Steel Bridge Erection Plan/Procedure: Safety and Performance

HIGH STEEL STRUCTURES INC.
An Affiliate of High Industries Inc.

INTERNATIONAL BRIDGE CONFERENCE
Bridge Construction Seminar
Pittsburgh, Pennsylvania
June 2nd, 2008

NATIONAL STEEL BRIDGE ALLIANCE

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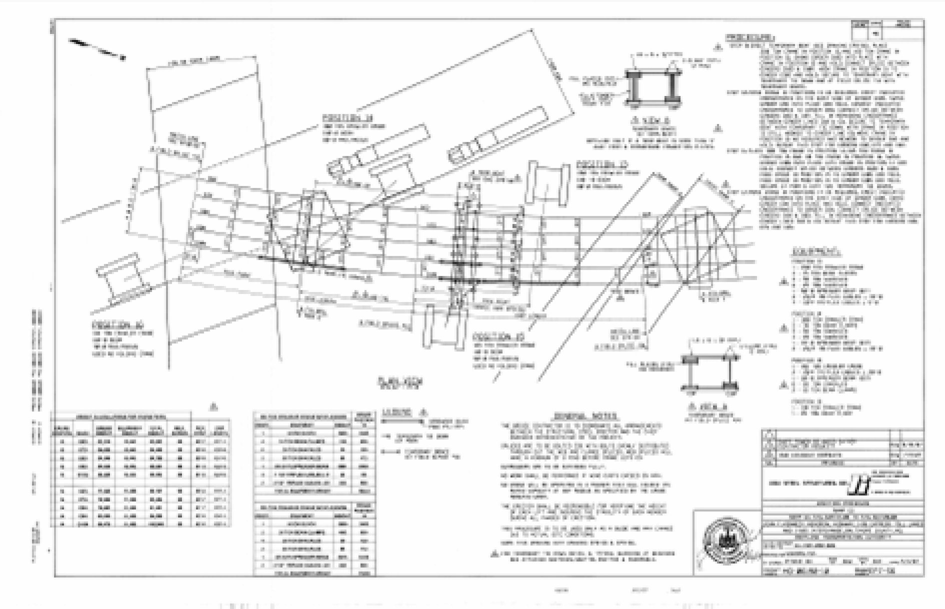
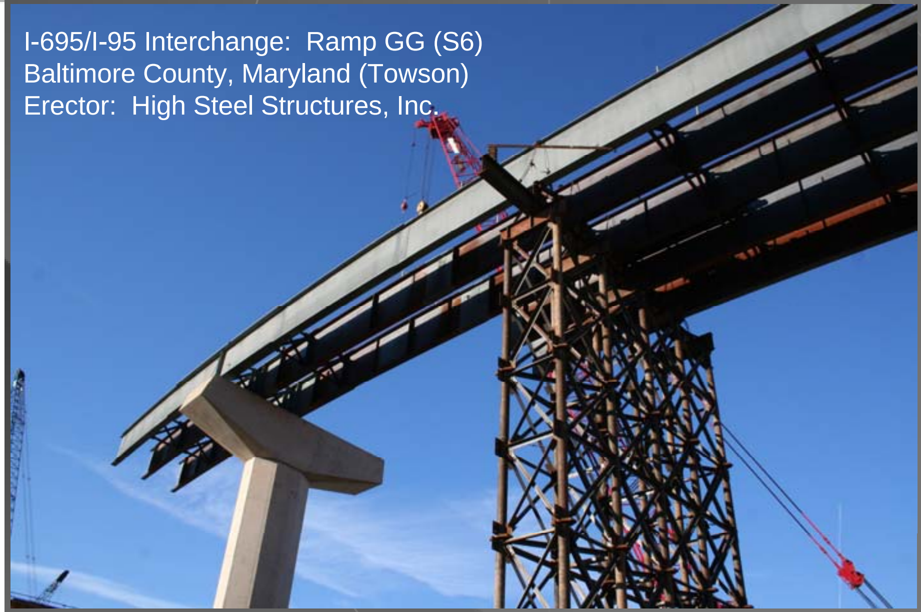
High Steel Structures Inc.

Original Session: 2007 World Steel Bridge Symposium
Session 5: Constructibility
New Orleans, Louisiana
Thursday, December 6th, 2007

Erection Plan/Procedure…

WHAT’S IN IT, WHAT DOES IT MEAN?





I-695/I-95 Interchange: Ramp GG (S6)
Baltimore County, Maryland (Towson)
Erector: High Steel Structures, Inc.

NOTE: this presentation illustrates one erector’s recent efforts to safely navigate their progress from the ASD/LFD to LRFD bridge environment; no guarantee is made regarding suitability of specific means/methods to particular projects. It is not a substitute for code requirements and engineering prudence.

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Introduction

CASE STUDY 1: short-span, LRFD Erection

SR 2024 over Towanda Creek
Bradford Co. PA
Erector: High Steel Structures, Inc.

CASE STUDY 2A: long span, crane only (no falsework)

I-95 SB over I-395NB
Springfield Interchange, VA
Erector: High Steel Structures, Inc.

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CASE STUDY 2B: long span, right-sized falsework

S.R. 6026 over S.R. 322
Centre County, PA
Erector: HSSI/Aycock joint effort

Introduction

CASE STUDY 3: environmentally sensitive area
Shorter span, limited falsework placement

U.S. Rte. 340 over Shenandoah River
Jefferson Co. WV
Erector: High Steel Structures, Inc.

CASE STUDY 4: curved tub girders
Shorter span, limited falsework placement

Washington Metro over I-95 Beltway
St. Georges County, MD
Erector: High Steel Structures, Inc.

Source: http://static-content.org/bigfoot/photos/at141-harpersferry

CASE STUDY 5: longer span
Liberal falsework placements

I-695/I-95 Interchange
Baltimore County, Maryland (Towson)
Erector: High Steel Structures, Inc.

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Erection Plan

The bridge erection plan drawing generally provides the erector with information such as the following:

- General framing layout
- Orientation of field section (pieces) with respect to marked end
- “Estimated weights” for main members
- Typical field bolted/welded connections w/details not otherwise indicated on the drawings
- Bearing point elevations (usually to bottom of steel/bearing)
- List of field bolt hardware and appropriate connection information

For additional information, please refer to:

AASHTO/NSBA Steel Bridge Collaboration G1.3-2002, Shop Detail Drawing Presentation Guidelines, Section 10 (www.steelbridges.org)

Erection Procedure

Provides the erector with a step-by-step direction to safely assemble the bridge. Often accompanied by calculations, technical data (“cut sheets”) & the following:

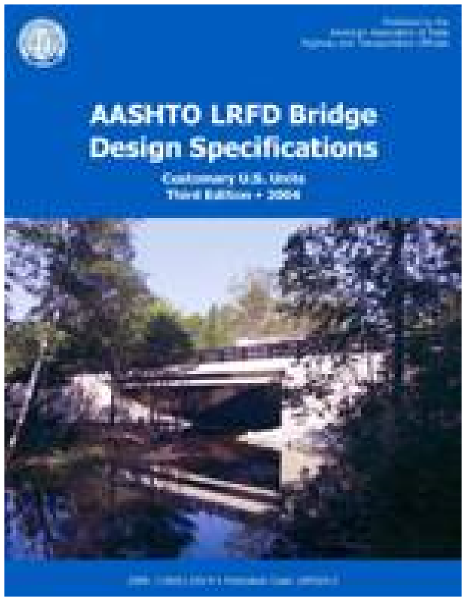
- Crane model, boom length & radius for indicated lifts
- Spreader beams, tie-downs, equipment to temporarily stabilize members
- Lifted section weights (include crossframes if to be lifted w/girders)
- Center(s) of gravity for complex/asymmetrical lifts
- Key equipment considerations (e.g., outriggers extended, 360º vs. rear-only lifting, hook block weight, etc)
- Minimum amount of sequential framing to maintain stable structure
- Quantity/location of bolts/pins before members are self-supporting
- Location of shoring towers/holding cranes
- Windspeed for safe erection of members & partially completed framing
- Traffic closure duration (location & delivered orientation of members)

For additional information, please refer to:

AASHTO/NSBA Steel Bridge Erection Guide Specification S10.1-2007

Case Study 1

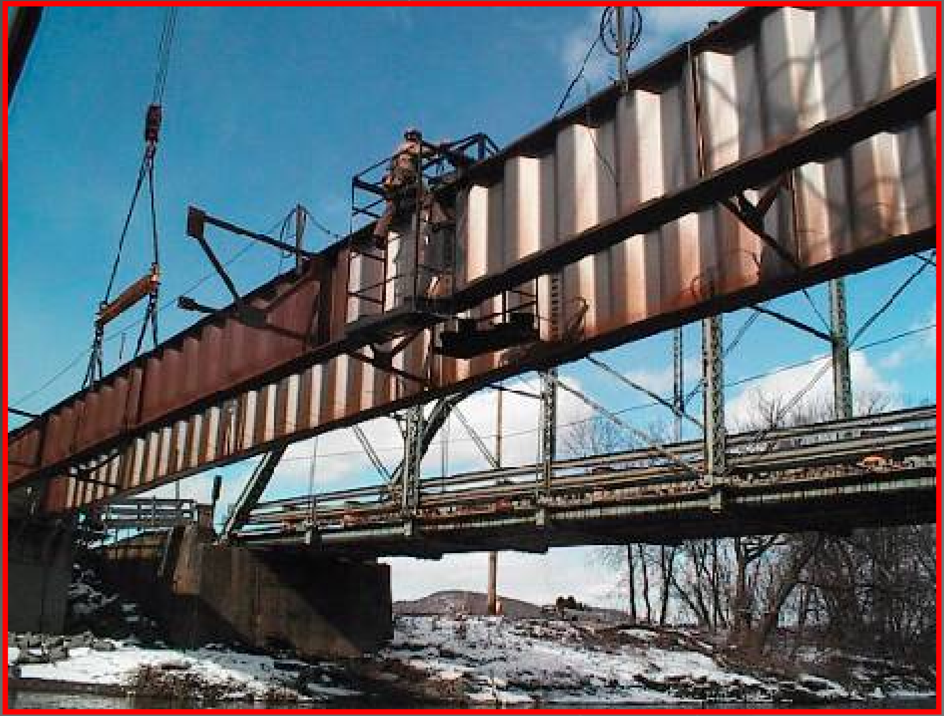
(Effective 2007) AASHTO LRFD Now Requires that One Plausible Method of Erection Be Investigated.



AASHTO LRFD Bridge Design Specifications

Customary U.S. Units
Third Edition • 2004

AASHTO LRFD 3rd ed. (interim) Specifications



SR 2024 over Towanda Creek
Bradford Co. PA
Contractor: Susquehanna Valley Construction Corp.
Erector: High Steel Structures, Inc.

Resources used for Case Studies:

- AASHTO/NSBA Steel Bridge Erection Guide Specification S10.1-2007
- AASHTO LRFD 2.5.3, Constructibility Considerations
- AASHTO LRFD 3.4.2, load factor $\gamma_{DC, WS} \geq 1.25$ was used for steel erection (*2008 note:* 1.5 may be required; under review by T-14 & market states)
- When performing ASD (SLD) erection analysis, for wind loadings:
  - ANSI/ASCE 7-95, Minimum Design Loads for Buildings and Other Structures
  - AASHTO Guide Design Specifications for Bridge Temporary Works
  - PADOT BD-620M (25psf, except 30psf over live traffic)
  - HSSI normally uses 25 MPH for picking/same shift erection; 40-50 MPH for overnight, & 25/30 psf (50-65 MPH) for partially erected steel beyond scope of local short-term weather forecast

Disclaimer: the above is what we used during the 2004-2007 period. As AASHTO LRFD becomes standard practice, the erection engineer must apply their own competent judgment for specific application: High Steel Structures, Inc. takes no responsibility for suitability of the above for a particular project.

!!!SPECIFICATIONS UPDATE!!!

2008 SUPPLEMENT (for IBC Bridge Construction Seminar)

This month, AASHTO has put out several pertinent publications including:

- LRFD BRIDGE DESIGN SPECIFICATIONS, 4TH EDITION
- LRFD BRIDGE CONSTRUCTION SPECIFICATIONS, 2ND EDITION
- GUIDE DESIGN SPECIFICATIONS FOR BRIDGE TEMPORARY WORKS, 2008 INTERIM
- CONSTRUCTION HANDBOOK FOR BRIDGE TEMPORARY WORKS, 2008 INTERIM

These documents may very well have new information (not yet included herein) which makes aspects of this presentation obsolete. The engineer is cautioned to become familiarized accordingly with the specifications & handbooks as applicable.

See: http://downloads.transportation.org/aashto_catalog.pdf (AASHTO 5-23-08 e-mail)

Fabricating and Erecting Skewed/curved Structures

- FABRICATION:
  - Girders profile usually similar to un-skewed structures (no-load "laydown" profile)
  - Cross-frame (CF) drops detailed to reflect no-load, steel DL or final position (owner preference)
  - Additional information may be found at:
    1) Symposium Session 2A, Erection of Skewed Bridges
    2) http://www.highsteel.com/contactus/newsroom/freeinfo
       Summer 2007 Newsletter: Skewed Bridge Presentation

- ERECTION:
  - Girders initially held to approximate no-load profile
  - Shop-welded/bolted CFs will normally force the required twist condition
  - Crossframe-braced girders released to steel DL state
  - Note: if knock-down (field-assembled) CF, must support member before it can withstand gravity.

Large/Curved Girder Erection

- As with fabrication, there are many means and methods:
  - NSBA/AISI/AISC effort: Advanced Erector Certification Program
  - Again, AASHTO/NSBA S10.1, Steel Bridge Erection Guide Specification

- States divided between two camps:
  1. Those who want engineered erection procedures (Complexity-based)
  2. Those who do not want the exposure of formal procedure/ calcs review.

I-95 SB Ramp over I-95/I-395/I-495
Fairfax County, VA
Fabricator, Erector: High Steel Structures Inc.

WMATA Blue Line over I-95
Prince George's County, MD
Fabricator, Erector: High Steel Structures Inc.

Case Study 2A

Erecting girders without shoring

- 250 ft spans
- R = 1000 ft (min)

IF JOBSITE CONDITIONS LIMIT EFFECTIVE TOWER PLACEMENTS, SPREADER BEAMS CAN STABILIZE THE GIRDERS...

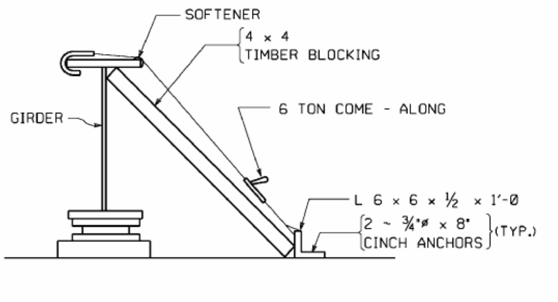
GIRDERS MAY BE SPLICED ON THE GROUND, THEN PICKED AS A UNIT. Example:
- L=300 ft
- Weight =115 Tons

...TWO CRANE PICKS MAY BE NECESSARY TO LIFT VERY LONG FIELD SECTIONS.

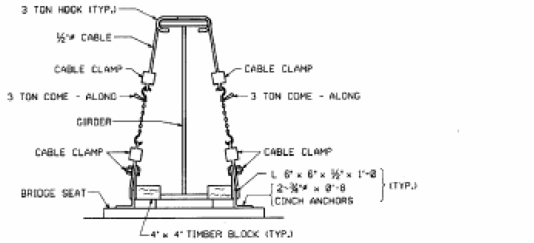
I-95SB over I-395NB (Bridge B610), Fairfax Co. VA **Erector: High Steel Structures, Inc.**

Sequential erection steps

TIE-DOWNS INITIALLY SECURE THE GIRDERS.



TEMPORARY TIE DOWN



TEMP. TIE DOWN

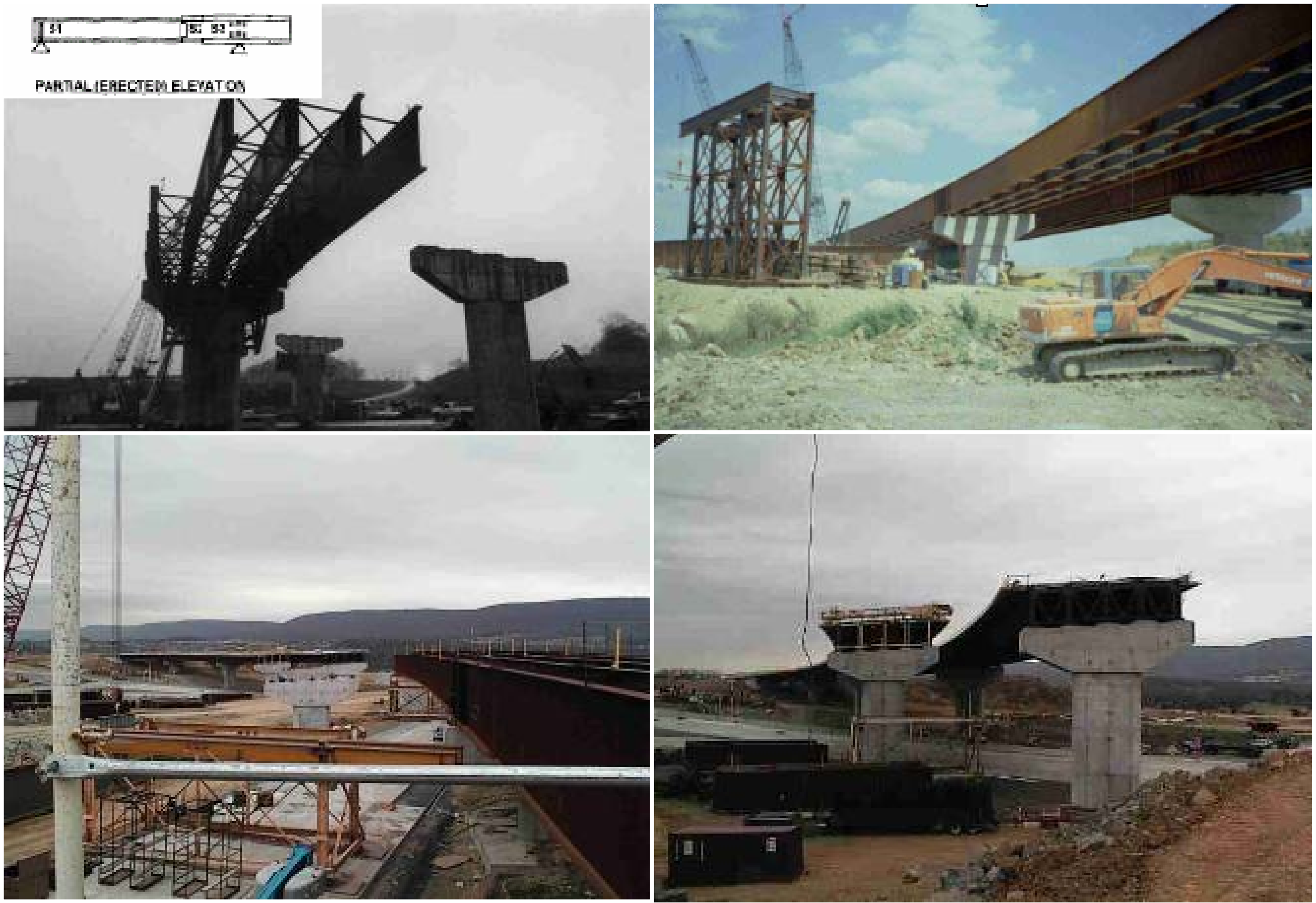
NOTES:
FOR GENERAL NOTES, SEE DRAWING EP201.
WORK THIS DRAWING WITH DRAWINGS EP201, EP202 AND EP203.

AS STABLE GIRDER PAIR IS LANDED, HOLDING CRANAGE IS FREED UP, TO EXPEDITE CROSSFRAME ERECTION.

Case Study 2B

Right-sized, adjustable shoring towers can facilitate alignment control as erection progresses.

As abandoned:
L_{OH} = 100 ft±
Δ_X = 1 FT
Δ_Y = 1 FT
Stability = 60mph±



At completion:
Length: 1,000 ft
Three Span (300ft – 330 ft – 270 ft) continuous unit

Radius: 1,900 ft
Depth: 10'-9"
Spacing: 9'-9"

Erected by High Steel Structures, Inc. & Aycock, Inc.

S.R. 6026 OVER S.R. 322 CENTRE COUNTY, PA (then-longest, curved steel girder span in PA)

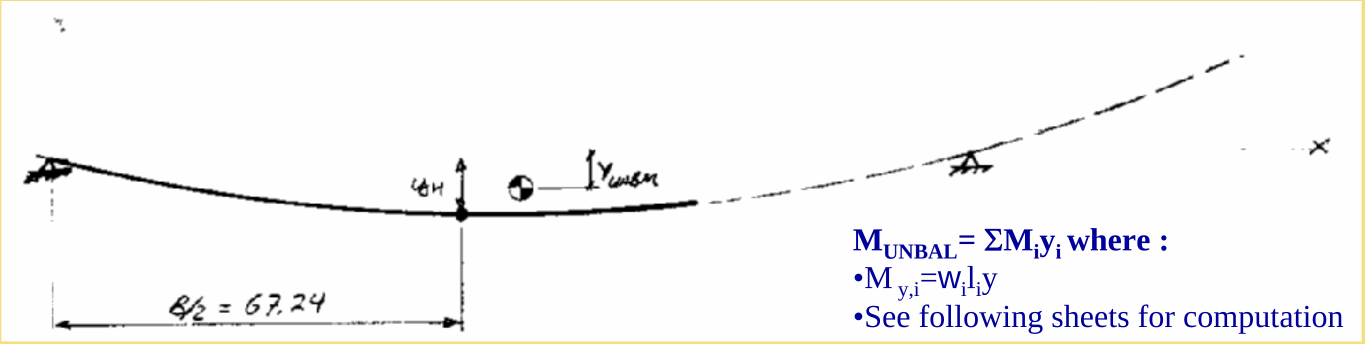
What one wants to avoid.

Critical stages and load combinations (e.g., D+W (temporary) may not always be immediately apparent, even when the structure is designed with lateral bracing.

In addition to conventional LTB AASHTO Standard Specifications for Highway Bridges, 17th Edition, Table 10.32.1A :

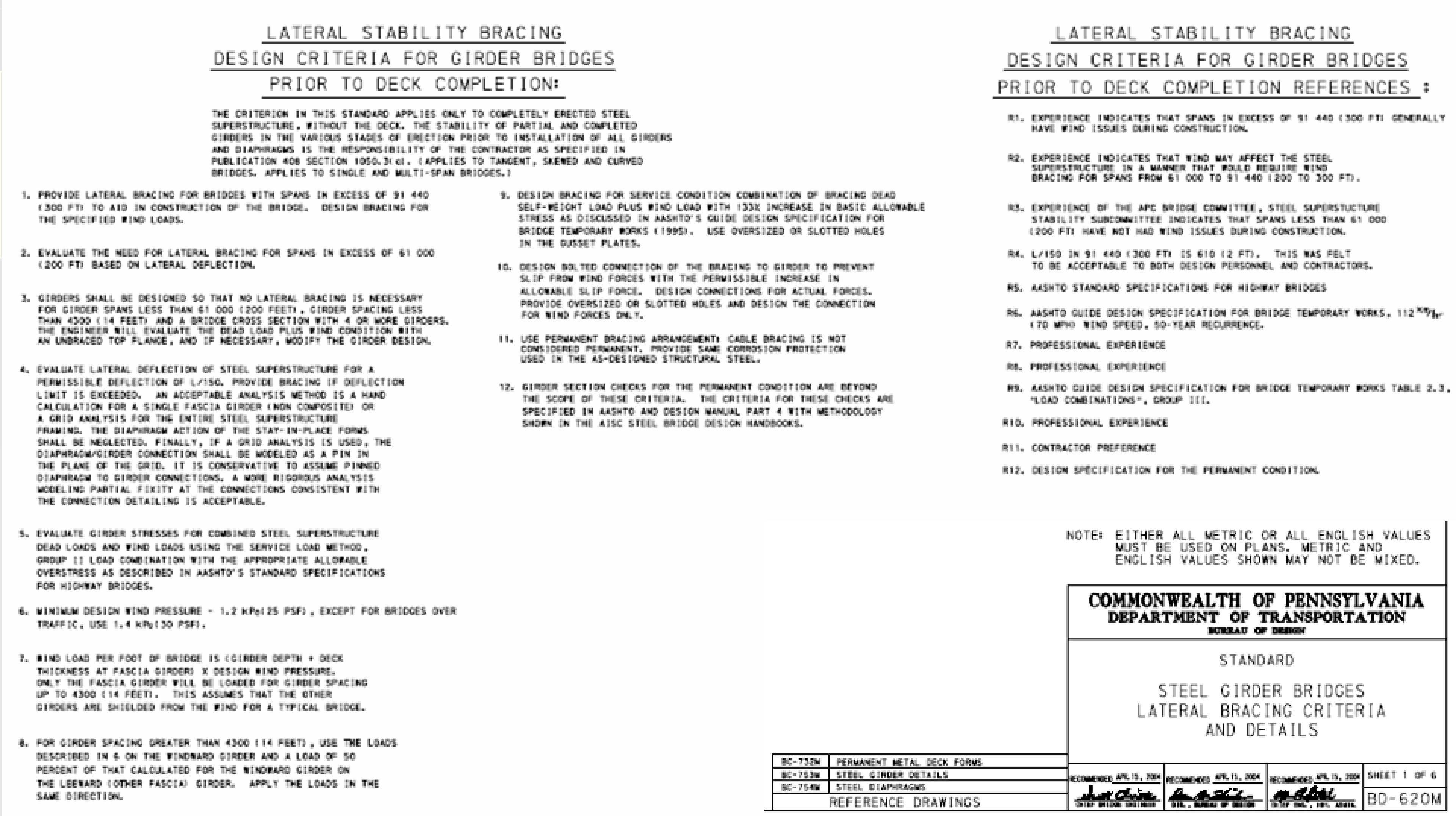
$$F_{b,c} = \frac{(50\times10^{3} C_b I_{yc})}{S_{xc} l_u} \sqrt{0.772 J/I_{yc} + 9.87(d/l)^{2}} < 0.55 F_y = \boxed{} \text{ ksi}$$

- for curved girders, lateral torsional effect is amplified and may be approximated as shown
- for simple and practical guidance, refer to PennDOT website BD 620M



$M_{UNBAL} = \Sigma M_i y_i$ **where :**
- $M_{y,i} = w_i l_i y$
- See following sheets for computation

Example: Quantifying Lateral Imbalance During Girder Erection



LATERAL STABILITY BRACING DESIGN CRITERIA FOR GIRDER BRIDGES PRIOR TO DECK COMPLETION:

THE CRITERION IN THIS STANDARD APPLIES ONLY TO COMPLETELY ERECTED STEEL SUPERSTRUCTURE, WITHOUT THE DECK. THE STABILITY OF PARTIAL AND COMPLETED GIRDERS IN THE VARIOUS STAGES OF ERECTION PRIOR TO INSTALLATION OF ALL GIRDERS AND DIAPHRAGMS IS THE RESPONSIBILITY OF THE CONTRACTOR AS SPECIFIED IN PUBLICATION 408 SECTION 1050.3(c). (APPLIES TO TANGENT, SKEWED AND CURVED BRIDGES. APPLIES TO SINGLE AND MULTI-SPAN BRIDGES.)

1. PROVIDE LATERAL BRACING FOR BRIDGES WITH SPANS IN EXCESS OF 91 440 (300 FT) TO AID IN CONSTRUCTION OF THE BRIDGE. DESIGN BRACING FOR THE SPECIFIED WIND LOADS.

2. EVALUATE THE NEED FOR LATERAL BRACING FOR SPANS IN EXCESS OF 61 000 (200 FT) BASED ON LATERAL DEFLECTION.

3. GIRDERS SHALL BE DESIGNED SO THAT NO LATERAL BRACING IS NECESSARY FOR GIRDER SPANS LESS THAN 61 000 (200 FEET), GIRDER SPACING LESS THAN 4300 (14 FEET) AND A BRIDGE CROSS SECTION WITH 4 OR MORE GIRDERS. THE ENGINEER WILL EVALUATE THE DEAD LOAD PLUS WIND CONDITION WITH AN UNBRACED TOP FLANGE, AND IF NECESSARY, MODIFY THE GIRDER DESIGN.

4. EVALUATE LATERAL DEFLECTION OF STEEL SUPERSTRUCTURE FOR A PERMISSIBLE DEFLECTION OF L/150. PROVIDE BRACING IF DEFLECTION LIMIT IS EXCEEDED. AN ACCEPTABLE ANALYSIS METHOD IS A HAND CALCULATION FOR A SINGLE FASCIA GIRDER (NON COMPOSITE) OR A GRID ANALYSIS FOR THE ENTIRE STEEL SUPERSTRUCTURE FRAMING. THE DIAPHRAGM ACTION OF THE STAY-IN-PLACE FORMS SHALL BE NEGLECTED. FINALLY, IF A GRID ANALYSIS IS USED, THE DIAPHRAGM/GIRDER CONNECTION SHALL BE MODELED AS A PIN IN THE PLANE OF THE GRID. IT IS CONSERVATIVE TO ASSUME PINNED DIAPHRAGM TO GIRDER CONNECTIONS. A MORE RIGOROUS ANALYSIS MODELING PARTIAL FIXITY AT THE CONNECTIONS CONSISTENT WITH THE CONNECTION DETAILING IS ACCEPTABLE.

5. EVALUATE GIRDER STRESSES FOR COMBINED STEEL SUPERSTRUCTURE DEAD LOADS AND WIND LOADS USING THE SERVICE LOAD METHOD, GROUP II LOAD COMBINATION WITH THE APPROPRIATE ALLOWABLE OVERSTRESS AS DESCRIBED IN AASHTO'S STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES.

6. MINIMUM DESIGN WIND PRESSURE - 1.2 kPa (25 PSF), EXCEPT FOR BRIDGES OVER TRAFFIC, USE 1.4 kPa (30 PSF).

7. WIND LOAD PER FOOT OF BRIDGE IS (GIRDER DEPTH + DECK THICKNESS AT FASCIA GIRDER) X DESIGN WIND PRESSURE. ONLY THE FASCIA GIRDER WILL BE LOADED FOR GIRDER SPACING UP TO 4300 (14 FEET). THIS ASSUMES THAT THE OTHER GIRDERS ARE SHIELDED FROM THE WIND FOR A TYPICAL BRIDGE.

8. FOR GIRDER SPACING GREATER THAN 4300 (14 FEET), USE THE LOADS DESCRIBED IN 6 ON THE WINDWARD GIRDER AND A LOAD OF 50 PERCENT OF THAT CALCULATED FOR THE WINDWARD GIRDER ON THE LEEWARD (OTHER FASCIA) GIRDER. APPLY THE LOADS IN THE SAME DIRECTION.

9. DESIGN BRACING FOR SERVICE CONDITION COMBINATION OF BRACING DEAD SELF-WEIGHT LOAD PLUS WIND LOAD WITH 133% INCREASE IN BASIC ALLOWABLE STRESS AS DISCUSSED IN AASHTO'S GUIDE DESIGN SPECIFICATION FOR BRIDGE TEMPORARY WORKS (1995). USE OVERSIZED OR SLOTTED HOLES IN THE GUSSET PLATES.

10. DESIGN BOLTED CONNECTION OF THE BRACING TO GIRDER TO PREVENT SLIP FROM WIND FORCES WITH THE PERMISSIBLE INCREASE IN ALLOWABLE SLIP FORCE. DESIGN CONNECTIONS FOR ACTUAL FORCES. PROVIDE OVERSIZED OR SLOTTED HOLES AND DESIGN THE CONNECTION FOR WIND FORCES ONLY.

11. USE PERMANENT BRACING ARRANGEMENT: CABLE BRACING IS NOT CONSIDERED PERMANENT. PROVIDE SAME CORROSION PROTECTION USED IN THE AS-DESIGNED STRUCTURAL STEEL.

12. GIRDER SECTION CHECKS FOR THE PERMANENT CONDITION ARE BEYOND THE SCOPE OF THESE CRITERIA. THE CRITERIA FOR THESE CHECKS ARE SPECIFIED IN AASHTO AND DESIGN MANUAL PART 4 WITH METHODOLOGY SHOWN IN THE AISC STEEL BRIDGE DESIGN HANDBOOKS.

LATERAL STABILITY BRACING DESIGN CRITERIA FOR GIRDER BRIDGES PRIOR TO DECK COMPLETION REFERENCES:

R1. EXPERIENCE INDICATES THAT SPANS IN EXCESS OF 91 440 (300 FT) GENERALLY HAVE WIND ISSUES DURING CONSTRUCTION.

R2. EXPERIENCE INDICATES THAT WIND MAY AFFECT THE STEEL SUPERSTRUCTURE IN A MANNER THAT WOULD REQUIRE WIND BRACING FOR SPANS FROM 61 000 TO 91 440 (200 TO 300 FT).

R3. EXPERIENCE OF THE APC BRIDGE COMMITTEE, STEEL SUPERSTUCTURE STABILITY SUBCOMMITTEE INDICATES THAT SPANS LESS THAN 61 000 (200 FT) HAVE NOT HAD WIND ISSUES DURING CONSTRUCTION.

R4. L/150 IN 91 440 (300 FT) IS 610 (2 FT). THIS WAS FELT TO BE ACCEPTABLE TO BOTH DESIGN PERSONNEL AND CONTRACTORS.

R5. AASHTO STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES

R6. AASHTO GUIDE DESIGN SPECIFICATION FOR BRIDGE TEMPORARY WORKS, 112 km/hr (70 MPH) WIND SPEED, 50-YEAR RECURRENCE.

R7. PROFESSIONAL EXPERIENCE

R8. PROFESSIONAL EXPERIENCE

R9. AASHTO GUIDE DESIGN SPECIFICATION FOR BRIDGE TEMPORARY WORKS TABLE 2.3, "LOAD COMBINATIONS", GROUP III.

R10. PROFESSIONAL EXPERIENCE

R11. CONTRACTOR PREFERENCE

R12. DESIGN SPECIFICATION FOR THE PERMANENT CONDITION.

NOTE: EITHER ALL METRIC OR ALL ENGLISH VALUES MUST BE USED ON PLANS. METRIC AND ENGLISH VALUES SHOWN MAY NOT BE MIXED.

BC-732M	PERMANENT METAL DECK FORMS
BC-753M	STEEL GIRDER DETAILS
BC-754M	STEEL DIAPHRAGMS
REFERENCE DRAWINGS	

COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION
BUREAU OF DESIGN

STANDARD

STEEL GIRDER BRIDGES
LATERAL BRACING CRITERIA
AND DETAILS

RECOMMENDED APR. 15, 2004

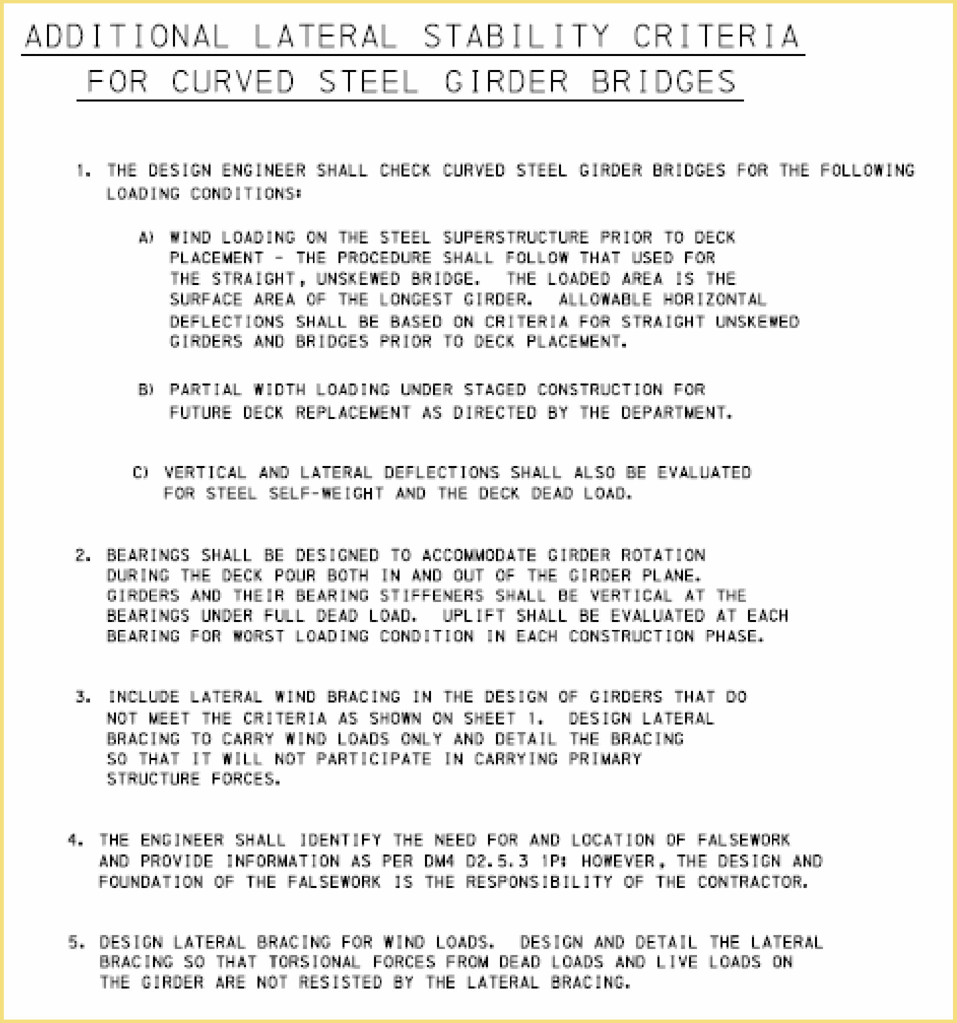
RECOMMENDED APR. 15, 2004

RECOMMENDED APR. 15, 2004

SHEET 1 OF 6

BD-620M

PADOT BD 620-M (page 1 of 6) Source: www.dot.state.pa.us/Internet/BQADStandards.nsf/home?Open Frameset



ADDITIONAL LATERAL STABILITY CRITERIA FOR CURVED STEEL GIRDER BRIDGES

1. THE DESIGN ENGINEER SHALL CHECK CURVED STEEL GIRDER BRIDGES FOR THE FOLLOWING LOADING CONDITIONS:

   A) WIND LOADING ON THE STEEL SUPERSTRUCTURE PRIOR TO DECK PLACEMENT - THE PROCEDURE SHALL FOLLOW THAT USED FOR THE STRAIGHT, UNSKEWED BRIDGE. THE LOADED AREA IS THE SURFACE AREA OF THE LONGEST GIRDER. ALLOWABLE HORIZONTAL DEFLECTIONS SHALL BE BASED ON CRITERIA FOR STRAIGHT UNSKEWED GIRDERS AND BRIDGES PRIOR TO DECK PLACEMENT.

   B) PARTIAL WIDTH LOADING UNDER STAGED CONSTRUCTION FOR FUTURE DECK REPLACEMENT AS DIRECTED BY THE DEPARTMENT.

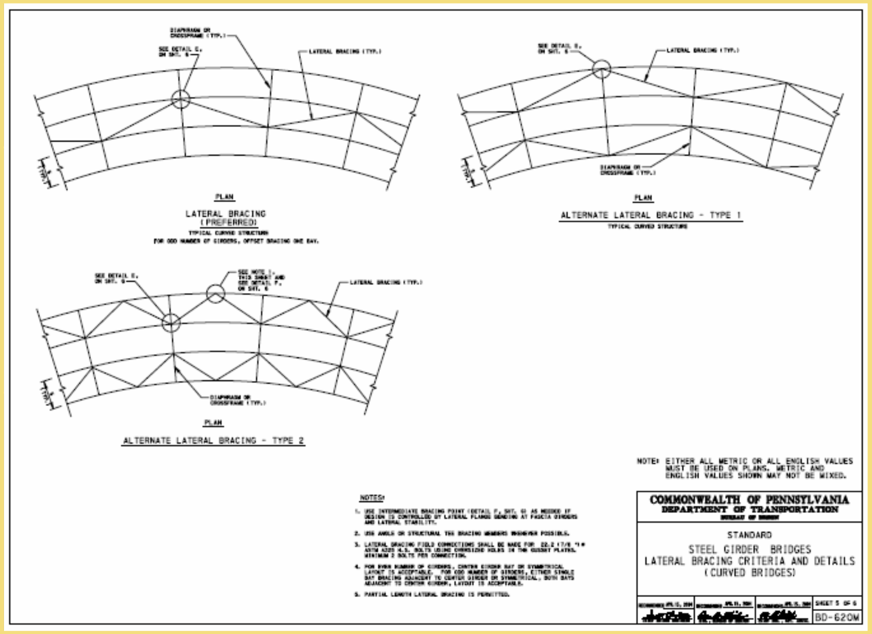
   C) VERTICAL AND LATERAL DEFLECTIONS SHALL ALSO BE EVALUATED FOR STEEL SELF-WEIGHT AND THE DECK DEAD LOAD.

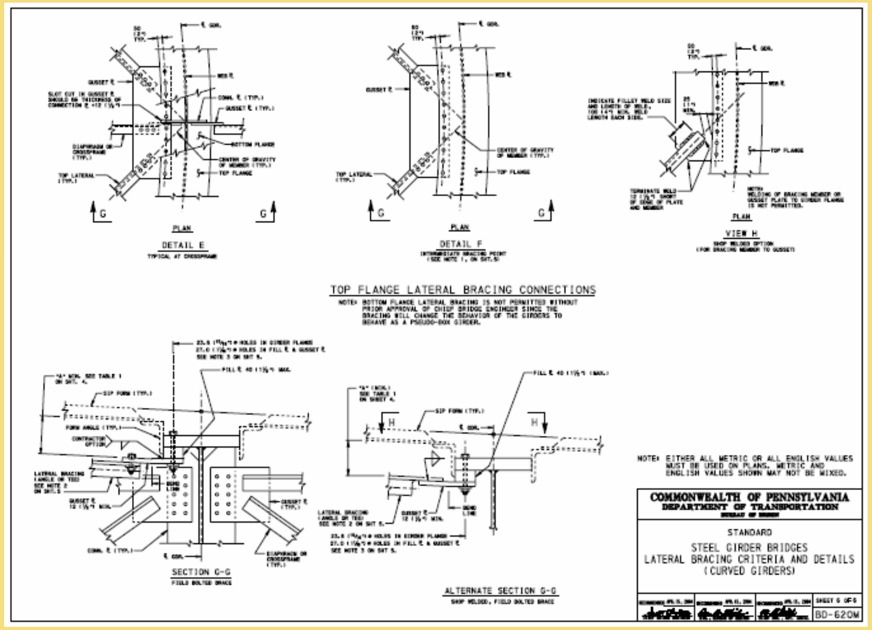
2. BEARINGS SHALL BE DESIGNED TO ACCOMMODATE GIRDER ROTATION DURING THE DECK POUR BOTH IN AND OUT OF THE GIRDER PLANE. GIRDERS AND THEIR BEARING STIFFENERS SHALL BE VERTICAL AT THE BEARINGS UNDER FULL DEAD LOAD. UPLIFT SHALL BE EVALUATED AT EACH BEARING FOR WORST LOADING CONDITION IN EACH CONSTRUCTION PHASE.

3. INCLUDE LATERAL WIND BRACING IN THE DESIGN OF GIRDERS THAT DO NOT MEET THE CRITERIA AS SHOWN ON SHEET 1. DESIGN LATERAL BRACING TO CARRY WIND LOADS ONLY AND DETAIL THE BRACING SO THAT IT WILL NOT PARTICIPATE IN CARRYING PRIMARY STRUCTURE FORCES.

4. THE ENGINEER SHALL IDENTIFY THE NEED FOR AND LOCATION OF FALSEWORK AND PROVIDE INFORMATION AS PER DM4 D2.5.3 1P: HOWEVER, THE DESIGN AND FOUNDATION OF THE FALSEWORK IS THE RESPONSIBILITY OF THE CONTRACTOR.

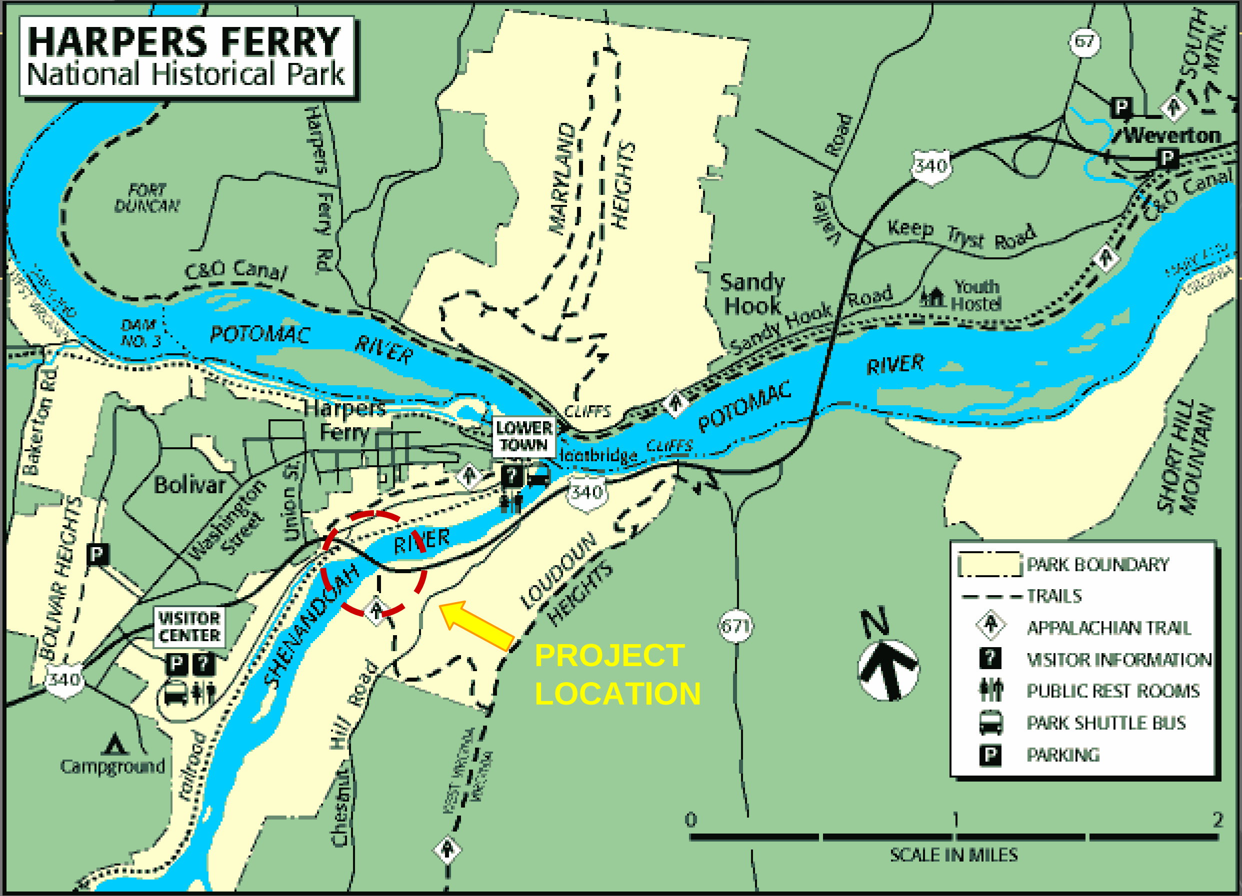
5. DESIGN LATERAL BRACING FOR WIND LOADS. DESIGN AND DETAIL THE LATERAL BRACING SO THAT TORSIONAL FORCES FROM DEAD LOADS AND LIVE LOADS ON THE GIRDER ARE NOT RESISTED BY THE LATERAL BRACING.

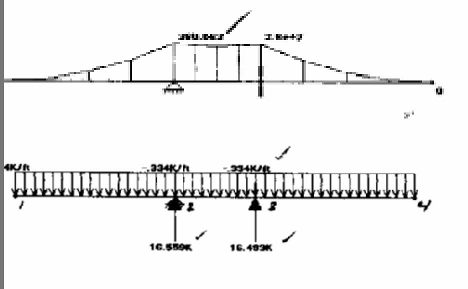


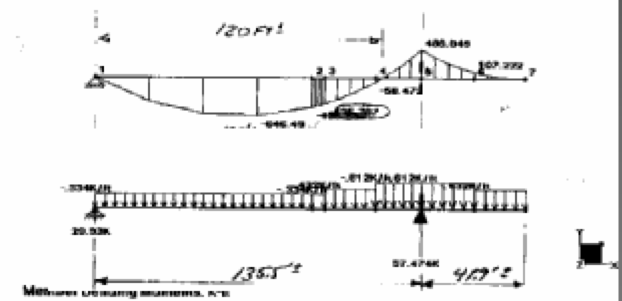


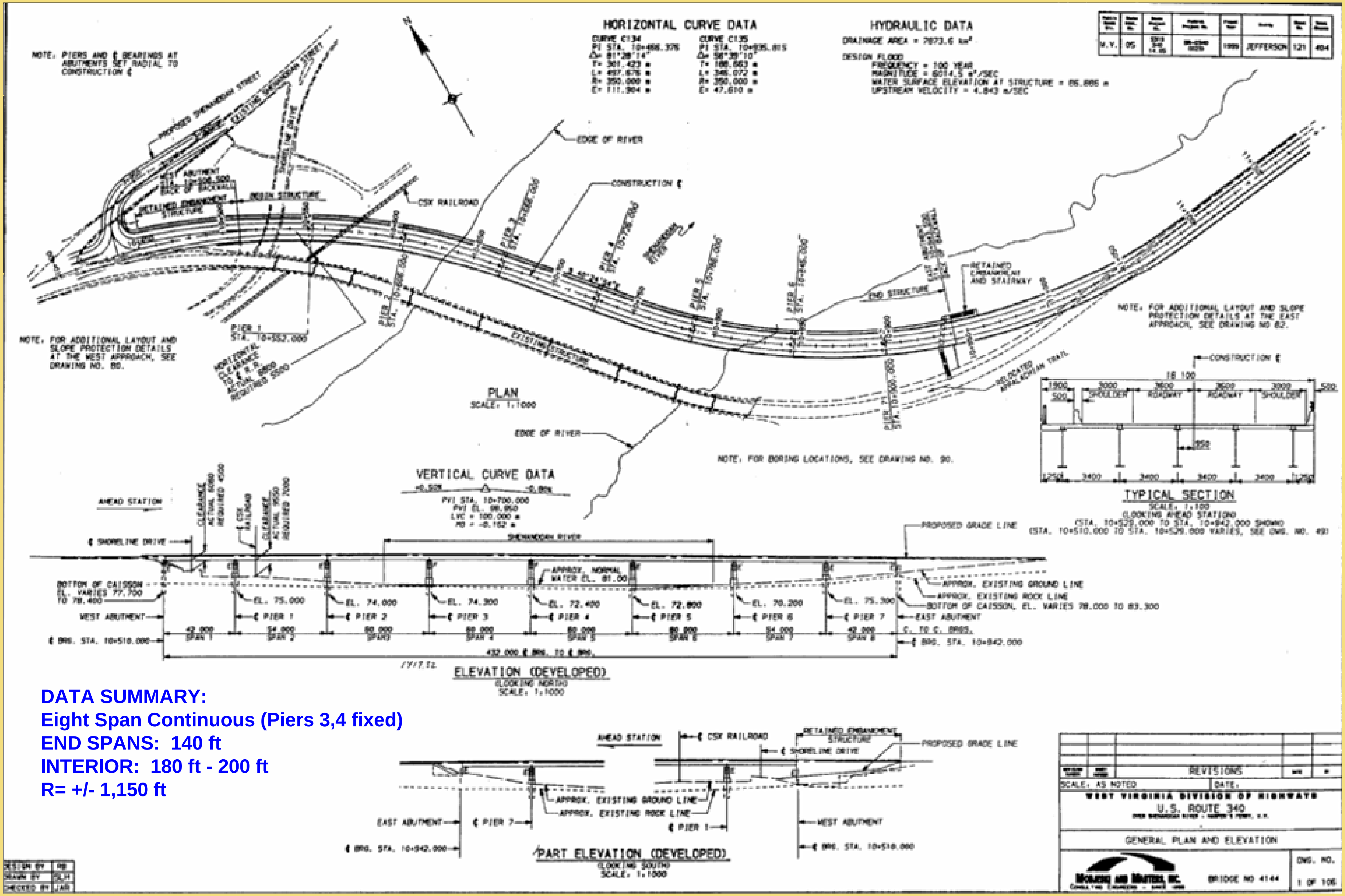
PADOT BD 620-M
(page 2 of 6)

Case Study 3: Harpers Ferry Bridge B4144

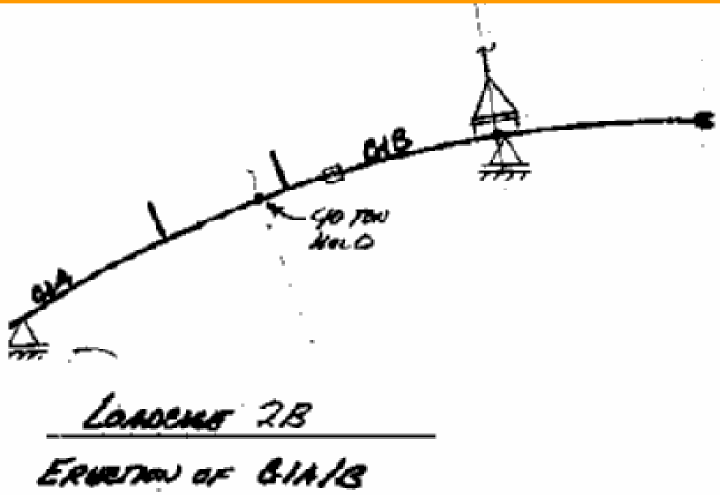




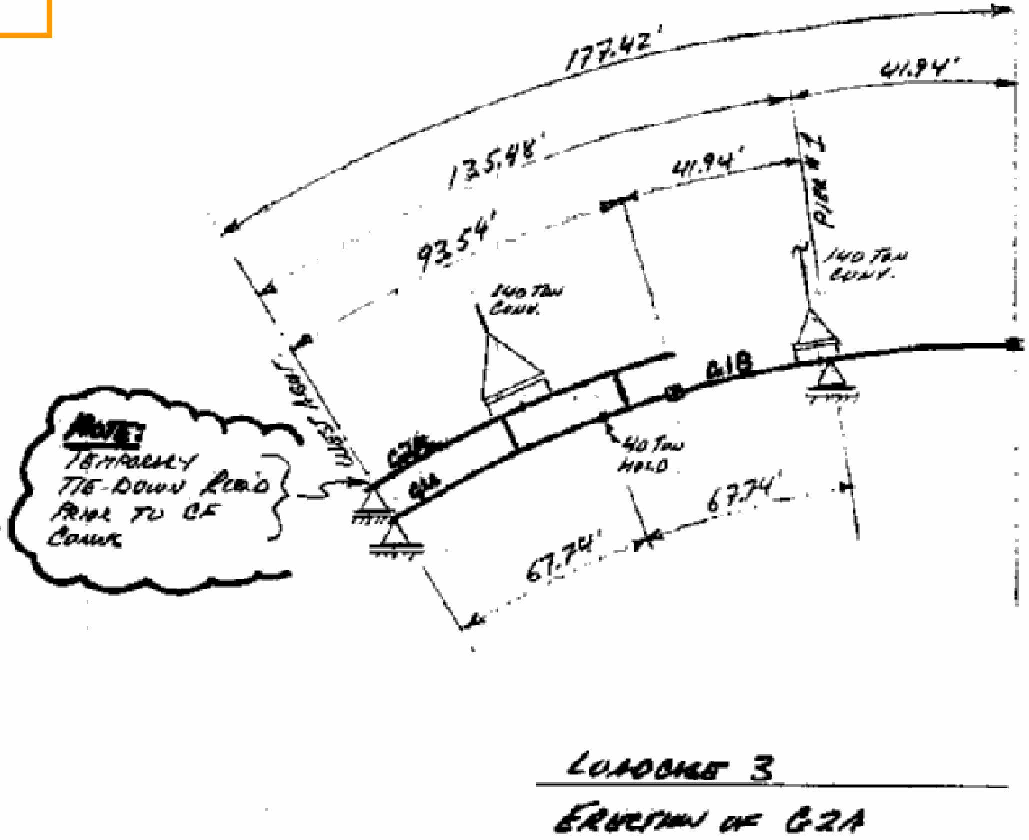




DATA SUMMARY:
Eight Span Continuous (Piers 3,4 fixed)
END SPANS: 140 ft
INTERIOR: 180 ft - 200 ft
R= +/- 1,150 ft

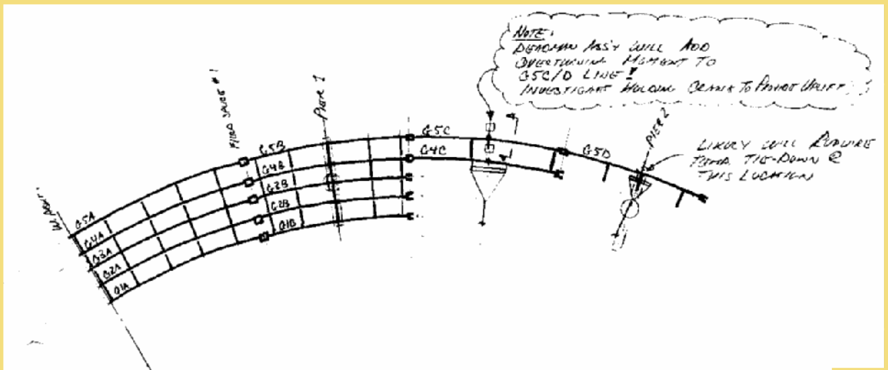


An erection stability analysis is often necessary during steel multi-girder bridge erection.



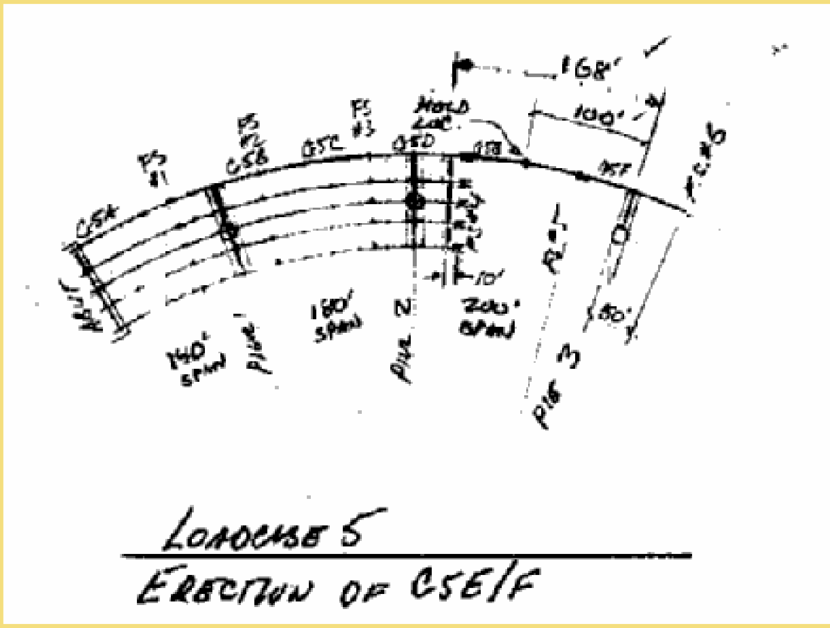
- For curved, multi-span structures over variable terrain, this often warrants a visual layout based upon feasible crane placements.
- The following must be checked:
  - D+W (temp: pick , initial, partial erection)
  - Curvature amplification (M_z) as applicable
  - Unusual (unique) component loads
  - $f_{bx}/F_{bx}+f_{by}/F_{by}<1$ (poss. 1.25, ASD)
  - Cranage & rigging loads
  - Splice & CF connections <50% (time window)
  - Often, global as well as local stability.
  - Shoring, tie-down reactions (vert/lat/longit.)
  - (if applicable) grade/superelev/thermal effects.

Additional crucial stages may also occur even after the initial spans have been set.



Loadcase 4

Erection of G4C



Loadcase 5

Erection of G5E/F

MODJESKI and MASTERS
Consulting Engineers
SINCE 1893

STATE OF WEST VIRGINIA DEPARTMENT OF TRANSPORTATION

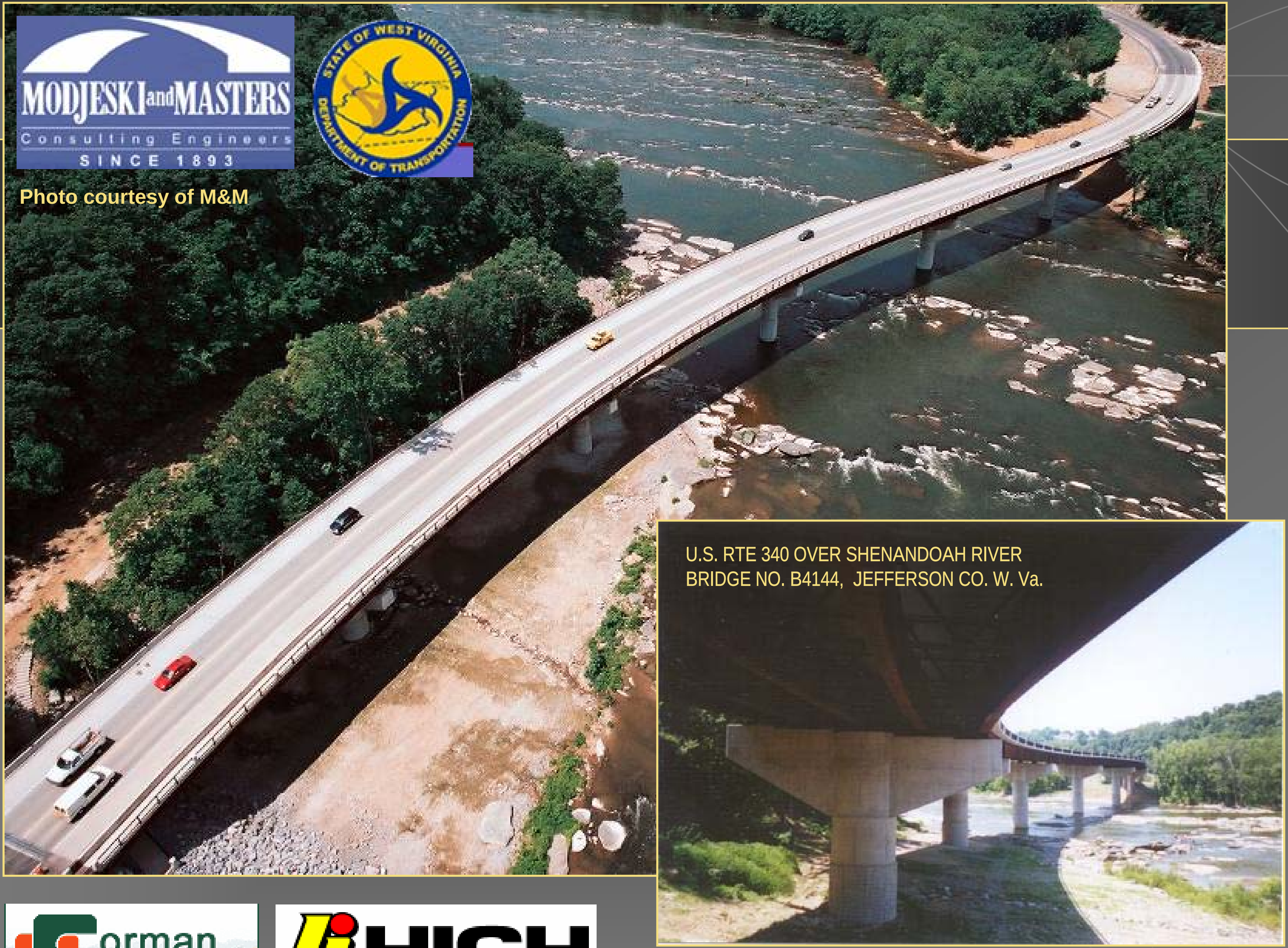


Photo courtesy of M&M

U.S. RTE 340 OVER SHENANDOAH RIVER
BRIDGE NO. B4144, JEFFERSON CO. W. Va.

Source: http://static-content.org/bigfoot/photos/at141-harpersferry_bridge

Corman
CONSTRUCTION
INCORPORATED

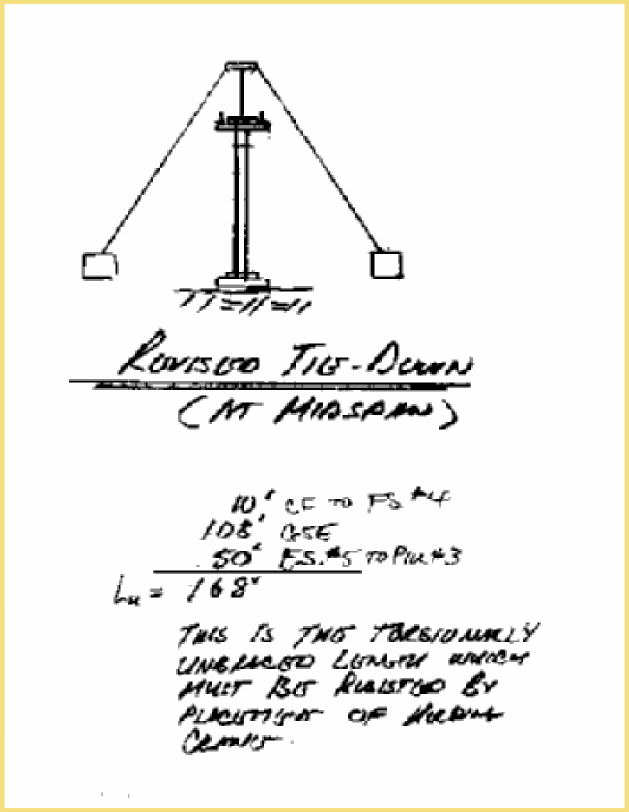
HIGH
STEEL
STRUCTURES, INC.

With careful planning, a beautifully crafted project is completed well.

Harpers Ferry Bridge Utilized a Combination of:

A) LATERAL STAYS (SHOWN HERE STABILIZING PRE-BD 620M TYPE PA STRUCTURE, CENTRE COUNTY PA)

B) SHORING TOWER FOR VERTICAL LOAD/DEFLECTION CONTROL (SIMILAR TO ABOVE CENTRE COUNTY, PA STRUCTURE TO LEFT OF DISTINGUISHED GENTLEMEN



REVISED TIE-DOWN
(AT MIDSPAN)

10' C.E. TO F.S. #4
108' G.S.E.
50' F.S. #5 TO PIER #3
L_u = 168'

THIS IS THE TORSIONALLY UNBRACED LENGTH WHICH MUST BE RESISTED BY PLACEMENT OF HOLDING CRANES.

WV 99017 TEMPORARY SHORED TIE-DOWN (MIDSPAN, IN LIEU OF HOLDING CRANE OR FULL TOWER ASSEMBLY)

Case Study 4

Trapezoidal Box Girders (Tub girders)

Tub girders generally erect more rapidly than I-girders

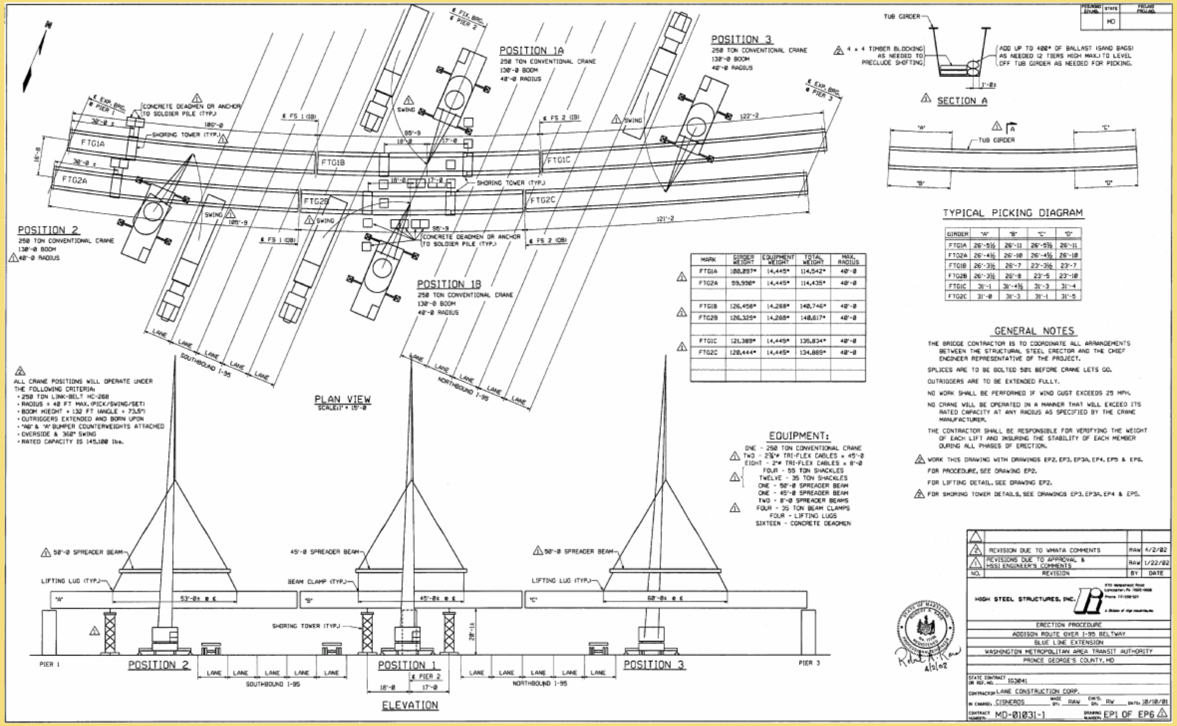
Washington Metro over I-95 Beltway, MD

Contractor: Lane Construction Corp.
Erector: High Steel Structures, Inc.

LARGE CURVED TRAPEZOIDAL BOX GIRDERS

GENERAL CONSIDERATIONS FOR:

- Internal stability
- Balance of curvature during handling



ALL CRANE POSITIONS WILL OPERATE UNDER THE FOLLOWING CRITERIA:
- 250 TON LINK-BELT HC-268
- RADIUS = 40 FT MAX. (PICK/SWING/SET)
- BOOM HIEGHT = 132 FT (ANGLE = 73.5°)
- OUTRIGGERS EXTENDED AND BORN UPON
- "AB" & "A" BUMPER COUNTERWEIGHTS ATTACHED
- OVERSIDE & 360° SWING
- RATED CAPACITY IS 145,100 lbs.

4 x 4 TIMBER BLOCKING AS NEEDED TO PRECLUDE SHIFTING

ADD UP TO 400# OF BALLAST (SAND BAGS) AS NEEDED (2 TIERS HIGH MAX.) TO LEVEL OFF TUB GIRDER AS NEEDED FOR PICKING.

MARK	GIRDER WEIGHT	EQUIPMENT WEIGHT	TOTAL WEIGHT	MAX. RADIUS
FTG1A	100,097#	14,445#	114,542#	40'-0
FTG2A	99,990#	14,445#	114,435#	40'-0
FTG1B	126,458#	14,288#	140,746#	40'-0
FTG2B	126,329#	14,288#	140,617#	40'-0
FTG1C	121,389#	14,445#	135,834#	40'-0
FTG2C	120,444#	14,445#	134,889#	40'-0

TYPICAL PICKING DIAGRAM

GIRDER	"A"	"B"	"C"	"D"
FTG1A	26'-5½	26'-11	26'-5½	26'-11
FTG2A	26'-4½	26'-10	26'-4½	26'-10
FTG1B	26'-3½	26'-7	23'-3½	23'-7
FTG2B	26'-3½	26'-8	23'-5	23'-10
FTG1C	31'-1	31'-4½	31'-3	31'-4
FTG2C	31'-0	31'-3	31'-1	31'-5

EQUIPMENT:

ONE - 250 TON CONVENTIONAL CRANE
TWO - 2¾"# TRI-FLEX CABLES x 45'-0
EIGHT - 2"# TRI-FLEX CABLES x 8'-0
FOUR - 55 TON SHACKLES
TWELVE - 35 TON SHACKLES
ONE - 50'-0 SPREADER BEAM
ONE - 45'-0 SPREADER BEAM
TWO - 8'-0 SPREADER BEAMS
FOUR - 35 TON BEAM CLAMPS
FOUR - LIFTING LUGS
SIXTEEN - CONCRETE DEADMEN

GENERAL NOTES

THE BRIDGE CONTRACTOR IS TO COORDINATE ALL ARRANGEMENTS BETWEEN THE STRUCTURAL STEEL ERECTOR AND THE CHIEF ENGINEER REPRESENTATIVE OF THE PROJECT.

SPLICES ARE TO BE BOLTED 50% BEFORE CRANE LETS GO.

OUTRIGGERS ARE TO BE EXTENDED FULLY.

NO WORK SHALL BE PERFORMED IF WIND GUST EXCEEDS 25 MPH.

NO CRANE WILL BE OPERATED IN A MANNER THAT WILL EXCEED ITS RATED CAPACITY AT ANY RADIUS AS SPECIFIED BY THE CRANE MANUFACTURER.

THE CONTRACTOR SHALL BE RESPONSIBLE FOR VERIFYING THE WEIGHT OF EACH LIFT AND INSURING THE STABILITY OF EACH MEMBER DURING ALL PHASES OF ERECTION.

WORK THIS DRAWING WITH DRAWINGS EP2, EP3, EP3A, EP4, EP5 & EP6.

FOR PROCEDURE, SEE DRAWING EP2.

FOR LIFTING DETAIL, SEE DRAWING EP2.

FOR SHORING TOWER DETAILS, SEE DRAWINGS EP3, EP3A, EP4 & EP5.

NO.	REVISION	BY	DATE
2	REVISION DUE TO WMATA COMMENTS	RAW	4/2/02
1	REVISIONS DUE TO APPROVAL & HSSI ENGINEER'S COMMENTS	RAW	1/22/02

HIGH STEEL STRUCTURES, INC.

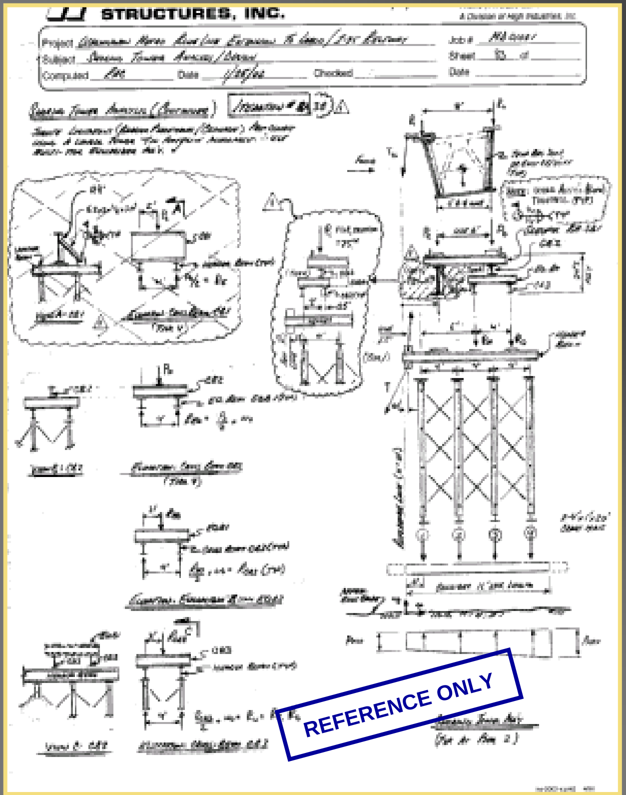
ERECTION PROCEDURE
ADDISON ROUTE OVER I-95 BELTWAY
BLUE LINE EXTENSION
WASHINGTON METROPOLITAN AREA TRANSIT AUTHORITY
PRINCE GEORGE'S COUNTY, MD

STATE CONTRACT OR REF. NO. 1G3041
CONTRACTOR: LANE CONSTRUCTION CORP.
IN CHARGE: CISNEROS MADE BY: RAW CHK'D. BY: RW DATE: 10/10/01
CONTRACT NUMBER: MD-01031-1 DRAWING NUMBER: EP1 OF EP6

Robert A. Raw
4/2/02

Tie-down Restraint May Be Needed (To Resist Lateral Roll Under D+W)

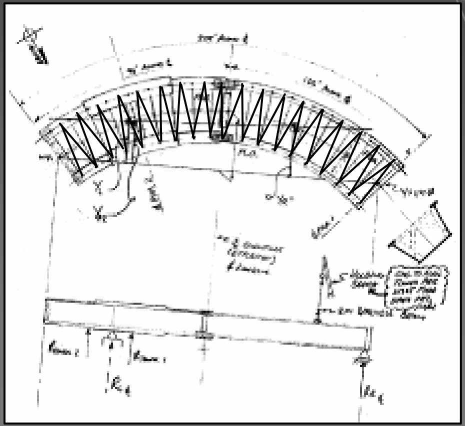
FOR AN INITIAL “QUICK CHECK”, EVALUATE ERECTION SPANS, OVERHANGS AND RADII , SOLVING FOR NET UNBALANCED M_{UNBAL}

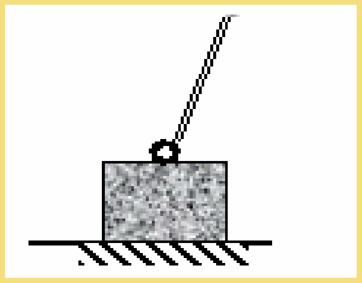


Balancing curved picks, etc.

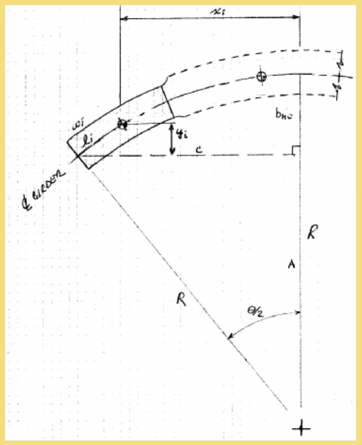
Compute $M_{UNBAL} = \Sigma M_i y_i$ where :

- $M_{y,i} = w_i l_i y$, where $\Sigma_i \ell_i = R\Theta_i$
- $A = R\cos(\Theta/2)$
- $b_{MO} = R - A$
- See AISC 9th Ed (ASD) p.6-16 to determine x_i & y_i





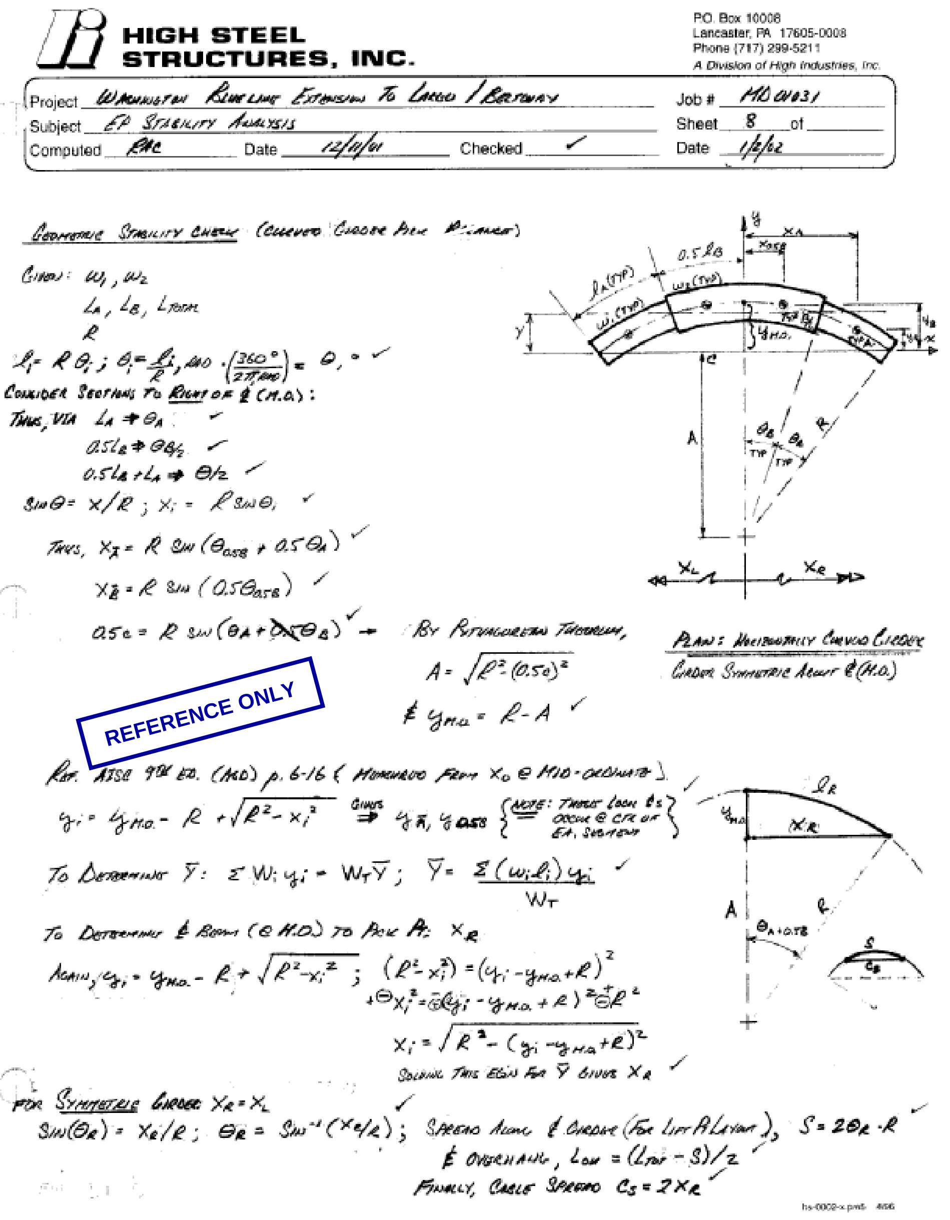
Tie-down to “deadman”



Computing M_y

UNLIKE FLEXIBLE, CURVED I-GIRDERS, WHICH MAY BE CHOKED (UNLESS PAINTED), CURVED TRAPEZOIDAL BOX GIRDERS MAY TEND TO LEAN (ROLL):

FIRST, ONE WAY DURING SHOP HANDLING...



HIGH STEEL STRUCTURES, INC.

P.O. Box 10008
Lancaster, PA 17605-0008
Phone (717) 299-5211
A Division of High Industries, Inc.

Project: Washington Blue Line Extension To Largo / Beltway — Job #: MD 01031
Subject: EP Stability Analysis — Sheet: 8 of
Computed: RMC — Date: 12/11/01 — Checked: ✓ — Date: 1/2/02

Geometric Stability Check (Curved Girder Pick Planar)

Given: w_1, w_2
L_A, L_B, L_{TOTAL}
R

$\ell_i = R\theta_i$; $\theta_i = \frac{\ell_i}{R}$, and $\cdot\left(\frac{360°}{2\pi \text{ rad}}\right) = \theta_i°$ ✓

Consider Sections to Right of ℄ (M.O.):

Thus, via $L_A \Rightarrow \theta_A$ ✓
$0.5\ell_B \Rightarrow \theta_{B/2}$ ✓
$0.5\ell_B + L_A \Rightarrow \theta/2$ ✓

$\sin\theta = x/R$; $x_i = R\sin\theta_i$ ✓

Thus, $x_{\bar{A}} = R\sin(\theta_{0.5B} + 0.5\theta_A)$ ✓
$x_{\bar{B}} = R\sin(0.5\theta_{0.5B})$ ✓
$0.5c = R\sin(\theta_A + \cancel{0.5}\theta_B)$ ✓ → By Pythagorean Theorem,

$$A = \sqrt{R^2 - (0.5c)^2}$$

℄ $y_{M.O.} = R - A$ ✓

Plan: Horizontally Curved Girder
Girder Symmetric About ℄ (M.O.)

REFERENCE ONLY

Ref. AISC 9th Ed. (ASD) p. 6-16 (Measured From X_0 @ Mid-Ordinate).

$y_i = y_{M.O.} - R + \sqrt{R^2 - x_i^2}$ Gives ⇒ $y_{\bar{A}}, y_{0.5B}$ {Note: These Loc. ℄s Occur @ CTR of Ea. Segment}

To Determine $\bar{Y}$: $\Sigma W_i y_i = W_T\bar{Y}$; $\bar{Y} = \frac{\Sigma(w_i\ell_i)y_i}{W_T}$ ✓

To Determine ℄ Boom (@ M.O.) to Pick Pt: X_R

Again, $y_i = y_{M.O.} - R + \sqrt{R^2 - x_i^2}$; $(R^2 - x_i^2) = (y_i - y_{M.O.} + R)^2$
$+\ominus x_i^2 = \ominus(y_i - y_{M.O.} + R)^2 \overset{+}{\ominus} R^2$
$x_i = \sqrt{R^2 - (y_i - y_{M.O.} + R)^2}$
Solving This Eq'n For $\bar{Y}$ Gives X_R ✓

For Symmetric Girder $X_R = X_L$ ✓
$\sin(\theta_R) = X_R/R$; $\theta_R = \sin^{-1}(X_R/R)$; Spread Along ℄ Girder (For Lift Plan Layout), $S = 2\theta_R \cdot R$ ✓
℄ Overhang, $L_{OH} = (L_{TOT} - S)/2$ ✓
Finally, Cable Spread $C_S = 2X_R$ ✓

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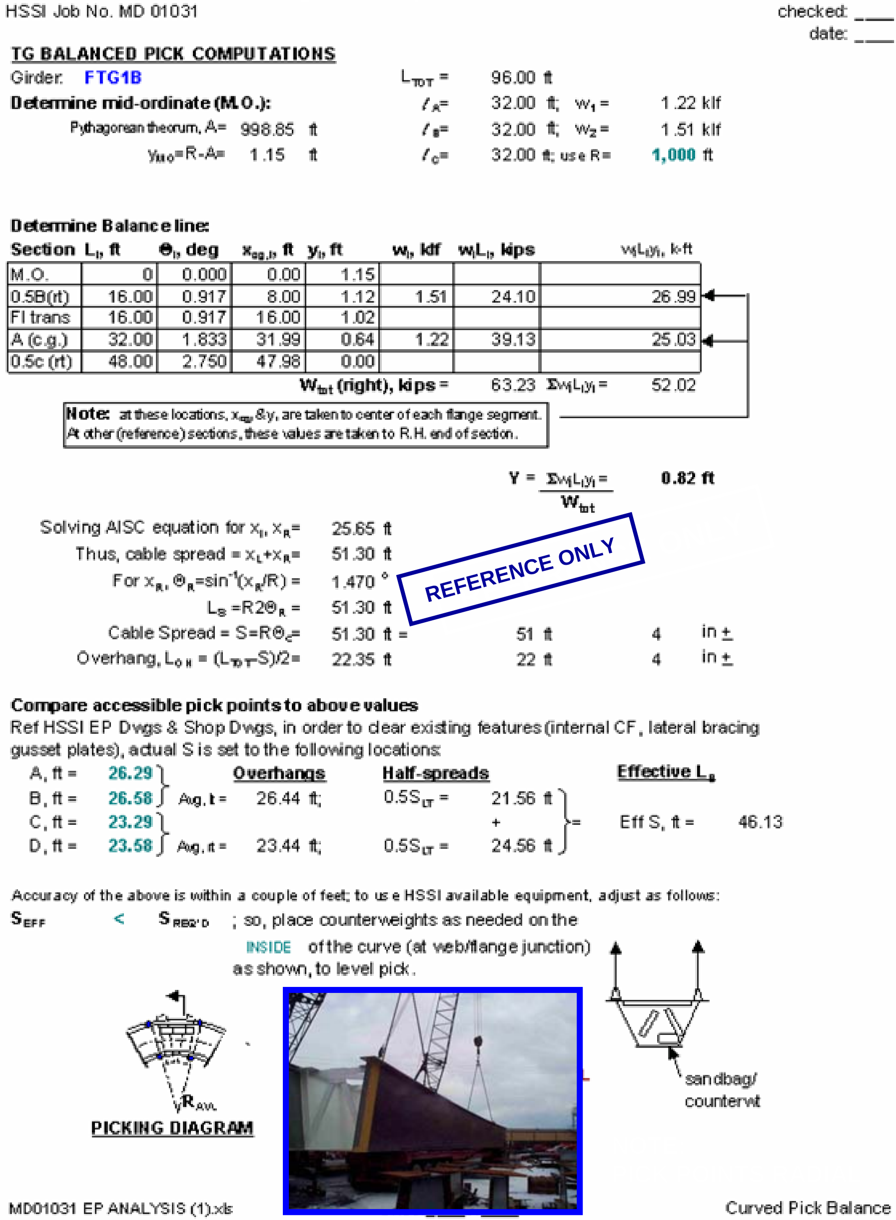


HSSI Job No. MD 01031

checked: ____
date: ____

TG BALANCED PICK COMPUTATIONS

Girder: **FTG1B**

L_{TOT} = 96.00 ft

Determine mid-ordinate (M.O.):

ℓ_A = 32.00 ft; w_1 = 1.22 klf
Pythagorean theorum, A = 998.85 ft
ℓ_B = 32.00 ft; w_2 = 1.51 klf
$y_{M.O.}$ = R - A = 1.15 ft
ℓ_C = 32.00 ft; use R = **1,000** ft

Determine Balance line:

Section	L_i, ft	θ_i, deg	$x_{cg,i}$, ft	y_i, ft	w_i, klf	w_iL_i, kips	$w_iL_iy_i$, k-ft
M.O.	0	0.000	0.00	1.15			
0.5B(rt)	16.00	0.917	8.00	1.12	1.51	24.10	26.99
Fl trans	16.00	0.917	16.00	1.02			
A (c.g.)	32.00	1.833	31.99	0.64	1.22	39.13	25.03
0.5c (rt)	48.00	2.750	47.98	0.00			

W_{tot} (right), kips = 63.23 $\Sigma w_iL_iy_i$ = 52.02

Note: at these locations, x_{cg} & y, are taken to center of each flange segment. At other (reference) sections, these values are taken to R.H. end of section.

$$Y = \frac{\Sigma w_iL_iy_i}{W_{tot}} = \mathbf{0.82\ ft}$$

Solving AISC equation for x_i, x_R = 25.65 ft
Thus, cable spread = $x_L + x_R$ = 51.30 ft
For x_R, $\theta_R = \sin^{-1}(x_R/R)$ = 1.470 °
$L_S = R2\theta_R$ = 51.30 ft
Cable Spread = $S = R\theta_c$ = 51.30 ft = 51 ft 4 in ±
Overhang, $L_{OH} = (L_{TOT} - S)/2$ = 22.35 ft = 22 ft 4 in ±

REFERENCE ONLY

Compare accessible pick points to above values

Ref HSSI EP Dwgs & Shop Dwgs, in order to clear existing features (internal CF, lateral bracing gusset plates), actual S is set to the following locations:

		Overhangs	Half-spreads		Effective L_s
A, ft =	**26.29**	Avg, lt = 26.44 ft;	$0.5S_{LT}$ =	21.56 ft	
B, ft =	**26.58**			+	Eff S, ft = 46.13
C, ft =	**23.29**	Avg, rt = 23.44 ft;	$0.5S_{LT}$ =	24.56 ft	
D, ft =	**23.58**				

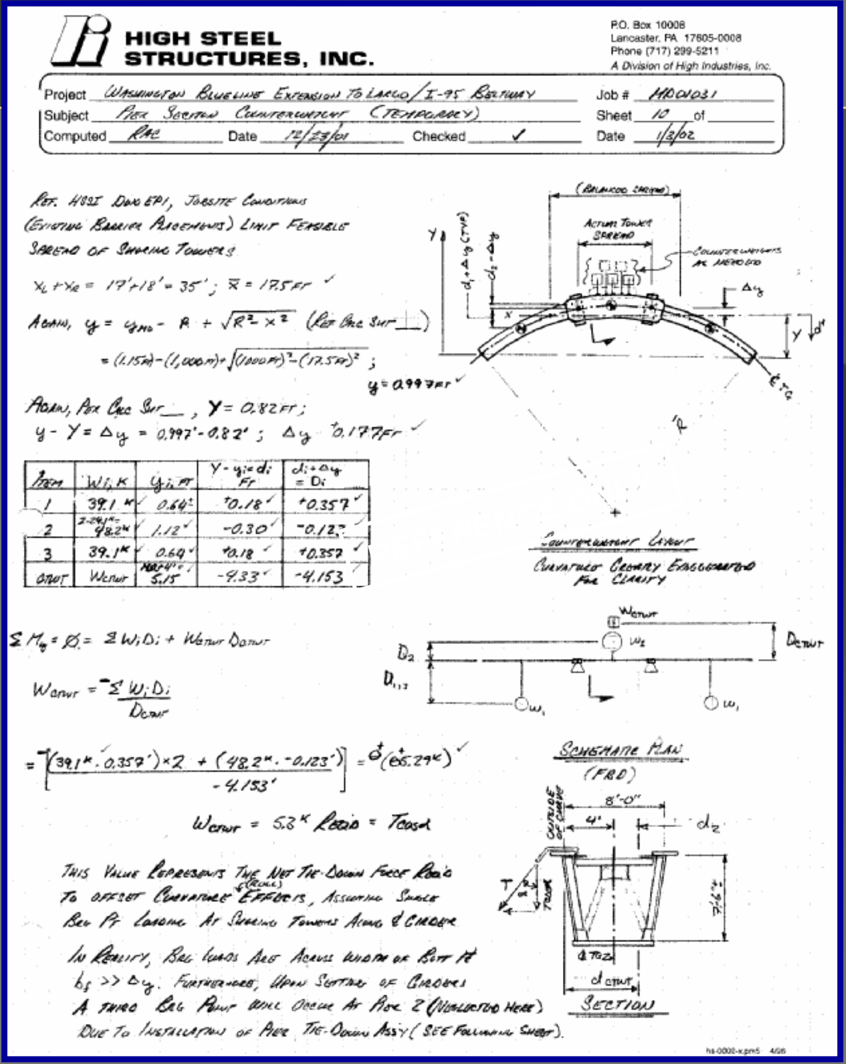
Accuracy of the above is within a couple of feet; to use HSSI available equipment, adjust as follows:

S_{EFF} < $S_{REQ'D}$; so, place counterweights as needed on the INSIDE of the curve (at web/flange junction) as shown, to level pick.

PICKING DIAGRAM

MD01031 EP ANALYSIS (1).xls — Curved Pick Balance

THEN, GIRDERS MAY LEAN (ROLL) IN THE OPPOSITE DIRECTION DURING SHIPPING/ERECTION, REQUIRING TEMPORARY/PERMANENT SUPPORTS…..



HIGH STEEL STRUCTURES, INC.

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Project: WASHINGTON BLUELINE EXTENSION TO LARGO/I-95 BELTWAY — Job #: HD01031
Subject: PIER SECTION COUNTERWEIGHT (TEMPORARY) — Sheet 10 of
Computed: RAC — Date: 12/23/01 — Checked: ✓ — Date: 1/3/02

REF. H031 DWG EP1, JOBSITE CONDITIONS (EXISTING BARRIER PLACEMENTS) LIMIT FEASIBLE SPREAD OF SHORING TOWERS

$x_L + x_R = 17' + 18' = 35'$; $\bar{x} = 17.5$ FT ✓

AGAIN, $y = y_{Mo} - R + \sqrt{R^2 - x^2}$ (REF CRC SHT ___)

$= (1.15\text{ft}) - (1{,}000\text{ft}) + \sqrt{(1000\text{ft})^2 - (17.5\text{ft})^2}$;

$y = 0.997$ FT ✓

AGAIN, PER CRC SHT ___, $Y = 0.82$ FT ;

$y - Y = \Delta y = 0.997' - 0.82'$; $\Delta y = {}^{+}0.177$ FT ✓

ITEM	W_i, K	y_i, FT	$Y - y_i = d_i$ FT	$d_i + \Delta y = D_i$
1	39.1 K ✓	0.64 ±	+0.18 ✓	+0.357 ✓
2	2·24.1 K = 48.2 K ✓	1.12 ✓	−0.30 ✓	−0.123 ✓
3	39.1 K ✓	0.64 ✓	+0.18 ✓	+0.357 ✓
CTWT	W_{CTWT}	5.15 ✓	−4.33 ✓	−4.153 ✓

$\Sigma M_y = \emptyset = \Sigma W_i D_i + W_{CTWT} D_{CTWT}$

$W_{CTWT} = \dfrac{-\Sigma W_i D_i}{D_{CTWT}}$

$= -\left[\dfrac{(39.1^K \cdot 0.357') \times 2 + (48.2^K \cdot -0.123')}{-4.153'}\right] = {}^{+}\emptyset\,({}^{+}5.29^K)$ ✓

$W_{CTWT} = 5.3^K$ REQ'D $= T\cos\alpha$

THIS VALUE REPRESENTS THE NET TIE-DOWN FORCE REQ'D TO OFFSET CURVATURE (ROLL) EFFECTS, ASSUMING SINGLE BRG PT LOADING AT SHORING TOWERS ALONG ℄ GIRDER

IN REALITY, BRG LOADS ARE ACROSS WIDTH OF BOTT FL $b_f \gg \Delta y$. FURTHERMORE, UPON SETTING OF GIRDERS A THIRD BRG POINT WILL OCCUR AT PIER 2 (NEGLECTED HERE) DUE TO INSTALLATION OF PIER TIE-DOWN ASSY (SEE FOLLOWING SHEET).

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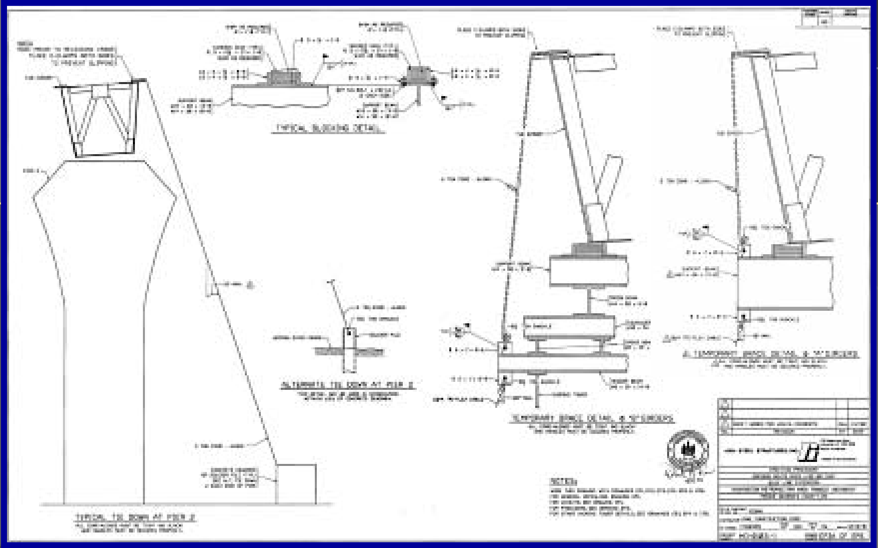
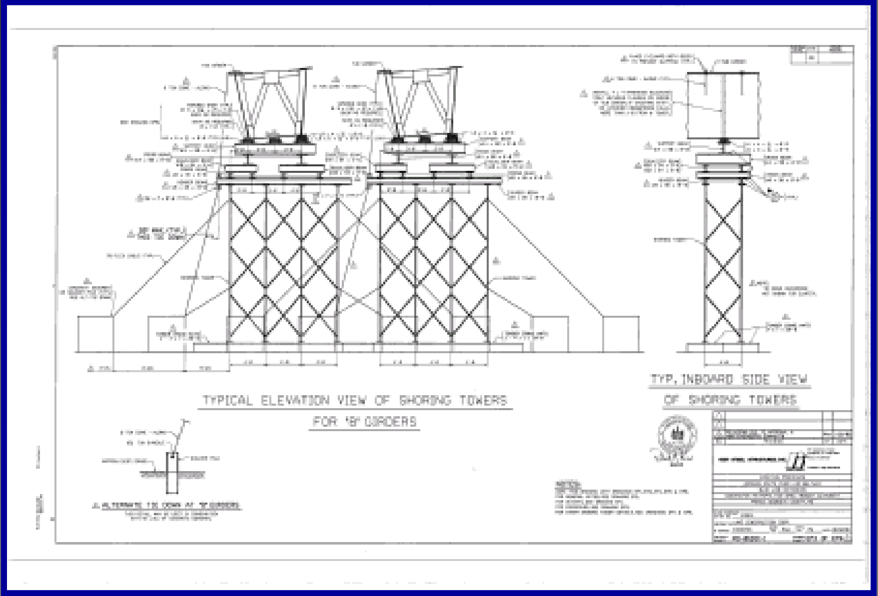
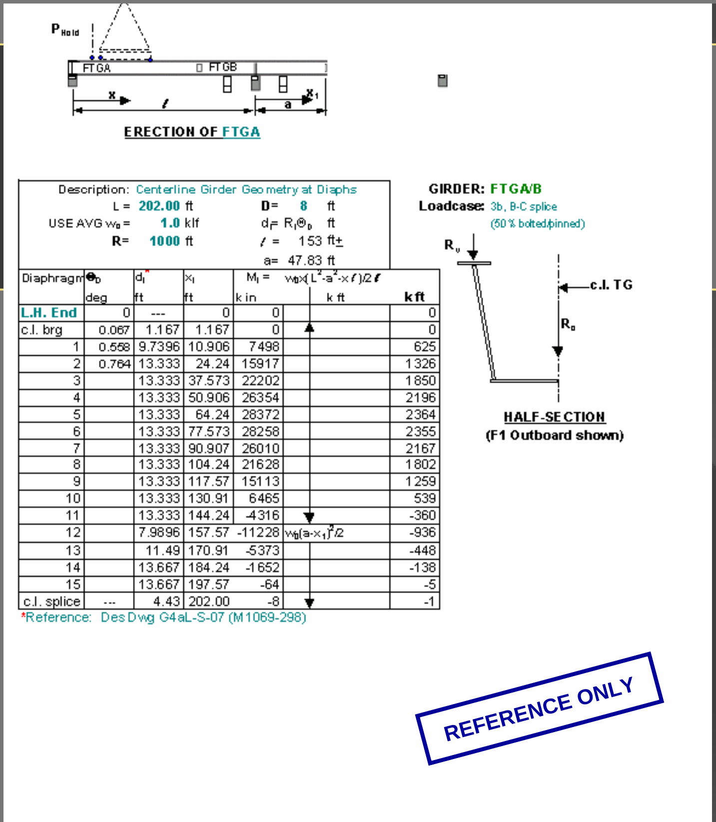




THE SAME GIRDER MAY THEN LEAN BACK IN THE ORIGINAL DIRECTION, DURING SPLICING OF SUBSEQUENT FIELD SECTIONS



ERECTION OF FTGA

Description: Centerline Girder Geometry at Diaphs

L = **202.00** ft
USE AVG w_0 = **1.0** klf
R= **1000** ft
D= **8** ft
$d_i = R_i\Theta_D$ ft
ℓ = 153 ft±
a= 47.83 ft

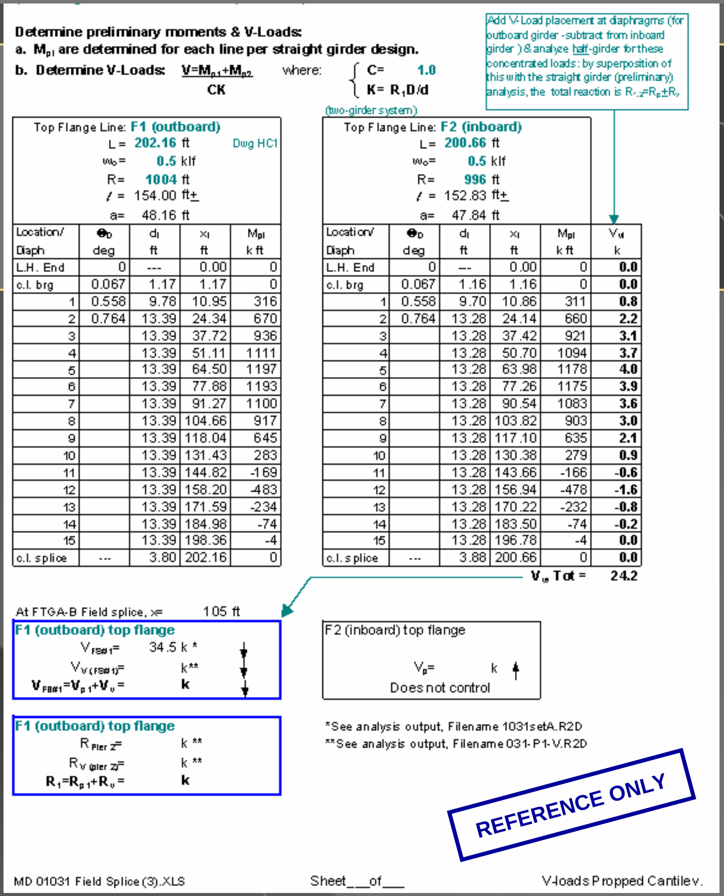
Diaphragm	Θ_D	d_i*	x_i	M_i =	$w_0\times(L^2-a^2-x\ell)/2\ell$	
	deg	ft	ft	k in	k ft	**k ft**
L.H. End	0	---	0	0		0
c.l. brg	0.067	1.167	1.167	0	▲	0
1	0.558	9.7396	10.906	7498		625
2	0.764	13.333	24.24	15917		1326
3		13.333	37.573	22202		1850
4		13.333	50.906	26354		2196
5		13.333	64.24	28372		2364
6		13.333	77.573	28258		2355
7		13.333	90.907	26010		2167
8		13.333	104.24	21628		1802
9		13.333	117.57	15113		1259
10		13.333	130.91	6465		539
11		13.333	144.24	-4316	▼	-360
12		7.9896	157.57	-11228	$w_0(a-x_1)^2/2$	-936
13		11.49	170.91	-5373		-448
14		13.667	184.24	-1652		-138
15		13.667	197.57	-64		-5
c.l. splice	---	4.43	202.00	-8	▼	-1

*Reference: Des Dwg G4aL-S-07 (M1069-298)

GIRDER: **FTGA/B**
Loadcase: 3b, B-C splice
(50 % bolted/pinned)

HALF-SECTION
(F1 Outboard shown)

REFERENCE ONLY



Determine preliminary moments & V-Loads:
a. M_{pl} are determined for each line per straight girder design.
b. Determine V-Loads: $V=\dfrac{M_{p1}+M_{p2}}{CK}$ where: C= **1.0**, K= R_1D/d
(two-girder system)

Add V-Load placement at diaphragms (for outboard girder - subtract from inboard girder) & analyze half-girder for these concentrated loads: by superposition of this with the straight girder (preliminary) analysis, the total reaction is $R_{1,2}=R_p\pm R_v$

Top Flange Line: **F1 (outboard)**
L = **202.16** ft   Dwg HC1
w_0= **0.5** klf
R= **1004** ft
ℓ = 154.00 ft±
a= 48.16 ft

Location/ Diaph	Θ_D deg	d_i ft	x_i ft	M_{pl} k ft
L.H. End	0	---	0.00	0
c.l. brg	0.067	1.17	1.17	0
1	0.558	9.78	10.95	316
2	0.764	13.39	24.34	670
3		13.39	37.72	936
4		13.39	51.11	1111
5		13.39	64.50	1197
6		13.39	77.88	1193
7		13.39	91.27	1100
8		13.39	104.66	917
9		13.39	118.04	645
10		13.39	131.43	283
11		13.39	144.82	-169
12		13.39	158.20	-483
13		13.39	171.59	-234
14		13.39	184.98	-74
15		13.39	198.36	-4
c.l. splice	---	3.80	202.16	0

Top Flange Line: **F2 (inboard)**
L = **200.66** ft
w_0= **0.5** klf
R= **996** ft
ℓ = 152.83 ft±
a= 47.84 ft

Location/ Diaph	Θ_D deg	d_i ft	x_i ft	M_{pl} k ft	V_{vi} k
L.H. End	0	---	0.00	0	**0.0**
c.l. brg	0.067	1.16	1.16	0	**0.0**
1	0.558	9.70	10.86	311	**0.8**
2	0.764	13.28	24.14	660	**2.2**
3		13.28	37.42	921	**3.1**
4		13.28	50.70	1094	**3.7**
5		13.28	63.98	1178	**4.0**
6		13.28	77.26	1175	**3.9**
7		13.28	90.54	1083	**3.6**
8		13.28	103.82	903	**3.0**
9		13.28	117.10	635	**2.1**
10		13.28	130.38	279	**0.9**
11		13.28	143.66	-166	**-0.6**
12		13.28	156.94	-478	**-1.6**
13		13.28	170.22	-232	**-0.8**
14		13.28	183.50	-74	**-0.2**
15		13.28	196.78	-4	**0.0**
c.l. splice	---	3.88	200.66	0	**0.0**

V_v Tot = **24.2**

At FTGA-B Field splice, x= 105 ft

F1 (outboard) top flange
$V_{FS\#1}$= 34.5 k *
$V_{V(FS\#1)}$= k**
$V_{FS\#1}=V_{p1}+V_v$ = k

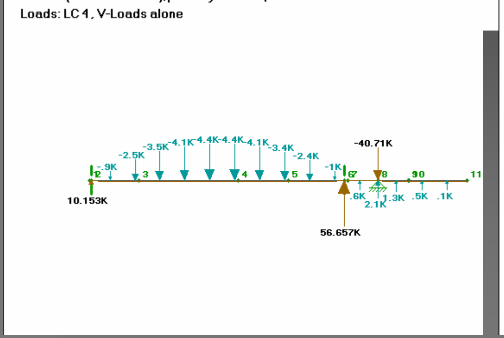
F2 (inboard) top flange
V_p= k
Does not control

F1 (outboard) top flange
$R_{Pier\ 2}$= k **
$R_{V(pier\ 2)}$= k **
$R_1=R_{p1}+R_v$ = k

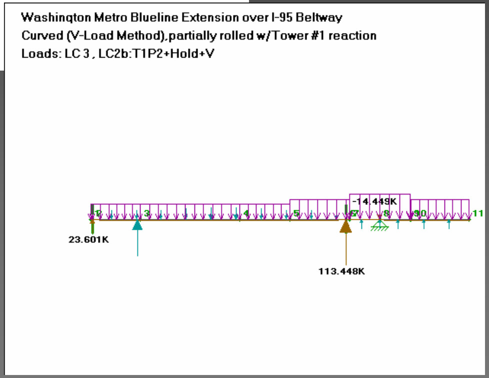
*See analysis output, Filename 1031setA.R2D
**See analysis output, Filename 031-P1-V.R2D

REFERENCE ONLY

MD 01031 Field Splice (3).XLS   Sheet___of___   V-loads Propped Cantilev.



Washington Metro Blueline Extension over I-95 Beltway
Curved (V-Load Method),partially rolled w/Tower #1 reaction
Loads: LC 4 , V-Loads alone

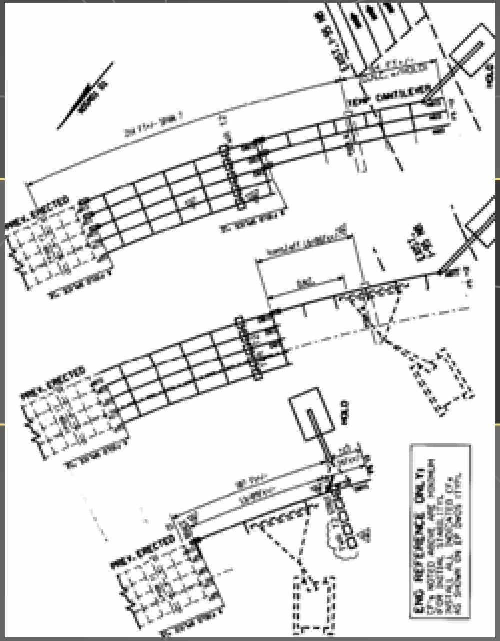


Washington Metro Blueline Extension over I-95 Beltway
Curved (V-Load Method),partially rolled w/Tower #1 reaction
Loads: LC 3 , LC2b:T1P2+Hold+V

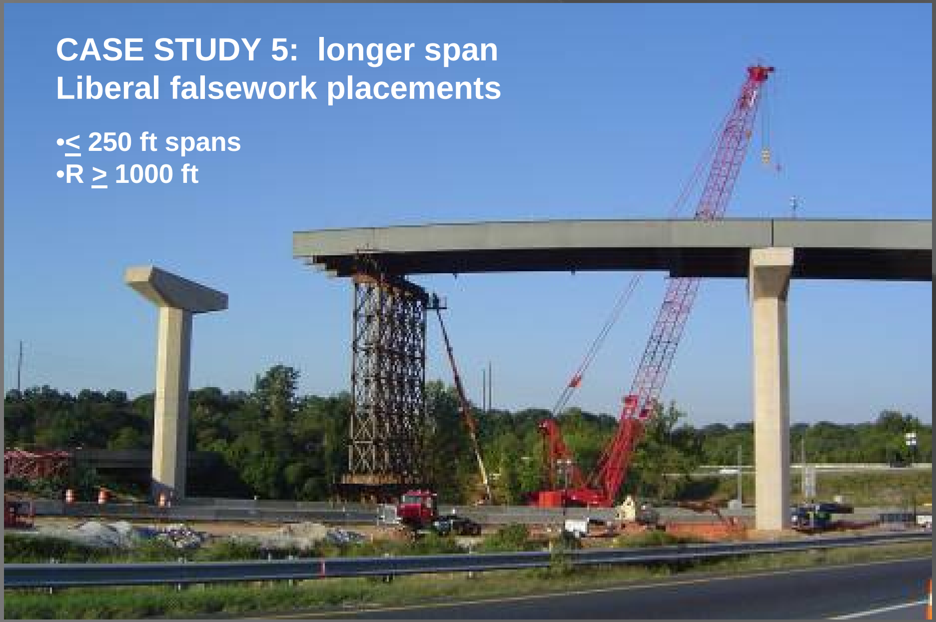
Thus, curved box girder rigidity warrants global (stability), in addition to the conventional torsional stability check.

Case Study 5

Erecting I-girders with temporary shoring when liberal placements are feasible



SUBJECT SEQUENCE UNIT



CASE STUDY 5: longer span
Liberal falsework placements

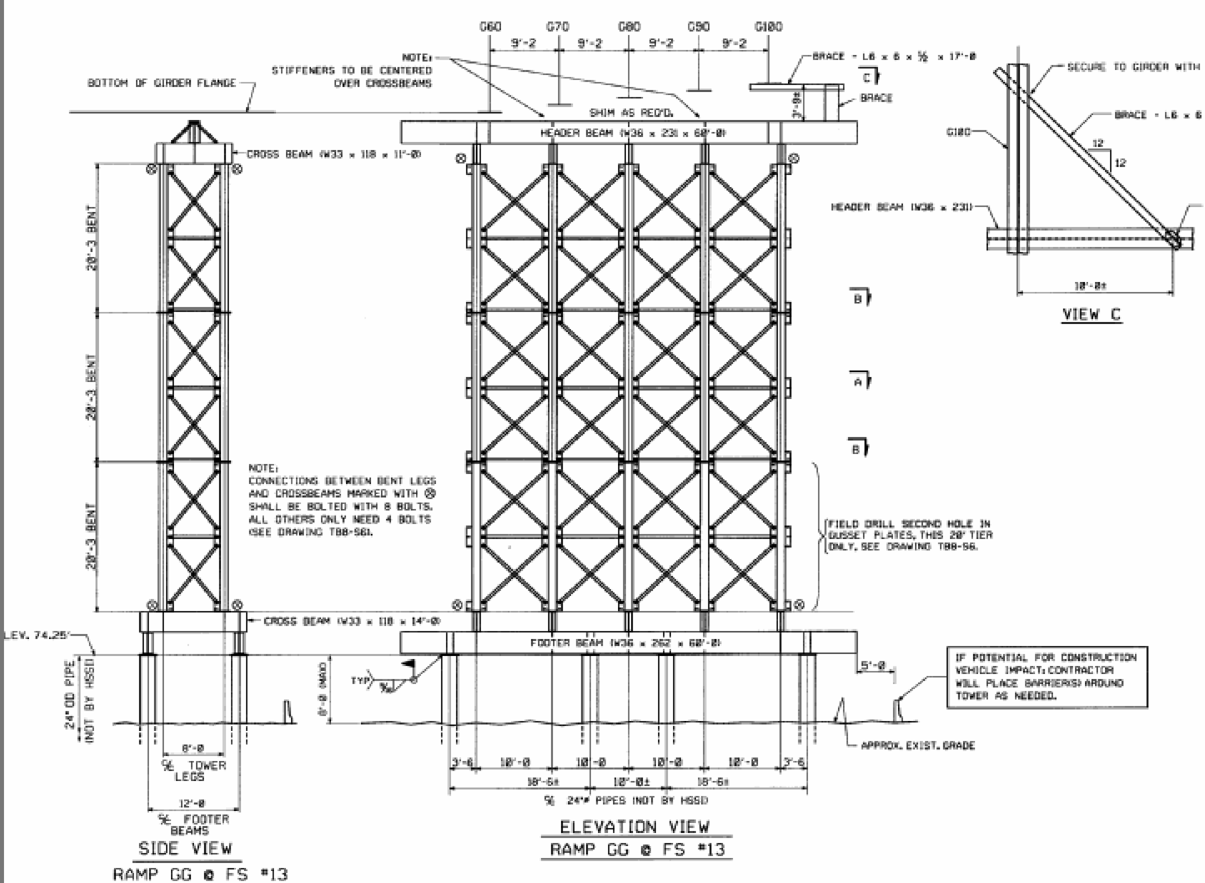
- ≤ 250 ft spans
- R ≥ 1000 ft

I-695/I-95 Interchange: Ramp GG, Structure S6
Baltimore County, Maryland (Towson)
Owner: Maryland Transportation Authority
Contractor: Wagman/Corman/McLean Tri-venture
Fabricator/Erector: High Steel Structures, Inc.

CASE STUDY 5 (continued)

BRIDGE TEMPORARY WORKS: FALSEWORK/SHORING

Reference: AASHTO LRFD BRIDGE CONSTRUCTION SPECIFICATIONS, 2^{ND} ED. (2004)



Sections 3.1.3 (Temporary Works) and 3.2 (Falsework) provide direction regarding established and generally accepted codes or specifications (verify acceptance criteria with the Engineer).

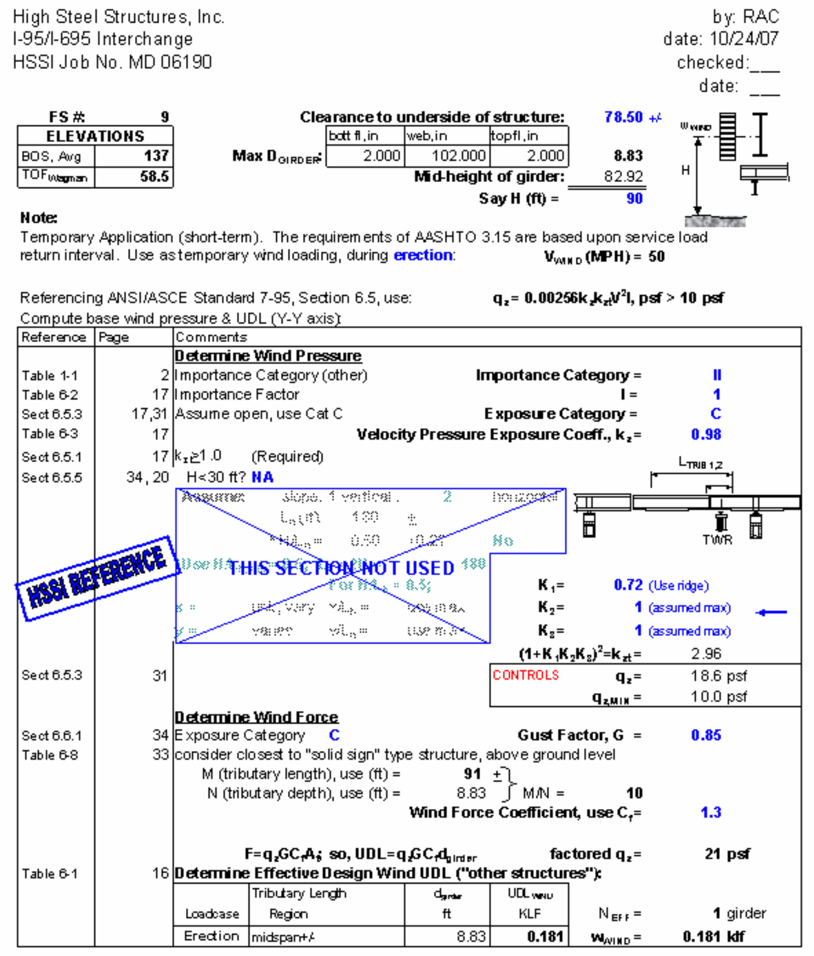
<u>CAUTION</u>: do not mix codes without due consideration of resistance level consistency among various falsework components, & erection equipment. Be especially alert for LFRD, LFD & SLD (ASD) based differences, such as:

- limit states
- resistance factors
- working loads
- explicit/inherent factors of safety.

CASE STUDY 5 (continued)

TOWER 2 (SUMMER):
- wind on tower
- wind on partially erected structure
- contingency (hurricane) tie-downs & anchorage

SOUND ENGINEERING JUDGEMENT IS KEY IN APPLYING APPROPRIATE, SITE-SPECIFIC WINDLOAD(S) FOR CONTROLLING ERECTION LOADCASES.



High Steel Structures, Inc.
I-95/I-695 Interchange
HSSI Job No. MD 06190

by: RAC
date: 10/24/07
checked:___
date: ___

FS #:	9
ELEVATIONS	
BOS, Avg	**137**
TOF$_{Wagman}$	**58.5**

Clearance to underside of structure: **78.50** +/-

	bott fl, in	web, in	topfl, in	
Max D$_{GIRDER}$:	2.000	102.000	2.000	**8.83**

Mid-height of girder: 82.92
Say H (ft) = **90**

Note:
Temporary Application (short-term). The requirements of AASHTO 3.15 are based upon service load return interval. Use as temporary wind loading, during **erection**: **V$_{WIND}$ (MPH) = 50**

Referencing ANSI/ASCE Standard 7-95, Section 6.5, use: **$q_z = 0.00256 k_z k_{zt} V^2 I$, psf > 10 psf**
Compute base wind pressure & UDL (Y-Y axis):

Reference	Page	Comments		
		Determine Wind Pressure		
Table 1-1	2	Importance Category (other)	**Importance Category =**	**II**
Table 6-2	17	Importance Factor	**I =**	**1**
Sect 6.5.3	17,31	Assume open, use Cat C	**Exposure Category =**	**C**
Table 6-3	17		**Velocity Pressure Exposure Coeff., k_z =**	**0.98**
Sect 6.5.1	17	$k_z \geq 1.0$ (Required)		
Sect 6.5.5	34, 20	H<30 ft? **NA**		
		THIS SECTION NOT USED	**K_1 =**	**0.72** (Use ridge)
			K_2 =	**1** (assumed max)
			K_3 =	**1** (assumed max)
			$(1+K_1K_2K_3)^2 = k_{zt}$ =	2.96
Sect 6.5.3	31	CONTROLS	**q_z =**	18.6 psf
			q_{zMIN} =	10.0 psf
		Determine Wind Force		
Sect 6.6.1	34	Exposure Category **C**	**Gust Factor, G =**	**0.85**
Table 6-8	33	consider closest to "solid sign" type structure, above ground level		
		M (tributary length), use (ft) = **91** ±		
		N (tributary depth), use (ft) = 8.83	M/N =	**10**
			Wind Force Coefficient, use C_f =	**1.3**
		$F = q_z G C_f A_f$; so, UDL $= q_z G C_f d_{girder}$	**factored q_z =**	**21 psf**
Table 6-1	16	**Determine Effective Design Wind UDL ("other structures"):**		

Loadcase	Tributary Length Region	d_{girder} ft	UDL$_{WIND}$ KLF		
				N$_{EFF}$ =	**1** girder
Erection	midspan+/-	8.83	**0.181**	**w_{WIND} =**	**0.181 klf**

TOWER 1 (SPRING):
- wind on tower
- wind on partially erected structure
- Skewed substructure considerations

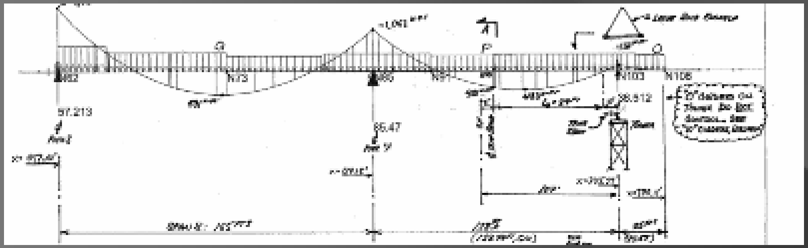
Subject structure used Engineer-accepted, site-specific (inland coastal) wind-levels for:
- tower alone (prior to girder erection)
- picking/setting girders (within single shift)
- partially erected structure (e.g., weekend)
- longer-term contingency (monitoring local weather)

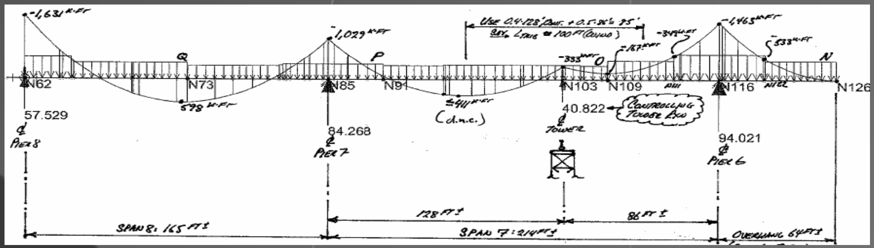
CASE STUDY 5 (continued)

SITE-SPECIFIC EXAMPLE: TOWER 2

STAGES CHECKED (CONTROLLING COMBINATIONS USED)

TOWER ALONE/CONST ($V_{\text{WIND, LONG/LAT}}$)





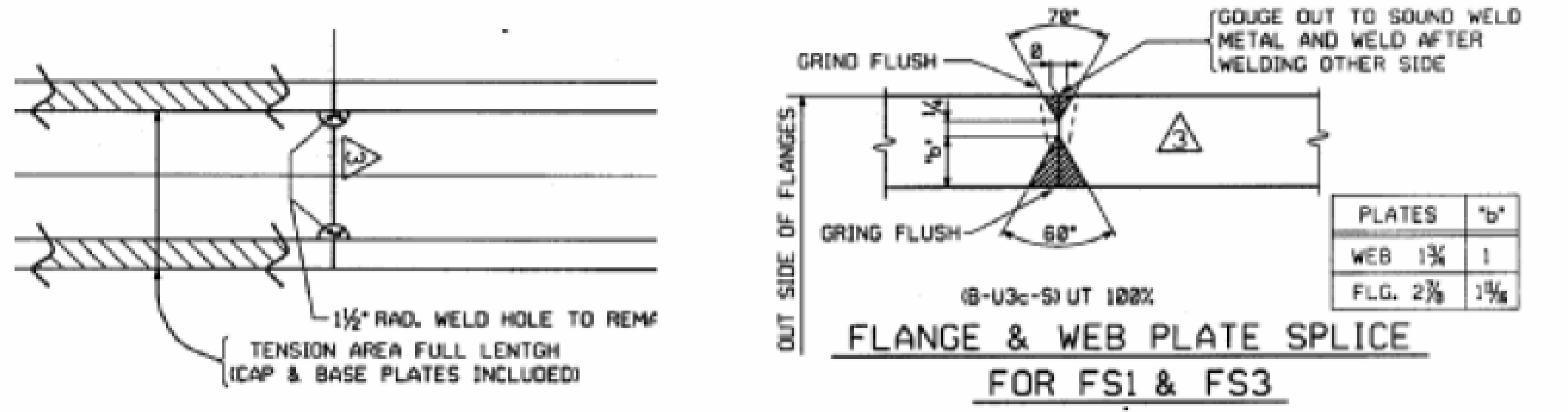
INITIAL GIRDER LINES
WEEKEND ($V_{\text{WIND}} \leq$ 40 MPH)

PARTIAL LOADING
SHORT-TERM ($V_{\text{WIND}} \leq$ 50 MPH)

PARTIAL LOADING
LONGER-TERM ($V_{\text{WIND}} \leq$ 80 MPH)

Field Welding

HSS FIELD SPLICE



WIDE FLANGE SHOP SPLICE

Bridge erection methodologies bear scrutiny & innovation. Example: consider field welding of I-girder/tub girder field splices (Texas practice). Advantages include smoother appearance than bolting (ratholes may be filled with caulk).

Thank you for your attention.

QUESTIONS?

HIGH STEEL STRUCTURES INC.
An Affiliate of High Industries Inc.

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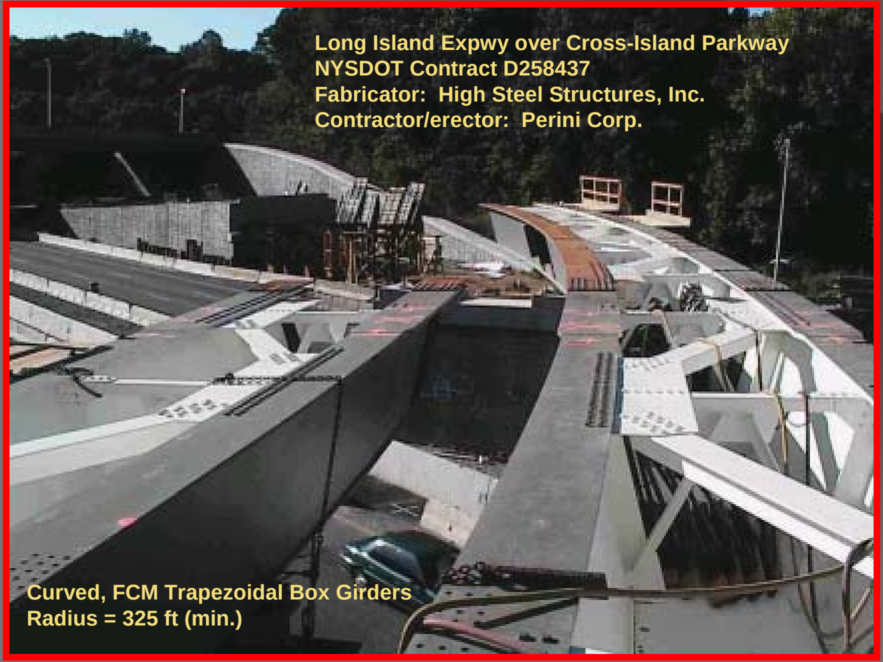

Long Island Expwy over Cross-Island Parkway
NYSDOT Contract D258437
Fabricator: High Steel Structures, Inc.
Contractor/erector: Perini Corp.

Curved, FCM Trapezoidal Box Girders
Radius = 325 ft (min.)

Presented by: Mike Alterio,
Alpha Structures, Inc.
IBC Bridge Const. Seminar
June 2nd, 2008

HIGH STEEL STRUCTURES INC.
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