibrahim, PE
HDR Engineering, Inc.
3 Gateway Center
Pittsburgh, PA 15222-1074
http://www.hdrinc.com/

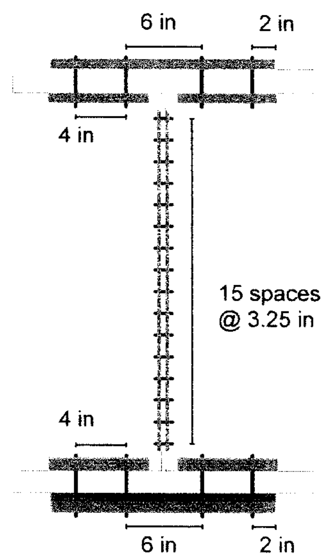
PROJECT NAME: SR 45 (Old Hickory Blvd.) over I-65
INPUT FILE NAME: C:\AISCSplice\Splice\sr42OLB.dat
RUN DATE & TIME: Thursday, Mar 27, 2003, 9:58 AM



Elevation



Left Cross Section



Right Cross Section

Girders
Splice Plates
Filler Plates

Top Flange Splice (in):
M270 Gr50W Plates
1- 0.5 × 18 × 27
2- 0.5 × 8.25 × 27
4 Rows of 4 Bolts @
3in Pitch

Bottom Flange Splice (in):
M270 Gr50W Plates
1- 0.625 × 18 × 31
2- 0.6875 × 8.25 × 31
5 Rows of 4 Bolts @
3in Pitch

Web Splice (in):
M270 Gr50W Plates
2- 0.375 × 13 × 51.75
2 Rows of 16 Bolts @
3.25in Spacing

ALL BOLTS:
7/8 in AASHTO M164 BOLTS
(A325-X Flanges)
(A325-X Web)
Faying Surface Class = B

PERFORMANCE RATIOS:
Top Flange Bolts........0.76
Top Flange Plates.......1.09(WARNING)
Bottom Flange Bolts.....1.00
Bottom Flange Plates....1.00
Web Bolts...............0.99
Web Plates..............0.70

REQUIRED FILLERS:
(1) 0.25 × 18 × 15.25 in

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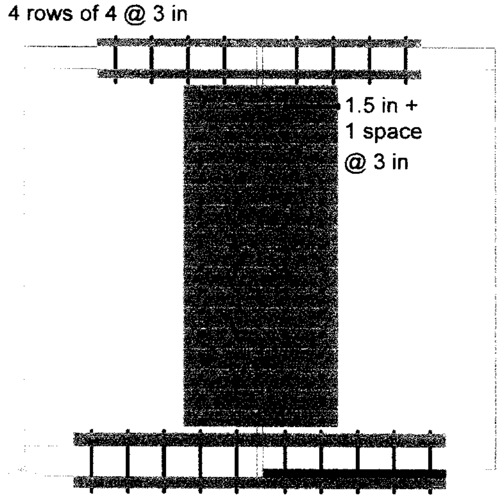
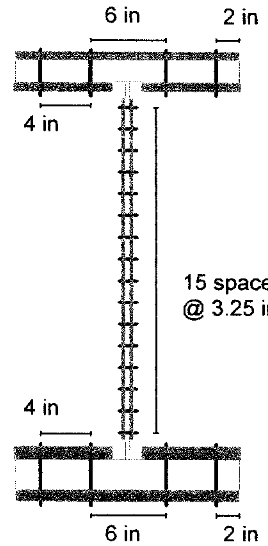
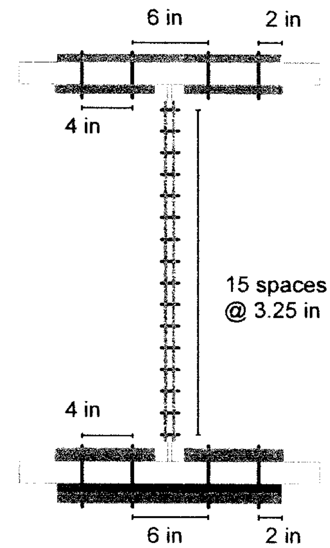
PROJECT NAME: SR 45 (Old Hickory Blvd.) over I-65
INPUT FILE NAME: C:\AISCSplice\Splice\sr42OLB.dat
RUN DATE & TIME: Thursday, Mar 27, 2003, 10:00 AM



Elevation



Left Cross Section



Right Cross Section

Top Flange Splice (in):
M270 Gr50W Plates
1- 0.5 × 18 × 27
2- 0.5625 × 8.25 × 27
4 Rows of 4 Bolts @
3in Pitch

Bottom Flange Splice (in):
M270 Gr50W Plates
1- 0.625 × 18 × 31
2- 0.6875 × 8.25 × 31
5 Rows of 4 Bolts @
3in Pitch

PERFORMANCE RATIOS:
Top Flange Bolts........0.76
Top Flange Plates.......1.00
Bottom Flange Bolts.....1.00
Bottom Flange Plates....1.00
Web Bolts...............0.99
Web Plates..............0.70

Web Splice (in):
M270 Gr50W Plates
2- 0.375 × 13 × 51.75
2 Rows of 16 Bolts @
3.25in Spacing

ALL BOLTS:
7/8 in AASHTO M164 BOLTS
(A325-X Flanges)
(A325-X Web)
Faying Surface Class = B

REQUIRED FILLERS:
(1) 0.25 × 18 × 15.25 in

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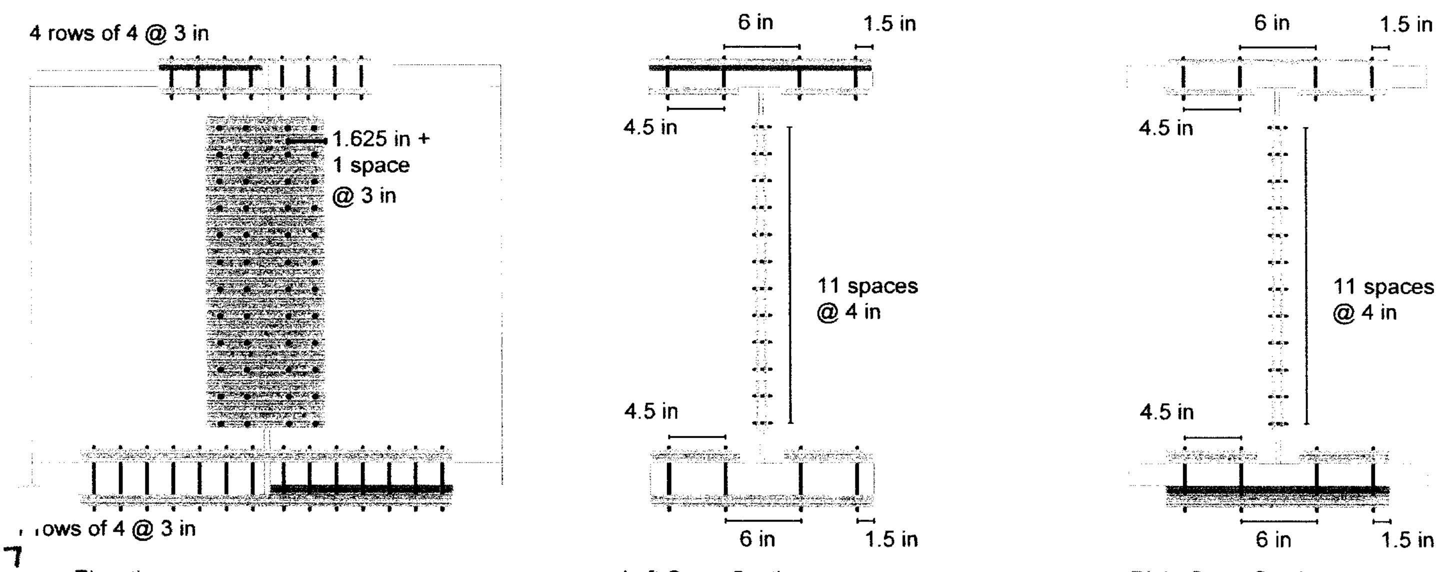
Revised Splice design due to plate changes in fatigue design

PROJECT NAME: SR 45 (Old Hickory Blvd.) over I-65
INPUT FILE NAME: C:\AISCSplice\Splice\sr42OLB.dat
RUN DATE & TIME: Tuesday, Aug 5, 2003, 1:48 PM



Elevation

Left Cross Section

Right Cross Section

Girders
Splice Plates
Filler Plates

Top Flange Splice (in):
M270 Gr50W Plates
1- 0.5 × 18 × 24.5
2- 0.625 × 7.5 × 24.5
4 Rows of 4 Bolts @
3in Pitch

Bottom Flange Splice (in):
M270 Gr50W Plates
1- 0.875 × 18 × 42.5
2- 1 × 7.5 × 42.5
7 Rows of 4 Bolts @
3in Pitch

Web Splice (in):
M270 Gr50W Plates
2- 0.375 × 14 × 47.25
2 Rows of 12 Bolts @
4in Spacing

ALL BOLTS:
7/8 in AASHTO M164 BOLTS
(A325-X Flanges)
(A325-X Web)
Faying Surface Class = B

PERFORMANCE RATIOS:
Top Flange Bolts........0.91
Top Flange Plates.......1.00
Bottom Flange Bolts.....0.92
Bottom Flange Plates....1.00
Web Bolts...............0.98
Web Plates.............0.76

REQUIRED FILLERS:
(1) 0.4375 × 18 × 21 in
(1) 0.25 × 18 × 12 in