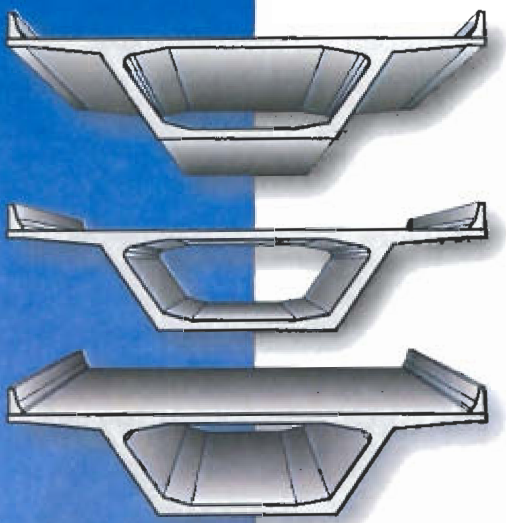
Draft Design Example

ASBI

AMERICAN SEGMENTAL BRIDGE INSTITUTE

PRECAST BALANCED CANTILEVER BRIDGE DESIGN USING AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS



Prepared by

Teddy S. Theryo, P.E.

Major Bridge Service Center

PARSONS BRINCKERHOFF

Prepared for

American Segmental Bridge Institute

July 2004

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NOTICE OF DISCLAIMER

This document is a draft effort at developing a design example of a precast balanced cantilever bridge design based on the third edition of the *AASHTO LRFD Bridge Design Specifications, 2004*. In its current form, it is not intended to represent a definitive reference for the design of either a segmental bridge or for the application of the LRFD Specifications to segmental bridge design. Additionally, the design steps shown in this example problem do not represent all of the steps that are required for the complete design of a segmental box girder bridge.

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1. INTRODUCTION

The AASHTO-PCI-ASBI Joint Committee was formed in Chicago, Illinois in October, 1994. The main goal of the committee was to develop a set of standard box sections for precast segmental grade separation bridges which would cover bridges of short to medium span ranges (approximately 200'-0" maximum span). The present practice in the industry shows that only sufficiently large projects can be competitively built using the precast segmental erection method due to the high cost of setting up a casting yard which is generally used only for one particular project. However, by using standardized cross sections, it is expected that precasters in the concrete industry could afford to build their own casting cells due to potential for repetitive work from contractors. In addition, the standard cross sections would be useful for structural engineers as an initial section for conceptual design and preliminary design stages.

Development of a family of standardized segmental box girder sections in metric units was completed and published by PCI / ASBI in 1998. The next step of the Committee's work is to evaluate the proposed standard sections through the creation of design examples. Three separate design examples were created: span-by-span erection with all external tendons, span-by-span erection with both external and internal tendons, and balanced cantilever erection. In additional to design issues, detailing will be discussed. The design examples will be done in accordance with the current *AASHTO LRFD Bridge Design Specifications, Third Edition, 2004.* This design example has been extensively used over the years in the annual ASBI "Design and Construction of Segmental Concrete Bridges" Seminar since its publication in 1996.

The following design report will cover only precast segmental balanced cantilever construction. The design example is a five-span precast segmental bridge with three 200'-0" interior spans and two 150'-0" end spans. The bridge will be supported on bearings, all of which are sliding bearings except for fixed bearings at Pier 4. The width of the bridge deck is 43'-0" which will accommodate two lanes of traffic plus inside and outside shoulders of an interstate ramp. The Type 2700-2 AASHTO-PCI-ASBI Standard Section was selected for this design example. The section depth is 9'-0" with a maximum span-to-depth ratio of 22. This report will also make a brief comparative study between AASHTO LRFD and LFD Design Specifications Load Combinations, including shear design.

The longitudinal analysis of the bridge will be performed using the Proprietary TANGO Program which enables the effects of stage-by-stage construction and time dependent analysis to be considered in the design. The transverse design will be accomplished with the aid of Proprietary GT-STRUDL and BDAC Programs.

2. DESIGN CRITERIA

The following criteria will be used for this design example:

A. Specifications, Codes, and Standards:

AASHTO LRFD Bridge Design Specifications, Third Edition 2004

B. Design Loadings:

1. Load Modifier:

A load modifier of 1.0 will be used for all limit states based on redundant members with the possibility of non-ductile components and connections, assuming an operational importance factor of 1.0 for all components.

2. Dead Load:

Unit Weight of Reinforced Concrete (DC):	0.150 KCF (23.5 KN/$m^{3)}$
Unit Weight of Post-Tensioned Concrete (DC):	0.155 KCF (24.3 KN/$m^{3)}$
Wearing Surface (DW):	0.015 KSF (0.72 KN/$m^{2)}$
Traffic Barriers (DC):	0.421 KLF (6.14 KN/m each)
Weight of Blisters (DC):	1 KIP each (4.4 KN each)

3. Live Load:

Vehicle: HL-93 (3 design lanes) using multiple presence factors and dynamic load allowance, as appropriate.

4. Wind Loads:

Design in accordance with LRFD Article 3.8.

5. Thermal Forces:

Seasonal Variation:

Mean Temperature:	70^{o} F (21^{o}C)
Thermal Coefficient:	6.5 x 10^{-6} ^{o} F (10.8 x 10^{-6} per ^{o}C)
Temperature Rise:	30^{o} F (17^{o} C)
Temperature Fall:	45^{o} F (25^{o} C)

Differential Temperature:

Longitudinal:

Non-linear temperature gradient as per LRFD Article 3.12.3 using a plain concrete surface for Solar Radiation Zone 3.

Transverse:

Reversible linear gradient of 10^{o} F (6^{o} C) between inside and outside of box girders.

6. Creep and Shrinkage:

Strains calculated in accordance with *CEB-FIP 1990 Model Code* for superstructures.

7. Earthquake:

Seismic Zone 1
Acceleration Coefficient: 0.06
Soil Type II

8. Construction Loads:

Construction loads are in accordance with LRFD Article 5.14.2.3. using the appropriate construction load combinations and allowable stresses. Load factor for temperature gradient during construction $\gamma_{TG} = 0.0$.

C. MATERIALS:

1. Concrete:

28 day Cylinder Compressive Strength:	6.0 KSI (42 Mpa)
Modulus of Elasticity:	4933 KSI (34,000 Mpa)
Allowable Stresses:	As per LRFD Article 5.9.4.

Superstructure concrete cover for main reinforcing, plastic (PE) ducts, and hardware:

Top riding surface	2 Inches (50 mm)
Exterior and interior	2 Inches (50 mm)

Concrete cover to plastic ducts shall not be less than one-half the diameter of the duct.

2. Reinforcing Steel:

Yield Strength:	60 KSI (400 Mpa)
Modulus of Elasticity:	29,000 KSI (200,000 Mpa)

3. Prestressing Steel:

Strand tendons shall consist of low-relaxation steel.

Material Properties:

Ultimate Tensile Strength (f_{pu}):	270 KSI (1860 Mpa)
Yield Strength (fpy):	243 KSI (1674 Mpa)
Apparent Modulus of Elasticity:	28,500 KSI (197,000 Mpa)
Friction Coefficient:	0.23 per RAD
Wobble Coefficient:	0.00020 per ft (0.00066 per m)
Anchor Set:	3/8 " (10 mm)

Allowable Stresses:

Jacking Force:	0.80 f_{pu}
At anchorages After Anchoring	0.70 f_{pu}
At other locations After Anchoring	0.74 f_{pu}
At Service Limit State After Losses	0.80 f_{py}

Bar tendons shall consist of high strength threaded bars.

Material Properties:

Ultimate Tensile Strength (f_{pu}):	150 KSI (1035 Mpa)
Yield Strength (f_{py}):	120 KSI (828 Mpa)
Modulus of Elasticity:	30,000 KSI (207,000 Mpa)
Friction Coefficient:	0.30 per RAD
Wobble Coefficient:	0.00020 per ft (0.00066 per m)
Anchor Set:	0.0625 inches (1.6 mm)

Allowable Stresses:

Permanent Bars:

Jacking Force:	0.75 f_{pu}
At Anchorages After Anchoring:	0.66 f_{pu}
At Service Limit State After Losses:	0.80 f_{py}

Temporary Bars for Reuse:

Jacking Force 0.50 f_{pu}

D. Design Method:

All applicable limit states (Strength, Extreme Event, Service, and Fatigue) will be satisfied in accordance with the LRFD Specifications.

3. SPAN CONFIGURATION AND TYPICAL SECTIONS

The structure is a five-span bridge with span configuration of 150’, 200’, 200’, 200’, 150’, producing a total length of 900 feet. The bridge carries two 12’-0” lanes of traffic in one direction with a left shoulder width of 6’-0” and a right shoulder width of 10’-0”. Expansion bearings are placed at all piers except Pier 4 which is fixed.

The typical section selected is the AASHTO-PCI-ASBI Segmental Box Girder Standard Type 2700-2, a single-cell concrete box girder with 43’-0” wide deck and 9’-0” in depth. Cantilevered overhangs are 10’-4.5” each. Minimum top slab thickness is 9”. The thickness of the bottom slab is 18” for three segments on both sides of each pier and 9” thick elsewhere. The thickness of the webs is 16”, which are sloped at 2.5:1.

The top slab can accommodate 12 tendons in each half of the box girder, for a total of 24 tendons in the top slab. The bottom slab can accommodate 6 tendons in each half of the box, for a total of 12 tendons in the bottom slab. Additional tendons may still be accommodated either in the top or bottom slab.

When dealing with development of a cross-section, it is important to investigate the efficiency of the proposed cross-section. The section efficiency of the AASHTO-PCI-ASBI 2700-2 section can be computed using Guyon’s formula:

$$\rho = \frac{I_c}{A_c y_t y_b}$$

where,

I_c = Moment of inertia of the section

A_c = Area of the section

y_t = Distance from the top fiber to the center of gravity of the section

y_b = Distance from the bottom fiber to the center of gravity of the section

The efficiency of the cross-section, ρ, is 0.6 which is considered to be high. For the sake of comparison, the flat slab is the most inefficient section with a ρ value of 0.33.

This design example utilizes a 12’-0” typical segment length, resulting in a maximum segment weight of 72.5 tons for the thin bottom slab segment and 80 tons for the thick bottom slab segment.

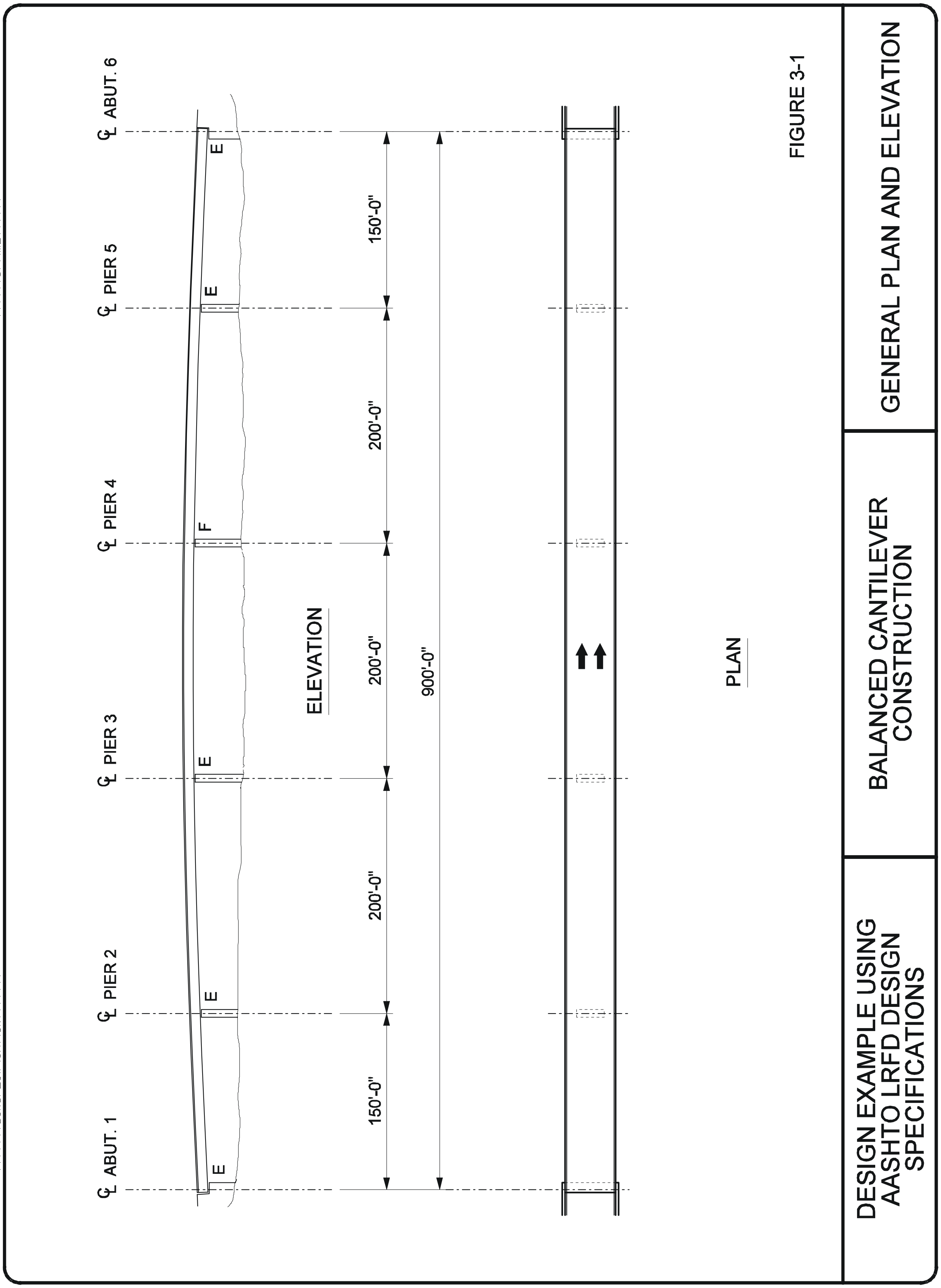
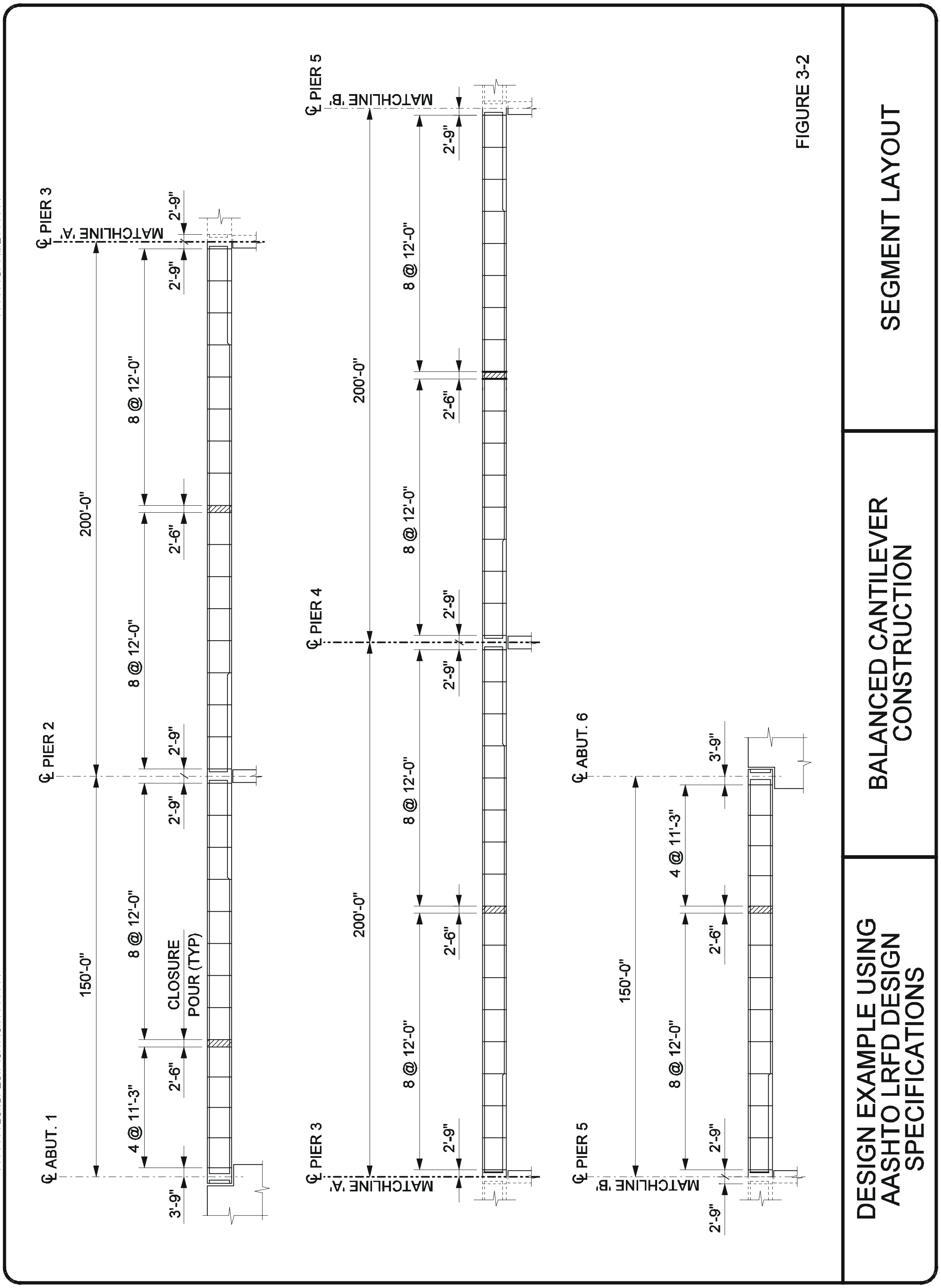
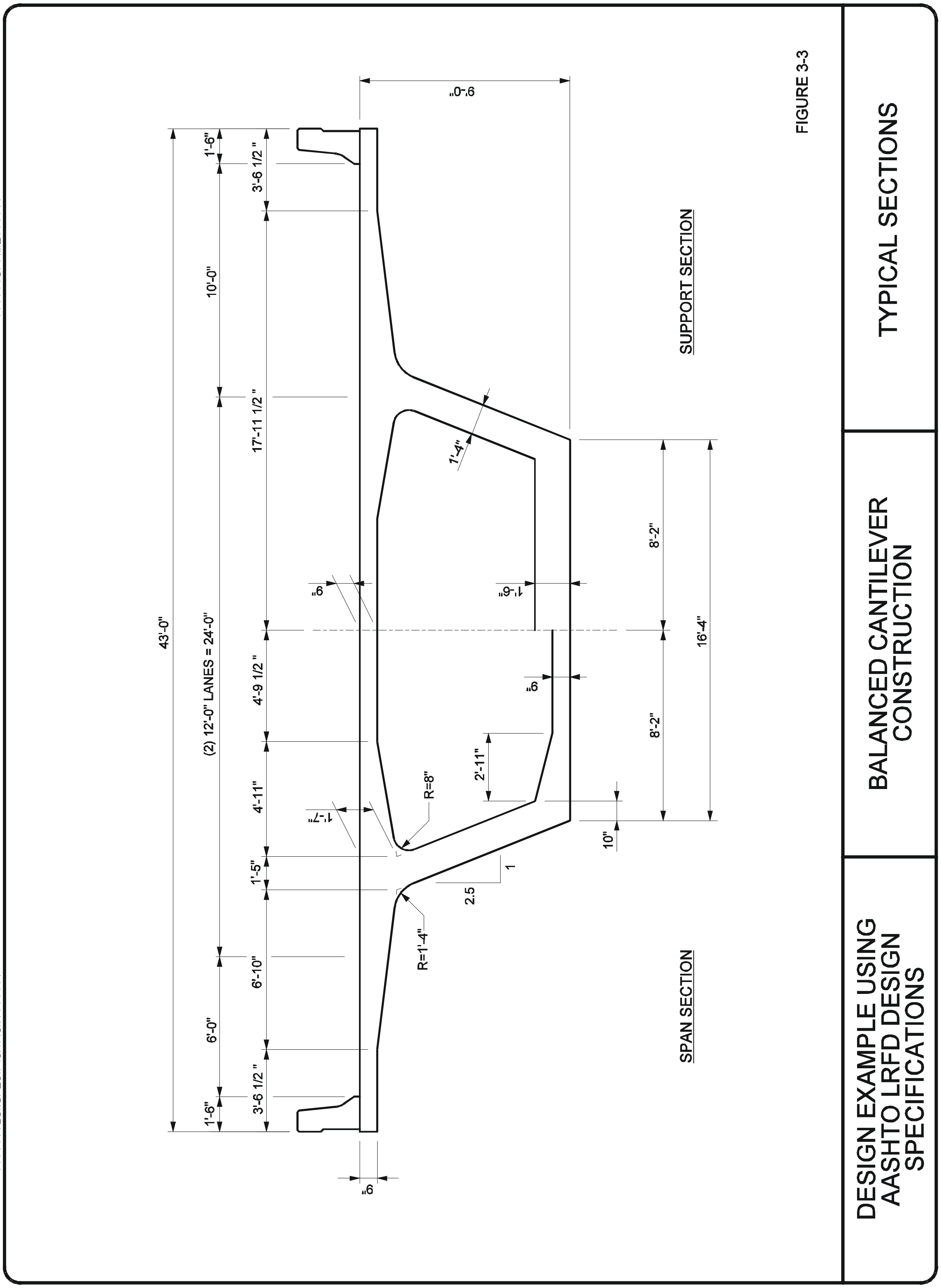

$$$$$$$DGNSPECIFICATION$$$$$$$
$$$$$$SYTIME$$$$$$
℄ ABUT. 1
℄ PIER 2
℄ PIER 3
℄ PIER 4
℄ PIER 5
℄ ABUT. 6
E
E
E
F
E
E
150'-0"
200'-0"
200'-0"
200'-0"
150'-0"
900'-0"
ELEVATION
PLAN
DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS
BALANCED CANTILEVER CONSTRUCTION
GENERAL PLAN AND ELEVATION

FIGURE 3-1


$$$$$$$DGNSPECIFICATION$$$$$$$$
$$$$$$$SYTIME$$$$$$
℄ ABUT. 1
℄ PIER 2
℄ PIER 3
℄ PIER 4
℄ PIER 5
℄ ABUT. 6
MATCHLINE 'A'
MATCHLINE 'B'
150'-0"
200'-0"
4 @ 11'-3"
8 @ 12'-0"
3'-9"
2'-6"
2'-9"
CLOSURE POUR (TYP)
FIGURE 3-2
DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS
BALANCED CANTILEVER CONSTRUCTION
SEGMENT LAYOUT


9'-0"
1'-6"
3'-6 1/2 "
10'-0"
17'-11 1/2 "
43'-0"
(2) 12'-0" LANES = 24'-0"
4'-9 1/2 "
4'-11"
1'-5"
6'-10"
6'-0"
3'-6 1/2 "
1'-6"
9"
SUPPORT SECTION
SPAN SECTION
1'-4"
8'-2"
16'-4"
8'-2"
1'-6"
9"
9"
2'-11"
R=8"
1'-7"
10"
1
2.5
R=1'-4"
FIGURE 3-3
TYPICAL SECTIONS
BALANCED CANTILEVER
CONSTRUCTION
DESIGN EXAMPLE USING
AASHTO LRFD DESIGN
SPECIFICATIONS

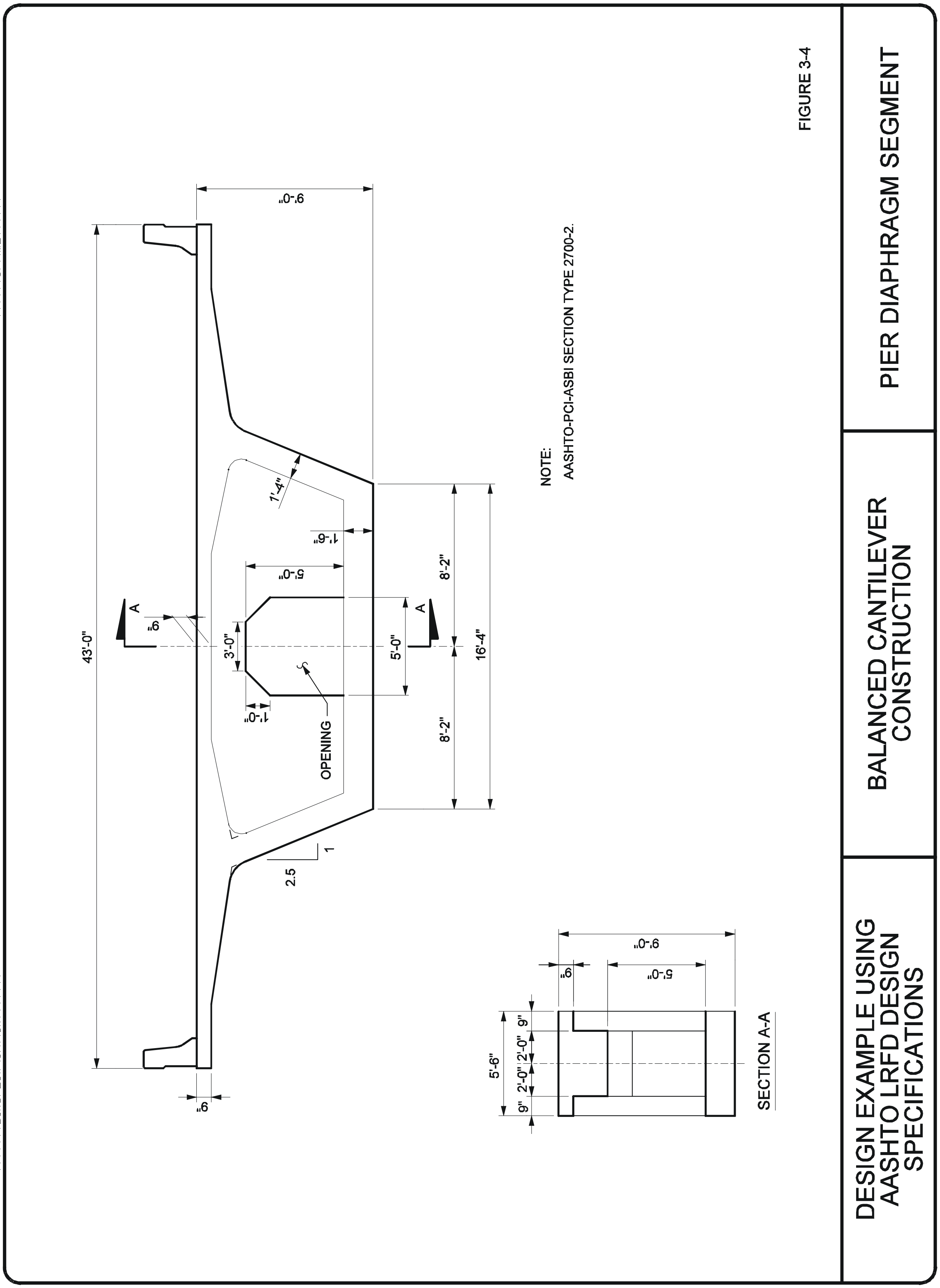
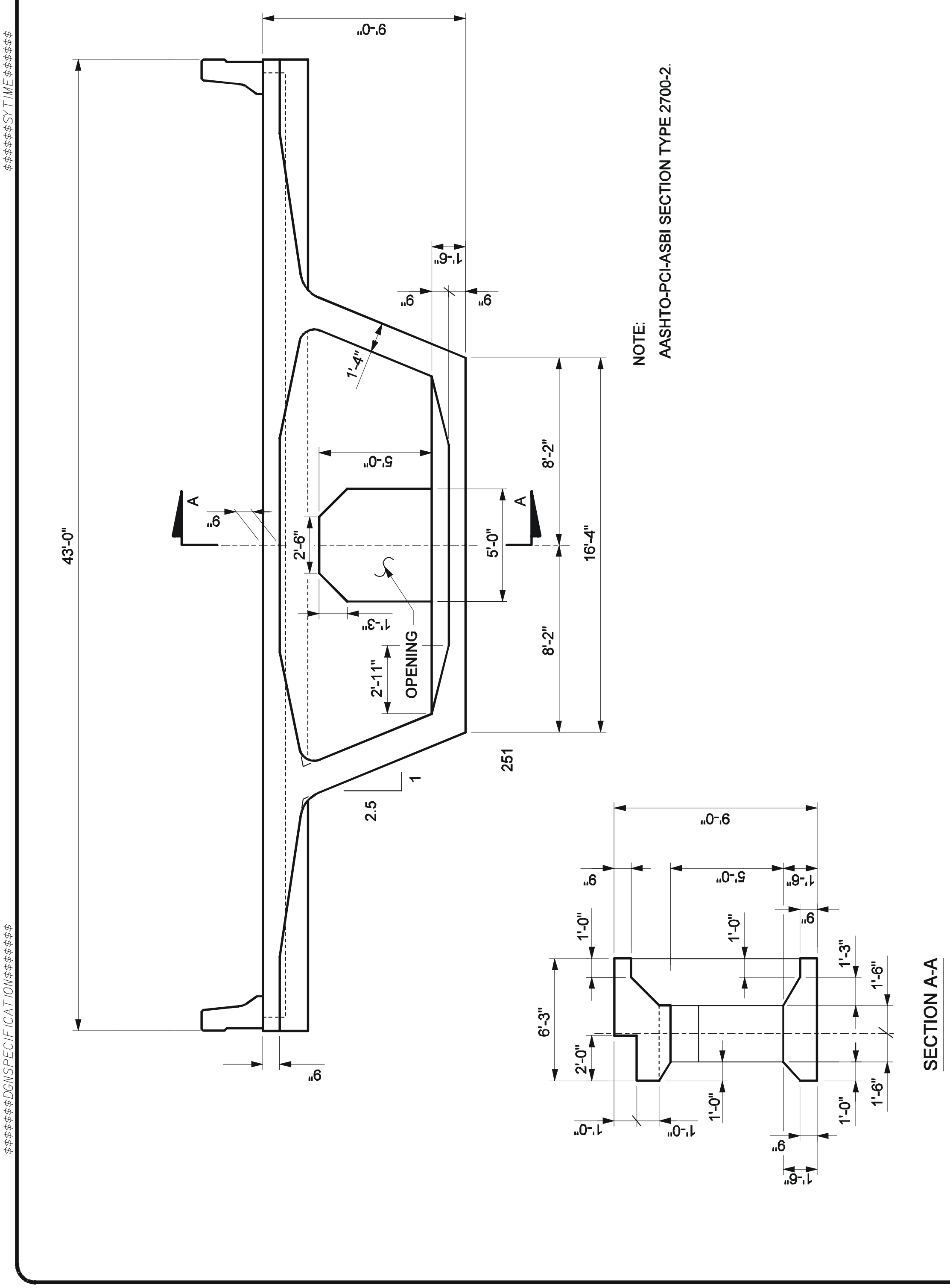


FIGURE 3-4
PIER DIAPHRAGM SEGMENT
BALANCED CANTILEVER CONSTRUCTION
DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS
NOTE:
AASHTO-PCI-ASBI SECTION TYPE 2700-2.
43'-0"
9'-0"
1'-4"
1'-6"
5'-0"
3'-0"
1'-0"
5'-0"
8'-2"
8'-2"
16'-4"
9"
9"
A
A
OPENING
2.5
1
SECTION A-A
9'-0"
5'-0"
9"
5'-6"
9"
2'-0"
2'-0"
9"



NOTE:
AASHTO-PCI-ASBI SECTION TYPE 2700-2.

FIGURE 3-5

EXPANSION JOINT SEGMENT

BALANCED CANTILEVER CONSTRUCTION

DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS

$$$$$$$DGNSPECIFICATION$$$$$$$ $$$$$$SYTIME$$$$$$

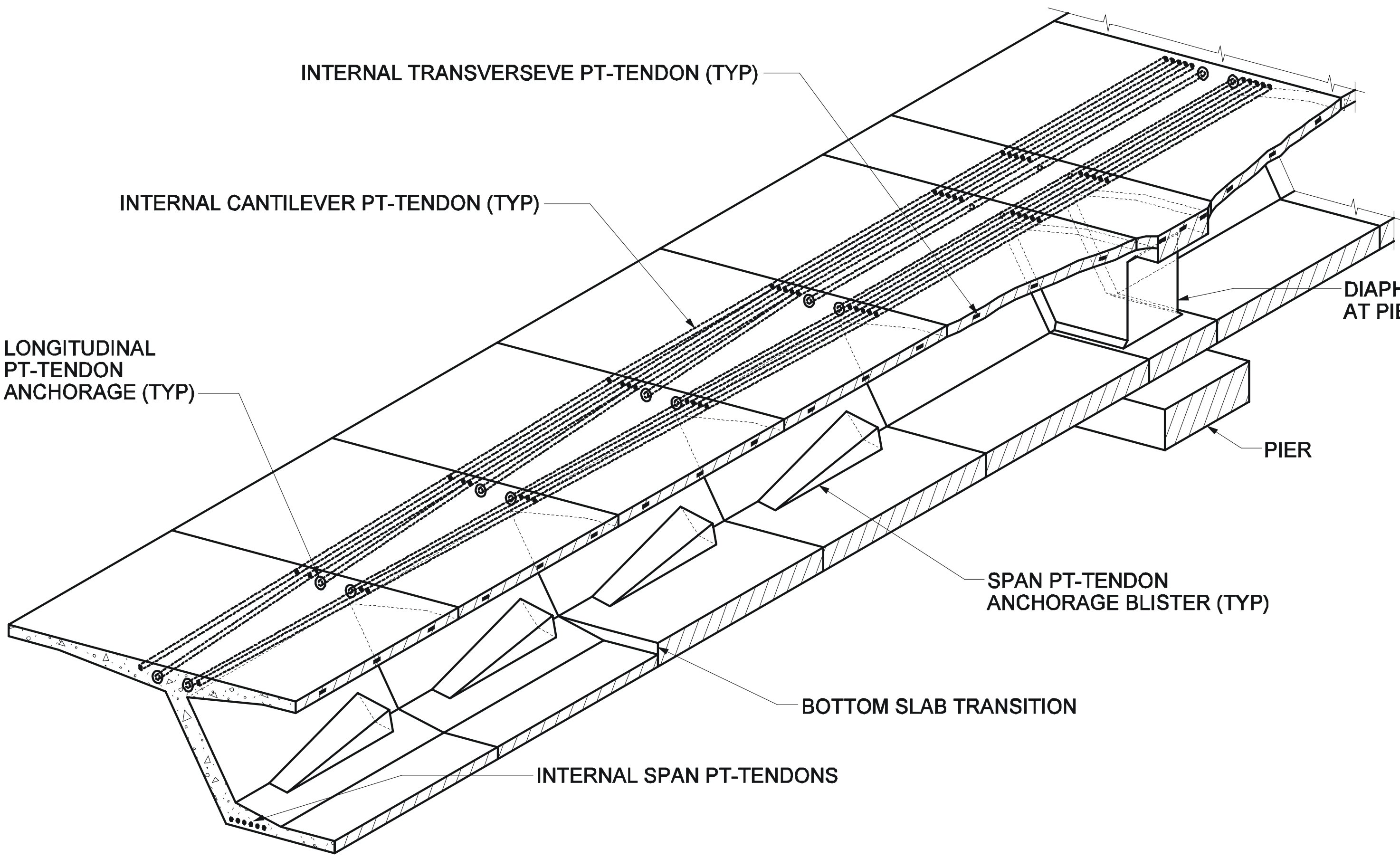
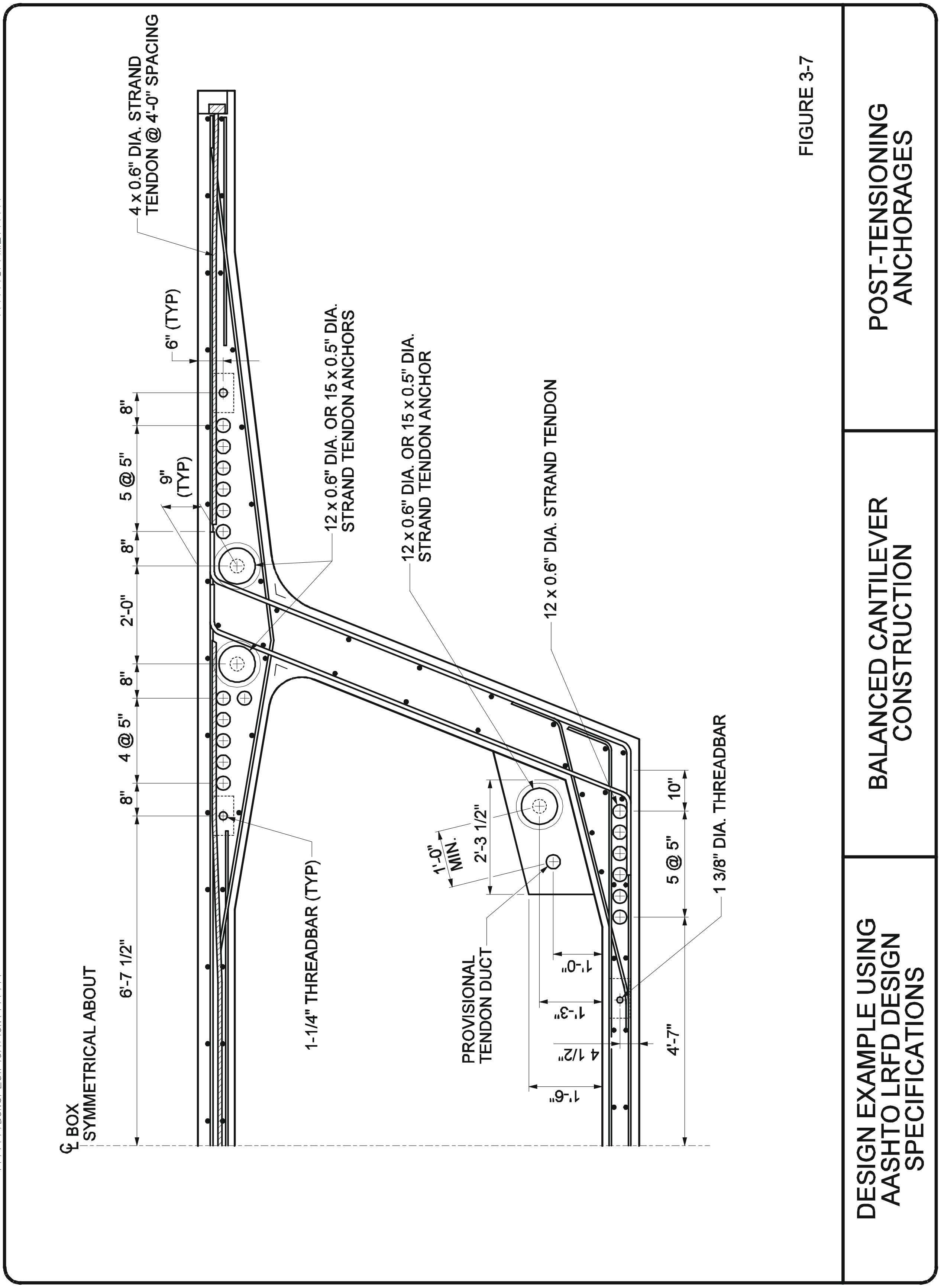


FIGURE 3-6

DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS	BALANCED CANTILEVER CONSTRUCTION	ISOMETRIC CROSS-SECTION AT TYPICAL PIER


4 x 0.6" DIA. STRAND TENDON @ 4'-0" SPACING
6" (TYP)
8"
5 @ 5"
9" (TYP)
8"
2'-0"
8"
4 @ 5"
8"
6'-7 1/2"
℄ BOX SYMMETRICAL ABOUT
12 x 0.6" DIA. OR 15 x 0.5" DIA. STRAND TENDON ANCHORS
12 x 0.6" DIA. OR 15 x 0.5" DIA. STRAND TENDON ANCHOR
12 x 0.6" DIA. STRAND TENDON
1-1/4" THREADBAR (TYP)
1'-0" MIN.
2'-3 1/2"
PROVISIONAL TENDON DUCT
1'-0"
1'-3"
4 1/2"
1'-6"
10"
5 @ 5"
4'-7"
1 3/8" DIA. THREADBAR
FIGURE 3-7
POST-TENSIONING ANCHORAGES
BALANCED CANTILEVER CONSTRUCTION
DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS

4. ERECTION SCHEME

The structure is erected using the precast balanced cantilever method of construction, where individual segments are placed successively on alternating sides of the cantilever. A segment is attached at either end of the cantilever by use of temporary post-tensioning bars after epoxy has been applied to the interface with the previously erected segment. In this example, temporary post-tensioning bars will be left in the segments and grouted afterward. Temporary post-tensioned bars may also be re-used. Cantilever tendons are then stressed, and the process is repeated for the entire cantilever.

The following erection stages were used for this example:

Stage	Day	Description
1.	180	Erect cantilever at pier 2 and stress cantilever tendons
2.	180	Erect span 1 segments on falsework, cast CIP closure, and stress span top and bottom tendons.
3.	200	Erect cantilever at pier 3 and stress cantilever tendons
4.	200	Cast span 2 closure, and stress span top and bottom tendons.
5.	220	Erect cantilever at pier 4 and stress cantilever tendons
6.	220	Cast span 3 closure, and stress span top and bottom tendons.
7.	240	Erect cantilever at pier 5 and stress cantilever tendons
8.	240	Cast span 4 closure, and stress span top and bottom tendons.
9.	250	Erect span 5 segments on falsework, cast CIP closure, and stress span top and bottom tendons
10.	300	Cast barriers, Install expansion joints, and place overlay if applicable
11.	350	Open bridge to traffic (End of Construction)
12.	500	Total forces and deformations after creep and shrinkage at day 500
13.	1000	Total forces and deformations after creep and shrinkage at day 1000
14.	2000	Total forces and deformations after creep and shrinkage at day 2000
15.	4000	Total forces and deformations after creep and shrinkage at day 4000
16.	10000	Total forces and deformations after creep and shrinkage at day 10000

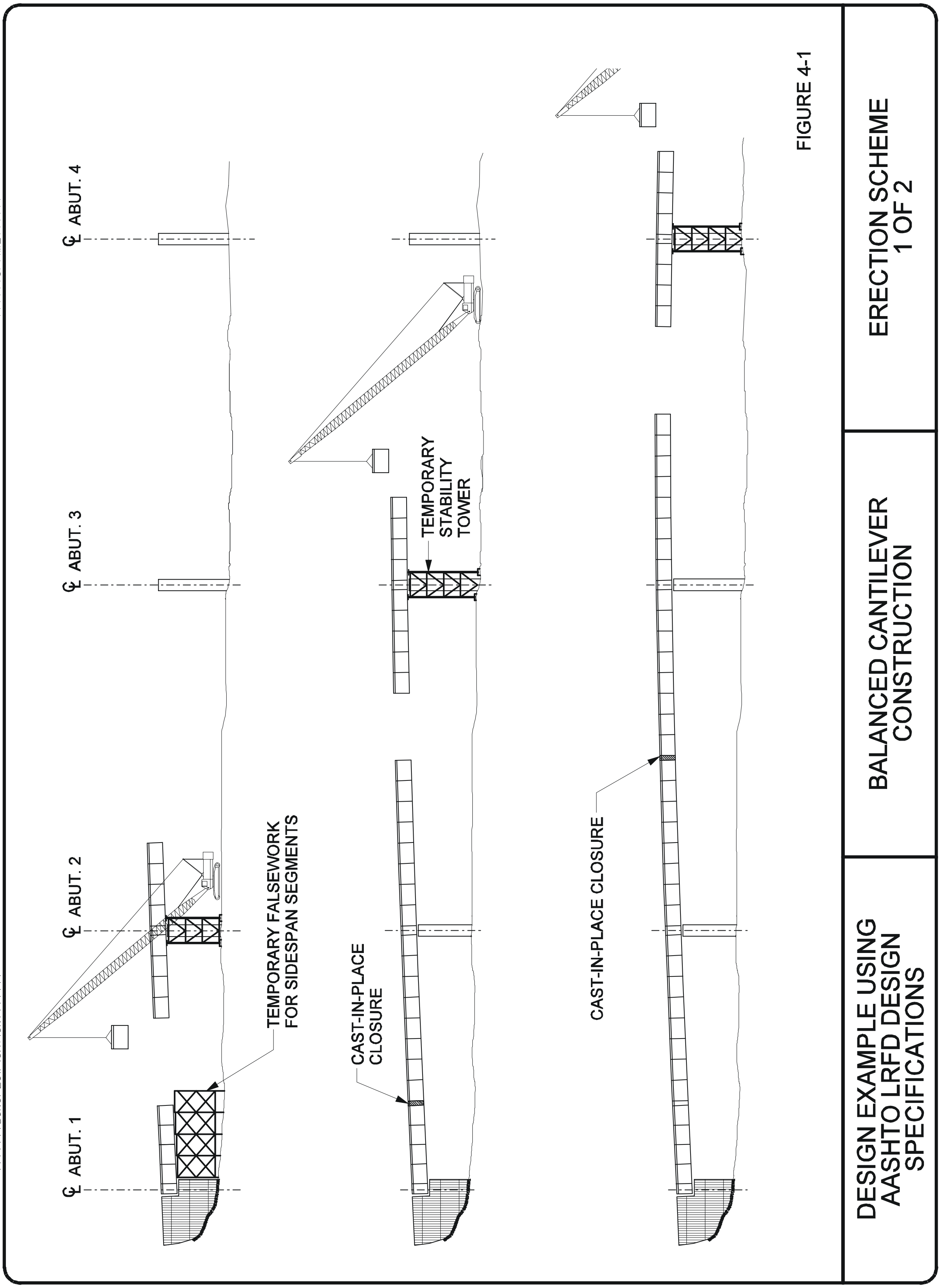
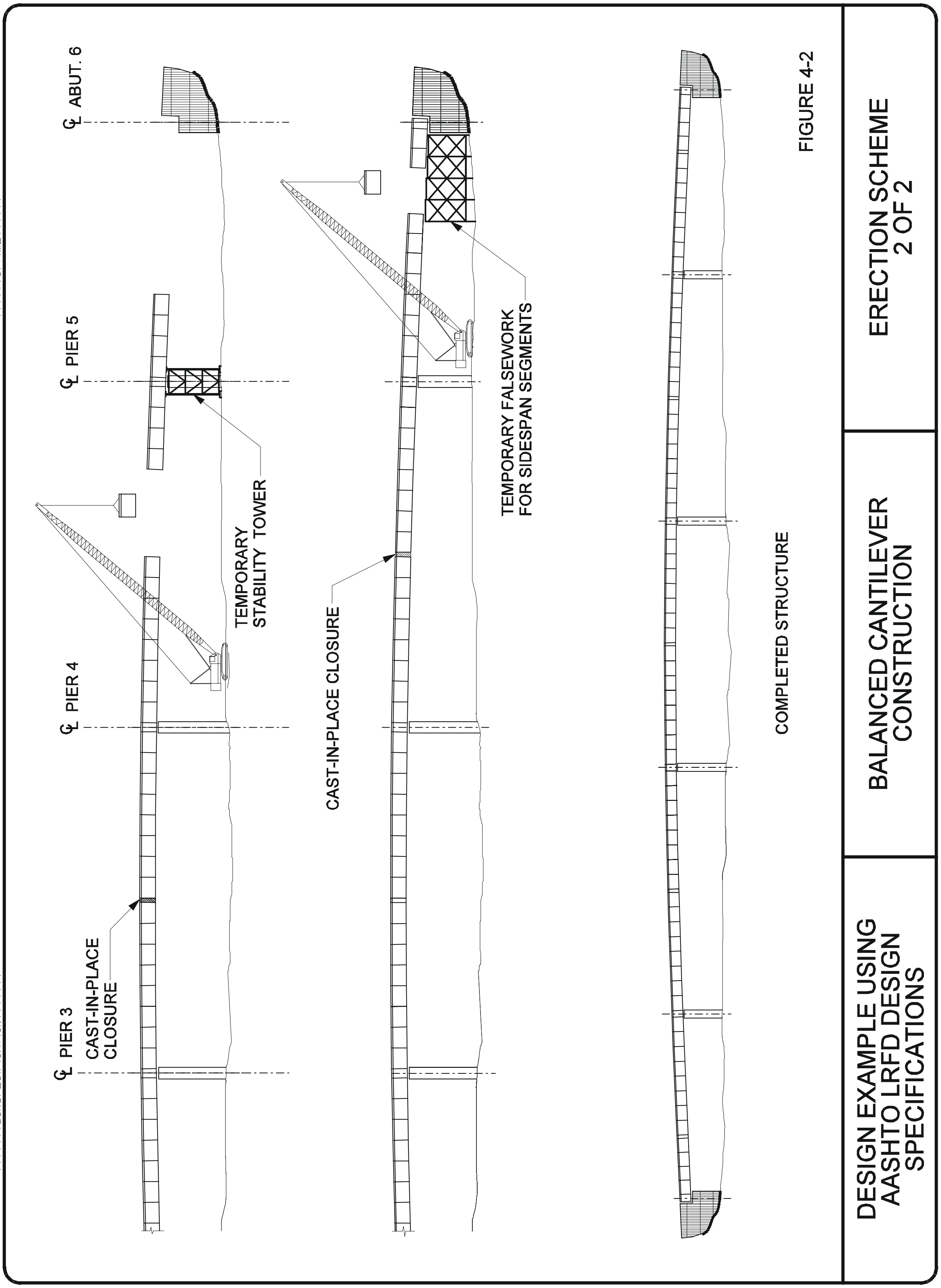

℄ ABUT. 1
℄ ABUT. 2
℄ ABUT. 3
℄ ABUT. 4
TEMPORARY FALSEWORK FOR SIDESPAN SEGMENTS
CAST-IN-PLACE CLOSURE
TEMPORARY STABILITY TOWER
CAST-IN-PLACE CLOSURE
DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS
BALANCED CANTILEVER CONSTRUCTION
ERECTION SCHEME 1 OF 2

FIGURE 4-1


$$$$$$$DGNSPECIFICATION$$$$$$$$
$$$$$$$SYTIME$$$$$$$
℄ PIER 3
CAST-IN-PLACE
CLOSURE
℄ PIER 4
TEMPORARY
STABILITY TOWER
℄ PIER 5
℄ ABUT. 6
CAST-IN-PLACE CLOSURE
TEMPORARY FALSEWORK
FOR SIDESPAN SEGMENTS
COMPLETED STRUCTURE
FIGURE 4-2
DESIGN EXAMPLE USING
AASHTO LRFD DESIGN
SPECIFICATIONS
BALANCED CANTILEVER
CONSTRUCTION
ERECTION SCHEME
2 OF 2

ch5. DECK DESIGN

5.1 Introduction

The top deck of a box girder is subjected to complex external forces, static and dynamic loads, thermal gradients, and creep and shrinkage effects. Proper consideration should be given to these effects to prevent cracking and deterioration. De-icing chemicals and freeze-thaw action should also be considered in design to counteract degradation.

Deck replacement is not only costly, but results in inconvenience to the traveling public. For segmental bridge superstructures, deck replacement is not practical and almost impossible to do without closing the entire bridge. Therefore, when designing decks for segmental bridges, it is always good strategy to be conservative and allow for reserved capacity.

Studies have shown that transverse post-tensioning of top decks improves long-term deck durability and results in low life cycle cost (See Reference 12). It is recommended that for all post-tensioned box girders the top deck be transversely post-tensioned, even for short overhangs. For bridges not subjected to freeze-thaw action and de-icing chemicals, at least the deck should be partially prestressed. The top deck should be designed using elastic methods and then checked for ultimate limit states, not the other way around.

In general it is standard practice to select a minimum top deck thickness of eight inches, although AASHTO-PCI-ASBI Standard Sections Committee recommends a minimum deck thickness of nine inches.

5.2 Design Approach

To correctly represent the final system of the box girder, one would need to do a three dimensional analysis and incorporate all loads the box is subjected to along with proper boundary conditions. Due to complexity of this type of analysis, in particular the application of prestressing to three dimensional systems, this is seldom done. In lieu of this complex analysis, it is common practice to model the box as a 2-D (two dimensional) plane frame of unit length, as shown in Figure 5.2-1. If the thicknesses of the web and bottom slab vary along the length of the bridge, several 2-D frames may have to be analyzed in order to obtain a more representative interpretation of these varying cross-sectional properties. The 2-D frame model allows for load distribution to the webs and slab members relative to their stiffness.

A typical 2-D frame model is assumed to be supported at the lower end of the webs as shown in Figure 5.2-1. While it could be argued that different boundary conditions exist for this model, this simplified assumption produces reasonable results.
The design loads considered in transverse design include, but are not limited to:

DC = Dead load of structural components and non-structural components, such as traffic barrier wall

DW = Dead load of wearing surface or future wearing surface and utilities if any

LL = Live load

IM = Dynamic load allowance

PT = Primary prestressing forces

EL = Accumulated locked-in force effects resulting from the construction process, including the secondary forces from post-tensioning

TG = Thermal gradient (+/- 10^{o}Fdifferential between the inside and outside of box girder) Note: currently not required by AASHTO LRFD Design Specifications, but commonly done in standard practice

CR = Creep effect of concrete

SH = Shrinkage effect of concrete

Secondary forces of post-tensioning shall be included in ultimate limit state load combinations with a load factor of 1.0.

In addition to service and strength limit state load combinations, the deck design should be checked for construction load combinations, such as segment lifting, construction equipment, and segment stacking (see LRFD Article 5.14.2).

06/25/2004

F:\$SEGMENTAL\LRFD Design Example\Fig 5-1 Plane Frame.dgn

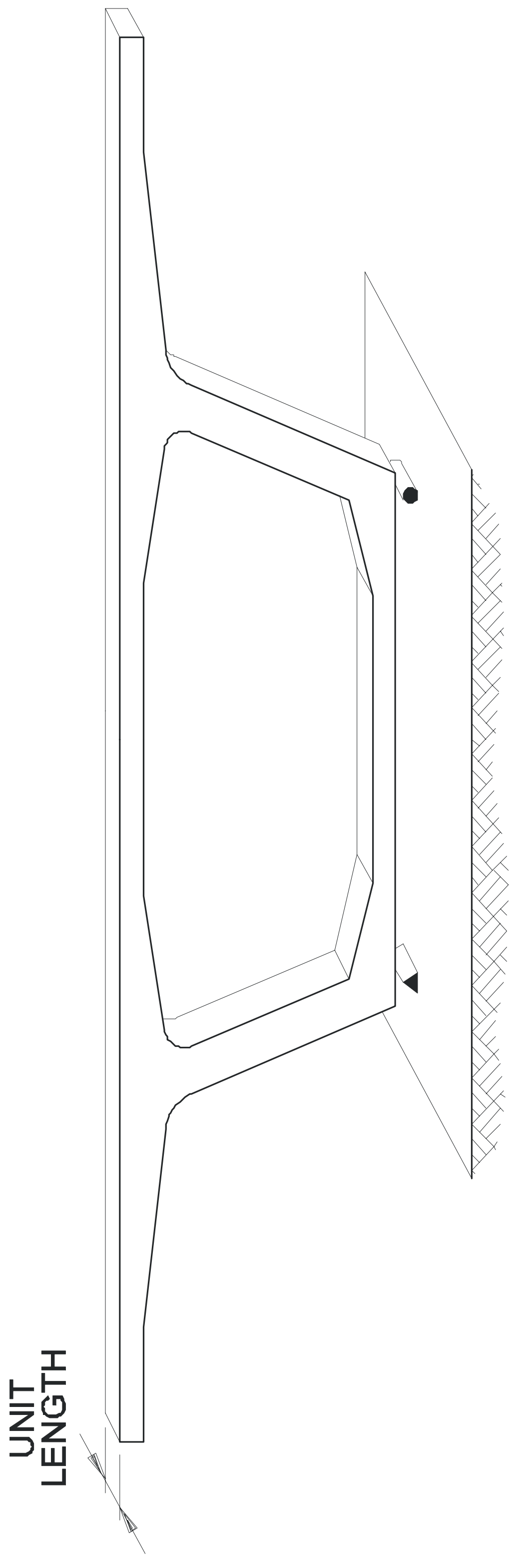


FIG. 5.2-1

DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS

BALANCED CANTILEVER CONSTRUCTION

SIMPLIFIED TWO-DIMENSIONAL PLANE FRAME OF UNIT LENGTH

5.3 Live Load Analysis

When a static concentrated load is applied on a deck, the deck will deflect transversely as well as longitudinally, similar to a two-way slab. The load distribution becomes more complex when multiple point loads are applied to the deck, such as a truck load. Since the structural model is simplified to a 2-D frame model, as stated in Section 5.2, it is important to obtain the resulting 3-D forces to the 2-D model.

Commonly, there are two ways of handling live load distributions in the transverse direction:

1. In the past, influence surfaces from Pucher or Homberg Charts have been extensively used in box girder transverse design. These charts are based on elastic theory of plates (homogeneous and isotropic). Some charts are valid for constant depth plate thickness and some for variable depth plate thickness with a parabolic soffit. Depending on the boundary conditions of the selected plate, the dimensionless charts provide bending moments per unit length at the fixed end and mid span only. The Fixed End Moments (FEM) are then applied as external forces to the 2D frame. The bending moments between supports are approximated by interpolation. The method has limitations for haunched deck slabs, regarding the support depth over mid span depth ratio. This method is approximate and can be useful for preliminary design.

2. A more accurate method is based on a partial 3-D (three dimensional) finite element model of the box girder. The term “partial” implies that the entire bridge superstructure need not be modeled; rather it should be interpreted as a partial length of the box that will be long enough to include three dimensional effects. From this model, influence lines can be generated at any section of interest. The influence lines should be generated using a line load consisting of front and rear wheels of a design truck. Since general finite element programs are readily available presently, it is recommended that this method be used for final design.

   It should be noted that theoretically, a continuous vehicle barrier could be incorporated into this model to further distribute live load longitudinally. However, due to discontinuities of the barrier and uncertain future quality, this edge stiffening effect is neglected and not recommended.

In this design example, the second method was implemented for analysis. Keep in mind the live load configuration should be strategically placed in order to produce the worst condition (see Figures 5.3-1 to 5.3-3). Listed below are some common points where stresses are checked:

- Maximum negative bending moment at the root of deck overhang
- Maximum positive and negative bending moments at the center line between two webs
- Maximum negative bending moment in the top deck at the interior face of the webs

- Maximum negative and positive bending moments in the webs and bottom slab
- Maximum negative moment in the deck overhang where the taper begins

See Figures 5.3-4 to 5.3-8 for influence lines corresponding to these locations.

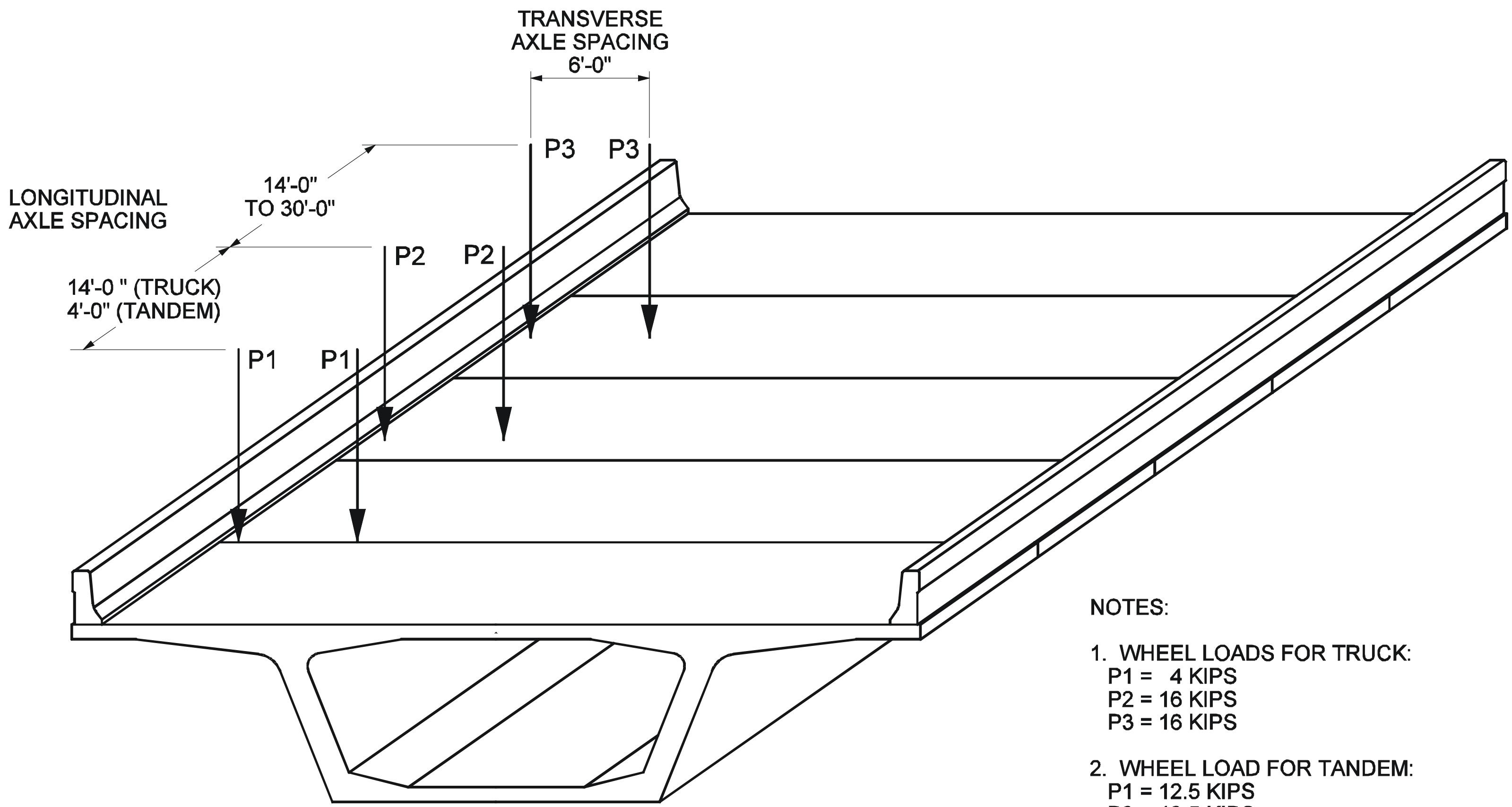
In the *AASHTO Standard Specifications* (LFD), only the effect of a design truck (or tandem) is to be considered for transverse design. However, the current 3rd Edition of LRFD requires the design truck and lane load to be combined to achieve maximum effects. In combination with this, if one truck controls, a multi-lane increase of 1.2 is to be applied. Due to these new requirements, LRFD will produce more conservative results when compared to the Standard Specification. Although impact and multi-lane factors have not been included, a live load moment envelope is given in Figure 5.3-9 to show the difference in codes.

In recent AASHTO T-5 and T-10 Committee meetings held in Orlando in June, 2004, revisions have been proposed for transverse deck design. In particular, the elimination of multi-lane factors, Service Limit State III, as well as lane load elimination have been proposed. Hence, only the design truck (or tandem) will be used to calculate maximum effects. Service Limit State III with a factor of 0.8 for live load will no longer be used for transverse deck design. Rather, it will be eliminated and Service Limit State I with a live load factor of 1.0 for both tension and compression will be checked. These revisions will produce results similar to that of the Standard Specifications and also have positive impacts on ultimate limit states.

For this design example, all limit states have been checked incorporating the proposed T-5 and T-10 Committee revisions for transverse deck design.

Please note that although the above deck design revisions have been proposed and approved by AASHTO T-5 and T-10 committees, they cannot be adopted until they are officially published in the *AASHTO LRFD Bridge Design Specifications, 3rd Edition* Interim Specifications.

F:$SEGMENTAL LRFD Design Example Fig 5-2-I LRFD LL Distribution for Transverse Design.dgn 06/25/2004



DISTRIBUTION OF LRFD LIVE LOAD
(EXAMPLE FOR TRANSVERSE DESIGN
AT ROOT OF OVERHANG)

NOTES:

1. WHEEL LOADS FOR TRUCK:
   P1 = 4 KIPS
   P2 = 16 KIPS
   P3 = 16 KIPS

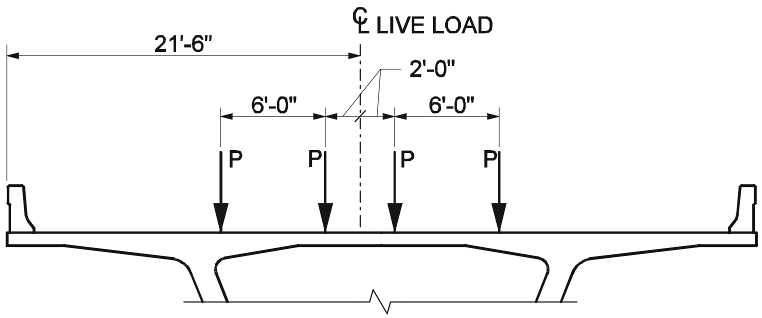
2. WHEEL LOAD FOR TANDEM:
   P1 = 12.5 KIPS
   P2 = 12.5 KIPS
   P3 = N/A

3. IN THIS DESIGN EXAMPLE, UNIFORM LANE LOAD IS ELIMINATED PER AASHTO T-5 AND T-10 PROPOSAL

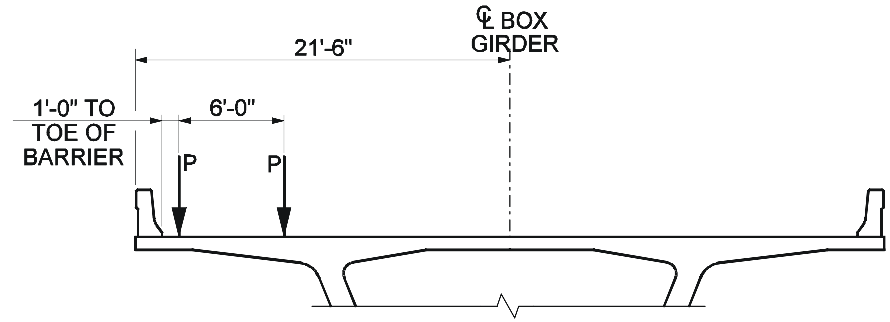
FIG. 5.3-1

DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS	BALANCED CANTILEVER CONSTRUCTION	LRFD LIVE LOAD DISTRIBUTION FOR TRANSVERSE DESIGN

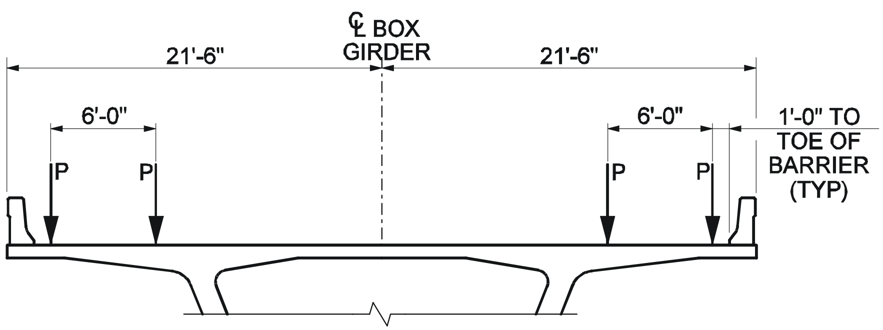
F:\$SEGMENTAL\LRFD Design Example\Fig 5-2-2 Transverse Configuration of LRFD Live Load 1.dgn 06/25/2004



LIVE LOAD CONFIGURATION FOR
MAXIMUM POSITIVE MOMENT AT ℄ SLAB



LIVE LOAD CONFIGURATION FOR
MAXIMUM NEGATIVE MOMENT AT OVERHANG



LIVE LOAD CONFIGURATION FOR
MAXIMUM NEGATIVE MOMENT AT ℄ SLAB

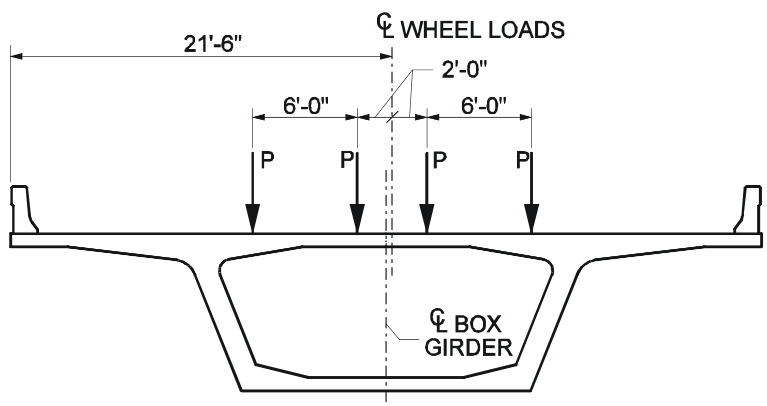
NOTES:

1. WHEEL LOADS FOR TRUCK:
   P = 4 OR 16 KIPS
2. WHEEL LOADS FOR TANDEM:
   P = 12.5 KIPS
3. IN THIS DESIGN EXAMPLE, UNIFORM LANE LOAD IS ELIMINATED PER AASHTO T-5 AND T-10 PROPOSAL

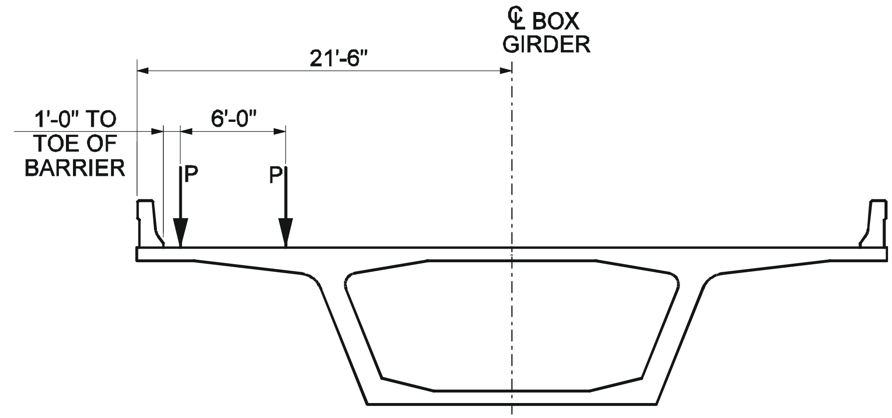
FIG. 5.3-2

DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS	BALANCED CANTILEVER CONSTRUCTION	TRANSVERSE CONFIGURATION OF LRFD LIVE LOAD 1 OF 2

F:\$SEGMENTAL\LRFD Design Example\Fig 5-2-3 Transverse Configuration of LRFD Live Load 2.dgn 06/25/2004



LIVE LOAD CONFIGURATION FOR MAXIMUM NEGATIVE MOMENT IN TOP SLAB AT INSIDE FACE OF WEB



LIVE LOAD CONFIGURATION FOR MAXIMUM NEGATIVE MOMENT IN BOTTOM SLAB AT INSIDE FACE OF WEB

NOTES:

1. WHEEL LOADS FOR TRUCK:
   P = 4 OR 16 KIPS
2. WHEEL LOADS FOR TANDEM:
   P = 12.5 KIPS
3. IN THIS DESIGN EXAMPLE, UNIFORM LANE LOAD IS ELIMINATED PER AASHTO T-5 AND T-10 PROPOSAL

FIG. 5.3-3

DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS	BALANCED CANTILEVER CONSTRUCTION	TRANSVERSE CONFIGURATION OF LRFD LIVE LOAD 2 OF 2

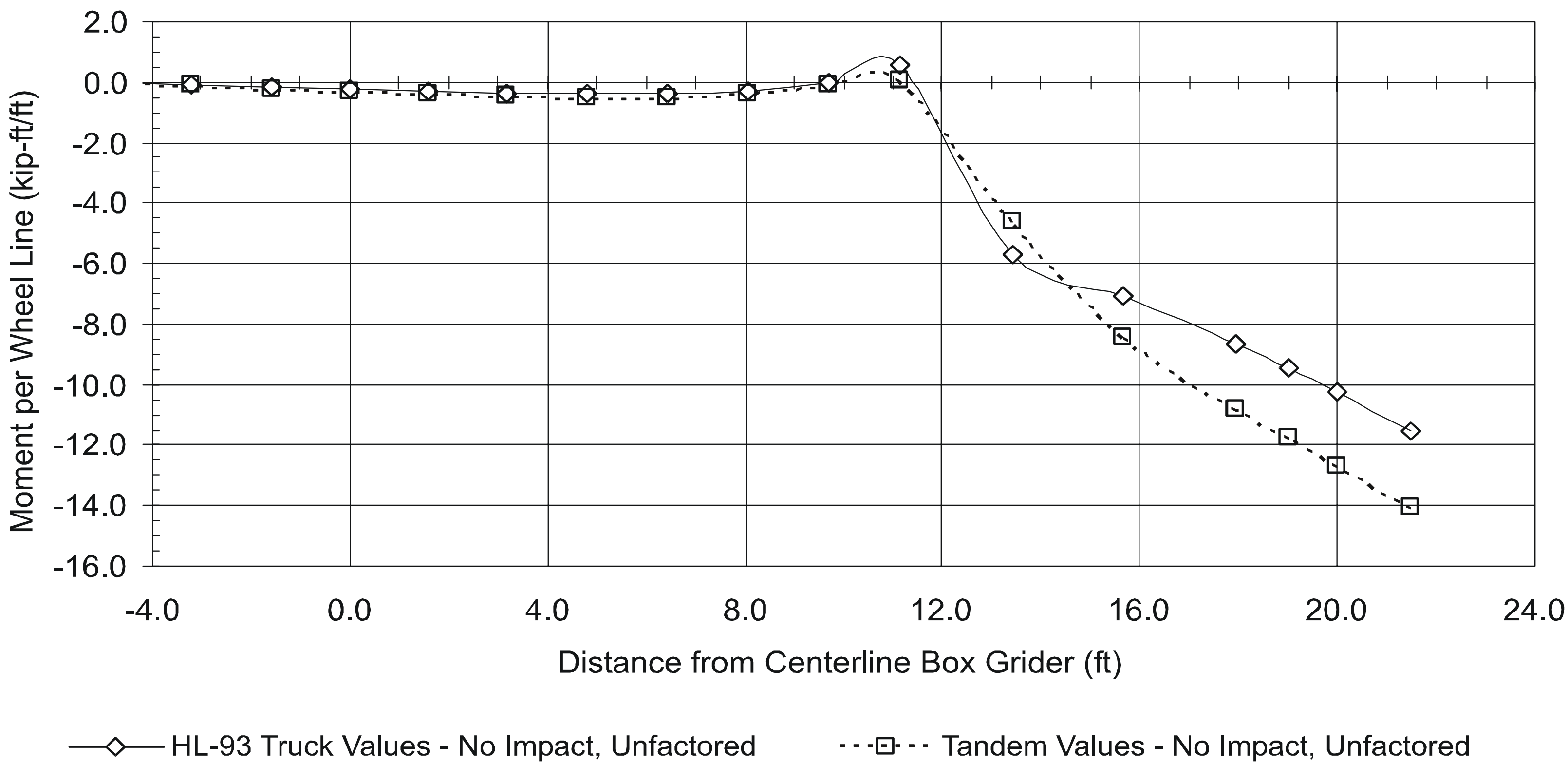


FIG. 5.3-4

DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS	BALANCED CANTILEVER CONSTRUCTION	LIVE LOAD INFLUENCE LINE FOR MOMENT: ROOT OF OUTSIDE WING

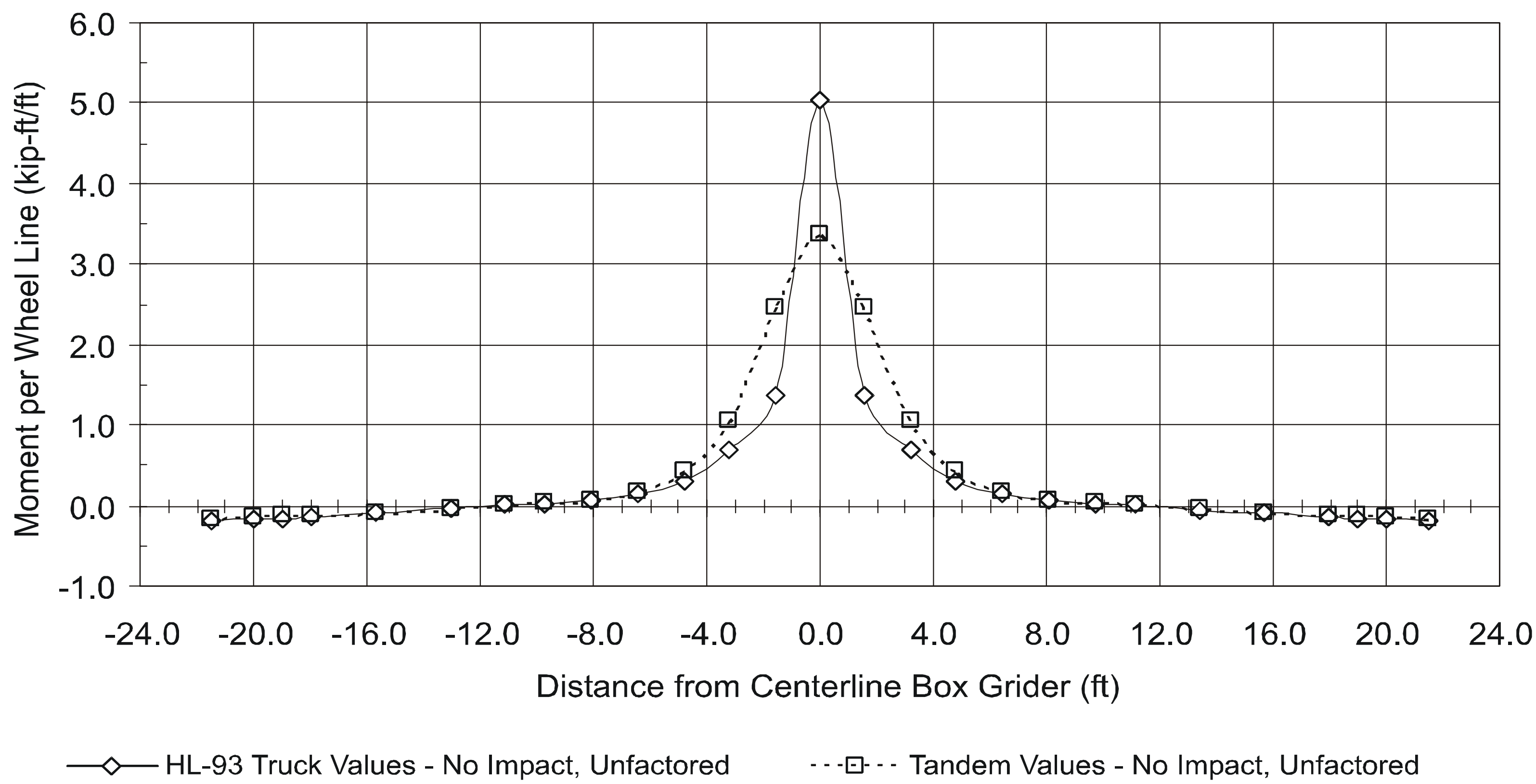


FIG. 5.3-5

DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS	BALANCED CANTILEVER CONSTRUCTION	LIVE LOAD INFLUENCE LINE FOR MOMENT: CENTERLINE BOX

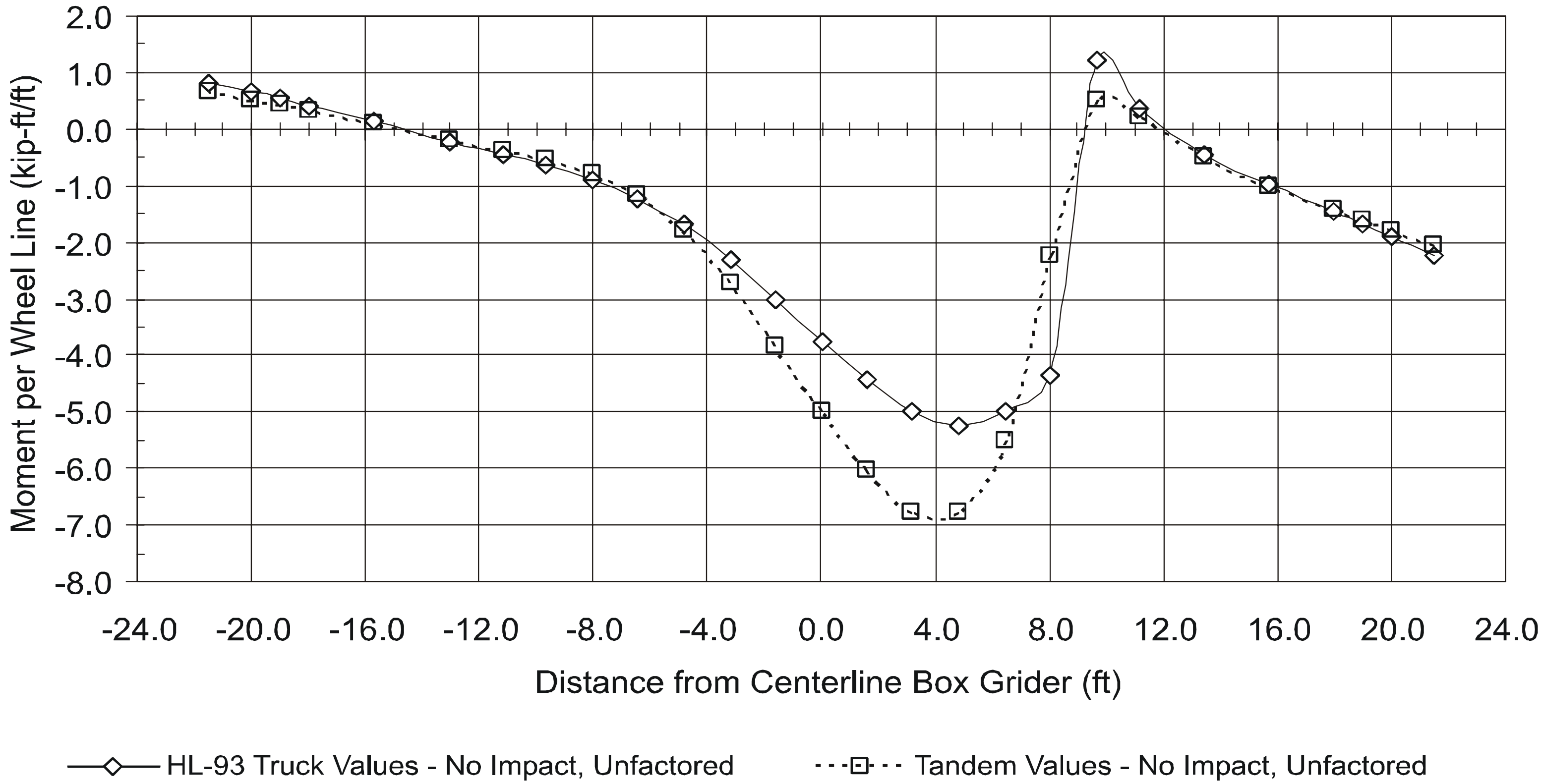


FIG. 5.3-6

DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS	BALANCED CANTILEVER CONSTRUCTION	LIVE LOAD INFLUENCE LINE FOR MOMENT: INSIDE FACE OF WEB

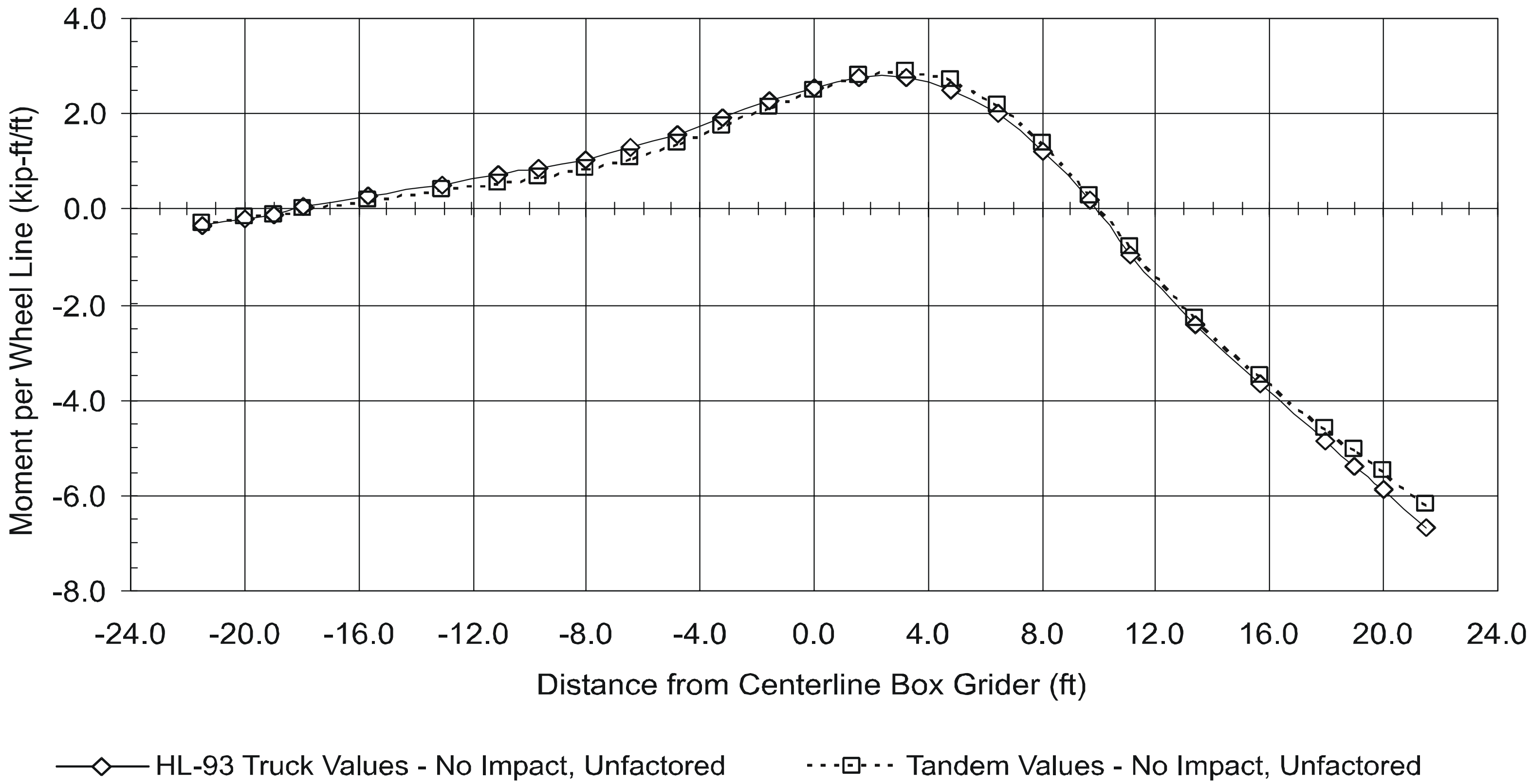


FIG. 5.3-7

DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS	BALANCED CANTILEVER CONSTRUCTION	LIVE LOAD INFLUENCE LINE FOR MOMENT: TOP OF WEB

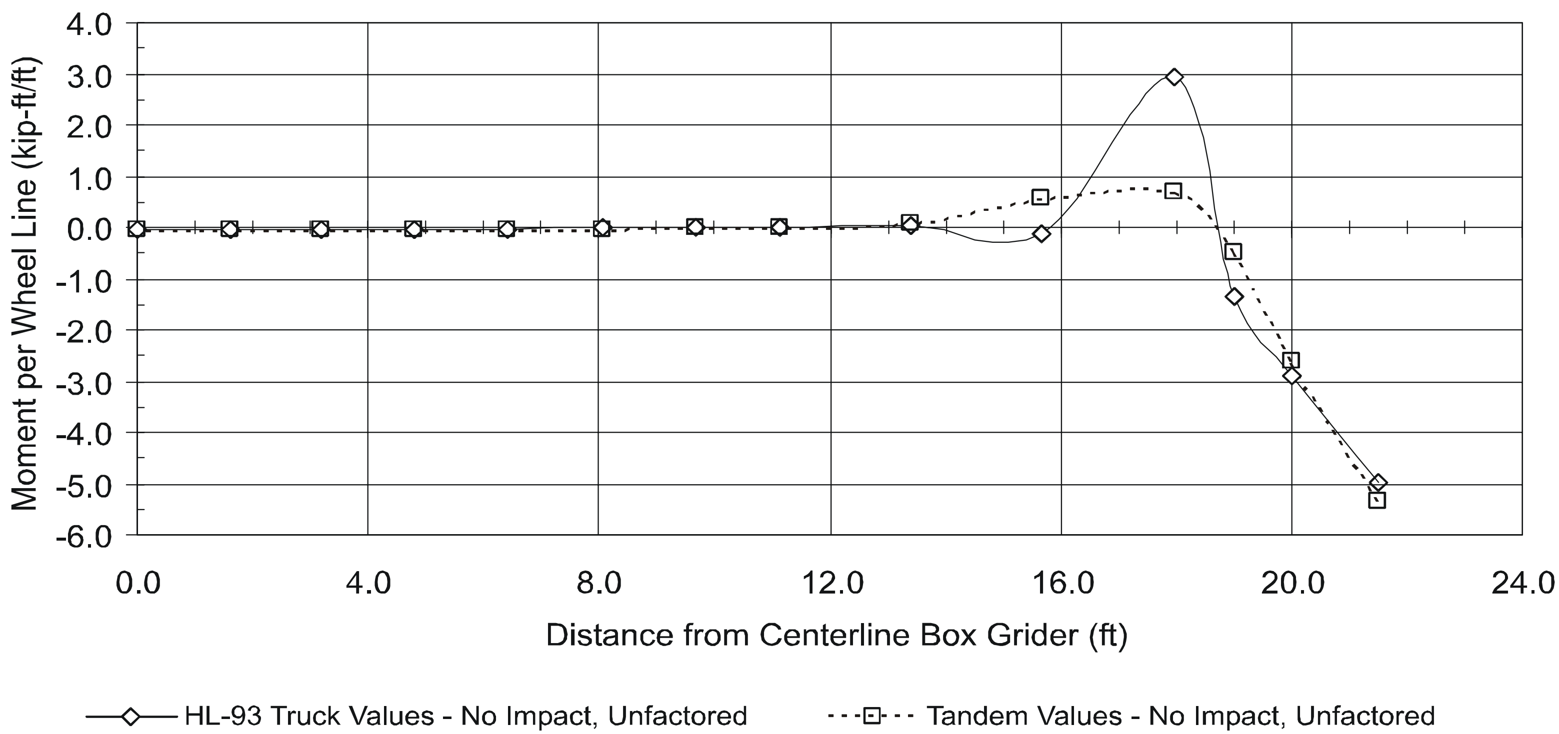


FIG. 5.3-8

DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS	BALANCED CANTILEVER CONSTRUCTION	LIVE LOAD INFLUENCE LINE FOR MOMENT: OUTSIDE WING AT THICKNESS TRANSITION

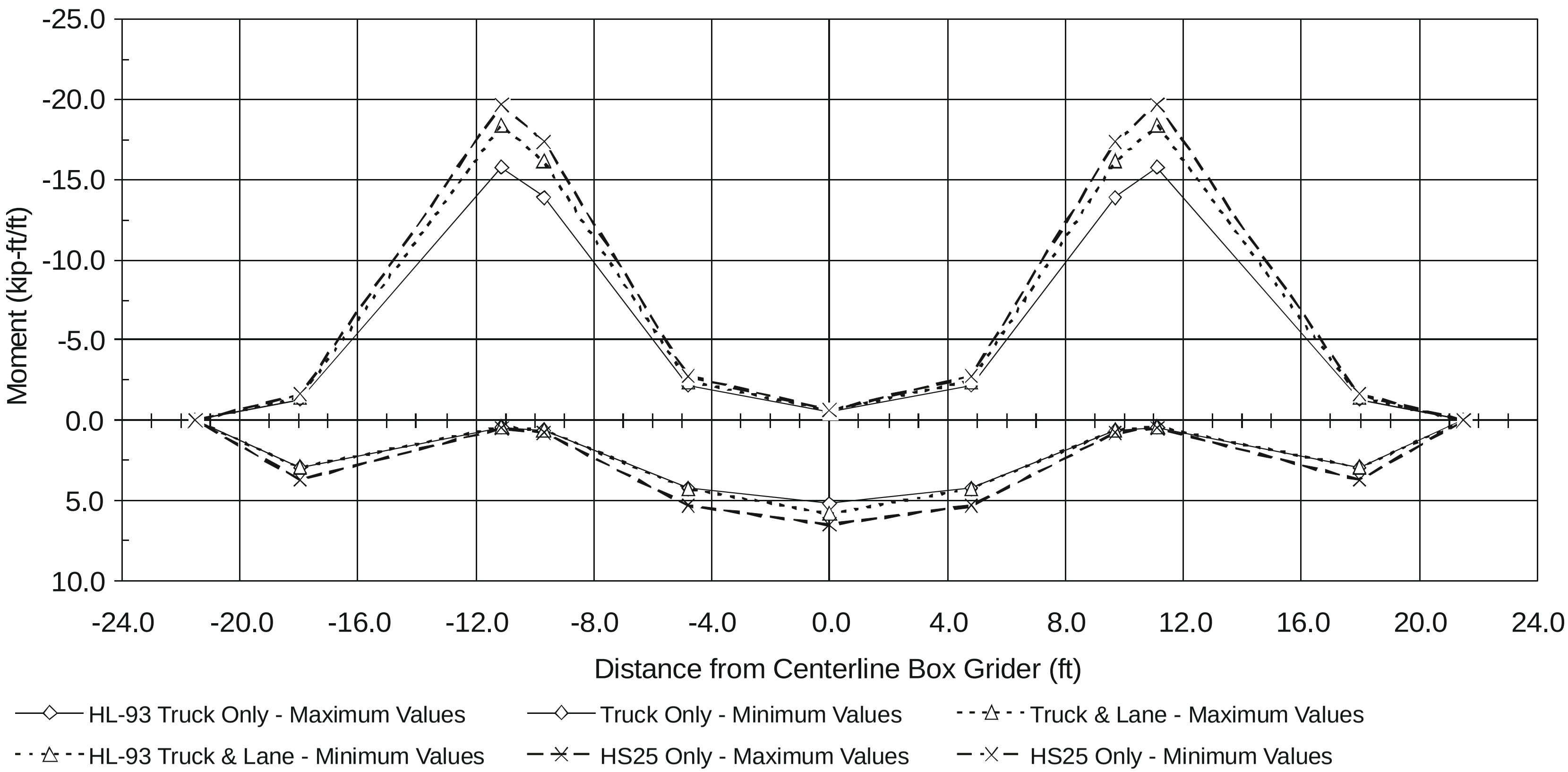


FIG. 5.3-9

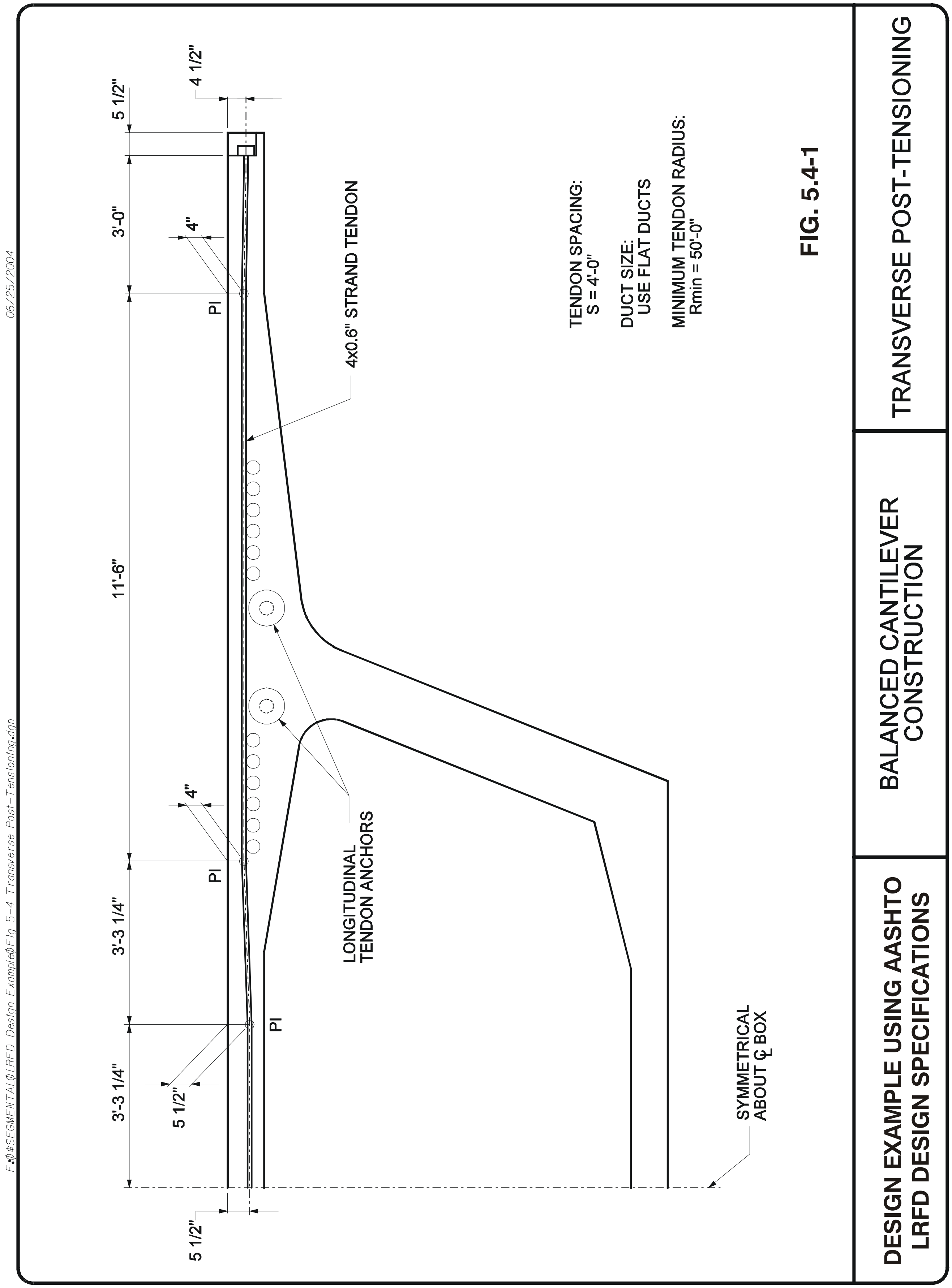
DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS	BALANCED CANTILEVER CONSTRUCTION	TRANSVERSE DECK LIVE LOAD MOMENT ENVELOPE (UNFACTORED W/O IMPACT)

5.4 Post-Tensioning Tendon Layout

Post-tensioning in the transverse direction typically consists of three to four 0.5” or 0.6” diameter strands per tendon passing through the top slab and anchored at the face of the overhang on each side of the box girder. These tendons are usually housed in flat ducts due to the thin top slab. To efficiently utilize the tendon, it should be suitably profiled for maximum structural efficiency.

A typical tendon is generally anchored at mid-height of the slab at wing tips and then gradually rises to a level above the neutral axis of the deck over the webs. This helps the tendon resist the negative moments at the webs. The tendon then gradually drops to a level below the neutral axis of the top slab near the centerline of the box girder in order to resist the positive bending in that region. The tendon path used for this example is shown in Figure 5.4-1.

Longitudinally, the tendon spacing is determined using the appropriate service and strength limit state checks. The maximum spacing of tendons is typically restricted to 4 feet in effort to limit shear lag effects between anchorages. If maximum tendon spacing is not addressed, zones near outside edges of the slab may be without effective prestressing.


06/25/2004
F:\0$SEGMENTAL\0LRFD Design Example\0Fig 5-4 Transverse Post-Tensioning.dgn
5 1/2"
4 1/2"
3'-0"
4"
PI
11'-6"
4"
PI
3'-3 1/4"
PI
3'-3 1/4"
5 1/2"
5 1/2"
4x0.6" STRAND TENDON
LONGITUDINAL TENDON ANCHORS
SYMMETRICAL ABOUT ℄ BOX
TENDON SPACING:
S = 4'-0"
DUCT SIZE:
USE FLAT DUCTS
MINIMUM TENDON RADIUS:
Rmin = 50'-0"
FIG. 5.4-1
TRANSVERSE POST-TENSIONING
BALANCED CANTILEVER CONSTRUCTION
DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS

5.5 Summary of Design Forces

The design forces obtained from the two-dimensional frame analysis and three-dimensional live load influence lines are combined in a spreadsheet using the LRFD Service Limit State and Strength Limit State combinations. The maximum tensile and compressive stresses at each predetermined section in the top slab are summarized and compared to the LRFD allowable stresses. In this example, the prestressing force is estimated in preliminary hand calculations, and then analyzed in a 2-D time dependent run using the BDAC program. All other loads are incorporated into the 2-D model, except live loads. The results are then compiled in a spreadsheet to check stresses. By varying the prestressing force, the combined stresses of service limit states are calculated. Using the selected tendon forces per unit length, the size and spacing of transverse tendons in the segment are determined.

The LRFD Strength Limit States are also tabulated in a spreadsheet and an envelope of maximum and minimum values is determined for each chosen section. The values in this moment envelope can then be compared to the calculated bending capacities for each of the corresponding transverse components.

5.6 Service Limit State Design

As stated in Section 5.3, only Service Limit State I will be checked with a live load factor of 1.0 for tension as well as compression. Also, a linear temperature gradient of 10 degrees Fahrenheit between interior and exterior surfaces of the box will be used in Service Limit State I. The current LRFD specification does not specify this loading, leaving it up to the owner or designer to establish if it should be included on a project-by-project basis. This example is based on a load factor of 0.5 for transverse temperature gradient when accompanying live load. Also, in addition to Service Limit State I, LRFD requires a check for service load stresses due to dead load and full temperature gradient. This limit state can often govern at locations where live load influences are small.

To show a comparison of the new proposed Service Limit State I verses the current LRFD Service Limit State III and Standard Specification, a graph of stresses is given in Figure 5.6-1. Since the box is symmetrical, minimum and maximum stresses for the top of the deck have been shown on one side and bottom deck stresses on the other. After examination of this figure, it can be seen that stresses resulting from the Committee T-5 and T-10 proposal closely follow those from the Standard Specification. The slight difference is due to the 1.33 impact factor from LRFD compared to 1.3 for the Standard Specification. It can also be seen that the stresses produced from the current LRFD specification are similar to those produced from a Standard Specification HS25 loading.

In addition to service limit states under maximum loading, temporary stresses such as those prior to barrier placement and vehicular traffic should be checked to ensure allowable stresses are not exceeded during the construction process.

Listed below are service load combinations used in this example:

<u>Service I (Tension & Compression)</u>

1.0(DC + DW + EL) + 1.0(PT) + 1.0(LL + IM) + 1.0(CR + SH) +/- 0.5(TG)

<u>Segmental Load Combination (LRFD Equation 3.4.1-2)</u>

1.0(DC + DW + EL) + 1.0(PT) + 1.0(CR + SH) +/- 1.0(TG)

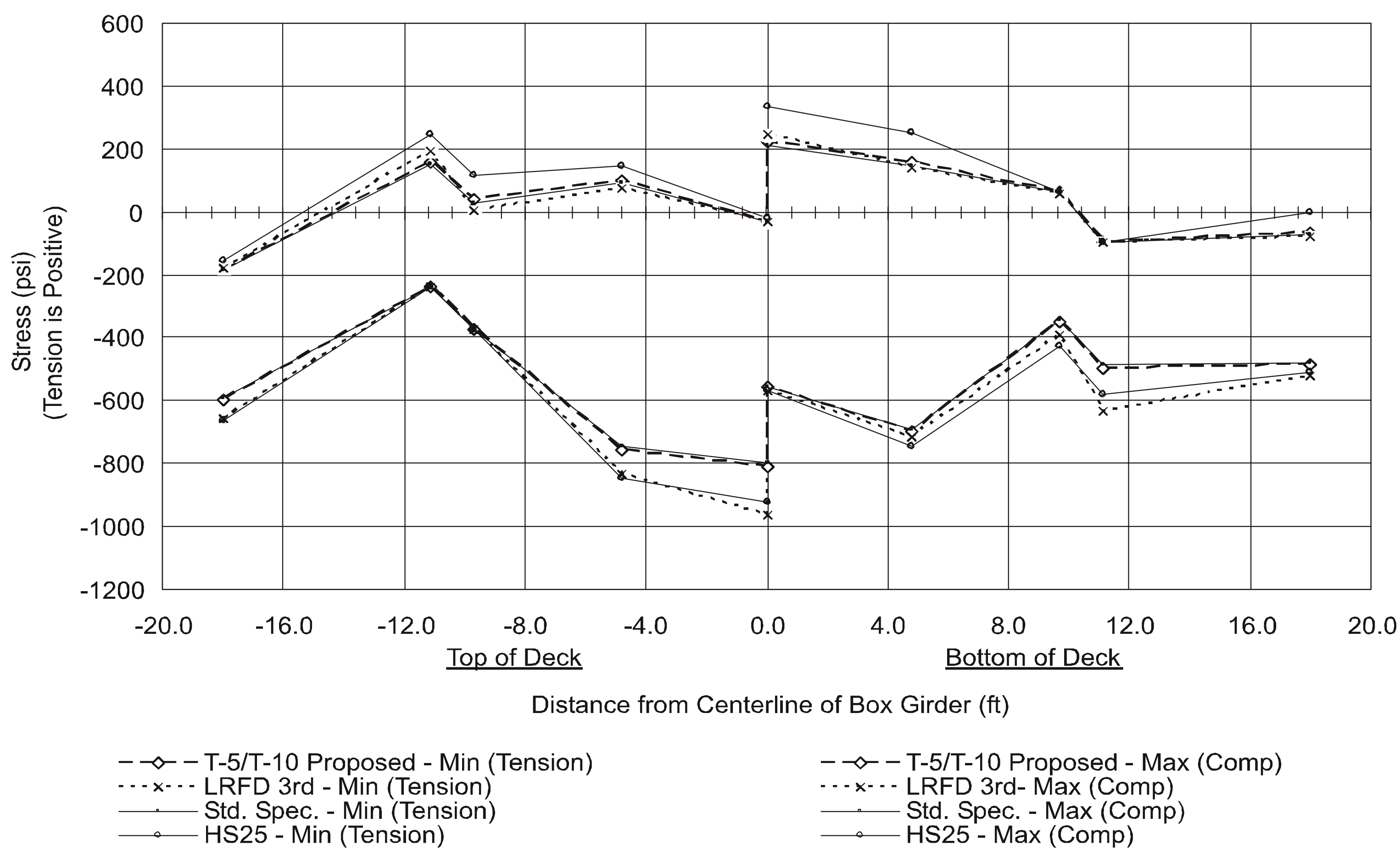


FIG. 5.6-1

DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS	BALANCED CANTILEVER CONSTRUCTION	CONCRETE SERVICE STRESSES: CODE COMPARISON

5.7 Ultimate Flexural Strength Check

For purposes of the transverse design, Strength Limit State IV is the same as Strength Limit State I without live load, with 25 percent more self-weight. This loading does not govern in this example.

For temperature gradient load factors, LRFD Specifications suggest determining a load factor on a project specific basis, with a recommendation of 0.0 for most instances. Since these loads are a result of restrained deformations, the loads should disappear if the reinforcement begins to yield at ultimate. In addition, the Segmental Guide Specifications does not include this component in ultimate load combinations. For these reasons, the temperature gradient was not used in the strength limit state combinations.

The LRFD specifications require minimum reinforcement equal to that required to resist 1.2 times the cracking moment. This requirement governed only for the bottom slab (soffit) design. To satisfy the minimum steel requirement, the transverse bar spacing in the bottom soffit was decreased from 12 inches to 8 inches, which represents an increase in reinforcement of 50 percent.

Also under ultimate flexure, the amount of web steel reinforcing required for transverse bending should be calculated. This should be combined in an appropriate manner with reinforcing required for longitudinal shear.

Listed below is the ultimate load combination used in this example:

Strength I

$\gamma_p DC + \gamma_p DW + 1.0EL + 1.75(LL + IM) + 0.5(CR + SH)$

5.8 Ultimate Shear Strength Check

Traditionally, shear behavior has been ignored in the design of concrete decks for AASHTO bridges. Box girder decks are similar in this sense, but can often have large construction loads placed on them. In these special cases, both one-way and two-way action shear should be investigated.

6. LONGITUDINAL DESIGN

6.1 Design Methodology

This structure is erected using the precast balanced cantilever method of construction. Due to changes in the statical system during erection, as cantilevers are made continuous through cast-in-place closure joints, it is necessary to analyze the structure for time-dependent effects. Time dependent analysis is a function of the segment casting date, times that the segments are incorporated into the structure, as well as dates associated with changes in the structural system throughout the construction process.

It is customary to establish an assumed sequence of construction and to estimate a reasonable construction schedule. Casting and erection dates of the segments are established based on construction schedule and production rate. Casting dates are a function of an assumed number of casting cells and time required to cast each segment. For purposes of estimating these dates, production rate is assumed as one typical segment per day per casting cell and one pier/expansion joint segment per week per casting cell. Segments are not to be erected earlier than one month after casting. During construction, when actual casting and erection dates become available, the stage-by-stage analysis should be re-run in order to obtain correct camber values.

Time dependent properties of concrete are established based on environmental humidity and dimensions of the cross-section, and can be adjusted for concrete composition (e.g. limestone aggregate), rate of hardening, and ambient temperature. Section properties shall be determined for each segment considering effects of shear lag in the top and bottom slab.

The above information is entered into time dependent analysis software such as TANGO, among others. A stage-by-stage analysis is performed using an assumed post-tensioning layout while carefully modeling appropriate boundary conditions for each step of the construction process. After the construction has been modeled, the structure is stepped through time to day 4000 or day 10000 to allow all time dependent effects to occur. It is also essential in statically indeterminate structures to sum up all locked-in forces that result from various stages of structural systems until day 10,000. Additional loads are placed on the structure such as live load, temperature gradient, and support settlement, as appropriate, and analyzed for initial (at end of construction) and final conditions at day 10,000.

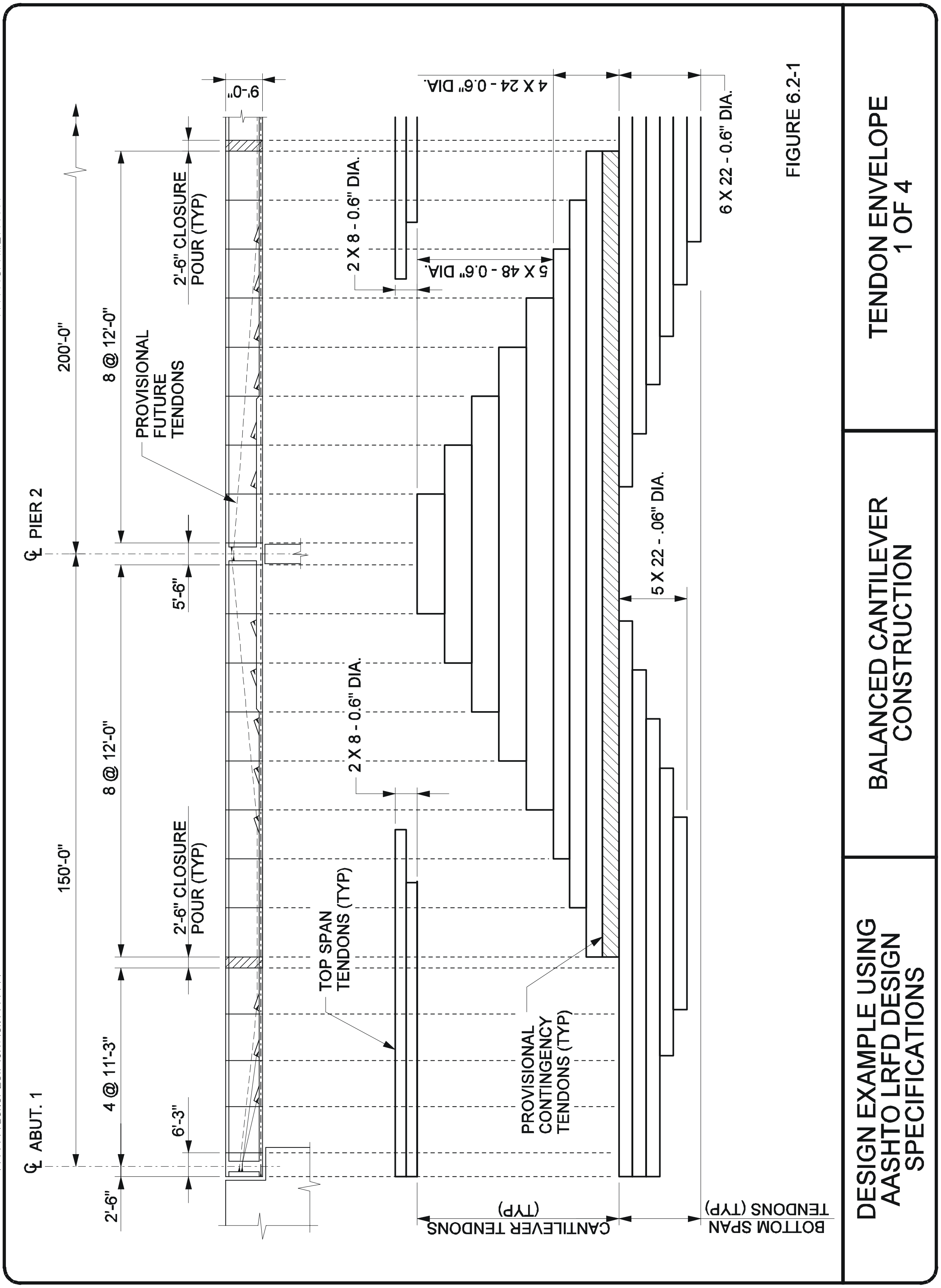
6.2 Tendon Layout / Envelope

An approximate tendon layout can be based on preliminary calculations for construction loading of a typical cantilever. Span continuity tendons can be estimated by preliminary design based on final structure approximate creep and shrinkage effects using load factor dead and live load combinations. The assumed layout can then easily be modified during final design to satisfy all applicable LRFD Limit State Load Combinations.

Preliminary design for this example indicated the need for twelve cantilever tendons and five bottom continuity tendons per web. Based on previous experience, two four strand continuity tendons were added in the top slab across the closure pour to control stresses resulting from temperature gradients. Final design resulted in an increase of one cantilever tendon and one bottom span continuity tendon at interior spans only.

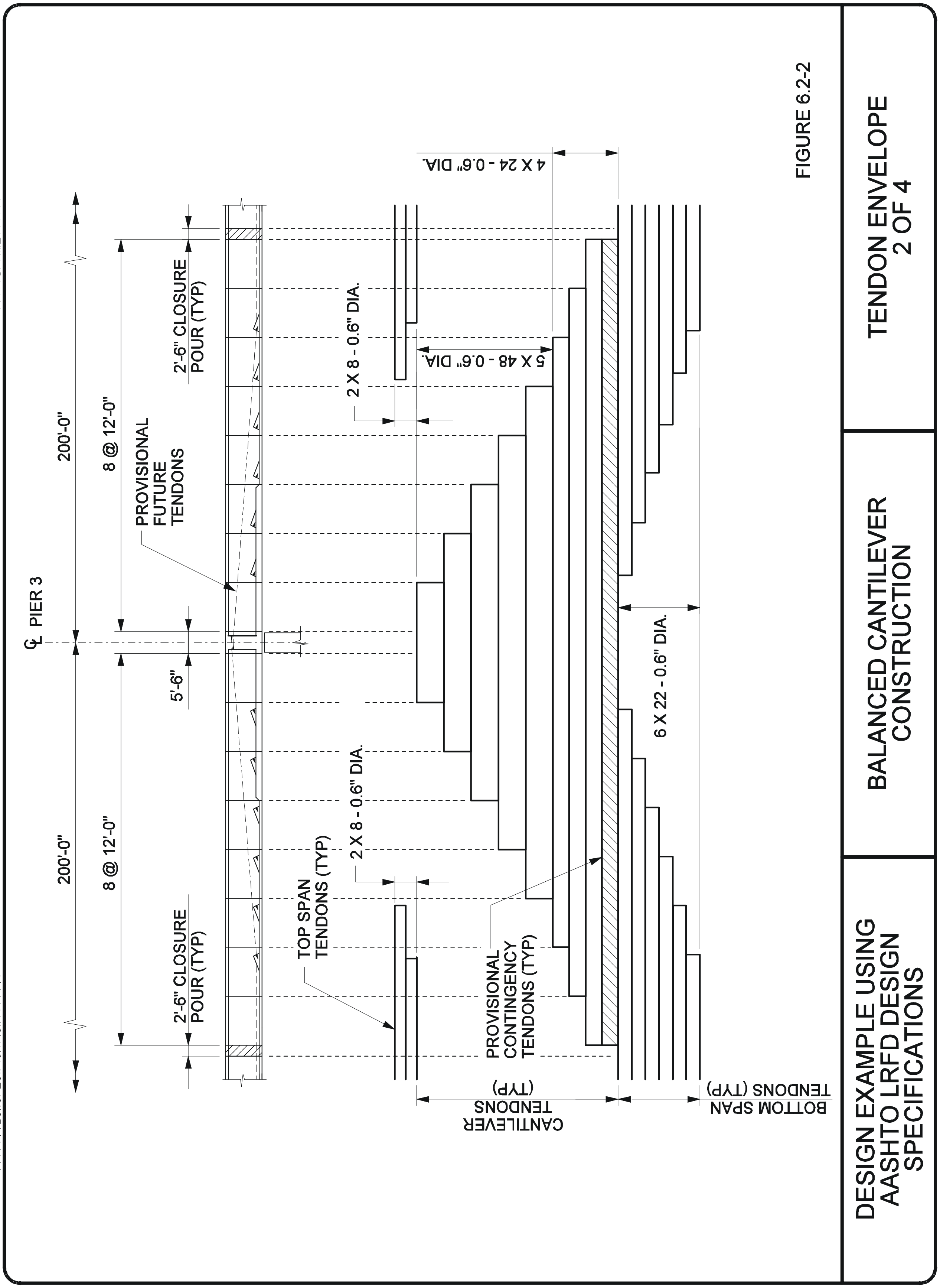
The tendons used are based on a twelve-strand system using 0.6” (15.24 mm) diameter strands. Only eleven strands were used for bottom continuity tendons to provide space for 5% contingency post-tensioning as required for internal tendons. One out of twelve strands will provide approximately 8% of the contingency post-tensioning if needed. An empty duct was provided for the cantilever tendons combined with an anchorage on the last segment of the cantilever in order to allow for contingency post-tensioning. This empty duct should be grouted if no contingency tendons are required.

Provisions are also made for future post-tensioning by addition of anchorages and deviation points for external tendons (inside the box section), which can be used for adjustment of deflections or for other unforeseen conditions. Provisional post-tensioning ducts and anchorages are covered under Article 5.14.2.3.8 of *AASHTO LRFD Bridge Design Specifications*.



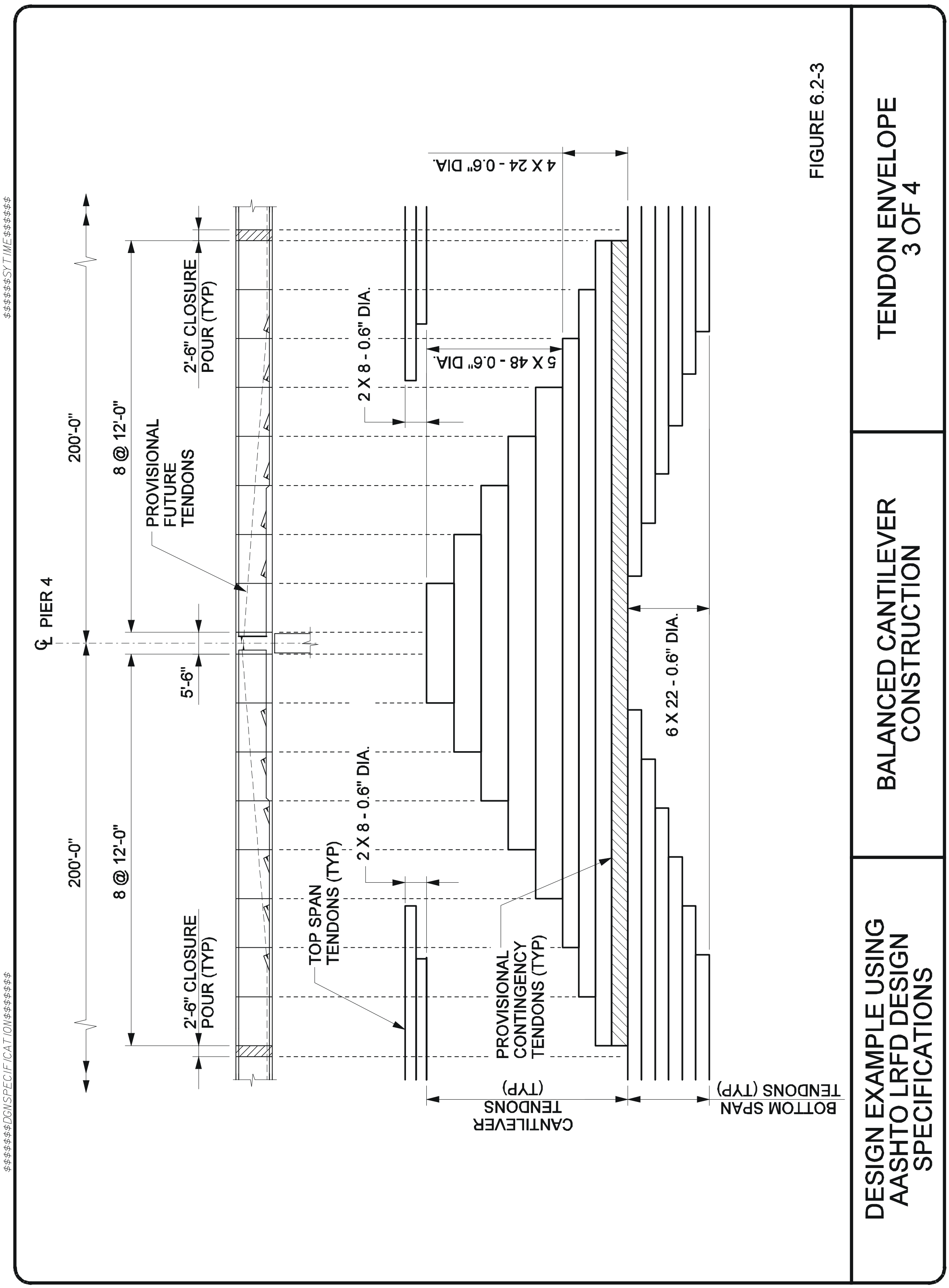
℄ ABUT. 1
℄ PIER 2
150'-0"
200'-0"
2'-6"
4 @ 11'-3"
8 @ 12'-0"
8 @ 12'-0"
6'-3"
5'-6"
9'-0"
2'-6" CLOSURE POUR (TYP)
2'-6" CLOSURE POUR (TYP)
PROVISIONAL FUTURE TENDONS
TOP SPAN TENDONS (TYP)
PROVISIONAL CONTINGENCY TENDONS (TYP)
2 X 8 - 0.6" DIA.
2 X 8 - 0.6" DIA.
5 X 48 - 0.6" DIA.
4 X 24 - 0.6" DIA.
5 X 22 - .06" DIA.
6 X 22 - 0.6" DIA.
CANTILEVER TENDONS (TYP)
BOTTOM SPAN TENDONS (TYP)
DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS
BALANCED CANTILEVER CONSTRUCTION
TENDON ENVELOPE 1 OF 4

FIGURE 6.2-1

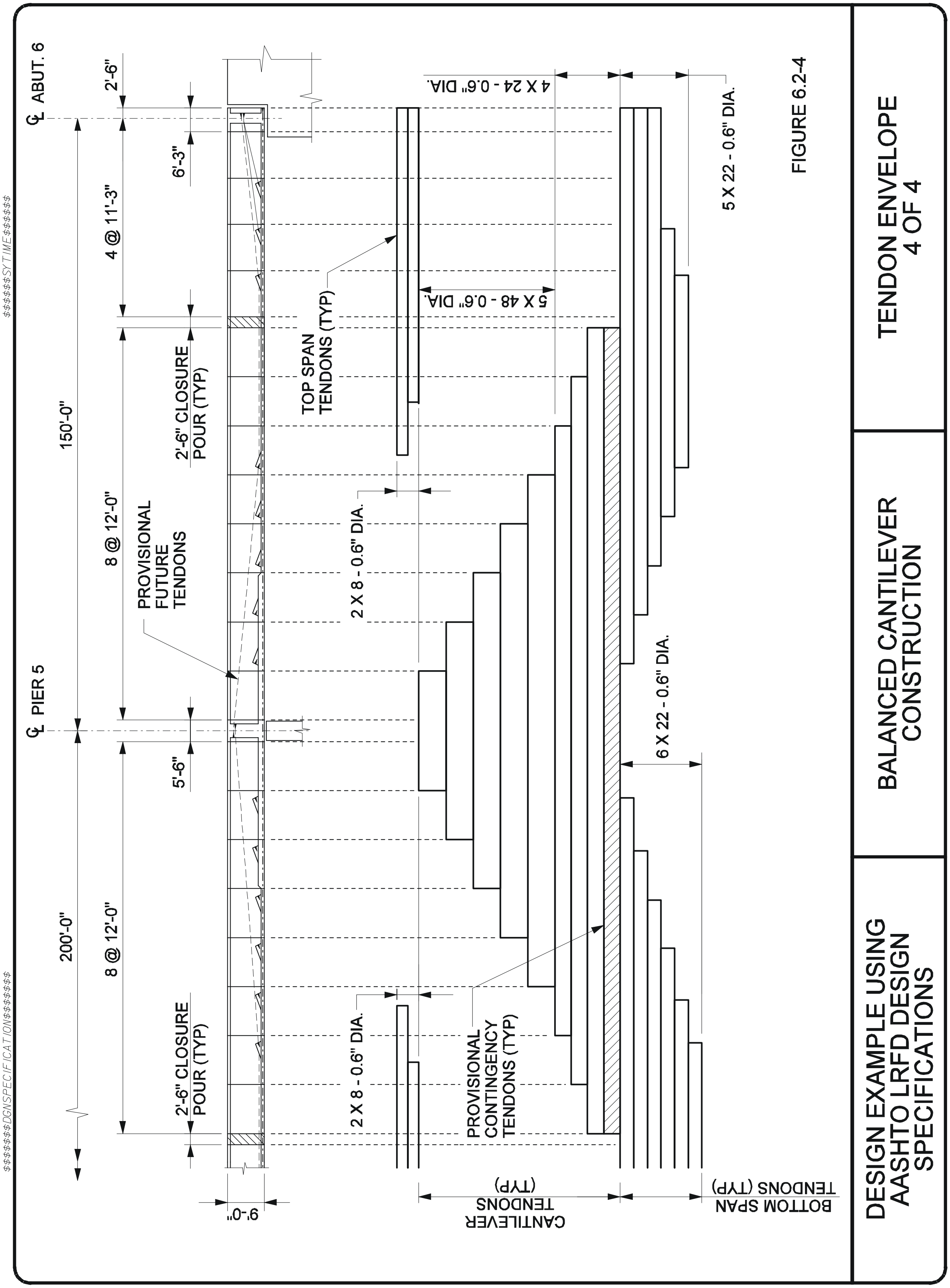


$$$$$$DGNSPECIFICATION$$$$$$$
$$$$$$SYTIME$$$$$$
℄ PIER 3
200'-0"
200'-0"
8 @ 12'-0"
8 @ 12'-0"
5'-6"
2'-6" CLOSURE POUR (TYP)
2'-6" CLOSURE POUR (TYP)
PROVISIONAL FUTURE TENDONS
TOP SPAN TENDONS (TYP)
2 X 8 - 0.6" DIA.
2 X 8 - 0.6" DIA.
PROVISIONAL CONTINGENCY TENDONS (TYP)
4 X 24 - 0.6" DIA.
5 X 48 - 0.6" DIA.
6 X 22 - 0.6" DIA.
CANTILEVER TENDONS (TYP)
BOTTOM SPAN TENDONS (TYP)
TENDON ENVELOPE
2 OF 4
BALANCED CANTILEVER
CONSTRUCTION
DESIGN EXAMPLE USING
AASHTO LRFD DESIGN
SPECIFICATIONS

FIGURE 6.2-2



℄ PIER 4
200'-0"
200'-0"
8 @ 12'-0"
8 @ 12'-0"
5'-6"
2'-6" CLOSURE POUR (TYP)
2'-6" CLOSURE POUR (TYP)
PROVISIONAL FUTURE TENDONS
TOP SPAN TENDONS (TYP)
PROVISIONAL CONTINGENCY TENDONS (TYP)
2 X 8 - 0.6" DIA.
2 X 8 - 0.6" DIA.
5 X 48 - 0.6" DIA.
4 X 24 - 0.6" DIA.
6 X 22 - 0.6" DIA.
CANTILEVER TENDONS (TYP)
BOTTOM SPAN TENDONS (TYP)
FIGURE 6.2-3
TENDON ENVELOPE 3 OF 4
BALANCED CANTILEVER CONSTRUCTION
DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS

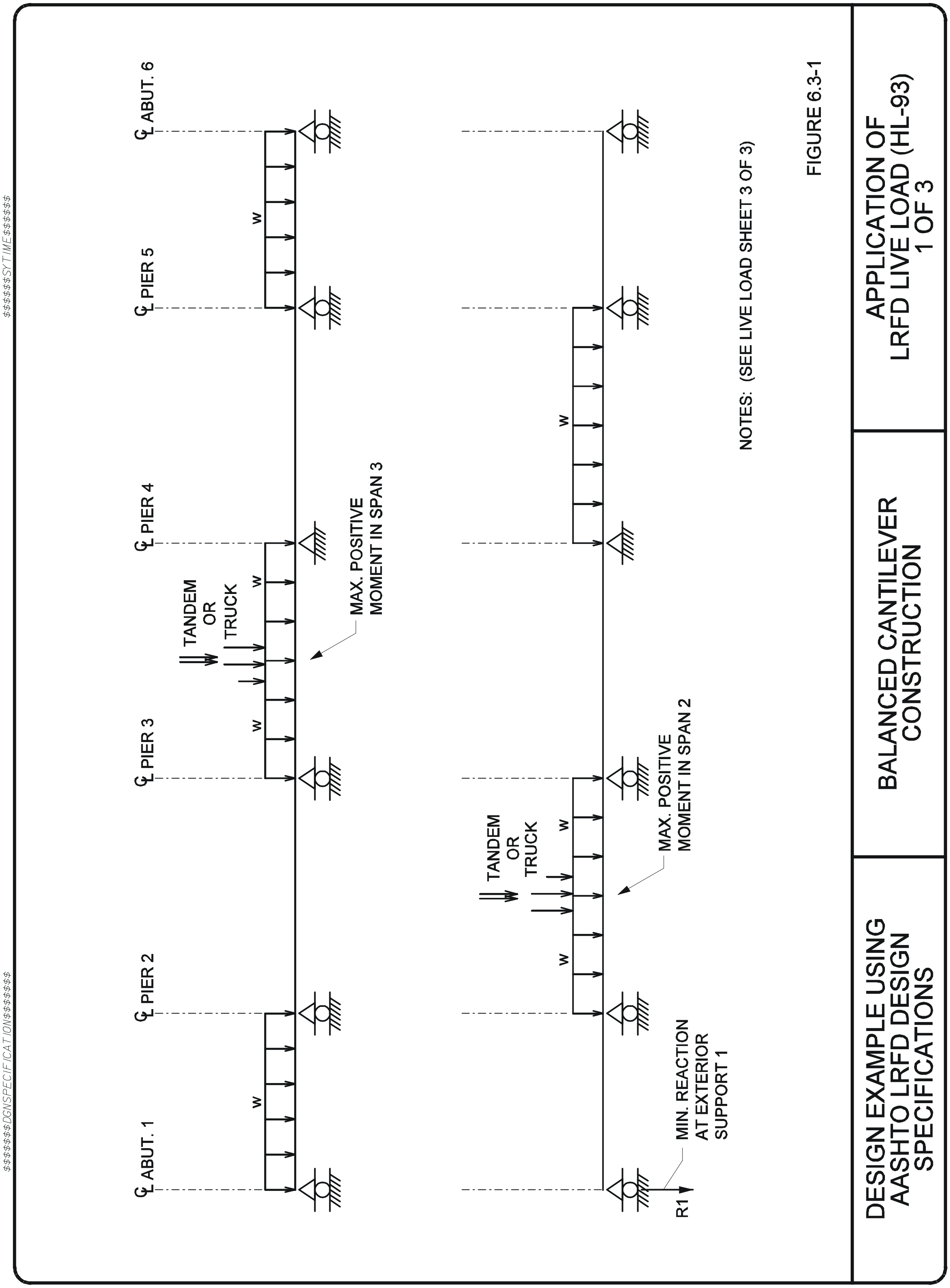

℄ ABUT. 6
℄ PIER 5
2'-6"
6'-3"
4 @ 11'-3"
150'-0"
8 @ 12'-0"
5'-6"
200'-0"
8 @ 12'-0"
2'-6" CLOSURE POUR (TYP)
2'-6" CLOSURE POUR (TYP)
PROVISIONAL FUTURE TENDONS
TOP SPAN TENDONS (TYP)
4 X 24 - 0.6" DIA.
5 X 48 - 0.6" DIA.
5 X 22 - 0.6" DIA.
2 X 8 - 0.6" DIA.
6 X 22 - 0.6" DIA.
2 X 8 - 0.6" DIA.
PROVISIONAL CONTINGENCY TENDONS (TYP)
9'-0"
CANTILEVER TENDONS (TYP)
BOTTOM SPAN TENDONS (TYP)
FIGURE 6.2-4
TENDON ENVELOPE 4 OF 4
BALANCED CANTILEVER CONSTRUCTION
DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS

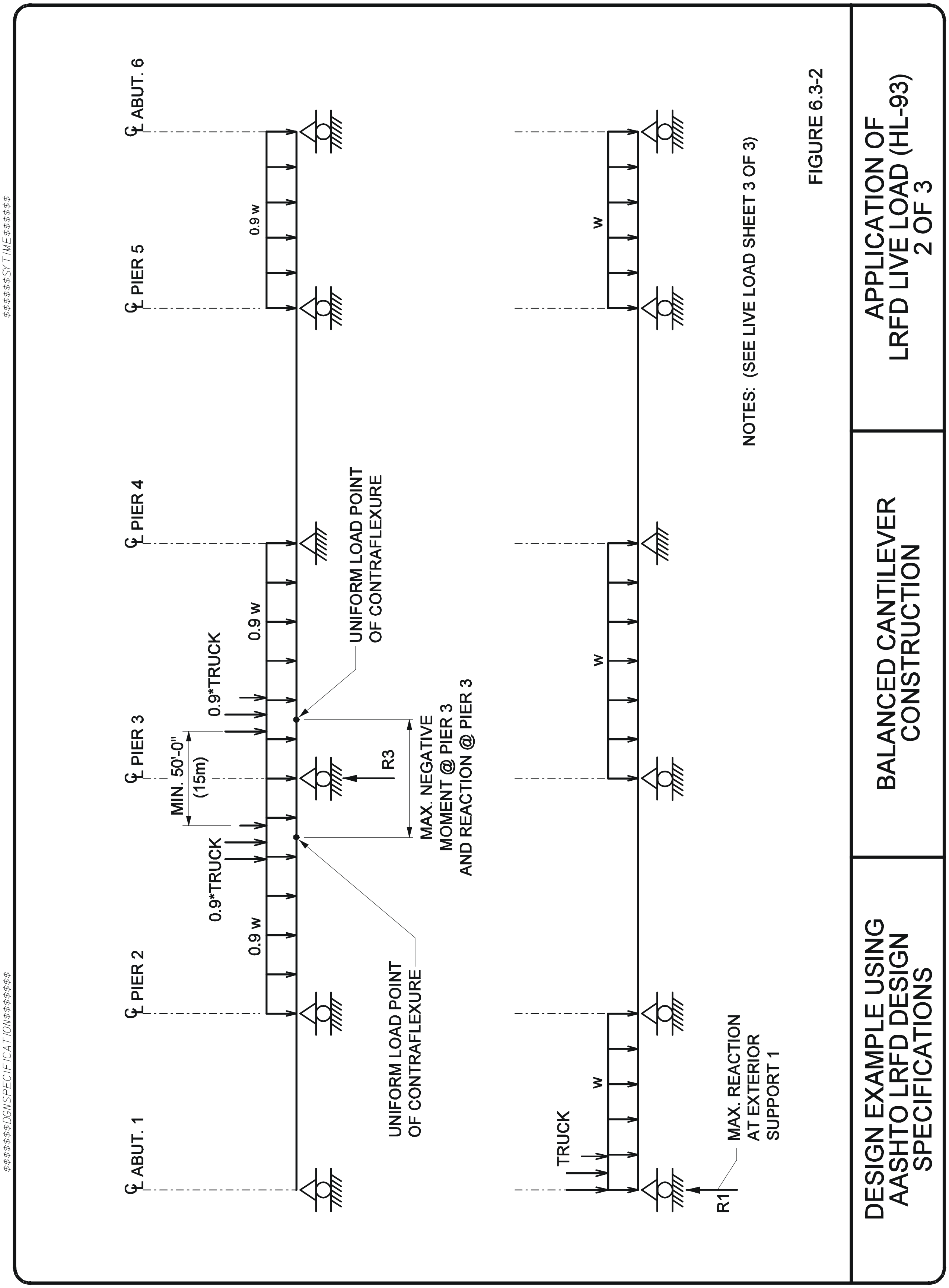
6.3 LRFD Live Load

LRFD live load (HL-93) consists of a single design truck per lane or tandem combined with a uniformly distributed lane load. For negative moments only, a second truck is added and the total effect is reduced by 10%. The second truck is required only between points of uniform load contraflexure, and should leave a space of at least 50 feet (15 meters) between trucks measured between the rear axle of the leading truck and the front axle of the trailing truck. A fatigue truck is also specified but was not considered for this example.

A dynamic load allowance (impact) of 33 percent is added to the design truck, but is not required for design lane load. Multiple presence factors range from 1.2 for a single lane to 0.85 for three lanes and 0.65 for more than three lanes. This example is based on 3 lanes, and has a multiple presence factor of 0.85 (the current *AASHTO Standard Specifications* would dictate an impact of 15% and a multiple presence factor of 0.90).

For comparison purpose, HS20-44 and HS25-44 AASHTO loadings were run in addition to the HL-93 LRFD loading. After impact and multiple presence factors are included, results for this example show that live load moments are increased by approximately 30% for negative moment and approximately 50% for positive moment when compared to HS 20-44 live load. Live load shears are increased by approximately 40% when compared to HS 20-44 live load. The HS25-44 loading increases the HS20-44 results by 25%, thus narrowing the difference, but HL-93 results remain slightly higher.

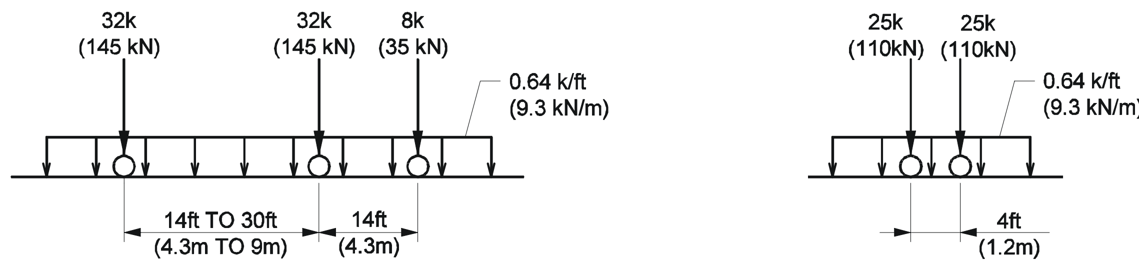

$$$$$$$DGNSPECIFICATION$$$$$$$
$$$$$$$SYTIME$$$$$$
℄ ABUT. 1
℄ PIER 2
℄ PIER 3
℄ PIER 4
℄ PIER 5
℄ ABUT. 6
w
TANDEM OR TRUCK
MAX. POSITIVE MOMENT IN SPAN 3
MAX. POSITIVE MOMENT IN SPAN 2
MIN. REACTION AT EXTERIOR SUPPORT 1
R1
NOTES: (SEE LIVE LOAD SHEET 3 OF 3)
FIGURE 6.3-1
DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS
BALANCED CANTILEVER CONSTRUCTION
APPLICATION OF LRFD LIVE LOAD (HL-93) 1 OF 3



$$$$$$$DGNSPECIFICATION$$$$$$$$
$$$$$$SYTIME$$$$$$
℄ ABUT. 1
℄ PIER 2
℄ PIER 3
℄ PIER 4
℄ PIER 5
℄ ABUT. 6
0.9 w
0.9*TRUCK
MIN. 50'-0" (15m)
R3
MAX. NEGATIVE MOMENT @ PIER 3 AND REACTION @ PIER 3
UNIFORM LOAD POINT OF CONTRAFLEXURE
TRUCK
w
R1
MAX. REACTION AT EXTERIOR SUPPORT 1
NOTES: (SEE LIVE LOAD SHEET 3 OF 3)
DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS
BALANCED CANTILEVER CONSTRUCTION
APPLICATION OF LRFD LIVE LOAD (HL-93) 2 OF 3

FIGURE 6.3-2

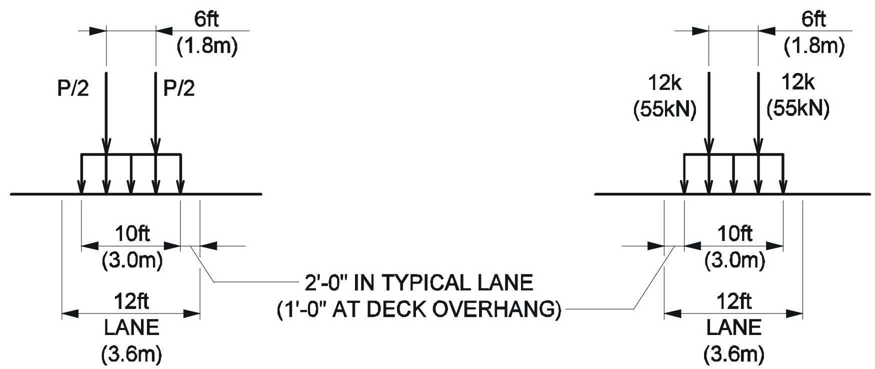
$$$$$$$DGNSPECIFICATION$$$$$$$ $$$$$$SYTIME$$$$$$



TRUCK AND LANE LOADS

TANDEM AND LANE LOADS

LONGITUDINAL LOAD CONFIGURATION



TRUCK LOAD

TANDEM LOAD

TRANSVERSE LOAD CONFIGURATION

NOTES:

1. w = LANE LOAD PER UNIT LENGTH = 0.64 k/ft (9.3 kN/m) PER LANE
2. DYNAMIC LOAD ALLOWANCE = 33%
3. THE DYNAMIC LOAD ALLOWANCE SHALL NOT BE APPLIED TO THE DESIGN LANE LOAD.
4. AXLES WHICH DO NOT CONTRIBUTE TO THE EXTREME FORCE EFFECT UNDER CONSIDERATION SHALL BE NEGLECTED.
5. UNIFORM LOAD FOR TRANSVERSE DESIGN TO BE ELIMINATED PER AASHTO T-5/T-10 COMMITTEE MEETING HELD IN JUNE 2004 IN ORLANDO, FLORIDA.

FIGURE 6.3-3

DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS	BALANCED CANTILEVER CONSTRUCTION	APPLICATION OF LRFD LIVE LOAD (HL-93) 3 OF 3

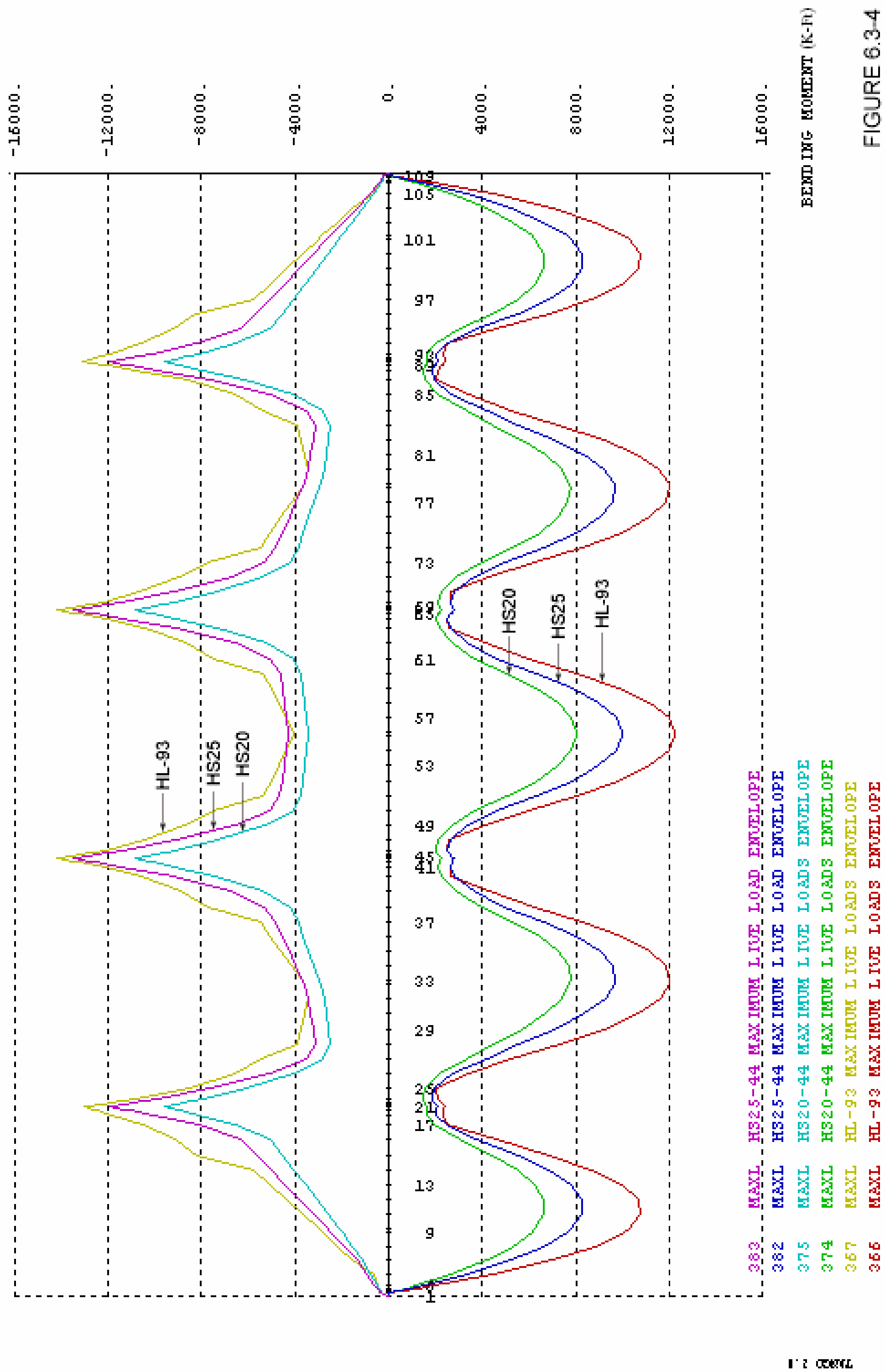
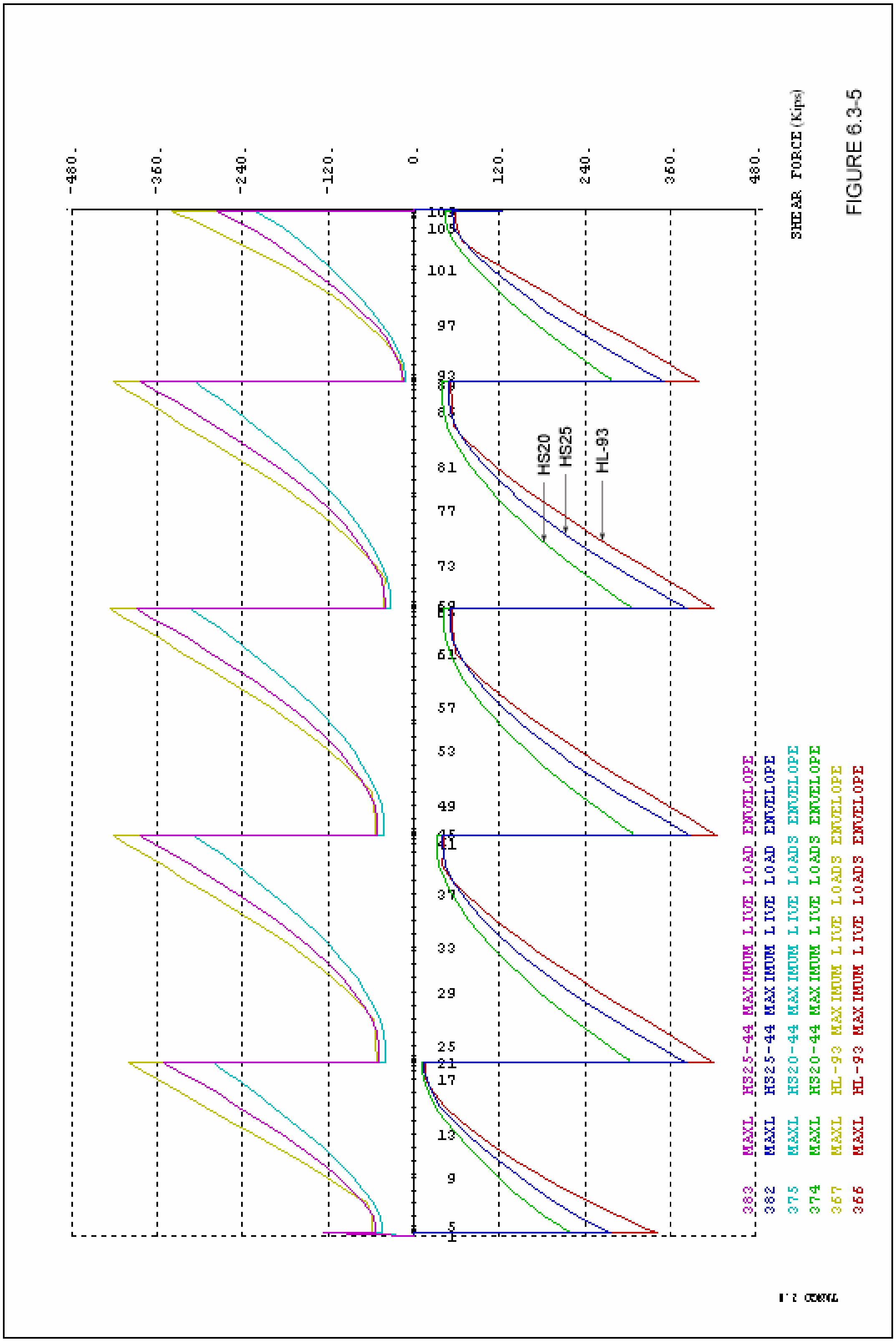
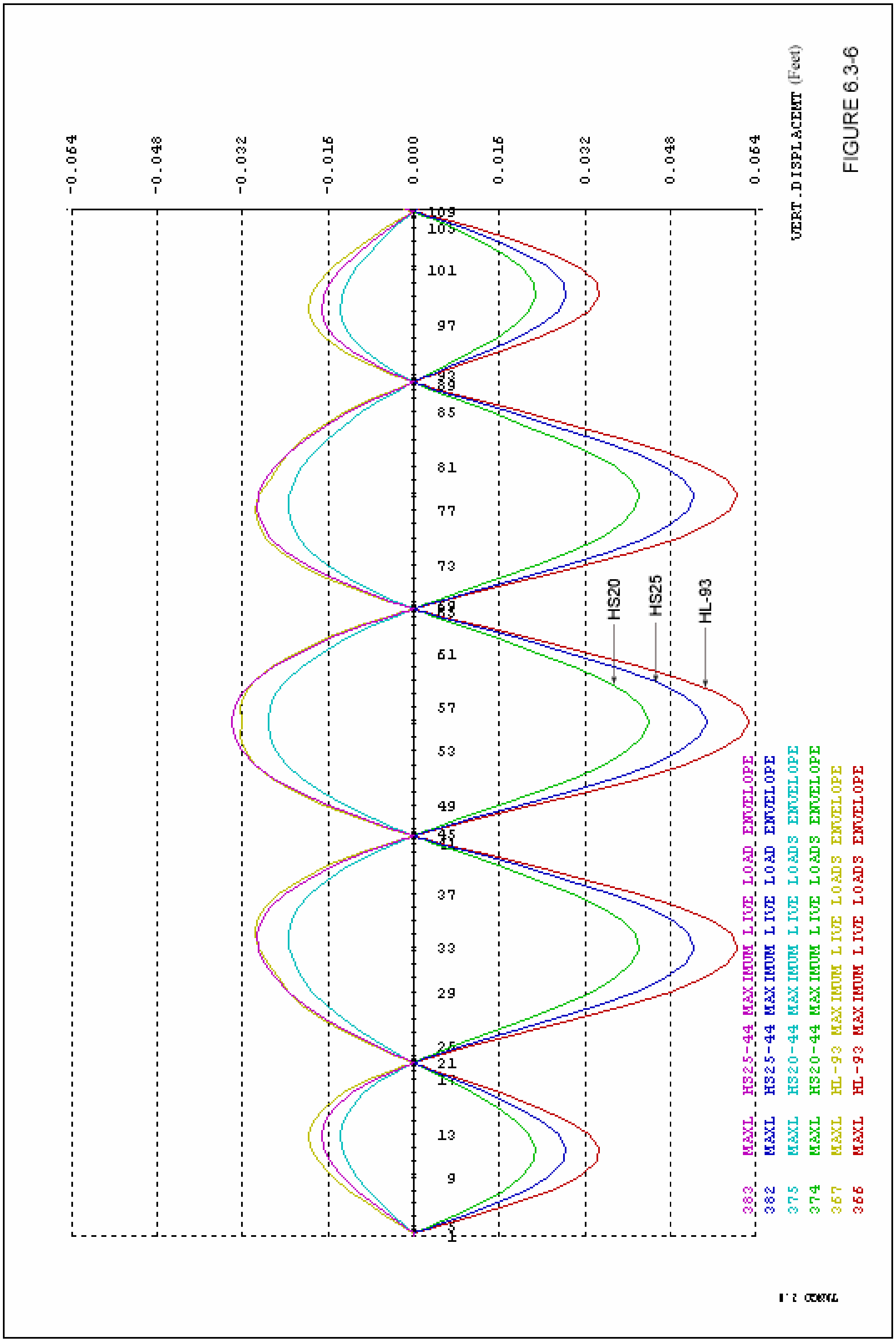

BENDING MOMENT (K-ft)
-16000.
-12000.
-8000.
-4000.
0.
4000.
8000.
12000.
16000.
HL-93
HS25
HS20
383 MAXL HS25-44 MAXIMUM LIVE LOAD ENVELOPE
382 MAXL HS25-44 MAXIMUM LIVE LOAD ENVELOPE
375 MAXL HS20-44 MAXIMUM LIVE LOADS ENVELOPE
374 MAXL HS20-44 MAXIMUM LIVE LOADS ENVELOPE
367 MAXL HL-93 MAXIMUM LIVE LOADS ENVELOPE
366 MAXL HL-93 MAXIMUM LIVE LOADS ENVELOPE

FIGURE 6.3-4



SHEAR FORCE (Kips)
-480.
-360.
-240.
-120.
0.
120.
240.
360.
480.
HS20
HS25
HL-93
383 MAXL HS25-44 MAXIMUM LIVE LOAD ENVELOPE
382 MAXL HS25-44 MAXIMUM LIVE LOAD ENVELOPE
375 MAXL HS20-44 MAXIMUM LIVE LOADS ENVELOPE
374 MAXL HS20-44 MAXIMUM LIVE LOADS ENVELOPE
367 MAXL HL-93 MAXIMUM LIVE LOADS ENVELOPE
366 MAXL HL-93 MAXIMUM LIVE LOADS ENVELOPE

FIGURE 6.3-5



-0.064
-0.048
-0.032
-0.016
0.000
0.016
0.032
0.048
0.064
VERT.DISPLACEMNT (Feet)
HS20
HS25
HL-93
383 MAXL HS25-44 MAXIMUM LIVE LOAD ENVELOPE
382 MAXL HS25-44 MAXIMUM LIVE LOAD ENVELOPE
375 MAXL HS20-44 MAXIMUM LIVE LOADS ENVELOPE
374 MAXL HS20-44 MAXIMUM LIVE LOADS ENVELOPE
367 MAXL HL-93 MAXIMUM LIVE LOADS ENVELOPE
366 MAXL HL-93 MAXIMUM LIVE LOADS ENVELOPE

FIGURE 6.3-6

6.4 Shear Lag Effect

The *AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges, First Edition* adopted shear lag provisions of *DIN 1075* (German Concrete Code) using linear transition of effective flanges. However, in the second edition, shear lag provision changed to a step function between span and support regions. In contrast to this change, the *AASHTO LRFD Bridge Design Specifications, Third Edition* adopted shear lag provisions similar to *DIN 1075*, as shown in Article 4.6.2.6.2. The difference between the two methods is insignificant, but the LRFD shear lag provision is considered more accurate.

When determining section properties, it is commonly assumed that shear lag applies to moment of inertia and location of the neutral axis of the section. However, cross-sectional area remains based on the full cross-section, so as to not overestimate the “P/A” component of post-tensioning stresses.

Shear lag is a function of the structural system at the time under consideration. If software permits, section properties can be changed in the construction model to approximate true statical conditions at all intermediate steps. This additional accuracy may not be warranted for all designs, but could be evaluated on a case-by-case basis.

The following shear lag effect calculation is in accordance with article 4.6.2.6 of *AASHTO LRFD Bridge Design Specifications, Third Edition 2004*.

I. COMPLETED STRUCTURE

<u>END SPAN</u>

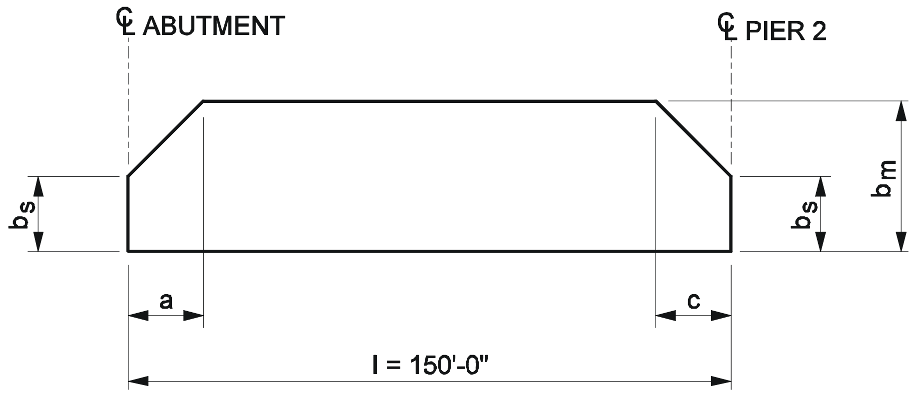


FIGURE 6.4-1

where: b = flange width on each side of web (See Figure 6.4-4)

b_1 = 10.37'

b_2 = 9.71'

b_3 = 7.34'

a = the largest of b, but not exceeding $0.25 \times l$

= 10.37' < 0.25 × 150' = 37.5'

c = $0.1 \times l$ = 0.1 (150') = 15' 0"

l_i = $0.8 \times l$ = 0.1 (150') = 120'

	b	b/l_i	b_s/b	b_m/b	b_{se}	b_{me}
b_1	10.37'	0.086	0.8	1.0	<u>8.3'</u>	10.37
b_2	9.71'	0.081	0.8	1.0	<u>7.77'</u>	9.71'
b_3	7.34'	0.061	1.0	1.00	7.34	7.34

Obtained b_s/b and b_m/b ratios from LRFD Figure 4.6.2.6.2-2.

Effective flange: b_{me} (No Reduction)

b_{s1e} =8.3'

b_{s2e} =7.77'

b_{s3e} =7.34' (No Reduction)

INNER SPAN

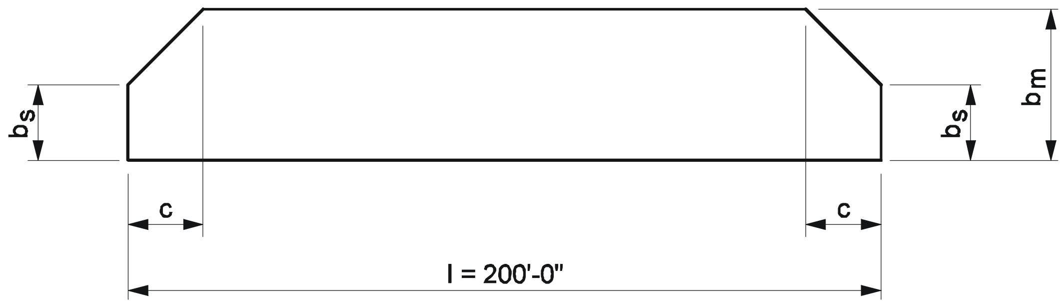


FIGURE 6.4-2

where: $c = 0.1 \times l = 0.1 \times 200' = 20'$

$l_i = 0.6 \times l = 0.6 \times 200' = 120'$

	b	b/l_i	b_s/b	b_m/b	$b_{s\,e}$	$b_{m\,e}$
b_1	10.37'	0.086	0.8	1.0	8.3'	10.37'
b_2	9.71'	0.080	0.8	1.0	7.77'	9.71'
b_3	7.34	0.060	1.0	1.00	7.34'	7.34'

Effective flange: b_{me} (No Reduction)

b_{s1e} =8.3'

b_{s2e} =7.77'

b_{s3e} =7.34' (No Reduction)

II . DURING CONSTRUCTION

CANTILEVER.

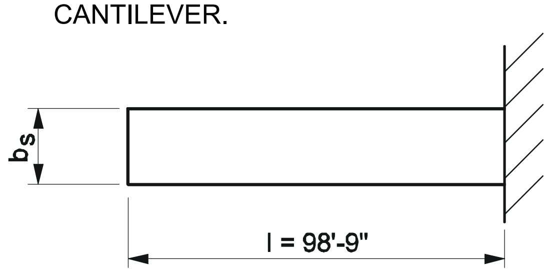


FIGURE 6.4-3

where: $l_i = 1.5 \times l = 1.5 \times 98.75 = 148.125'$

	b	b/l_i	b_s/b	b_{se}
b_1	10.37	0.07	0.75	7.77’
b_2	9.71	0.07	0.75	7.28’
b_3	7.34	0.05	1.00	7.34’

Effective flange: b_{s1e} =7.77’

b_{s2e} =7.28’

b_{s3e} =7.34’ (No Reduction)

$$$$$$$DGNSPECIFICATION$$$$$$$ $$$$$$SYTIME$$$$$$

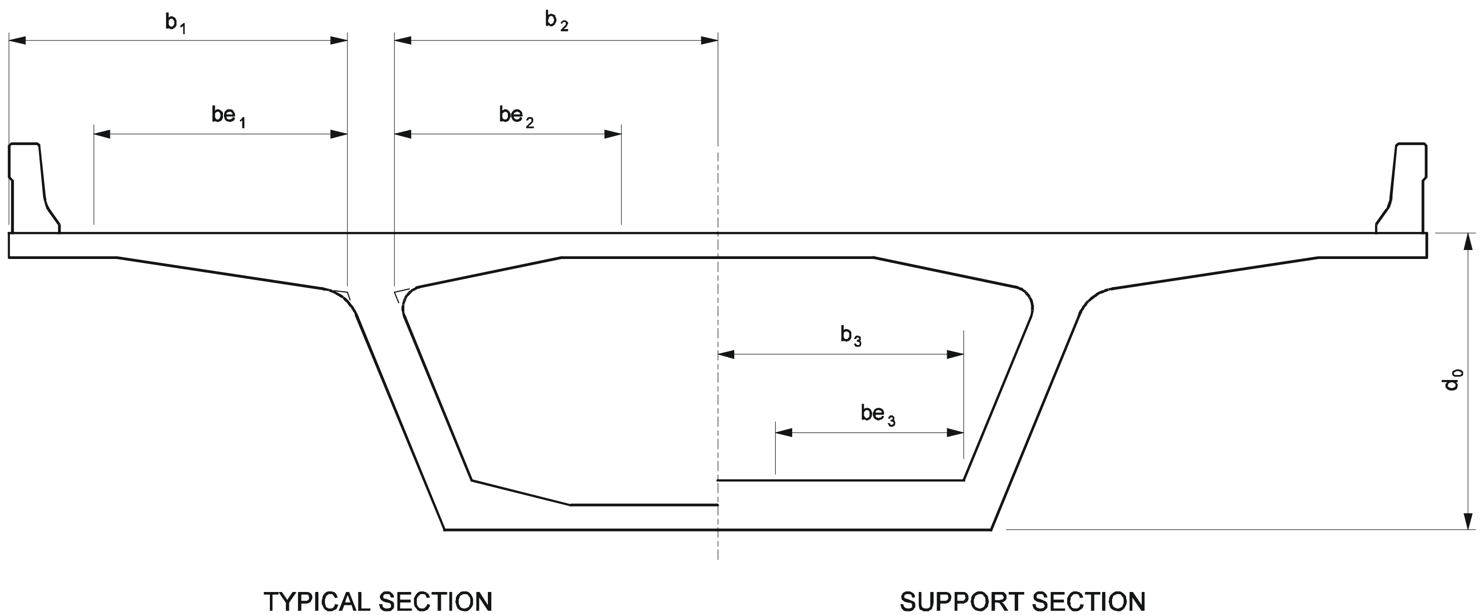


FIGURE 6.4-4

DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS

BALANCED CANTILEVER CONSTRUCTION

SHEAR LAG EFFECT

6.5 Temperature Load

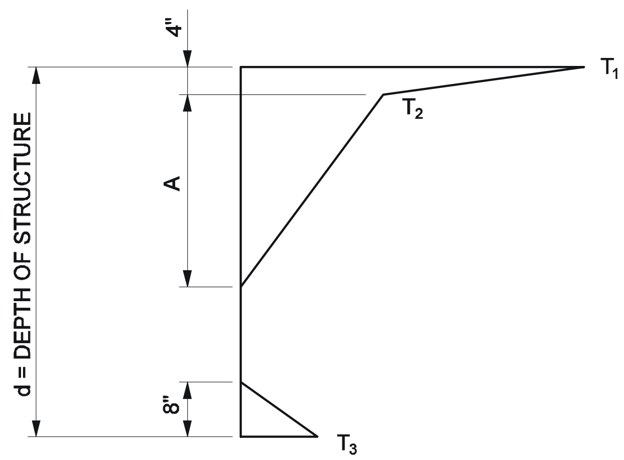
Temperature loads for superstructures consist of uniform temperature change as well as temperature gradients. A uniform temperature change of the superstructure is defined as the entire cross-section heating or cooling at the same rate. In contrast to this, a temperature gradient is defined as a vertical temperature change from top to bottom of the box. A positive temperature gradient results from solar heating of the deck surface and will cause higher temperatures in the top deck. A negative temperature gradient results from rapid cooling of deck concrete while ground temperatures may remain relatively unchanged from daytime conditions. The aforementioned gradients vary in a non-linear fashion with respect to depth of the superstructure, which requires a rather complex method of analysis to determine resulting stresses. The *AASHTO LRFD Bridge Design Specifications, Third Edition* adopted a temperature gradient profile (see Figure 6.5-1) that differs from that used by the *AASHTO Guide Specifications for Thermal Effects in Concrete Bridge Superstructures*, which is an abridged version of *NCHRP Report 276*.

Both uniform temperature and temperature gradient shall be included in service limit state load combinations. Temperature gradient may be reduced by 50% if live load is present in service load combinations. For segmental bridge design only, a special load combination (LRFD equation 3.4.1-2) for service shall be checked. This load combination has no live load; therefore 100% of the temperature gradient shall be included. In general, this load combination controls for segmental concrete bridges where live load force effects are small. In this example, such an area occurs at closure pours in the top of the box. Please note, for uniform temperature use a load factor of 1.0 when checking stresses, and 1.2 for structural deformations.

Temperature gradient shall not be included in strength limit state load combinations, while uniform temperature shall be included. Two load factors are assigned to uniform temperature in strength limit states. A factor of 0.5 shall be used for strength capacity calculations and 1.2 for structural deformations.

$$$$$$$DGNSPECIFICATION$$$$$$$ $$$$$$SYTIME$$$$$$

AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, 3RD EDITION 2004 (3.12.3).



POSITIVE VERTICAL TEMPERATURE GRADIENT

TEMPERATURES		
ZONE	T_1 (DEG F)	T_2 (DEG F)
1	54	14
2	46	12
3	41	11
4	38	9

NOTES:

1. T_3 = 0 OR T $\leq$ 5 DEGREES F
2. A = 1'-0" FOR d $\geq$ 1'-4"
   A = 4" FOR d $<$ 1'-4"
3. FOR NEGATIVE VERTICAL TEMPERATURE GRADIENT, MULTIPLY T BY -0.30 FOR PLAIN CONCRETE AND -0.20 FOR DECK WITH AN ASPHALT OVERLAY.

FIGURE 6.5-1

DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS	BALANCED CANTILEVER CONSTRUCTION	VERTICAL TEMPERATURE GRADIENT PROFILES

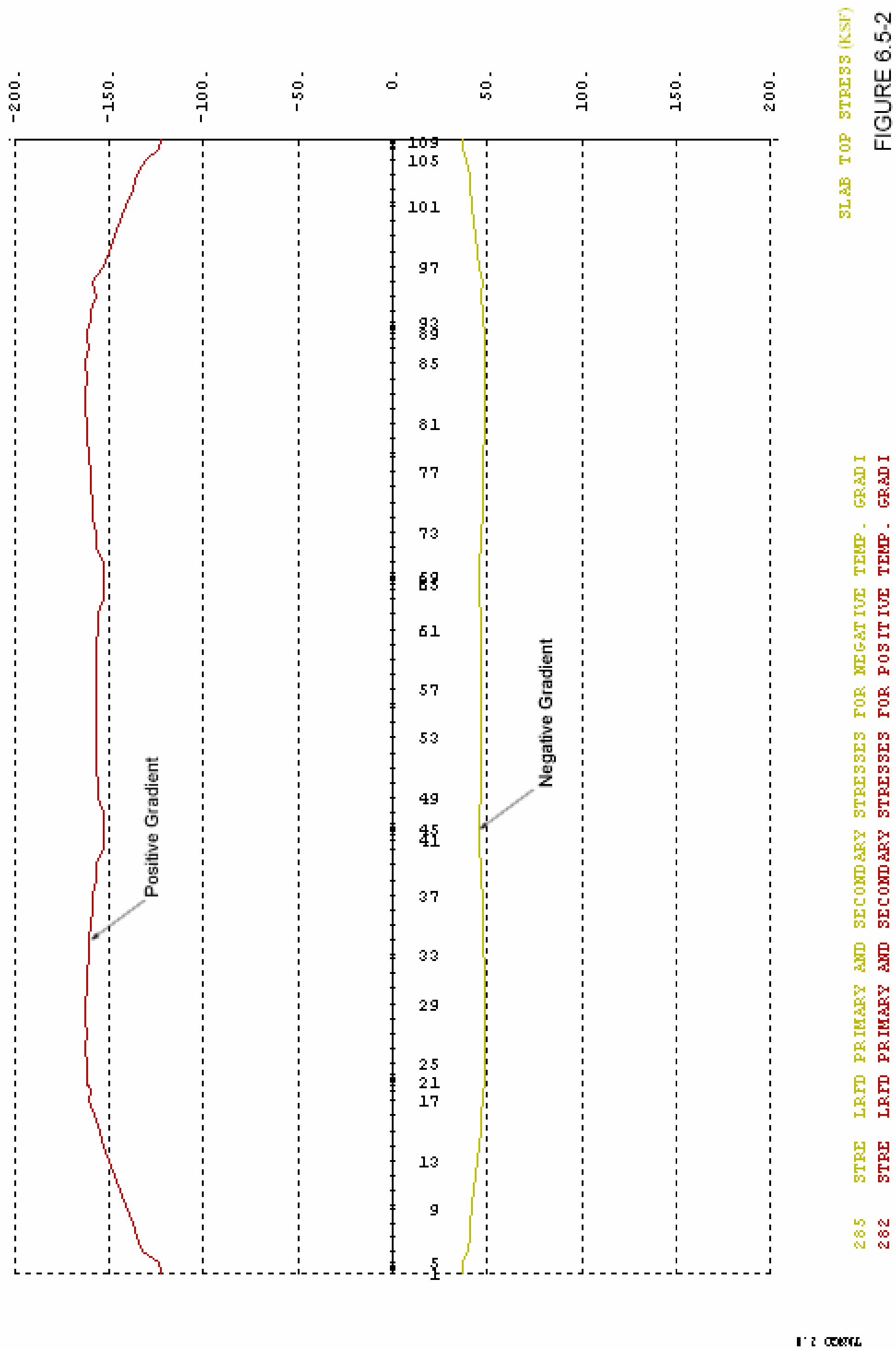
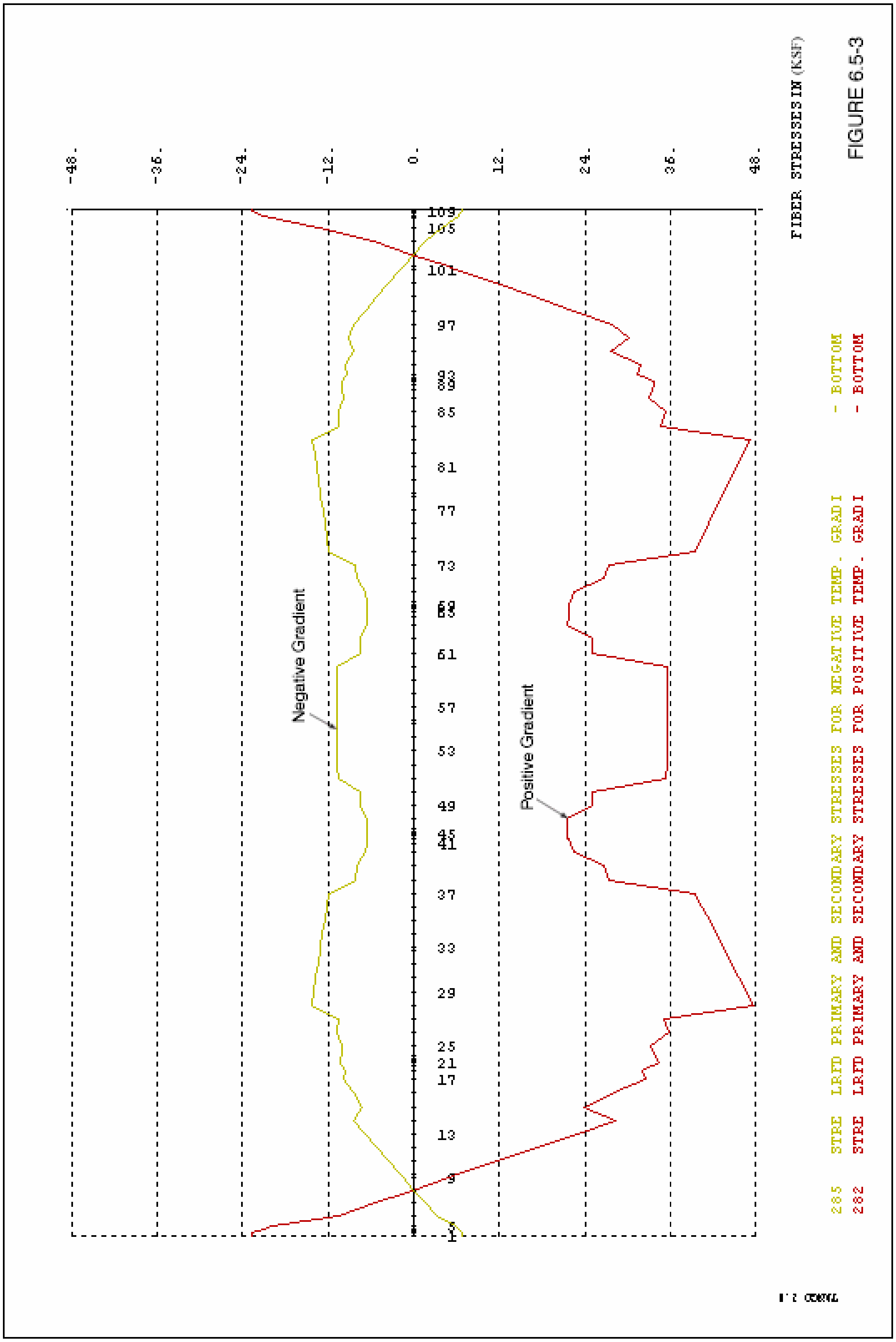

SLAB TOP STRESS (KSF)
-200.
-150.
-100.
-50.
0.
50.
100.
150.
200.
Positive Gradient
Negative Gradient
285 STRE LRFD PRIMARY AND SECONDARY STRESSES FOR NEGATIVE TEMP. GRADI
282 STRE LRFD PRIMARY AND SECONDARY STRESSES FOR POSITIVE TEMP. GRADI

FIGURE 6.5-2



FIBER STRESSES IN (KSF)
-48.
-36.
-24.
-12.
0.
12.
24.
36.
48.
Negative Gradient
Positive Gradient
285 STRE LRFD PRIMARY AND SECONDARY STRESSES FOR NEGATIVE TEMP. GRADI - BOTTOM
282 STRE LRFD PRIMARY AND SECONDARY STRESSES FOR POSITIVE TEMP. GRADI - BOTTOM

FIGURE 6.5-3

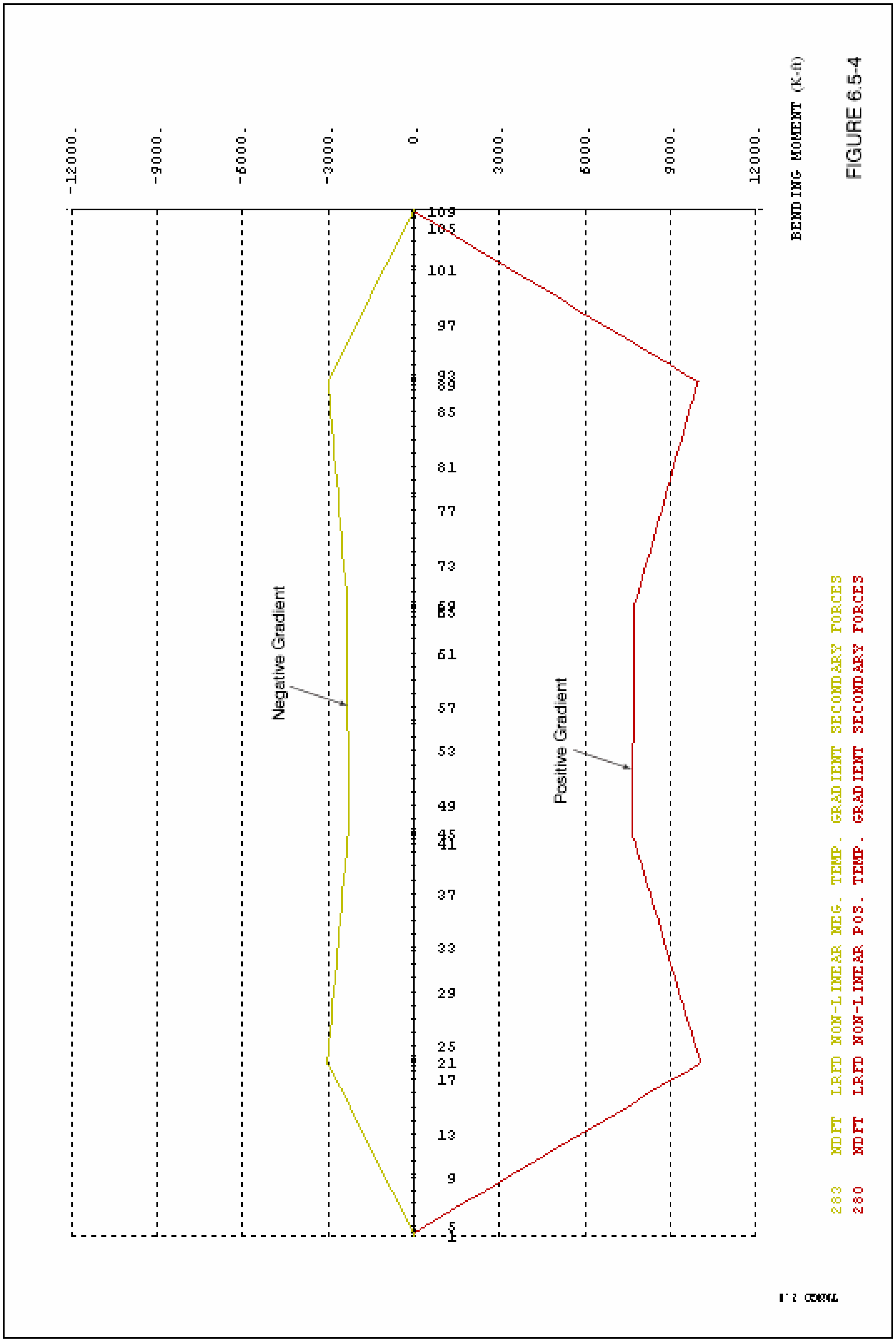

BENDING MOMENT (K-ft)
-12000.
-9000.
-6000.
-3000.
0.
3000.
6000.
9000.
12000.
Negative Gradient
Positive Gradient
283 NDFT LRFD NON-LINEAR NEG. TEMP. GRADIENT SECONDARY FORCES
280 NDFT LRFD NON-LINEAR POS. TEMP. GRADIENT SECONDARY FORCES

FIGURE 6.5-4

6.6 Time Dependent Effect

Creep and shrinkage of concrete, including relaxation of prestressing steel are commonly referred to as time dependent longterm effects. These effects are important factors that demand consideration in design of segmental bridges. Non-linear time dependent deformations will result in force redistribution due to changes in statical system during the course of the construction, and continue through day 10,000 when longterm effects are considered diminished.

Shrinkage, which causes shortening of concrete due to dehydration, is independent of stress (applied loads). Creep is a result of concrete deformation under permanent stress (loads) in addition to elastic deformation.

The redistribution of sectional forces due to change in statical system and creep effect can be estimated by Dischinger's equation.

$M_f = M_{II} + (M_I - M_{II})\, e^{-\varphi}$

Where:

M_f = Final moment at day 10,000

M_I = Moment as constructed at the end of construction

M_{II} = Moment assuming the bridge is constructed on false work

φ = Creep coefficient

M_{cr}= Moment due to creep effect

The above equation can be re-written to obtain M due to creep effects: $M_{cr} = (1\text{-}e^{\varphi})(M_{II} - M_I)$

Steel relaxation is the loss of tension in prestressing steel under constant length and temperature over a period of time. To prevent excessive relaxation loss in segmental bridges, low relaxation strand shall be used. The low relaxation strands shall meet the ASTM Standard requirement that relaxation loss after 1000 hours under 70˚ F shall be no more than 2.5% when initially stressed to 70% G.U.T.S. (Guaranteed Ultimate Tensile Strength) and not more than 3.5% when stressed to 80% G.U.T.S.

Although *AASHTO LRFD Bridge Design Specifications* allow creep and shrinkage effects to be evaluated using the provisions of *CEB-FIP Model Code* or *ACI 209*, for segmental bridge design, the *CEB-FIP Mode Code* provisions are commonly used. This design example utilizes the *CEB-FIP Model Code 1990*.

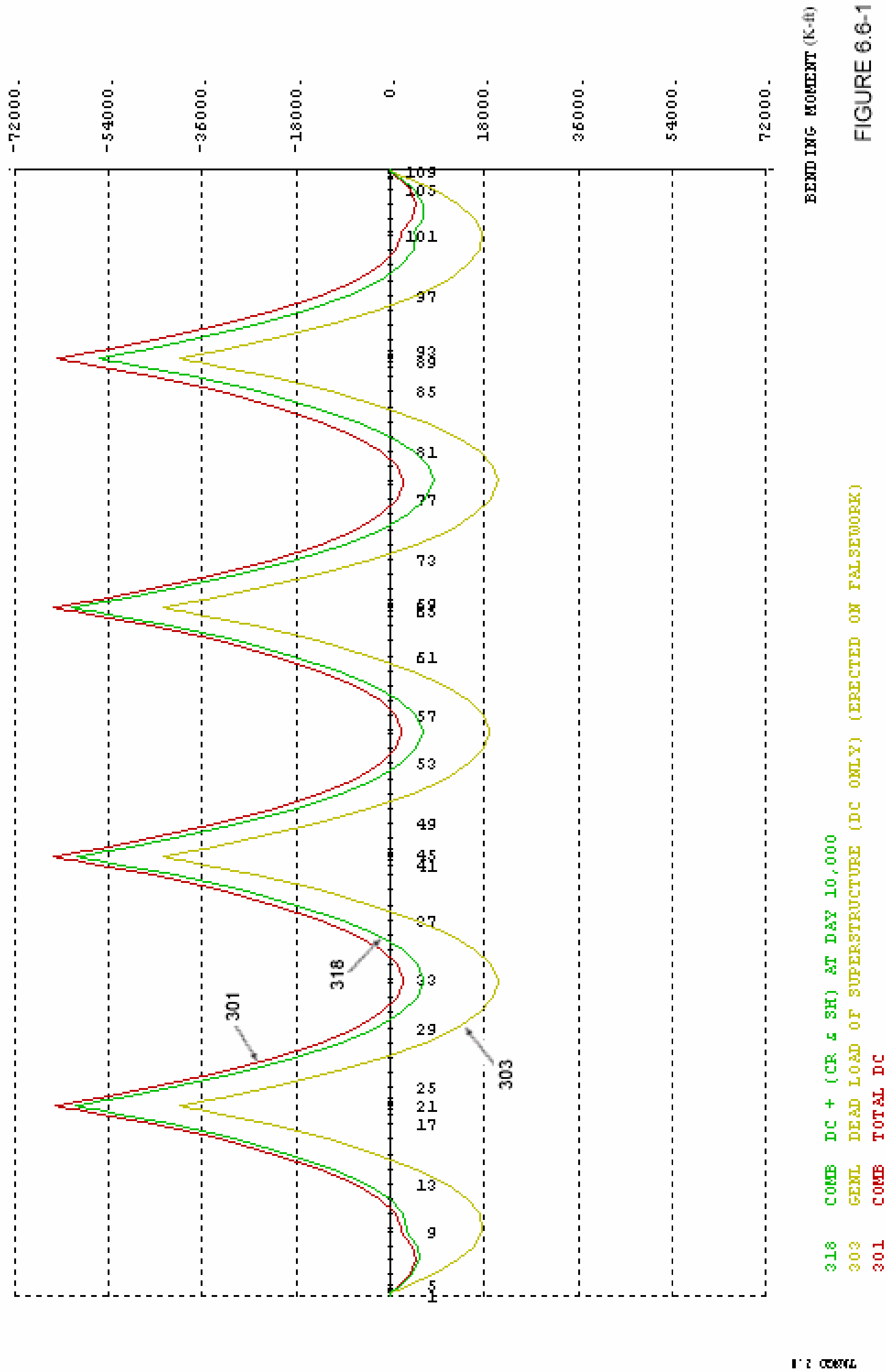
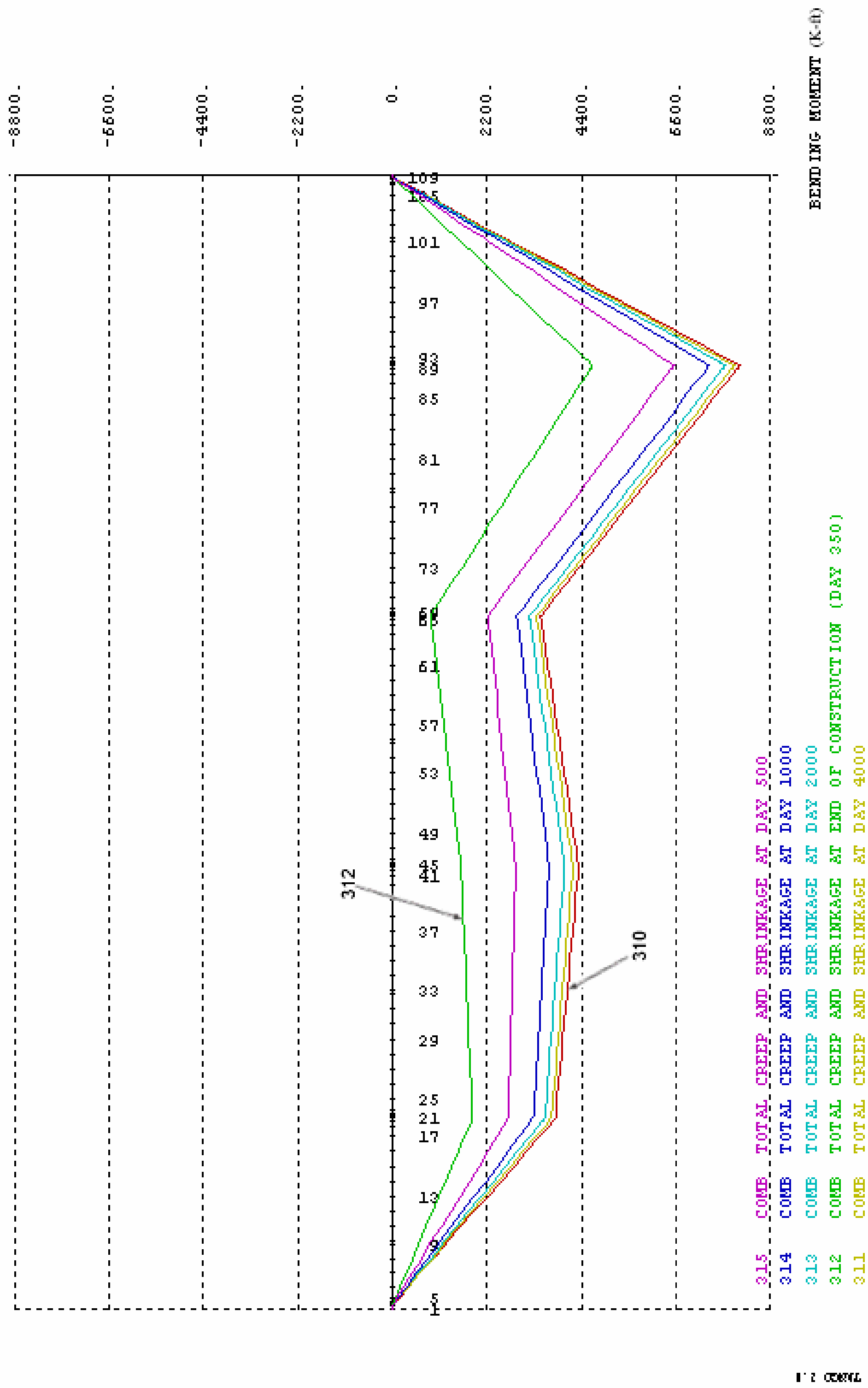

-72000.
-54000.
-36000.
-18000.
0.
18000.
36000.
54000.
72000.
BENDING MOMENT (K-ft)
109
105
101
97
93
89
85
81
77
73
69
65
61
57
53
49
45
41
37
33
29
25
21
17
13
9
5
1
301
318
303
318 COMB DC + (CR & SH) AT DAY 10,000
303 GENL DEAD LOAD OF SUPERSTRUCTURE (DC ONLY) (ERECTED ON FALSEWORK)
301 COMB TOTAL DC

FIGURE 6.6-1



BENDING MOMENT (K-ft)
-8800.
-6600.
-4400.
-2200.
0.
2200.
4400.
6600.
8800.
109
105
101
97
93
89
85
81
77
73
69
65
61
57
53
49
45
41
37
33
29
25
21
17
13
9
5
1
312
310
315 COMB TOTAL CREEP AND SHRINKAGE AT DAY 500
314 COMB TOTAL CREEP AND SHRINKAGE AT DAY 1000
313 COMB TOTAL CREEP AND SHRINKAGE AT DAY 2000
312 COMB TOTAL CREEP AND SHRINKAGE AT END OF CONSTRUCTION (DAY 350)
311 COMB TOTAL CREEP AND SHRINKAGE AT DAY 4000
310 COMB TOTAL CREEP AND SHRINKAGE AT DAY 10000

FIGURE 6.6-2

6.7 Secondary Forces

Secondary forces are internal forces generated as a result of applied deformations or imposed loads to statically indeterminate systems.

Listed below are several recognized secondary forces in segmental bridge design:

- Secondary forces due to primary post-tensioning
- Secondary forces due to construction process such as locked-in forces
- Secondary forces due to creep and shrinkage effects
- Secondary forces due to temperature loads (uniform and gradient temperature)
- Secondary forces due to support settlement

All of the above secondary forces shall be included in service limit state load combinations without exception. However, inclusion of different types of secondary forces in strength limit state load combinations may differ from code to code.

For instance, in the *AASHTO LRFD Bridge Design Specifications, Third Edition, 2004*, the secondary forces due to prestressing and erection loads (locked-in forces) are lumped together as “EL” with a permanent load factor γ_p equal to 1.0 for all strength limit state load combinations. On the other hand, in the *AASHTO Guide Specification for Design and Construction of Segmental Concrete Bridges, Second Edition*, the erection loads (locked-in forces) are lumped together with permanent dead loads, receiving a factor higher than 1.0. Under this assumption, since temporary loads are added during construction and then removed, only the effects due to permanent load will receive a load factor higher than 1.0.

The combination of prestressing and construction process secondary forces under “EL” as shown in *AASHTO LRFD Bridge Design Specifications, Third Edition* serves little merit in segmental bridge design. The author of this example recommends that the secondary forces due to prestressing and erection loads be separated and applied in accordance with *AASHTO Guide Specification for Design and Construction of Segmental Concrete Bridges, Second Edition*. For purposes of service limit state combinations, the separation of prestressing secondary forces and locked-in forces will make no difference in stresses. However, for ultimate limit state load combinations, a difference will occur. In most segmental software, dead loads are not distinguished from locked-in forces. Due to many construction stages during the erection process, it is possible to accumulate large quantities of dead load cases and locked-in force load cases. Once completing the construction process, backtracking to separate dead load cases from locked-in load cases creates complex book-keeping, and serves little benefit to end results.

Secondary forces due to temperature gradient are not included in strength limit state load combinations, while support settlement secondary forces are to be considered on a project specific basis.

Uniform temperature secondary forces, including creep and shrinkage effects, are included in strength limit state load combinations with load factor of 0.5.

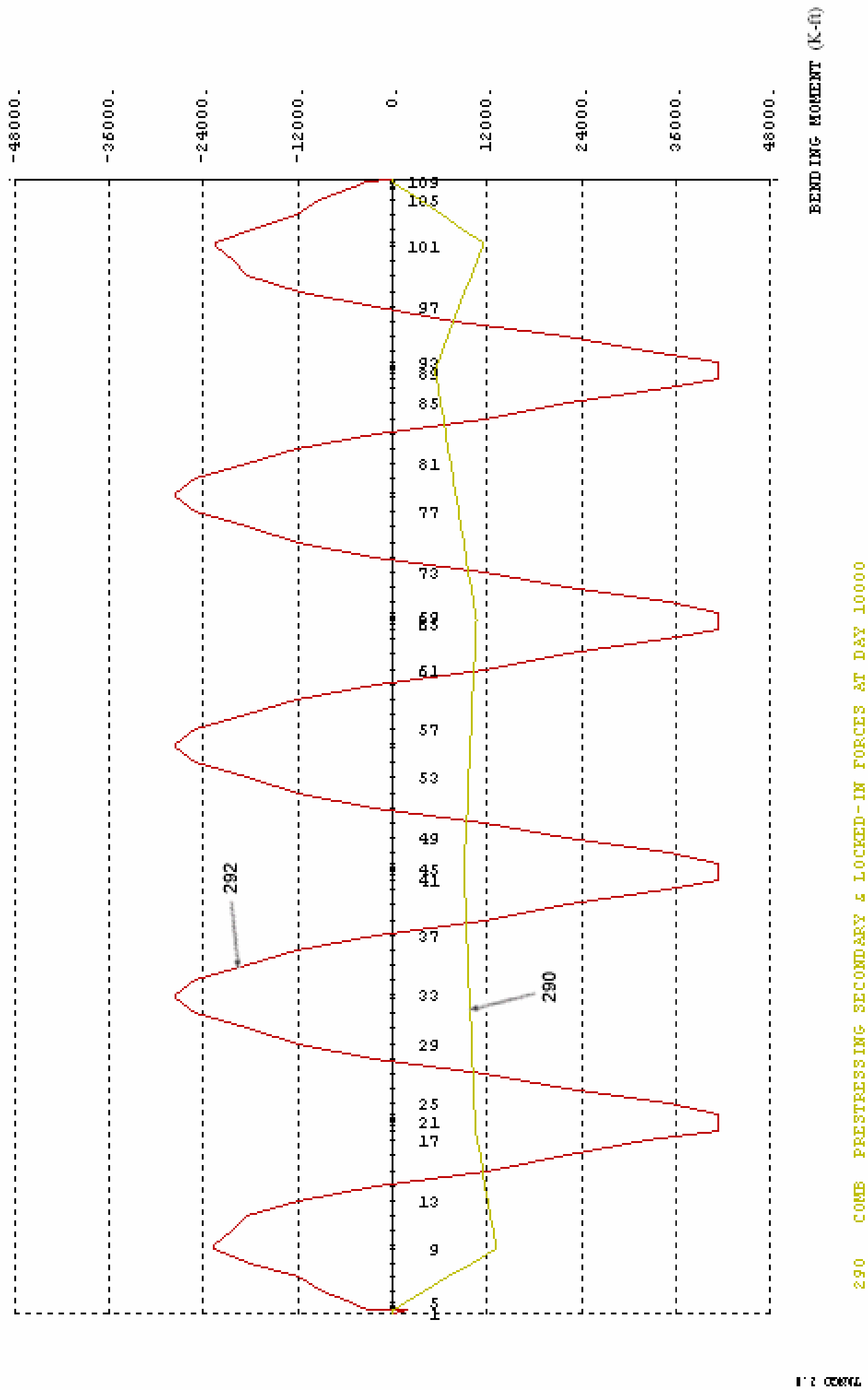

BENDING MOMENT (K-ft)
-48000.
-36000.
-24000.
-12000.
0.
12000.
24000.
36000.
48000.
292
290
290 COMB PRESTRESSING SECONDARY & LOCKED-IN FORCES AT DAY 10000
292 COMB TOTAL PT PRIMARY MOMENT

FIGURE 6.7-1

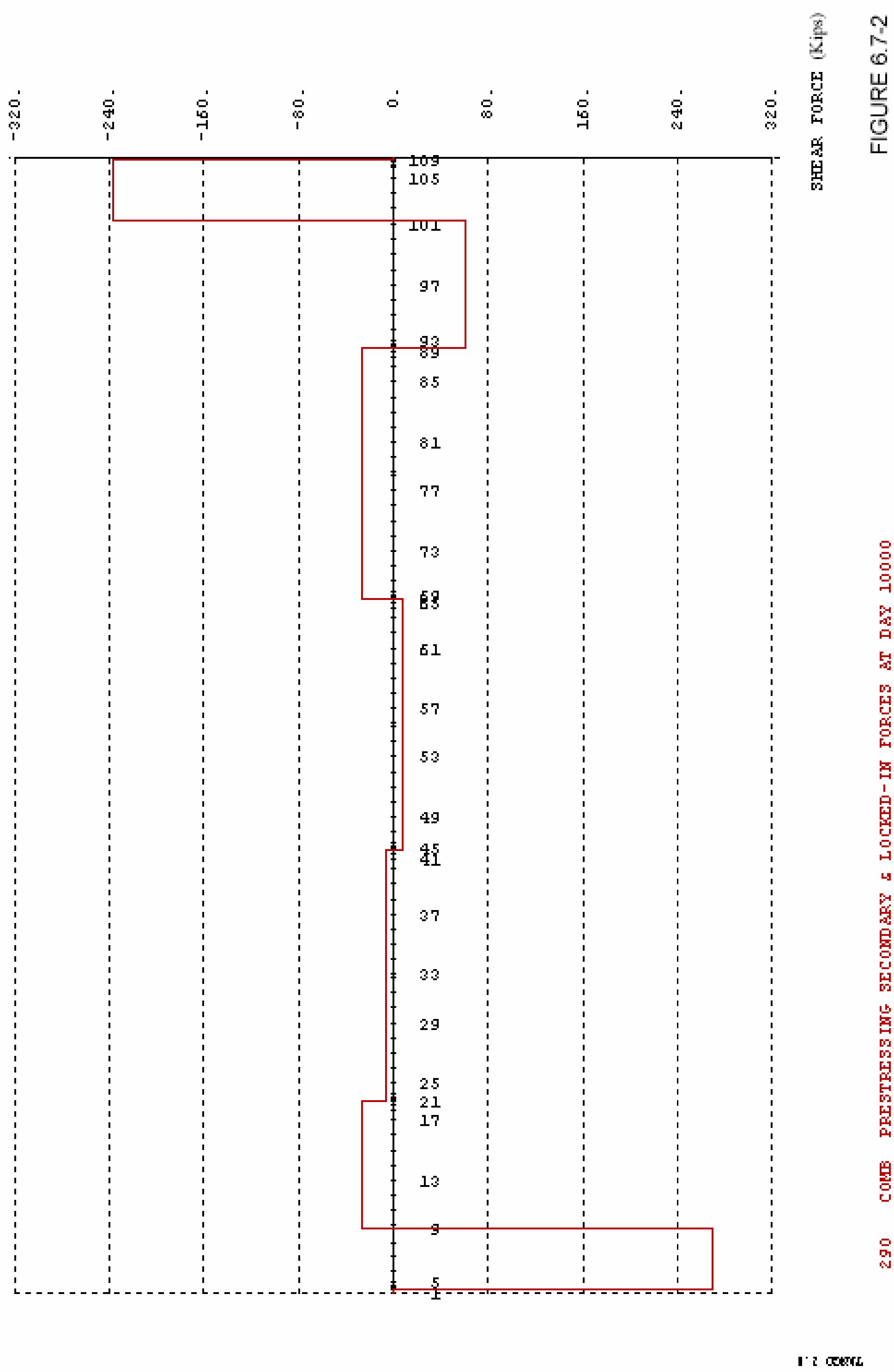

-320.
-240.
-160.
-80.
0.
80.
160.
240.
320.
SHEAR FORCE (Kips)
290 COMB PRESTRESSING SECONDARY & LOCKED-IN FORCES AT DAY 10000

FIGURE 6.7-2

6.8 Summary of Design Forces

As mentioned previously, a comparison of service live load forces was conducted for *AASHTO LRFD* and the *AASHTO Standard Specification*. This was done to get an idea of how much larger forces will be for the HL-93 loading. At maximum locations, the differences in positive and negative moments were 50% and 30% respectively. The difference in shear was 40%.

Even though these numbers represent large differences, for the span lengths under consideration live load only constitutes approximately 25% of the total factored load. This occurrence combined with lower ultimate load factors used by *AASHTO LRFD* will bring the ultimate limit states for the two codes very close to one another.

The results of the different load combination envelopes can be observed in Figures 6.8-1 to 6.8-7. It is interesting to note that the negative bending moments of the three groups only differ by 5%, with the largest value coming from the *AASHTO Standard Specification* HS25-44 loading. The positive bending moments of the HL-93 load combination are approximately 7% higher than the HS20-44 load combination, while the HS25-44 load combination is about 12% higher than the HS20-44 load combination. The shear forces of the HL-93 load combination are comparable to the HS20-44 load combination, while the HS25-44 shear force is about 6% higher than the HL-93 and HS20-44 load combinations.

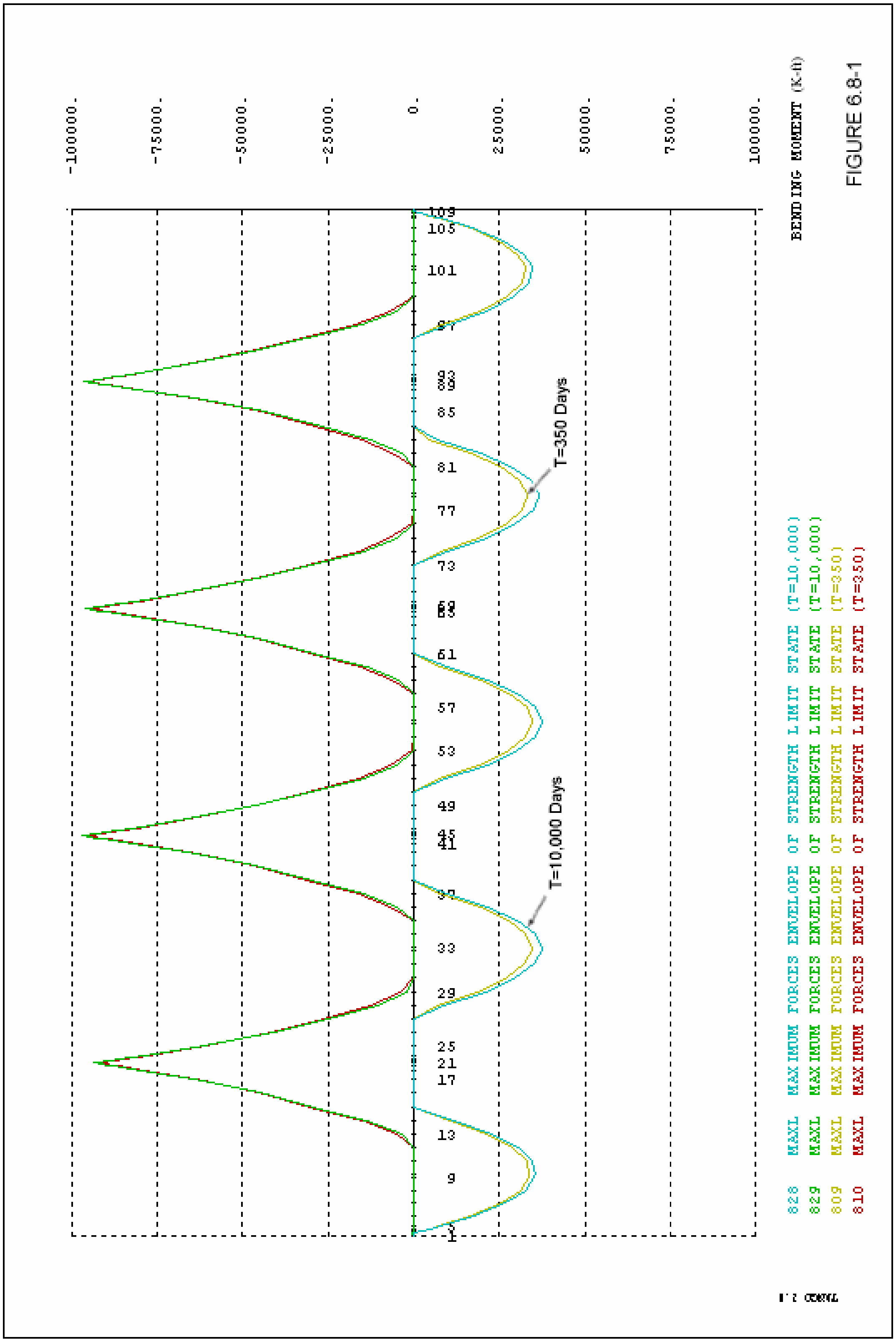

BENDING MOMENT (K-ft)
-100000.
-75000.
-50000.
-25000.
0.
25000.
50000.
75000.
100000.
T=350 Days
T=10,000 Days
828 MAXL MAXIMUM FORCES ENVELOPE OF STRENGTH LIMIT STATE (T=10,000)
829 MAXL MAXIMUM FORCES ENVELOPE OF STRENGTH LIMIT STATE (T=10,000)
809 MAXL MAXIMUM FORCES ENVELOPE OF STRENGTH LIMIT STATE (T=350)
810 MAXL MAXIMUM FORCES ENVELOPE OF STRENGTH LIMIT STATE (T=350)

FIGURE 6.8-1

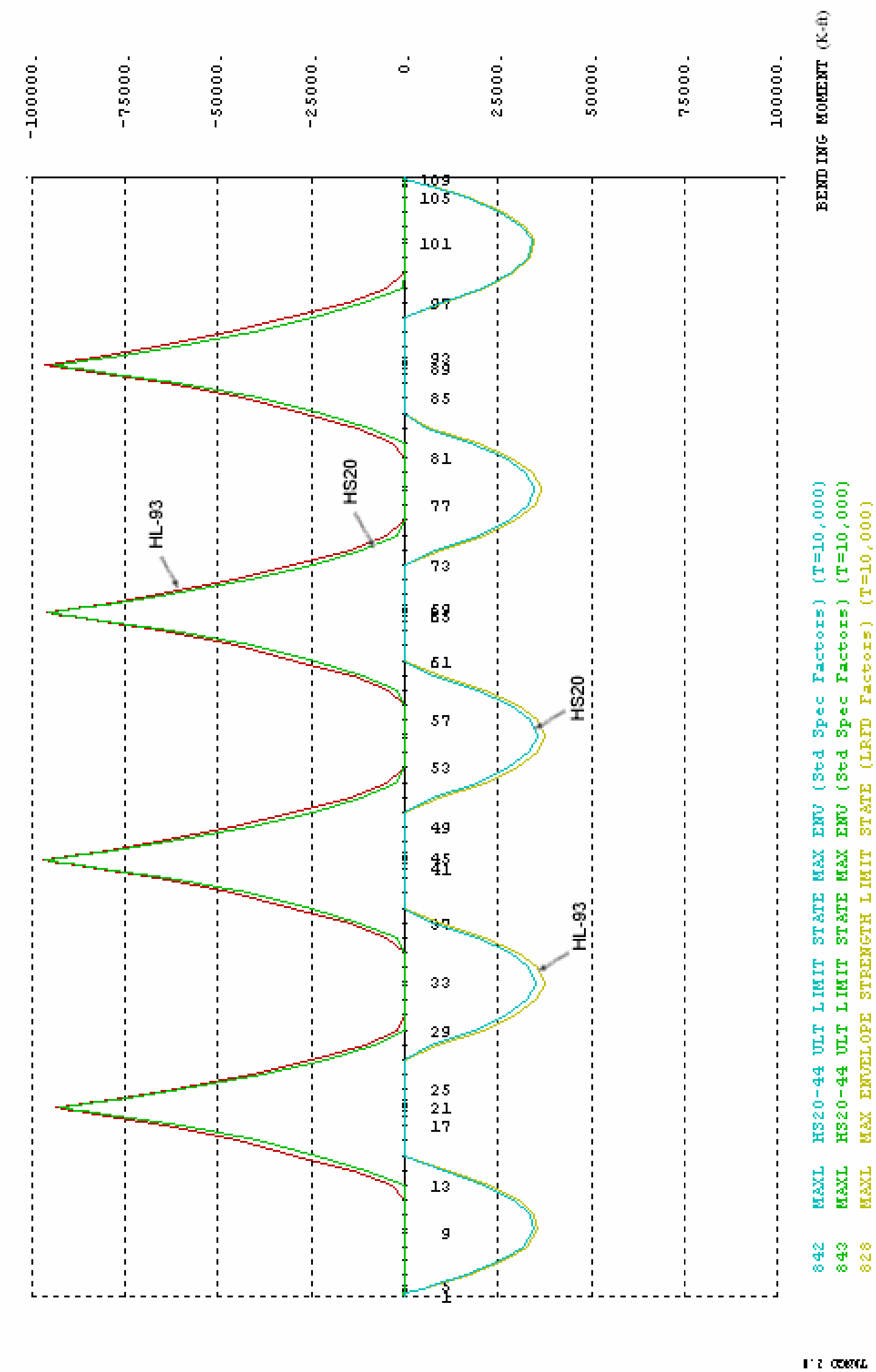

-100000.
-75000.
-50000.
-25000.
0.
25000.
50000.
75000.
100000.
BENDING MOMENT (K-ft)
HL-93
HS20
HS20
HL-93
842 MAXL HS20-44 ULT LIMIT STATE MAX ENV (Std Spec Factors) (T=10,000)
843 MAXL HS20-44 ULT LIMIT STATE MAX ENV (Std Spec Factors) (T=10,000)
828 MAXL MAX ENVELOPE STRENGTH LIMIT STATE (LRFD Factors) (T=10,000)
829 MAXL MAX ENVELOPE STRENGTH LIMIT STATE (LRFD Factors) (T=10,000)

FIGURE 6.8-2

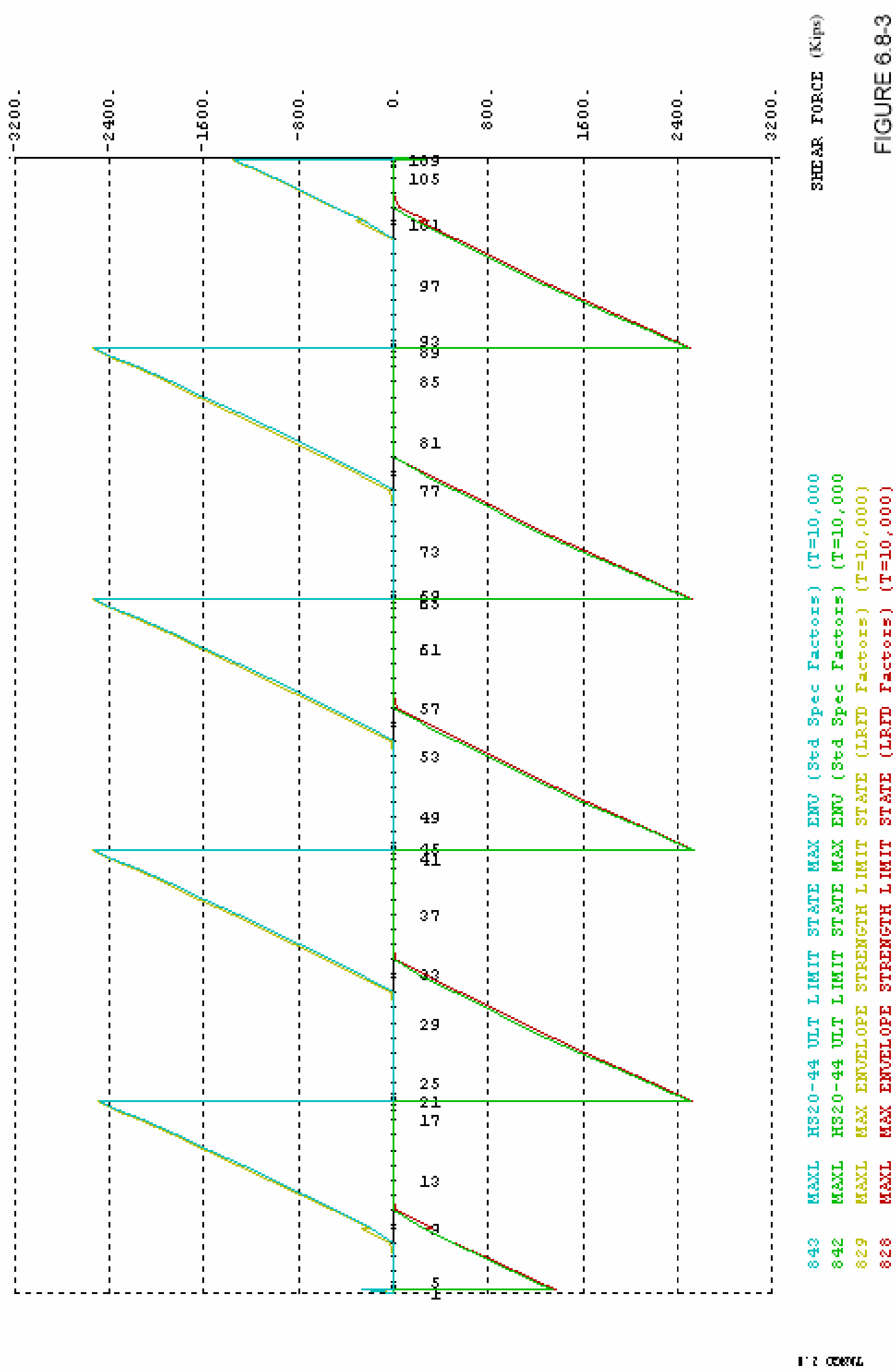

-3200.
-2400.
-1600.
-800.
0.
800.
1600.
2400.
3200.
SHEAR FORCE (Kips)
843 MAXL HS20-44 ULT LIMIT STATE MAX ENV (Std Spec Factors) (T=10,000
842 MAXL HS20-44 ULT LIMIT STATE MAX ENV (Std Spec Factors) (T=10,000
829 MAXL MAX ENVELOPE STRENGTH LIMIT STATE (LRFD Factors) (T=10,000)
828 MAXL MAX ENVELOPE STRENGTH LIMIT STATE (LRFD Factors) (T=10,000)

FIGURE 6.8-3

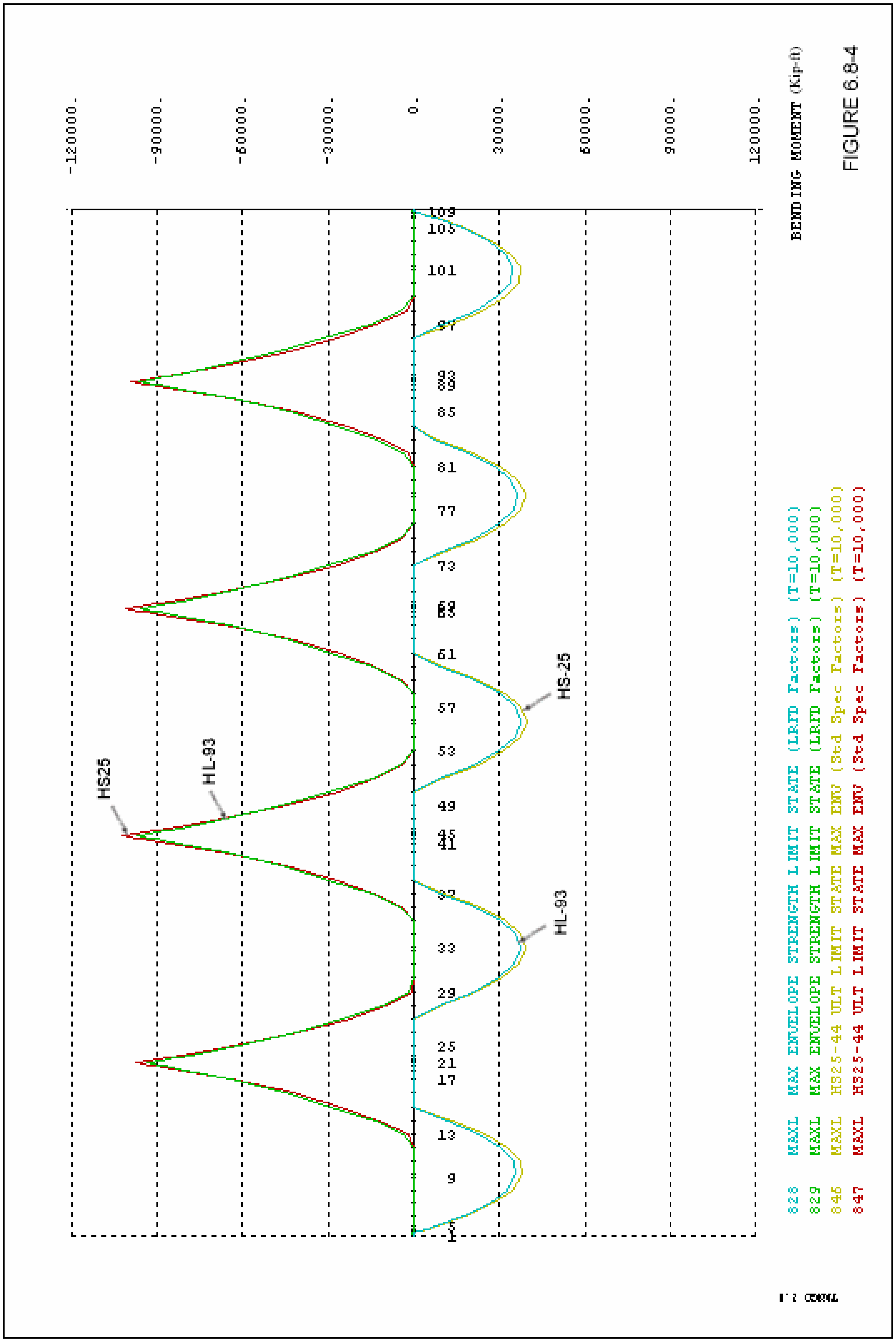
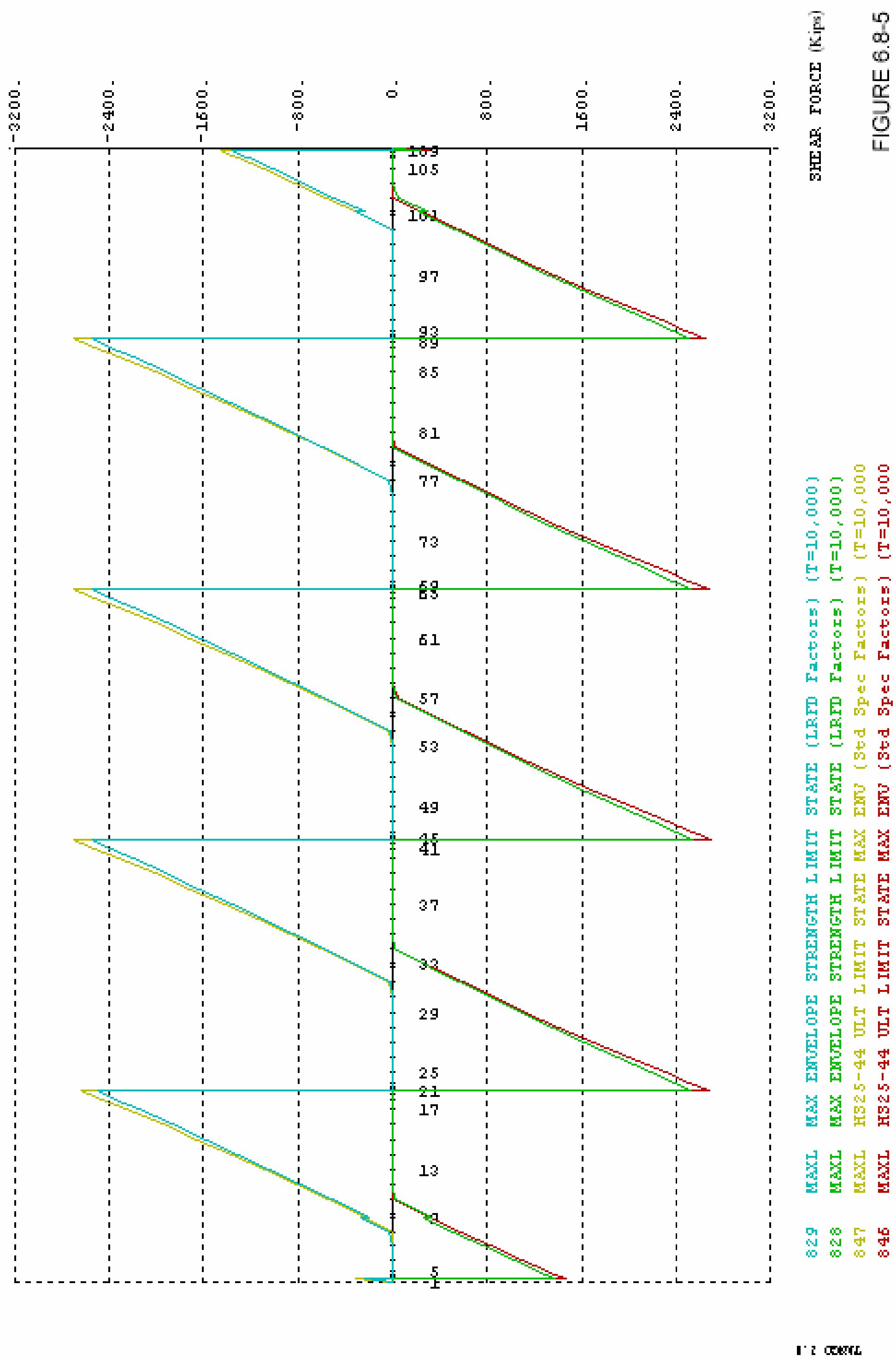
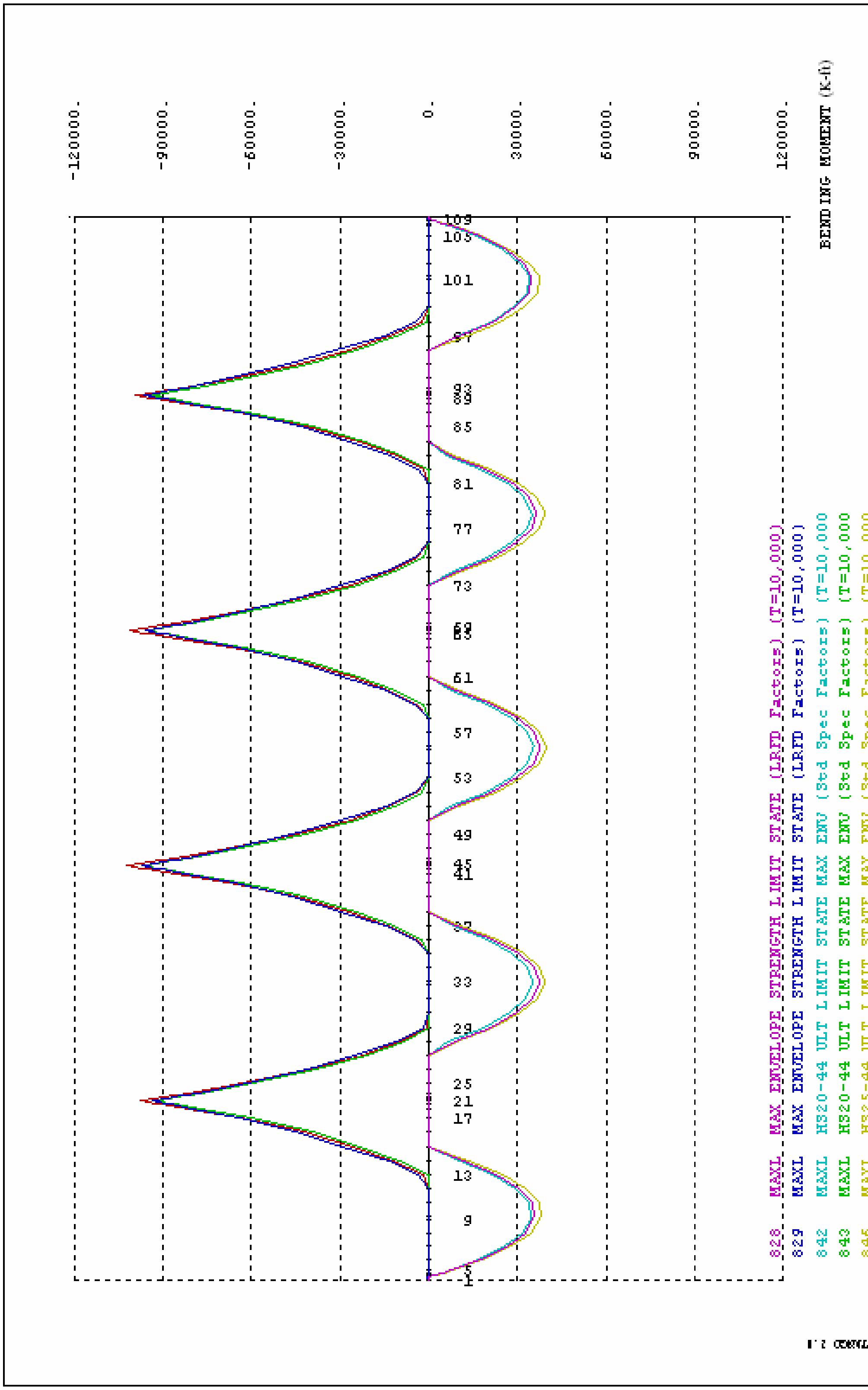

-120000.
-90000.
-60000.
-30000.
0.
30000.
60000.
90000.
120000.
BENDING MOMENT (Kip-ft)
HS25
HL-93
HS-25
HL-93
828 MAXL MAX ENVELOPE STRENGTH LIMIT STATE (LRFD Factors) (T=10,000)
829 MAXL MAX ENVELOPE STRENGTH LIMIT STATE (LRFD Factors) (T=10,000)
846 MAXL HS25-44 ULT LIMIT STATE MAX ENV (Std Spec Factors) (T=10,000)
847 MAXL HS25-44 ULT LIMIT STATE MAX ENV (Std Spec Factors) (T=10,000)

FIGURE 6.8-4



SHEAR FORCE (Kips)
-3200.
-2400.
-1600.
-800.
0.
800.
1600.
2400.
3200.
829 MAXL MAX ENVELOPE STRENGTH LIMIT STATE (LRFD Factors) (T=10,000)
828 MAXL MAX ENVELOPE STRENGTH LIMIT STATE (LRFD Factors) (T=10,000)
847 MAXL HS25-44 ULT LIMIT STATE MAX ENV (Std Spec Factors) (T=10,000
846 MAXL HS25-44 ULT LIMIT STATE MAX ENV (Std Spec Factors) (T=10,000

FIGURE 6.8-5



-120000.
-90000.
-60000.
-30000.
0.
30000.
60000.
90000.
120000.
BENDING MOMENT (K-ft)
828 MAXL MAX ENVELOPE STRENGTH LIMIT STATE (LRFD Factors) (T=10,000)
829 MAXL MAX ENVELOPE STRENGTH LIMIT STATE (LRFD Factors) (T=10,000)
842 MAXL HS20-44 ULT LIMIT STATE MAX ENV (Std Spec Factors) (T=10,000
843 MAXL HS20-44 ULT LIMIT STATE MAX ENV (Std Spec Factors) (T=10,000
846 MAXL HS25-44 ULT LIMIT STATE MAX ENV (Std Spec Factors) (T=10,000
847 MAXL HS25-44 ULT LIMIT STATE MAX ENV (Std Spec Factors) (T=10,000

FIGURE 6.8-6

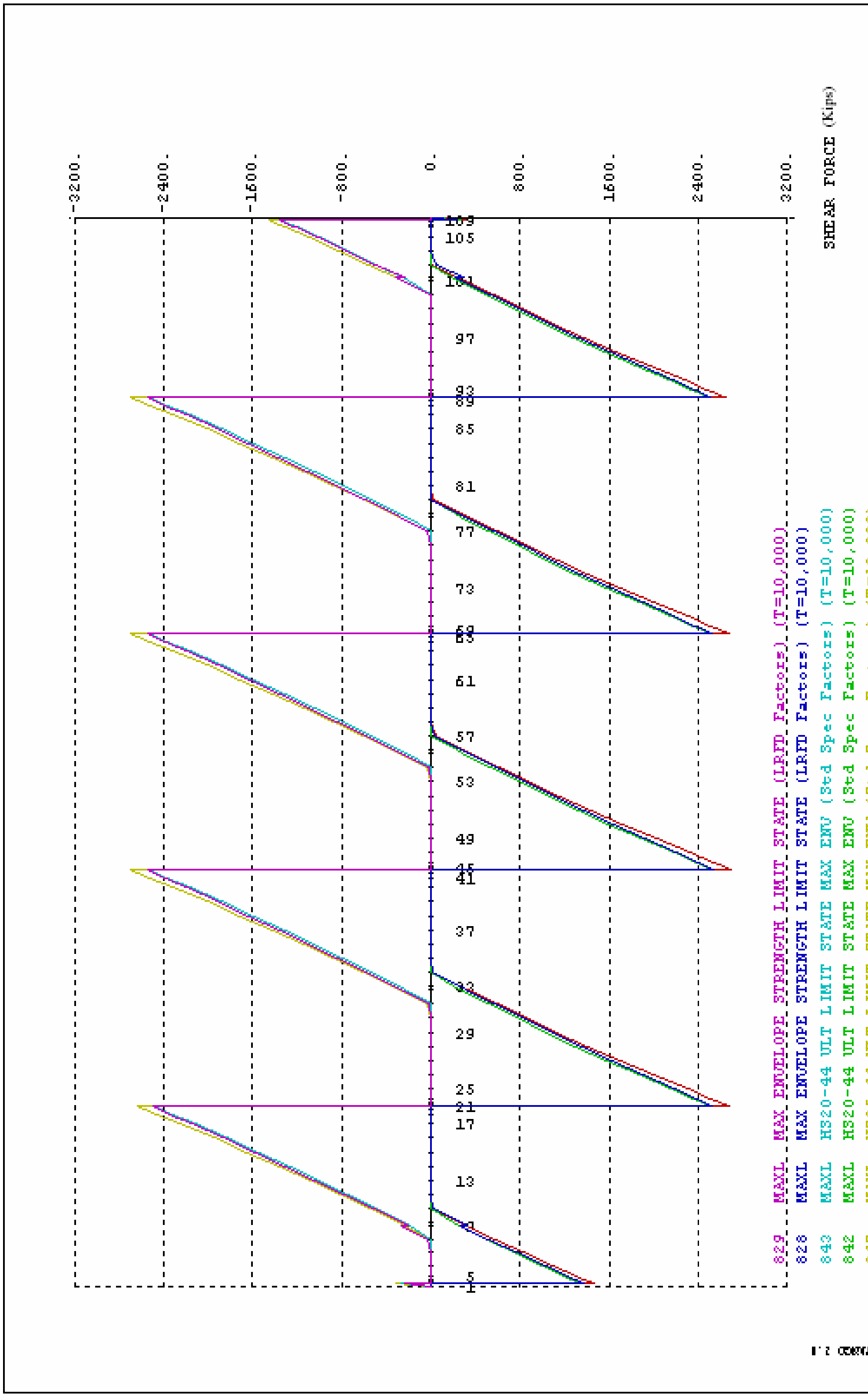

SHEAR FORCE (Kips)
-3200. -2400. -1600. -800. 0. 800. 1600. 2400. 3200.
829 MAXL MAX ENVELOPE STRENGTH LIMIT STATE (LRFD Factors) (T=10,000)
828 MAXL MAX ENVELOPE STRENGTH LIMIT STATE (LRFD Factors) (T=10,000)
843 MAXL HS20-44 ULT LIMIT STATE MAX ENV (Std Spec Factors) (T=10,000)
842 MAXL HS20-44 ULT LIMIT STATE MAX ENV (Std Spec Factors) (T=10,000)
847 MAXL HS25-44 ULT LIMIT STATE MAX ENV (Std Spec Factors) (T=10,000)
846 MAXL HS25-44 ULT LIMIT STATE MAX ENV (Std Spec Factors) (T=10,000)

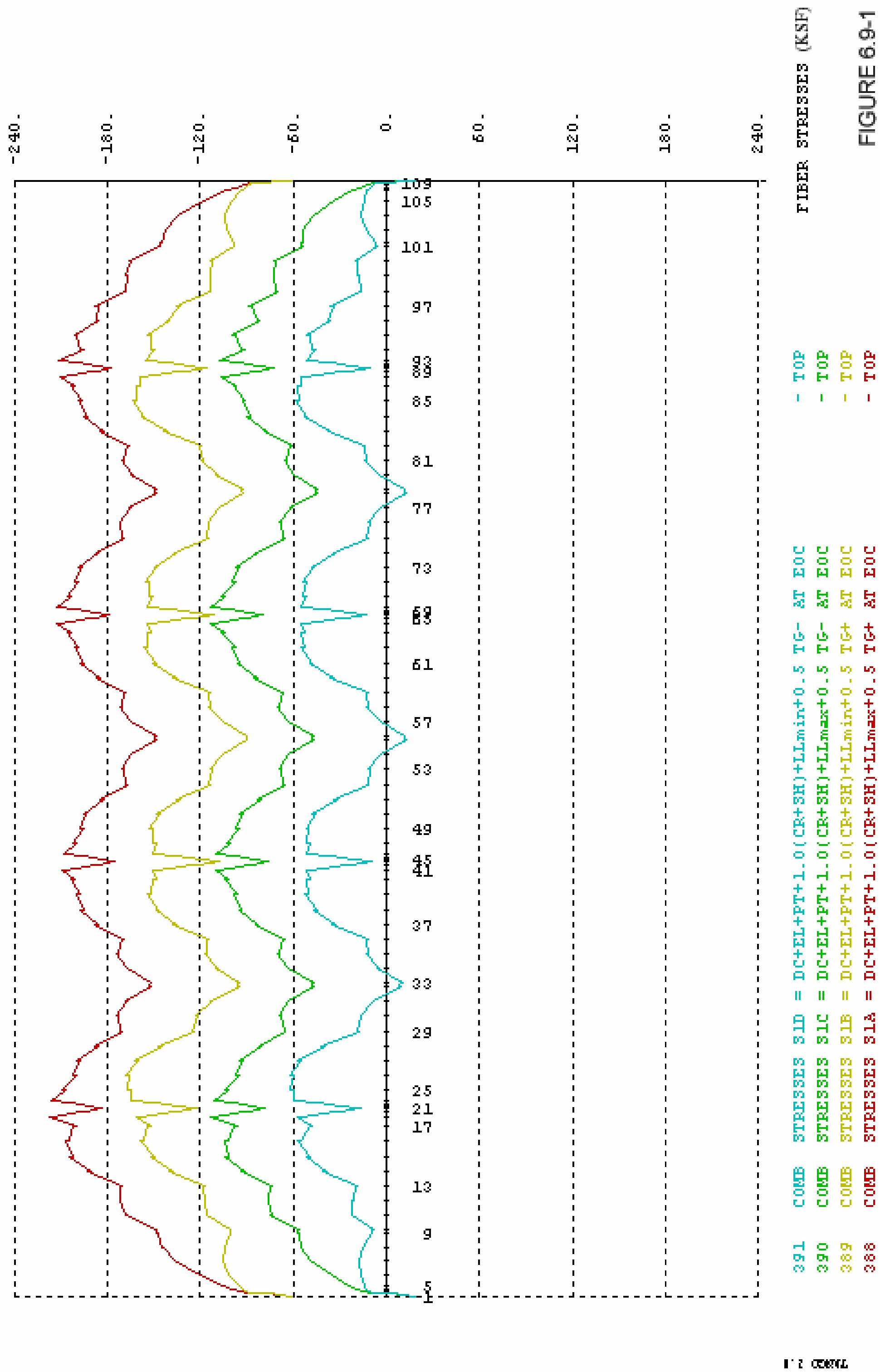
FIGURE 6.8-7

6.9 Service Limit State Design

Service limit state design of the superstructure requires a stress check for three load combinations. These consist of Service Limit State I, Service Limit State III, and a special load case for segmental bridges. Service Limit State III allows tension to be evaluated using a 0.8 live load factor, while Service Limit State I checks compression with a 1.0 live load factor. In combination with these three limit states, a non-linear temperature gradient will be applied. For Service Limit States I & III, which use maximum live load influence, LRFD recommends a factor of 0.5 for temperature gradient in lieu of project-specific data. For the special load case applying to segmental bridges, temperature gradient receives a load factor of 1.0, since live load is not included. For a description of this load case, see LRFD Equation 3.4.1-2.

It is important to note that although the special load case may not control at locations where large amounts of post-tensioning are present, it may indeed control at locations where live load effects are small or at locations outside of the precompressed tensile zone. Such locations for this example include tension in the top of closure pours and compression in the top of the box at pier locations. For this example, tendons were added in the top of the box crossing the closure pour to counteract tension produced by the bottom of the box being warmer than the top.

Results from the service stress load combinations can be referenced in Figures 6.9-1 to 6.9-12. It can be seen that small amounts of tension exist at nodes 8 and 104 under Service Load Case III at day 10,000. Due to the conservative boundary conditions assumed while erecting the end spans, this tension is acceptable.



-240.
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65
61
57
53
49
45
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29
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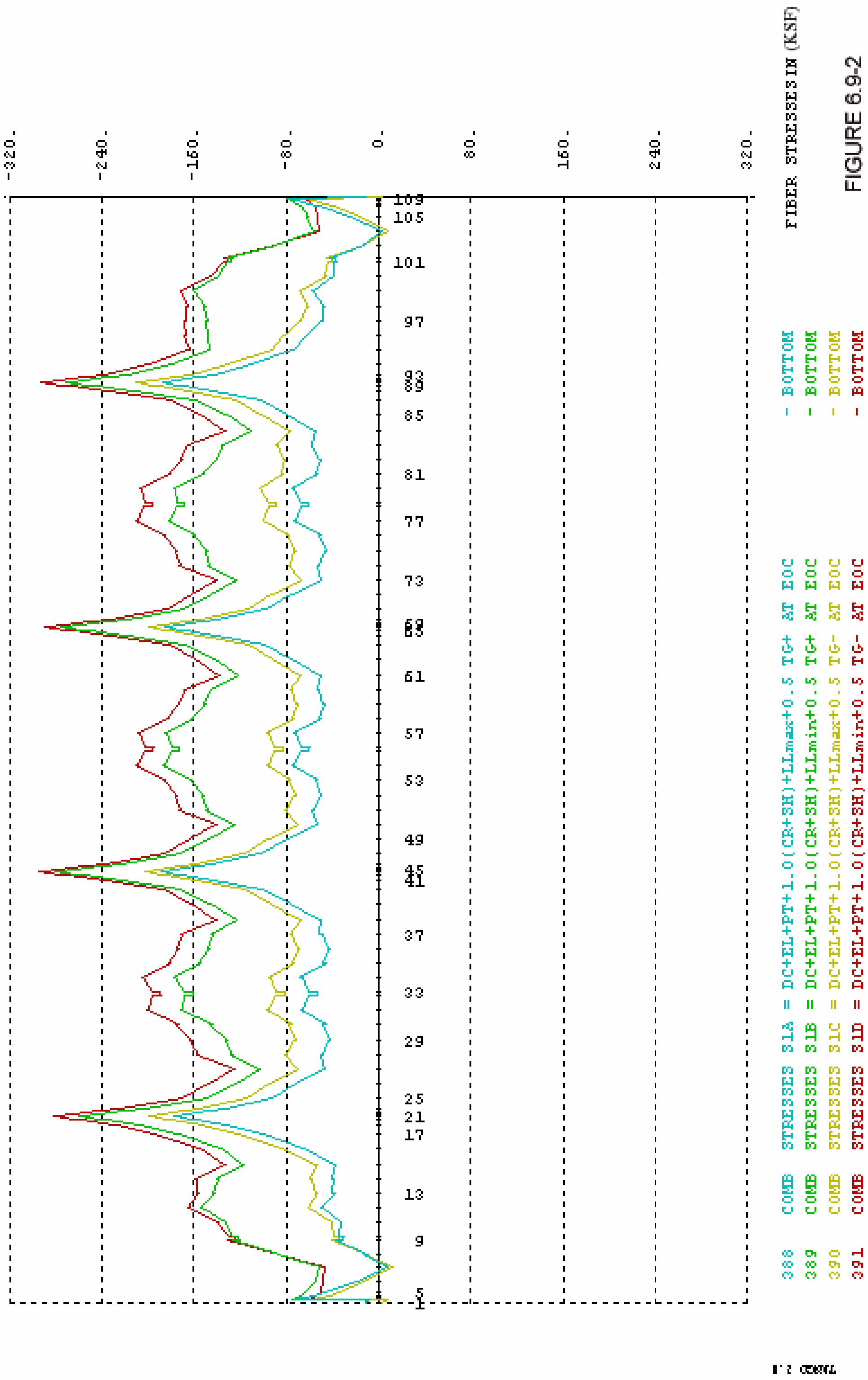
21
17
13
9
5
1
FIBER STRESSES (KSF)
391 COMB STRESSES 31D = DC+EL+PT+1.0(CR+SH)+LLmin+0.5 TG- AT EOC - TOP
390 COMB STRESSES 31C = DC+EL+PT+1.0(CR+SH)+LLmax+0.5 TG- AT EOC - TOP
389 COMB STRESSES 31B = DC+EL+PT+1.0(CR+SH)+LLmin+0.5 TG+ AT EOC - TOP
388 COMB STRESSES 31A = DC+EL+PT+1.0(CR+SH)+LLmax+0.5 TG+ AT EOC - TOP

FIGURE 6.9-1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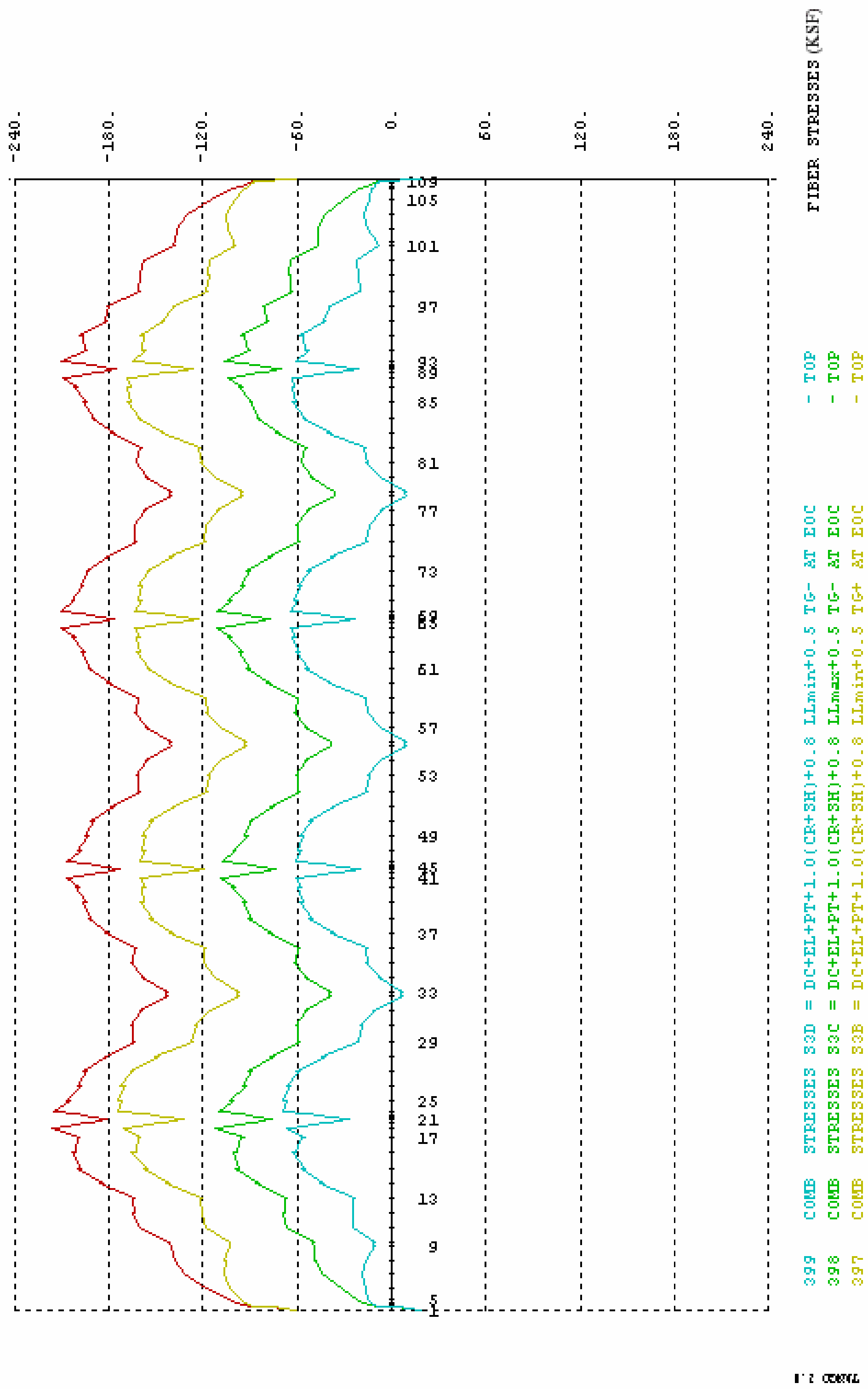
-160.
-80.
0.
80.
160.
240.
320.
388 COMB STRESSES 31A = DC+EL+PT+1.0(CR+SH)+LLmax+0.5 TG+ AT EOC - BOTTOM
389 COMB STRESSES 31B = DC+EL+PT+1.0(CR+SH)+LLmin+0.5 TG+ AT EOC - BOTTOM
390 COMB STRESSES 31C = DC+EL+PT+1.0(CR+SH)+LLmax+0.5 TG- AT EOC - BOTTOM
391 COMB STRESSES 31D = DC+EL+PT+1.0(CR+SH)+LLmin+0.5 TG- AT EOC - BOTTOM
FIBER STRESSES IN (KSF)

FIGURE 6.9-2


-240.
-180.
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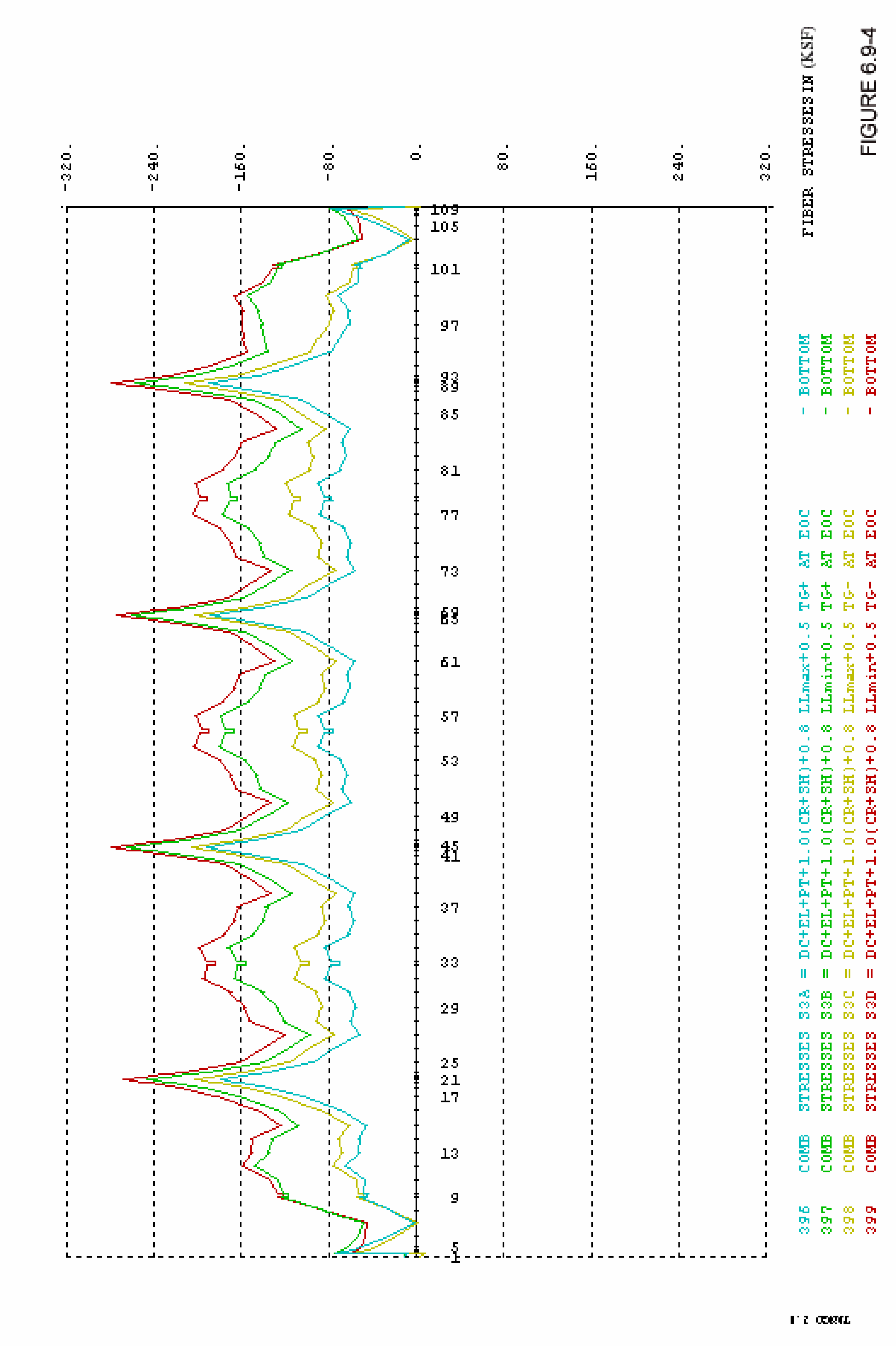
-60.
0.
60.
120.
180.
240.
FIBER STRESSES (KSF)
399 COMB STRESSES S3D = DC+EL+PT+1.0(CR+SH)+0.8 LLmin+0.5 TG- AT EOC - TOP
398 COMB STRESSES S3C = DC+EL+PT+1.0(CR+SH)+0.8 LLmax+0.5 TG- AT EOC - TOP
397 COMB STRESSES S3B = DC+EL+PT+1.0(CR+SH)+0.8 LLmin+0.5 TG+ AT EOC - TOP
396 COMB STRESSES S3A = DC+EL+PT+1.0(CR+SH)+0.8 LLmax+0.5 TG+ AT EOC - TOP

FIGURE 6.9-3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37
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FIBER STRESSES IN (KSF)
396 COMB STRESSES 33A = DC+EL+PT+1.0(CR+SH)+0.8 LLmax+0.5 TG+ AT EOC - BOTTOM
397 COMB STRESSES 33B = DC+EL+PT+1.0(CR+SH)+0.8 LLmin+0.5 TG+ AT EOC - BOTTOM
398 COMB STRESSES 33C = DC+EL+PT+1.0(CR+SH)+0.8 LLmax+0.5 TG- AT EOC - BOTTOM
399 COMB STRESSES 33D = DC+EL+PT+1.0(CR+SH)+0.8 LLmin+0.5 TG- AT EOC - BOTTOM

FIGURE 6.9-4

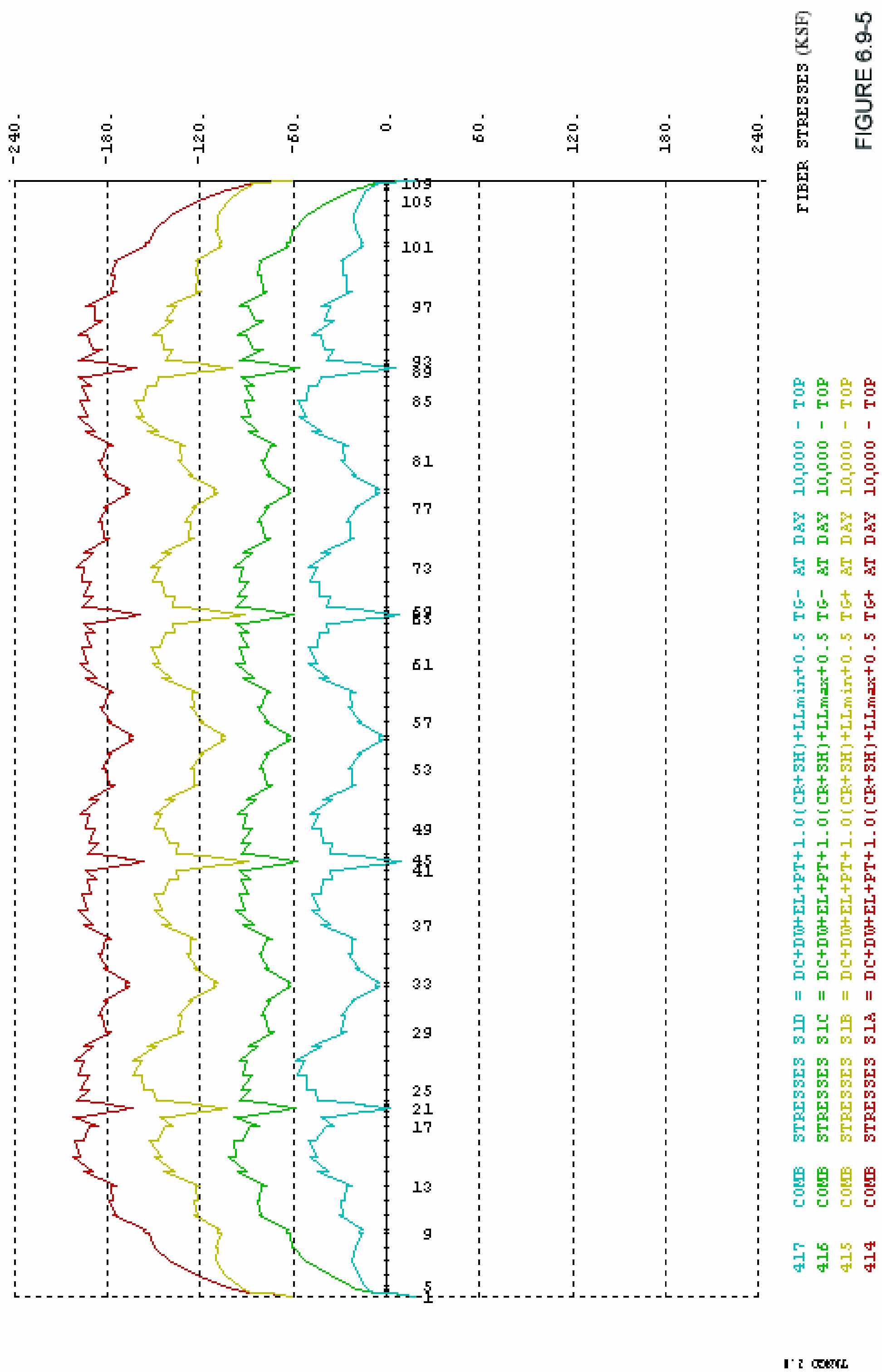
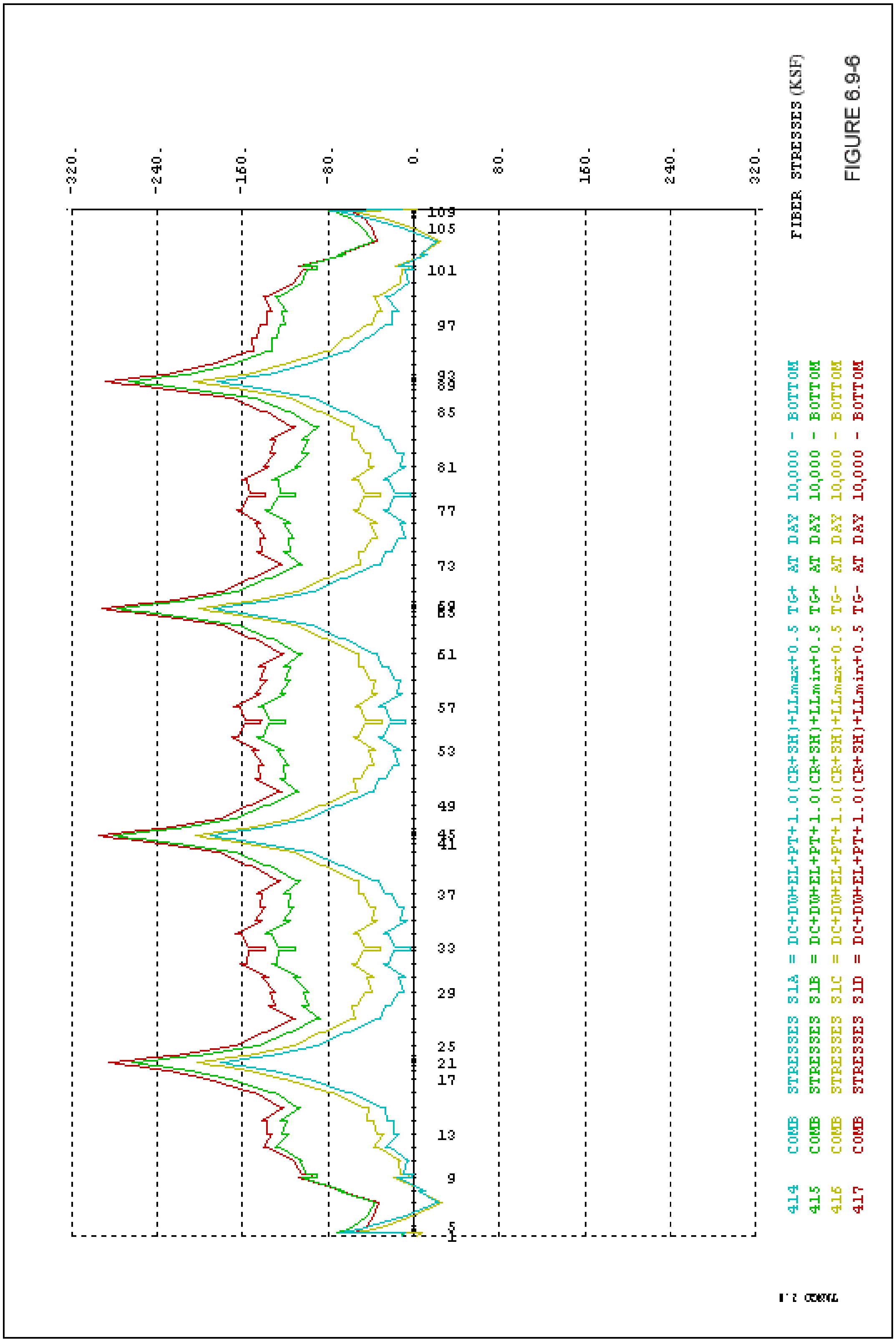

FIBER STRESSES (KSF)
417 COMB STRESSES 31D = DC+DW+EL+PT+1.0(CR+SH)+LLmin+0.5 TG- AT DAY 10,000 - TOP
416 COMB STRESSES 31C = DC+DW+EL+PT+1.0(CR+SH)+LLmax+0.5 TG- AT DAY 10,000 - TOP
415 COMB STRESSES 31B = DC+DW+EL+PT+1.0(CR+SH)+LLmin+0.5 TG+ AT DAY 10,000 - TOP
414 COMB STRESSES 31A = DC+DW+EL+PT+1.0(CR+SH)+LLmax+0.5 TG+ AT DAY 10,000 - TOP

FIGURE 6.9-5



-320.
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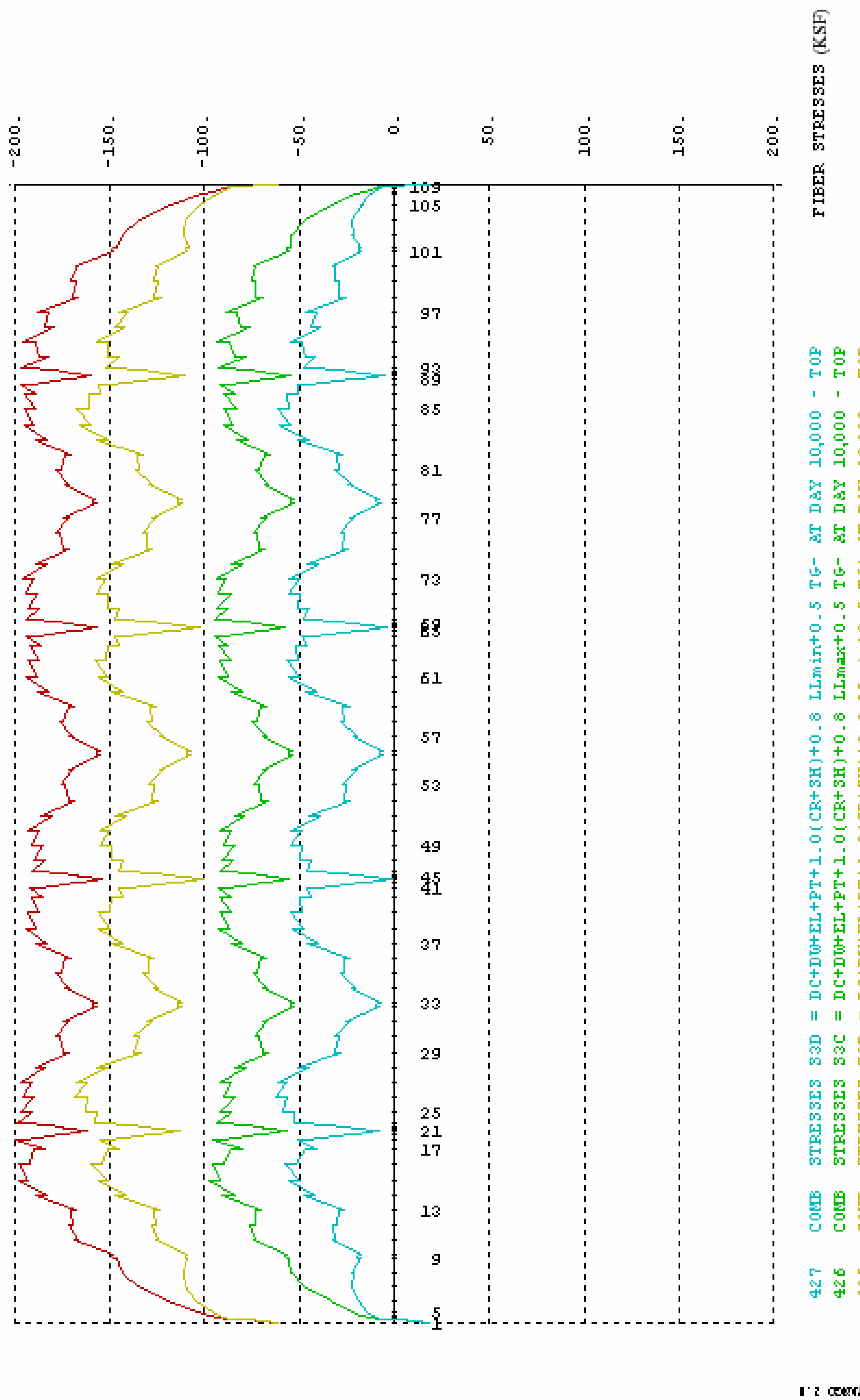
93
97
101
105
109
FIBER STRESSES (KSF)
414 COMB STRESSES 31A = DC+DW+EL+PT+1.0(CR+SH)+LLmax+0.5 TG+ AT DAY 10,000 - BOTTOM
415 COMB STRESSES 31B = DC+DW+EL+PT+1.0(CR+SH)+LLmin+0.5 TG+ AT DAY 10,000 - BOTTOM
416 COMB STRESSES 31C = DC+DW+EL+PT+1.0(CR+SH)+LLmax+0.5 TG- AT DAY 10,000 - BOTTOM
417 COMB STRESSES 31D = DC+DW+EL+PT+1.0(CR+SH)+LLmin+0.5 TG- AT DAY 10,000 - BOTTOM

FIGURE 6.9-6



-200.
-150.
-100.
-50.
0.
50.
100.
150.
200.
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45
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53
57
61
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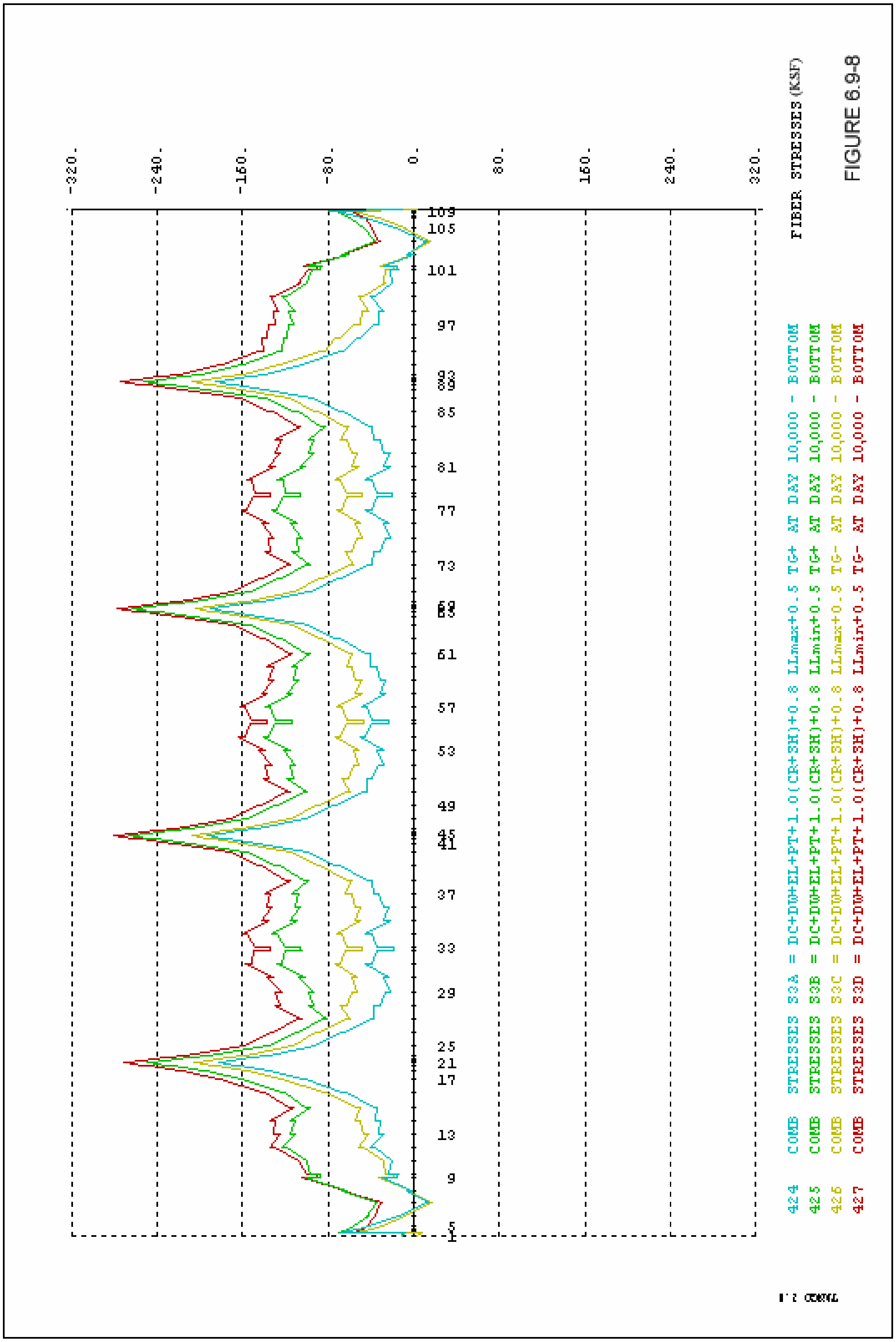
69
73
77
81
85
89
93
97
101
105
109
FIBER STRESSES (KSF)
427 COMB STRESSES 33D = DC+DW+EL+PT+1.0(CR+SH)+0.8 LLmin+0.5 TG- AT DAY 10,000 - TOP
426 COMB STRESSES 33C = DC+DW+EL+PT+1.0(CR+SH)+0.8 LLmax+0.5 TG- AT DAY 10,000 - TOP
425 COMB STRESSES 33B = DC+DW+EL+PT+1.0(CR+SH)+0.8 LLmin+0.5 TG+ AT DAY 10,000 - TOP
424 COMB STRESSES 33A = DC+DW+EL+PT+1.0(CR+SH)+0.8 LLmax+0.5 TG+ AT DAY 10,000 - TOP

FIGURE 6.9-7



FIBER STRESSES (KSF)
424 COMB STRESSES 33A = DC+DW+EL+PT+1.0(CR+SH)+0.8 LLmax+0.5 TG+ AT DAY 10,000 - BOTTOM
425 COMB STRESSES 33B = DC+DW+EL+PT+1.0(CR+SH)+0.8 LLmin+0.5 TG+ AT DAY 10,000 - BOTTOM
426 COMB STRESSES 33C = DC+DW+EL+PT+1.0(CR+SH)+0.8 LLmax+0.5 TG- AT DAY 10,000 - BOTTOM
427 COMB STRESSES 33D = DC+DW+EL+PT+1.0(CR+SH)+0.8 LLmin+0.5 TG- AT DAY 10,000 - BOTTOM

FIGURE 6.9-8

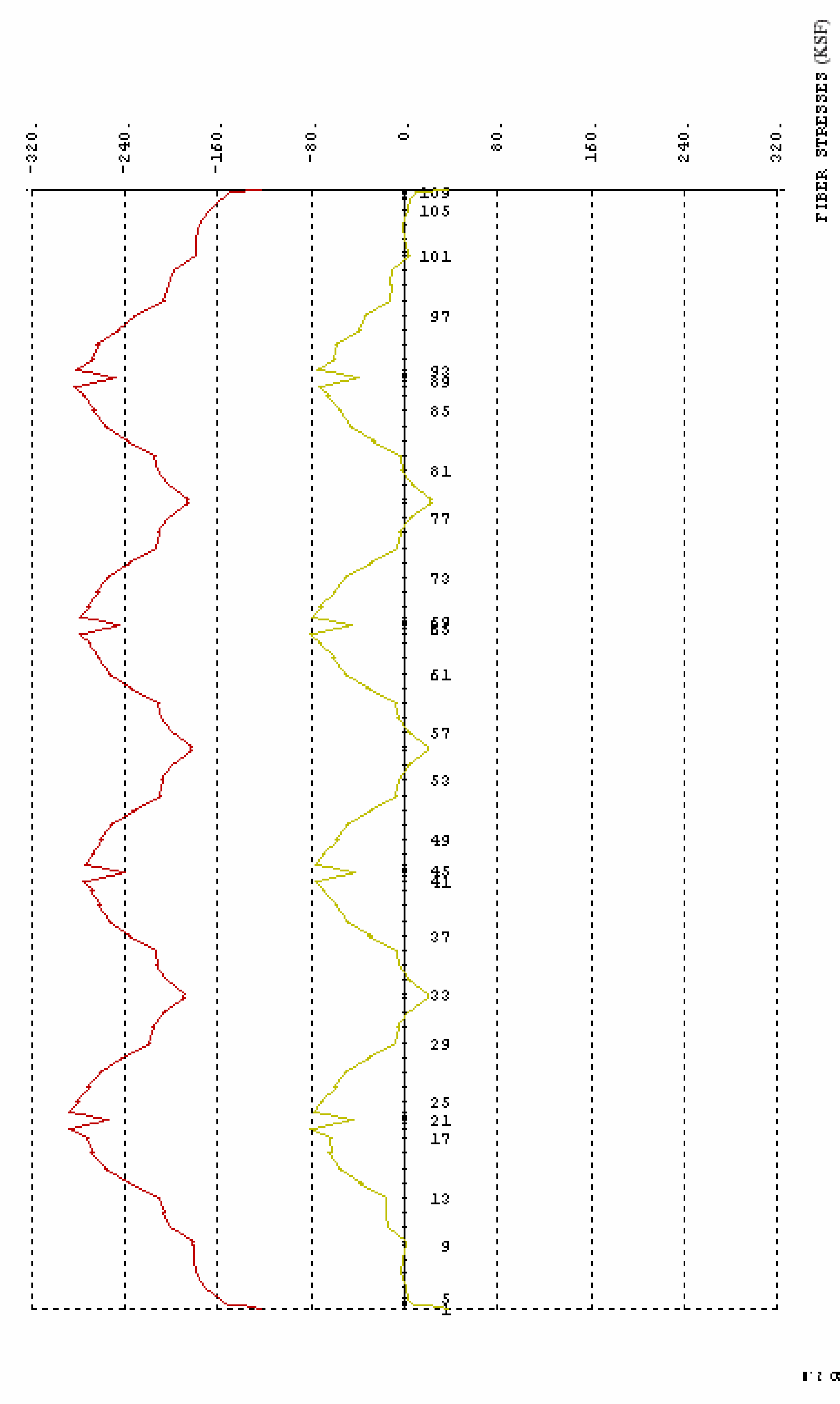
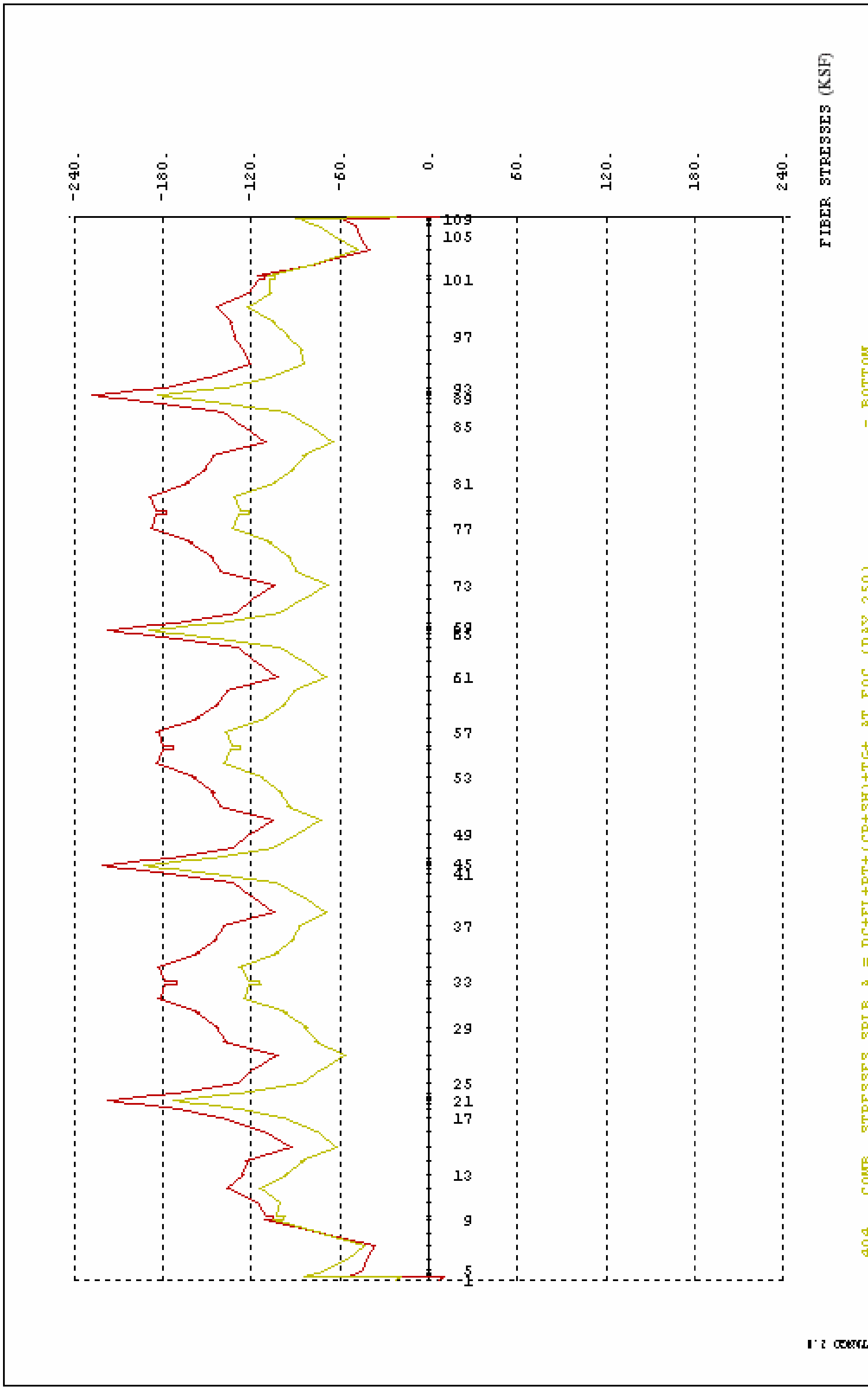

FIBER STRESSES (KSF)
-320.
-240.
-160.
-80.
0.
80.
160.
240.
320.
405 COMB STRESSES SPLB B = DC+EL+PT+(CR+SH)+TG- AT EOC (DAY 350) - TOP
404 COMB STRESSES SPLB A = DC+EL+PT+(CR+SH)+TG+ AT EOC (DAY 350) - TOP

FIGURE 6.9-9



FIBER STRESSES (KSF)
-240.
-180.
-120.
-60.
0.
60.
120.
180.
240.
404 COMB STRESSES SPLB A = DC+EL+PT+(CR+SH)+TG+ AT EOC (DAY 350) - BOTTOM
405 COMB STRESSES SPLB B = DC+EL+PT+(CR+SH)+TG- AT EOC (DAY 350) - BOTTOM

FIGURE 6.9-10

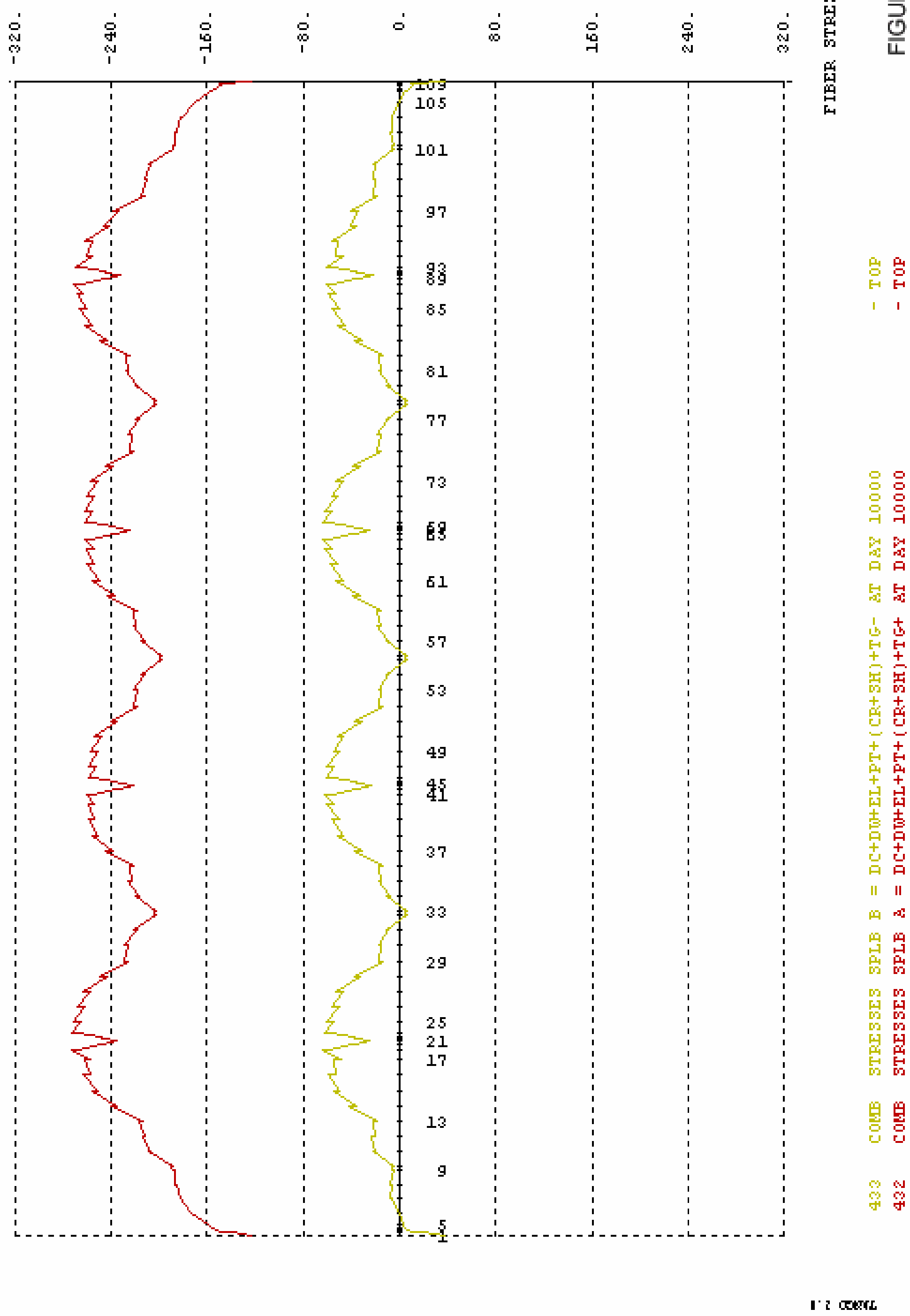

FIBER STRESSES (KSF)
-320.
-240.
-160.
-80.
0.
80.
160.
240.
320.
1
5
9
13
17
21
25
29
33
37
41
45
49
53
57
61
65
69
73
77
81
85
89
93
97
101
105
109
433 COMB STRESSES SPLB B = DC+DW+EL+PT+(CR+SH)+TG- AT DAY 10000 - TOP
432 COMB STRESSES SPLB A = DC+DW+EL+PT+(CR+SH)+TG+ AT DAY 10000 - TOP

FIGURE 6.9-11

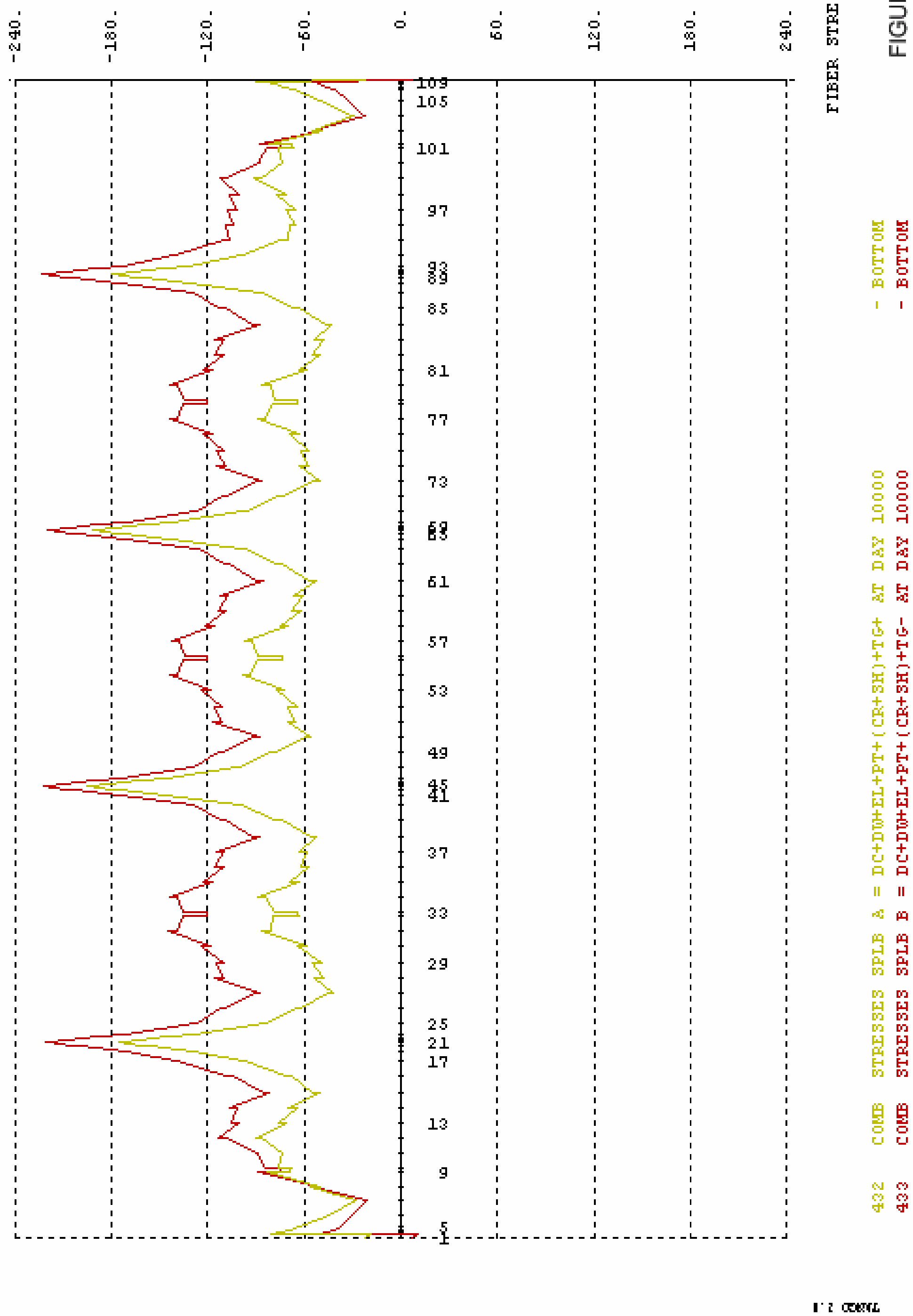

-240.
-180.
-120.
-60.
0.
60.
120.
180.
240.
FIBER STRESSES (KSF)
432 COMB STRESSES SPLB A = DC+DW+EL+PT+(CR+SH)+TG+ AT DAY 10000 - BOTTOM
433 COMB STRESSES SPLB B = DC+DW+EL+PT+(CR+SH)+TG- AT DAY 10000 - BOTTOM

FIGURE 6.9-12

6.10 Principal Tension Stress Check

A principal tensile stress check in shear design is not yet specified by code, but is typically performed as a method to prevent cracking during service load conditions. Stresses are calculated using Mohr's circle to determine principle tension. If the allowable tensile capacity of the concrete is exceeded, diagonal tension cracks may be anticipated. Typically the maximum principal tension stress is limited from $3\sqrt{f'c}$ to $4\sqrt{f'c}$ (psi). Based on information from AASHTO T-5 & T-10 Committee meetings held in June, 2004, principle tension stress will be limited to a value of $3.5\sqrt{f'c}$ for segmental bridges. It is anticipated that this check will be adopted by *AASHTO LRFD Bridge Design Specifications* in the near future. Although it is likely that this check will only be required at the neutral axis of the web, it is recommended that the top slab and web interface location be investigated as well. For this example, $3.5\sqrt{f'c}$ tension will be used as a maximum allowable value under service loading.

Since principal stress is a function of longitudinal, vertical, and shear stress, it is necessary to determine concurrent moments for the maximum live load shear. It should be noted that high principle stresses commonly occur at interior pier locations, and the HL-93 live load moment corresponding to shear should only use one truck, rather than two as used in calculating negative moment at interior piers. The live load shall also have a load factor of 0.8 similar to Service III Limit State or it would be practically impossible to satisfy principal stresses while the extreme fiber could be in tension.

The maximum principal stresses in this example occurred near the interior piers at the top of the web for final conditions. From analysis at the critical section, the maximum principle tension stress was approximately $4.5\sqrt{f'c}$; larger than the previously discussed limit. For this particular example, vertical post-tensioning bars will be used to control the principal tension stress. Calculations show that (3) 1¼" diameter bars, as shown in Figure 6.10-3, will be needed in each web to reduce principle tension to an acceptable value. The overstress could also be addressed by modifying the cross-section (web thickness) or adding more longitudinal compressive stress (additional strands). The solution presented was deemed acceptable since only a small number of segments will require vertical post-tensioning. A graph of principle stress prior to addition of vertical post-tensioned bars can be seen in Figure 6.10-2.

Principal Tensile Stress Check

$$v = \frac{VQ}{Ib}$$

where

V = Vertical shear force

Q = First moment of an area with respect to C.G. of section

I = Moment of inertia about C.G. of section

b = Perpendicular web thickness

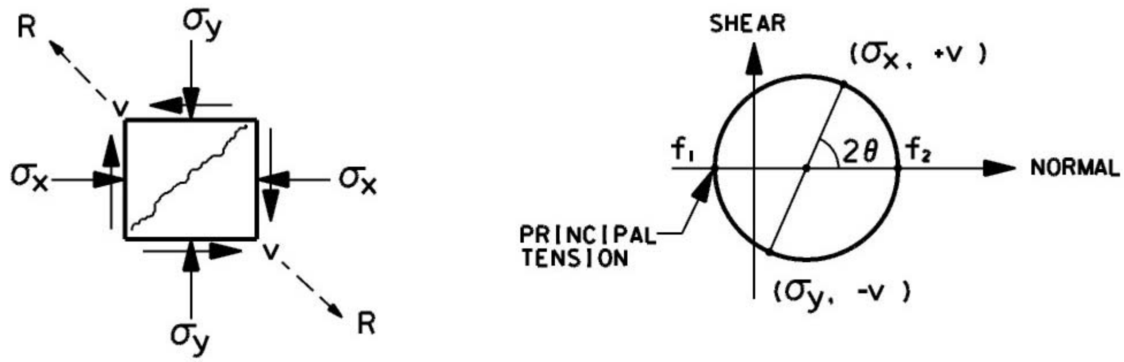


Figure 6.10-1: Principal Stresses and Mohr's Circle

$$f_1 = \frac{\sigma_x + \sigma_y}{2} - \frac{1}{2}\sqrt{4v^2 + \left(\sigma_x - \sigma_y\right)^2}$$

where compression stress is positive

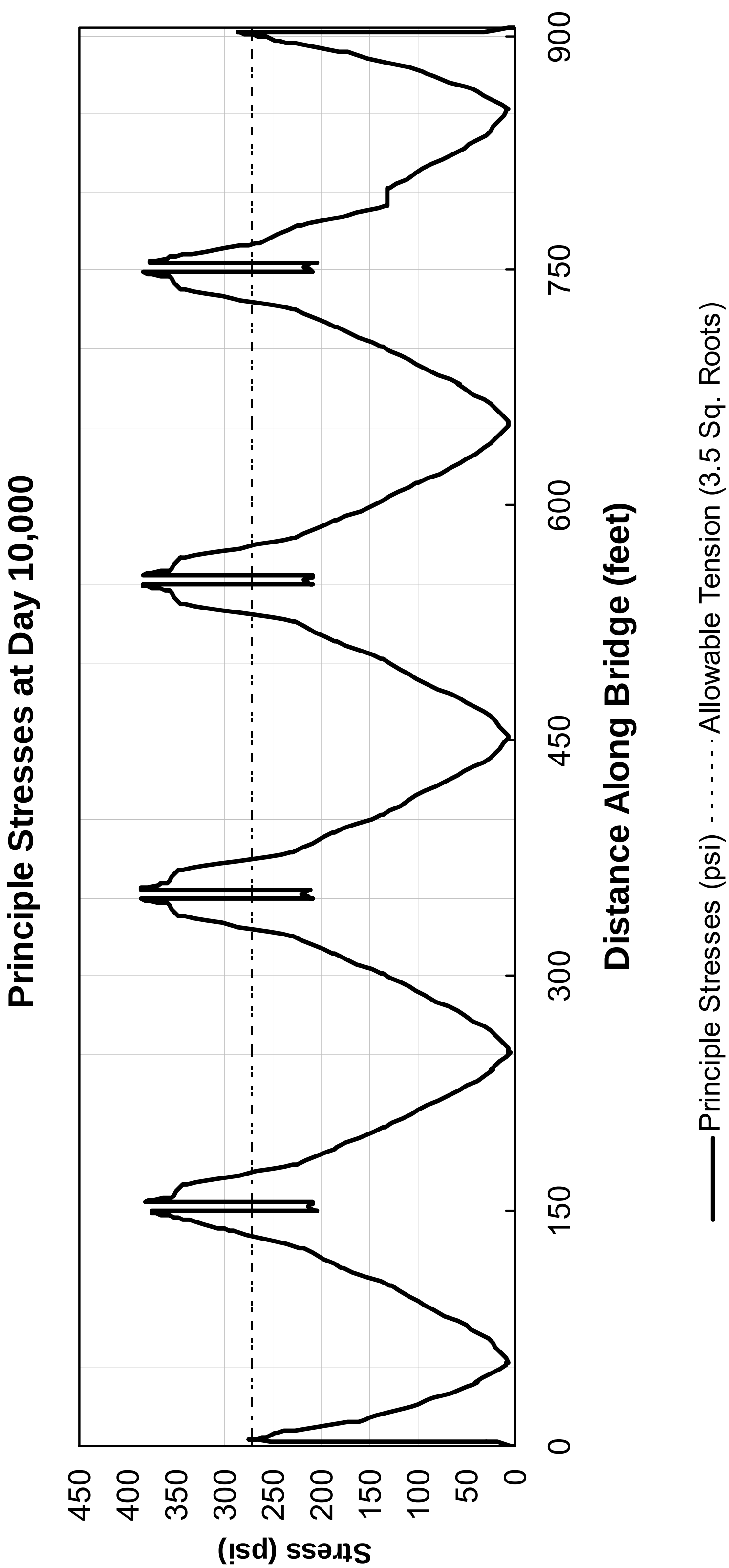
For $\sigma_y = 0$: (at sections where no vertical web post-tensioning is present)

$$v_a = \sqrt{f_a \times \left(f_a + f\right)}$$

where

f_a = Allowable principal tension

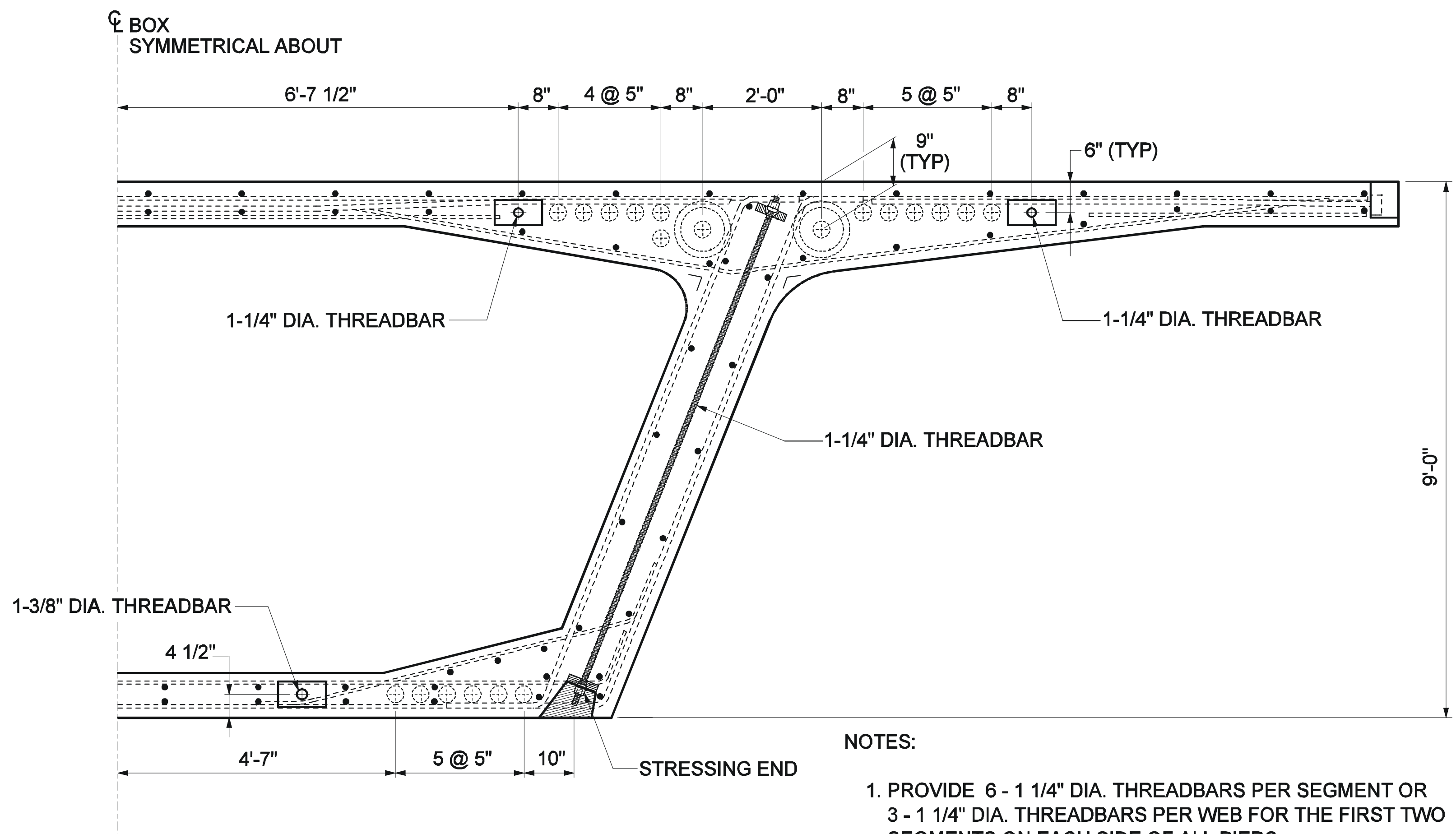
f = Compressive stress at level on web under investigation



Principle Stresses at Day 10,000
Stress (psi)
450
400
350
300
250
200
150
100
50
0
0
150
300
450
600
750
900
Distance Along Bridge (feet)
Principle Stresses (psi)
Allowable Tension (3.5 Sq. Roots)

FIGURE 6.10-2

$$$$$$$DGNSPECIFICATION$$$$$$$ $$$$$$SYTIME$$$$$$



NOTES:

1. PROVIDE 6 - 1 1/4" DIA. THREADBARS PER SEGMENT OR 3 - 1 1/4" DIA. THREADBARS PER WEB FOR THE FIRST TWO SEGMENTS ON EACH SIDE OF ALL PIERS.

2. STRESS EACH BAR TO 75% OF G.U.T.S.

FIGURE 6.10-3

DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS	BALANCED CANTILEVER CONSTRUCTION	SHEAR POST-TENSIONING BARS

6.11 Flexural Strength Check

Once service stresses are satisfied in the superstructure, the limit state of flexural strength must be checked. For most cases with superstructures, Strength Limit State I is the only load combination that needs to be considered. However, for longer spans where the ratio of dead load to live load is large, Strength Limit State IV may control. For this example, the magnitudes of live load force effects are greater than a 25% difference in structural component dead load. Hence, Strength Limit State IV will not control.

The load factor for support settlement and temperature gradient are not provided by LRFD. Rather, they are to be determined on a project-specific basis. In lieu of project-specific data, LRFD recommends using a load factor of 0.0 for temperature gradient. With regard to temperature gradient, the loads imposed result from restrained deformations and should disappear if the reinforcement starts to yield at ultimate. Due to this occurrence, temperature gradient is not considered in strength limit states. Also, support settlements are not considered in this example.

The LRFD specifications require minimum reinforcement equal to that required to resist 1.2 times the cracking moment. All sections in the example satisfy this requirement.

In the following pages of this section, example calculations for ultimate flexural capacity are given for an individual node in the bridge.

Flexural Capacity Design Example

Node number: 42 (Maximum negative moment at pier section joint)

Ultimate moment: $M_u = -90{,}565 kip - ft$ (negative moment, bottom slab is in compression)

$\phi = 0.95$ for bonded tendons

$A_{ps} = 0.217(5 \times 48 + 3 \times 24) = 67.704 in^2$

$b = 196 in$, bottom soffit width

$b_w = 32 in$, effective web width

$h_f = 18 in$, bottom soffit depth

$f_c' = 6 ksi$, compression strength of concrete

$d_p = 102 in$, effective depth of tendons

$\beta_1 = 0.75$, stress block factor

$f_{pu} = 270 ksi$, strand ultimate strength

$k = 0.28$, low relaxation strands

Find compression block depth:

$$c = \frac{A_{ps} f_{pu}}{0.85 f_c' \beta_1 b + k A_{ps} \dfrac{f_{pu}}{d_p}} = \frac{67.704 \times 270}{0.85 \times 6 \times 0.75 \times 196 + 0.28 \times 67.704 \dfrac{270}{102}} = 22.85 in$$

$a = \beta_1 b = 0.75 \times 22.85 = 17.14 in$, less than bottom slab thick., mod. comp. block is rectangular

Find stress in strands at ultimate (per LRFD 5.7.3):

$$f_{ps} = f_{pu} \times (1 - k \frac{c}{d_p}) = 270 \times (1 - 0.28 \frac{22.85}{102}) = 253 ksi$$

Find ultimate moment strength:

$$\phi M_n = \phi A_{ps} f_{ps} (d_p - a/2) = 0.95 \times 67.704 \times 253 \times (102 - 17.1/2) = 126{,}696 kip - ft$$

$\phi M_n > M_u$, O.K.

Check reinforcement ratio:

$d_e = 102 in$, effective depth of tendons

$$\frac{c}{d_e} < 0.42$$

$$\frac{c}{d_e} = \frac{22.85}{102} = 0.224 < 0.42,$$ O.K. Section is under-reinforced

Check that 1.2 times the cracking moment is satisfied:

$\sigma_t = 580\,psi$, compressive stress at top of section due to permanent loads at day 10,000

$f_r = 7.5\sqrt{f_c'} = 7.5 \times \sqrt{6000} = 581 psi$

$S_t = 435740 in^3$

$1.2M_{cr} = 1.2 \times (\sigma_t + f_t) \times S_t = 1.2 \times (580 + 581) \times 435740 = 50{,}589 kip - ft$

$\phi M_n > 1.2M_{cr}$, O.K.

6.12 Shear and Torsion Design

From recent AASHTO T-10 Committee meetings on June 21, 2004 in Orlando, Florida, it was proposed that for post-tensioned box girder bridges, including segmental bridges, the design procedure similar to *AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges*, Article 12.0 may be elected. The current edition of *AASHTO LRFD Bridge Design Specifications, Third Edition* uses modified compression field theory for shear and torsion design. Both shear design methods will be presented in this design example.

6.12.1 AASHTO T-10 Proposed Shear and Torsion Design Procedure

$$V_u \leq \phi V_n$$

$$V_n = V_c + V_s + V_p$$

Where:

Φ = Resistance factors (LRFD 5.5.4.2)

V_u = Factored shear force

V_n = Total nominal shear resistance

V_c = Concrete shear resistance

V_s = Shear resistance provided by shear reinforcement

V_p = Shear resistance provided by effective prestressing force component

$$V_c = 2K\sqrt{f_c'}b_w d \text{ (lbs)}$$

Where $K = \sqrt{1 + f_{pc}/2\sqrt{f_c'}} \leq 2.0$

b_w = effective web width

d = effective shear depth

$$V_s = \frac{A_v f_y d}{s} \text{ (lbs)}$$

Where A_v = Area of transverse reinforcement within a distance s (in^2)

f_{pc} = Compressive stress in concrete after allowance for all prestress losses at the centroid of cross-section resisting shear (psi)

f'_c = Specified concrete strength (psi)

$K \le$ 1.0 at any section where stress in the extreme tension fiber due to factored load and effective prestress force exceeds $6\sqrt{f'c}$ (psi)

f_y = Specified yield strength of non-prestressed reinforcement (psi)

$V_n \le 10\sqrt{f'_c}$ for sections without torsion or where torsion can be neglected

$\sqrt{(V_n / b_w d)^2 + (T_n / 2A_o b_e)^2} \le 15\sqrt{f'_c}$ for sections where torsion is considered

$$T_u \le \phi T_n$$

$$T_n = 2A_o A_t f_y / s$$

$$A_l = \frac{T_n p_h}{2A_o f_y}$$

Where T_u = Factored torsional moment (in-lb.)

T_n = Nominal torsional resistance (in-lb.)

A_t = Area of one leg of closed transverse torsion reinforcement within a distance s (in^2)

A_l = Total additional longitudinal reinforcement required for torsion (in^2)

A_o = Area enclosed by shear flow path (in^2)

p_h = Perimeter of centerline outermost continuous closed transverse reinforcement (in)

b_e = Minimum effective shear flow web or flange width to resist torsional stresses (in)

6.12.2 AASHTO LRFD Shear and Torsion Design Procedure

The Modified Compression Field Theory (MCFT) was developed by Dr. Michael P. Collins, Dr. Frank J. Vecchio of University of Toronto and Dr. Denis Mitchell of McGill University in Canada. The MCFT for shear and torsion design was adopted for the first time by the *Ontario Highway Bridge Design Code* in 1991. The 1994 *AASHTO LRFD Bridge Design Specifications* also adopted the new method of shear and torsion design in lieu of the traditional ACI empirical equations. The new method is a simple, unified method which is applicable to both prestressed and nonprestressed members. Unlike previous empirical methods, MCFT is a rational method which gives physical significance to the parameters being calculated.

The MCFT is based on variable-angle truss instead of a 45^o truss model. Due to this truss model, the longitudinal reinforcement becomes an important element of shear design. However, in light of the iterative procedure required in the new design procedure, hand calculation is no longer practical, and a computer program should be utilized.

<u>Sections Subjected to Shear Only</u>

In a box girder, the stresses due to shear and torsion will be additive on one side of the web and will counteract each other on the other side. Therefore, the final transverse web reinforcement should be based on the summation of reinforcement due to shear and torsion.

Normally, the loading which produces the maximum shear will not be the same loading which produces the maximum torsion. Therefore, it is conservative to design based on the maximum shear and maximum torsion. However, it is sufficient to design using the maximum shear with its associated torsion and the maximum torsion with its associated shear.

For shear design, the following basic relationship must be satisfied at each section:

$$V_u \leq \phi V_n$$

where,

$$V_n = V_c + V_s + V_p \qquad \text{(LRFD 5.8.3.3-1)}$$

This relationship is similar to the method of shear design prescribed in the *AASHTO Standard Specifications*. However, with LRFD, V_c is computed in an entirely different manner. The equation for V_c is now:

$$V_c = 0.0316\beta\sqrt{f_c^{'}}\,b_v d_v \qquad \text{(LRFD 5.8.3.3-3)}$$

The value of β at a given section must be obtained through an iterative process. The following two parameters must be computed as part of this process:

$$v_u = \frac{V_u - \phi V_p}{\phi b_v d_v} \qquad \text{(LRFD 5.8.2.9-1)}$$

$$\varepsilon_x = \frac{M_u/d_v + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps}f_{po}}{2(E_s A_s + E_p A_{ps})} \leq 0.001 \qquad \text{(LRFD 5.8.3.4.2-1)}$$

A first trial value of θ is assumed to compute the initial value of ε_x. Then, knowing v and ε_x, Table 5.8.3.4.2-1 is used to look up the corresponding values of β and θ. If θ is not within a reasonable tolerance of the assumed θ, then the current value of θ is used to compute a new ε_x, and a new look-up in Table 5.8.3.4.2-1 is performed. When convergence is reached, V_c can be then be calculated.

<u>Longitudinal Reinforcement</u>

One of the cornerstone principles of modified compression field theory is the recognition that shear causes tension in longitudinal steel. At each section of the beam not subjected to torsion, the capacity of the longitudinal reinforcement must be checked for sufficiency. This relationship is expressed as follows:

$$A_s f_y + A_{ps} f_{ps} \geq \left[\frac{M_u}{d_v \phi_f} + 0.5 \frac{N_u}{\phi_c} + \left(\frac{V_u}{\phi_v} - 0.5 V_s - V_p \right) \cot\theta \right] \qquad \text{(LRFD 5.8.3.5-1)}$$

Procedure to Determine β and θ

Step 1: Compute v/f'_c ratio

where: $v_u = \dfrac{V_u - \phi V_p}{\phi b_v d_v}$

Step 2: Estimate θ, say 28°

Step 3: Compute ε_x at mid-depth of member

$$\varepsilon_x = \frac{M_u / d_v + 0.5 N_u + 0.5 (V_u - V_p) \cot\theta - A_{ps} f_{po}}{2(E_s A_s + E_p A_{ps})} \leq 0.001$$

where:

M_u = factored moment at the section (Kips - in)

N_u = factored axial force normal to the cross-section, assuming simultaneously with V_u (Kips)

A_{ps} = area of prestressing steel on the flexural tension side of members (in^2)

A_s = area of reinforcing steel (in^2)

f_{po} = a parameter taken as modules of elasticity of prestressing tendons multiplied by the locked-in difference in strain between the prestressing tendons and the surrounding concrete (KSI). For the Kips – in. Usual levels of prestressing, a value of 0.7 f_{pu} will be appropriate for both pretensioned and post-tensioned members.

E_p = 197,000 MPa, Modulus of elasticity of prestressing tendons

Step 4: Select β and θ from Table 5.8.3.4.2-1.

<u>Step 5</u>: Repeat the calculation from step 2 with the latest θ from step 4 until θ in step 4 matches close to θ in step 2, then select the new β.

<u>Step 6</u>: Compute steel and concrete contributions for nominal capacity

<u>Sections Subjected to Combined Shear and Torsion</u>

For sections subjected to combined shear and torsion, reference Article 5.8.3.6.2. Strain will need to be calculated taking into account the combination of these effects. Shear stress, longitudinal reinforcing, and area of shear reinforcing will also need to be modified.

Design Examples (Using LRFD Modified Compression Field Theory)

Node number: 41 (at critical shear section)

Ultimate moment: $M_u = 82091 kip - ft$ (negative moment, bottom slab is in compression)

$V_u = 2391 kip$

$\phi = 0.90$ for shear

Nominal shear resistance:

$V_n = V_c + V_s + V_p$

or $V_n = 0.25 f_c' b_v d_v + V_p$

where:

$f_c' = 6 ksi$, compression strength of concrete

$b_v = 32 in$, effective web width

$d_v = 108 - 6 - 17.1/2 = 93.4 in = 7.79\,ft > Max\{0.9(108-6), 0.72 \times 108\}$, effective shear depth

$V_p = 0$

$V_n = 0.25 \times 6 \times 32 \times 93.4 = 4483 kip$

$V_n = 4483 kip > V_u / \phi = 2391 kip / 0.9 = 2657 kip$ O.K.

Cross section dimension is sufficient.

Concrete Contribution:

$V_c = 0.0316 \beta \sqrt{f_c'} b_v d_v$

Transverse reinforcement contribution:

$$V_s = \frac{A_v f_y d_v \cot\theta}{s}$$

where:

β = factor indicating the ability of diagonally cracked concrete to transmit tension

θ = angle of inclination of diagonal compressive stresses

General Procedure to Determine β and θ

Step 1: Compute the v/f'_c ratio

$$v = \frac{V_u - \phi V_p}{\phi b_v d_v} = \frac{2391}{0.90 \times 32 \times 93.4} = 0.889 ksi$$

$$v/f'_c = 0.889/6 = 0.148$$

Step 2: Calculate the strain in the reinforcement on the flexural tension side of the member:

Assume $\theta = 27$ degrees

$$\varepsilon_x = \frac{\frac{M_u}{d_v} + 0.5N_u + 0.5V_u \cot\theta - A_{ps} f_{po}}{2(E_s A_s + E_p A_{ps})}$$

$$= \frac{\frac{82091}{7.79} + 0.5 \times 2391 \times \cot 27 - 67.7 \times 189}{2(28500 \times 67.7)}$$

$$= \frac{-2257 + 1195 \times \cot 27}{3858900} = 0.000023$$

Step 3: Find the values of θ and $\varepsilon_x \times 1000$ in Table 5.8.3.4.2-1 which correspond to $v/f'_c = 0.148 \,\&\, \varepsilon_x * 1000 = 0.023$. If θ corresponds to the assumed value, iteration is complete. If not, choose another θ value and repeat until convergence is achieved. If $\varepsilon_x < 0.0$, equation 5.8.3.4.2-3 shall be used to calculate strain:

$$\varepsilon_x = \frac{\frac{M_u}{d_v} + 0.5N_u + 0.5V_u \cot\theta - A_{ps} f_{po}}{2(E_c A_c + E_s A_s + E_p A_{ps})}$$

Finally, we obtain $\theta = 26.9°$ and $\beta = 2.6$

$$V_c = 0.0316\beta\sqrt{f_c'}b_v d_v$$
$$= 0.0316 \times 2.60\sqrt{6} \times 32 \times 93.4 = 602kip$$

$$V_s = V_n - V_c = V_u/\phi - V_c$$
$$= 2391/0.9 - 602 = 2055kip = 1028kip/web$$

$$\frac{A_v}{s} = \frac{V_s}{f_y d_v \cot\theta}$$
$$= \frac{1028}{60 \times 93.4 \times \cot 26.9°} = 0.093in^2/in = 1.12in^2/ft$$

Use double #6 bars at 9" centers per web $A_v = 1.17\, in^2/ft$

Longitudinal Reinforcement

For sections not subjected to torsion, longitudinal reinforcement needs to satisfy:

$$A_s f_y + A_{ps} f_{ps} \geq \left[\frac{M_u}{d_v\phi} + 0.5\frac{N_u}{\phi} + \left(\frac{V_u}{\phi} - 0.5V_s - V_p\right)\cot\theta\right]$$ (LRFD 5.8.3.5-1)

$\phi = 0.95$ for flexure; (Table 5.5.4.2.2-1)

$\phi = 0.90$ for shear; (Table 5.5.4.2.2-1)

$$A_s f_y + A_{ps} f_{ps} = 0 \times 0 + 67.7 \times 253 = 17136kip$$

$$\frac{M_u}{d_v\phi} + 0.5\frac{N_u}{\phi} + \left(\frac{V_u}{\phi} - 0.5V_s - V_p\right)\cot\theta$$
$$= \frac{82091}{7.79 \times 0.95} + 0 + \left(\frac{2391}{0.90} - 0.5 \times 2055 - 0\right) \times \cot 26.9°$$
$$= 14304kip$$

Therefore, the condition (5.8.3.5-1) is satisfied.

Node number: 29 (at section 60 feet from the face of diaphragm)

Ultimate moment: $M_u = 20816kip - ft$ (positive moment, top slab is in compression)

$V_u = 1087kip$
$\phi = 0.90$

Nominal shear resistance:

$V_n = V_c + V_s + V_p$

or $V_n = 0.25 f_c' b_v d_v + V_p$

where:

$f_c' = 6ksi$, compression strength of concrete

$b_v = 32in$, effective web width

$d_v = 108 - 5 - 2.6/2 = 101.7in = 8.48 ft > Max\{0.9(108-5), 0.72 \times 108\}$, effective shear depth;

$V_p = 0$

$V_n = 0.25 \times 6 \times 32 \times 101.7 = 4882kip$

$V_n = 4882kip > V_u/\phi = 1087/0.9 = 1208kip$ O.K.

Cross section dimensions are sufficient

Concrete Contribution:

$V_c = 0.0316\beta\sqrt{f_c'} b_v d_v$

Transverse reinforcement contribution:

$$V_s = \frac{A_v f_y d_v \cot\theta}{s}$$

where:

β = factor indicating ability of diagonally cracked concrete to transmit tension

θ = angle of inclination of diagonal compressive stresses

General Procedure to Determine β and θ

Step 1: Compute the v/f_c' ratio

$$v = \frac{V_u - \phi V_p}{\phi b_v d_v} = \frac{1087}{0.9 \times 32 \times 101.7} = 0.371ksi$$

$v/f_c' = 0.371/6 = 0.062$

Step 2: Calculate the strain in the reinforcement on the flexural tension side of the member.

Use Equation 1 since strain will be positive:

Assume $\theta = 24.3$ degrees

$$\varepsilon_x = \frac{\frac{M_u}{d_v} + 0.5N_u + 0.5V_u \cot\theta - A_{ps} f_{po}}{2(E_s A_s + E_p A_{ps})}$$

$$= \frac{\frac{20816}{8.48} + 0.5 \times 1087 \times \cot 24.3 - 19.1 \times 189}{2(19.1 \times 28500)}$$

$$= \frac{-1154 \times 543 \cot 24.3}{1088472} = 0.0000453$$

Step 3: Find the values θ and $\varepsilon_x \times 1000$ in Table 5.8.3.4.2-1 which correspond to $v/f_c' = 0.062$ & $\varepsilon_x \times 1000 = 0.045$. If θ corresponds to the assumed value, iteration is complete. If not, choose another θ value and repeat until convergence is achieved.

We obtain $\theta = 24.3°$ and $\beta = 3.24$

$$V_c = 0.0316\beta\sqrt{f_c'}b_v d_v$$

$$= 0.0316 \times 3.24\sqrt{6} \times 32 \times 101.7 = 816kip$$

$$V_s = V_n - V_c = V_u/\phi - V_c$$

$$= 1087/0.90 - 816 = 392kip = 196kip/web$$

$$\frac{A_v}{s} = \frac{V_s}{f_y d_v \cot\theta}$$

$$= \frac{196}{60 \times 101.7 \times \cot 24.3°} = 0.0145 in^2/in = 0.174 in^2/ft$$

Minimum reinforcing $A_v = 0.0316\sqrt{f_c'}b_v \frac{s}{f_y} = 0.0316\sqrt{6} \times 16 \times \frac{12}{60} = 0.248\, in^2/ft$

<u>Conservatively use double #5 at 18” centers</u> $A_v = 0.413\, in^2/ft$

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} = \frac{2 \times 0.413 \times 60 \times 101.7 \times \cot 24.3°}{12} = 930kip$$

Longitudinal Reinforcement

For sections not subjected to torsion, longitudinal reinforcement needs to satisfy:

$$A_s f_y + A_{ps} f_{ps} \geq \left[\frac{M_u}{d_v \phi} + 0.5 \frac{N_u}{\phi} + \left(\frac{V_u}{\phi} - 0.5 V_s - V_p \right) \cot\theta \right] \quad \text{(LRFD 5.8.3.5-1)}$$

$\phi = 0.95$ for flexure; (Table 5.5.4.2.2-1)

$\phi = 0.90$ for shear; (Table 5.5.4.2.2-1)

$$A_s f_y + A_{ps} f_{ps} = 0 \times 0 + 19.1 \times 268 = 5118 kip$$

$$\frac{M_u}{d_v \phi} + 0.5 \frac{N_u}{\phi} + \left(\frac{V_u}{\phi} - 0.5 V_s - V_p \right) \cot\theta$$
$$= \frac{20816}{8.48 \times 0.95} + 0 + \left(\frac{1087}{0.90} - 0.5 \times 930 - 0 \right) \times \cot 24.3°$$
$$= 4229 kip$$

Therefore, the condition (5.8.3.5-1) is satisfied.

Design Examples (Using AASHTO Segmental Spec modified in accordance with AASHTO T-5 & T-10 Committee)

Node number: 41 (at critical shear section)

$V_u = 2391 kip$

$\phi = 0.90$ for shear

$f_{pc} = 906\, psi$ at neutral axis

$f_c' = 6ksi$, compression strength of concrete

$b_v = 32in$, effective web width

Concrete Contribution:

$$V_c = 2K\sqrt{f_c'} b_w d$$

$$K = \sqrt{1 + f_{pc} / 2\sqrt{f_c'}} \leq 2.0$$

$$K = \sqrt{1 + 906 \ / 2\sqrt{6000}} = 2.62 ==> 2.0$$

Note: Tensile stress at the extreme fiber under factored loads with effective prestressing was checked to insure it was under $6\sqrt{f_c'}$.

$$V_c = 2 \times 2.0\sqrt{6000} \times 32 \times 102 = 1011 kip$$

Transverse reinforcement contribution:

$$V_s = V_n - V_c = V_u / \phi - V_c$$
$$= 2391 / 0.90 - 1011 = 1646 kip = 823 kip / web$$

$$\frac{A_v}{s} = \frac{V_s}{f_y d}$$
$$= \frac{823}{60 \times 102} = 0.134 in^2 / in = 1.61 in^2 / ft$$

<u>Use double #6 bar at 6" centers per web</u> $A_v = 1.76\, in^2 / ft$

$$V_s = \frac{A_v f_y d}{s}$$

$$V_s = \frac{2 \times 1.76 \times 60 \times 102}{12} = 1795 kip$$

Ultimate shear resistance:

$$\phi V_n = \phi (V_c + V_s + V_p)$$

$V_p = 0$

$\phi V_n = 0.9(1011 + 1795) \ = 2525 kip$

$V_u = 2391 kip$ O.K.

Check maximum nominal shear resistance:

$V_n = V_c + V_s + V_p \leq 10\sqrt{f_c'} b\, d$

$V_n = 1011 + 1795 \ = 2806 kip$

$$\# Roots V_n = \frac{V_n}{\sqrt{f_c'} b\, d} = \frac{2806 \times 1000}{\sqrt{6000} \ \times 32 \ \times 102} = 11.1\sqrt{f_c'} > 10\sqrt{f_c'}$$

Therefore the section is inadequate to carry the factored shear force. Consider increasing web thickness or going to deeper section. Note that in the current *Guide Specification for Design and Construction of Segmental Bridges, Second Edition, 1999*, a maximum of $V_n = 12\sqrt{f_c'} b\, d$ is recommended. If using this code, the section would be adequate.

Node number: 29 (at section 60 feet from the face of diaphragm)

$V_u = 1087 kip$

$\phi = 0.90$ for shear

$f_{pc} = 533 psi$ at neutral axis

$f_c' = 6 ksi$, compression strength of concrete

$b_v = 32 in$, effective web width

Concrete Contribution:

$$V_c = 2K\sqrt{f_c'}b_w d$$

$$K = \sqrt{1 + f_{pc}/2\sqrt{f_c'}} \leq 2.0$$

$$K = \sqrt{1 + 533 \ / 2\sqrt{6000}} = 2.11 ==> 2.0$$

Note: Tensile stress at the extreme fiber under factored loads with effective prestressing was checked to insure it was under $6\sqrt{f_c'}$.

$$V_c = 2 \times 2.0\sqrt{6000} \times 32 \times 103 = 1021 kip$$

Transverse reinforcement contribution:

$$V_s = V_n - V_c = V_u / \phi - V_c$$
$$= 1087 / 0.90 - 1021 = 187 kip = 93 kip / web$$

$$\frac{A_v}{s} = \frac{V_s}{f_y d}$$
$$= \frac{93}{60 \times 103} = 0.015 in^2/in = 0.18\, in^2/ft$$

Minimum reinforcing $A_v = \frac{50 b_w s}{f_y} = \frac{50 \times 16 \times 12}{60{,}000} = 0.16\, in^2/ft$

<u>Minimum reinforcing does not control. However, conservatively use double #5 at 18" centers</u>

$$A_v = 0.413\, in^2/ft$$

$$V_s = \frac{A_v f_y d}{s}$$

$$V_s = \frac{2 \times 0.413 \times 60 \times 103}{12} = 425 kip$$

Ultimate shear resistance:

$$\phi V_n = \phi(V_c + V_s + V_p)$$

$$V_p = 0$$

$$\phi V_n = 0.9(1021 + 425) \quad = 1301 kip$$

$V_u = 1087 kip$ O.K.

Check maximum nominal shear resistance:

$$V_n = V_c + V_s + V_p \leq 10\sqrt{f_c'} b\ d$$

$$V_n = 1021 + 425 \quad = 1446 kip$$

$\# Roots V_n = \dfrac{V_n}{\sqrt{f_c'} b\ d} = \dfrac{1446 \times 1000}{\sqrt{6000} \times 32 \times 103} = 5.7\sqrt{f_c'} \leq 10\sqrt{f_c'}$, O.K.

7. CONSTRUCTION STAGE ANALYSIS

7.1 Stability During Construction

A stability analysis during construction is one of the design criteria for segmental bridge design. During the construction of a segmental bridge, the boundary conditions constantly change from the beginning of construction to the end. At any time during construction, the structure and foundation must be in a stable state and have ample safety factors against material failure, over-turning, and buckling. Stability analysis, therefore, becomes an important design issue due to the lower degree of redundancy and the load imbalance of the structure during this period.

A free cantilever structure is one example that requires a stability check during erection of a segment. The longer the span length, the larger the unbalanced forces. In many cases, temporary supports are required to handle the load imbalance during erection. In addition to balanced cantilever conditions, other partially completed structures may also need to be investigated.

It is important that the engineer specify on design plans the construction loads that were assumed during design. The limits of these loads and locations where loads are applied on the structure should also be shown. Additionally, the engineer's construction schemes should be clearly stated, including approximate support reactions due to construction equipment. The stresses caused by critical construction loads and strengths of the members should also be checked.

The stability analysis specification was originally covered in article 7.4 of the *AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges*, *Second Edition 1999*. Later, those specifications were adopted by the *AASHTO LRFD Bridge Design Specifications, Third Edition, 2004*, under Article 5.14.2.3.

The following construction loads should be considered in a stability analysis:

DC = Weight of the supported structure, (kips)

DIFF = Differential load: applicable only to balanced cantilever construction, taken as 2% of the dead load applied to one cantilever, (kips)

DW = Superimposed dead load, (kips or klf)

CLL = Distributed construction live load; taken as 0.01ksf of deck area applied to one side of cantilever and 0.005 ksf on the other side

CE = Specialized construction equipment, load from launching gantry, formtraveller, beam and winch, etc., (kips)

IE = Dynamic load from equipment; determined according to the type of machinery (For gradual lifting, it may be taken as 10% of the lifting load)

CLE = Longitudinal construction equipment loads, (kips)

U = Segment unbalanced load, (kips)

WS = Horizontal wind load on structure in accordance with the provisions of Section 3, (ksf)

WE = Horizontal wind load on equipment taken as 0.1 ksf of exposed surface

WUP = Wind uplift on cantilever taken as 0.005 ksf of deck area applied to one side only

A = Static weight of precast segment being handled, (kips)

AI = Dynamic response due to accidental release of precast segment taken as static load to be added to the dead load as 100% of load A, (kips)

CR = Creep effects in accordance with Article 5.14.2.3.6

SH = Shrinkage in accordance with Article 5.14.2.3.6

T = Thermal loads; the sum of the effects due to uniform temperature variation (TU) and temperature gradients (TG)

WA = Water load and stream pressure

Table 7.1-1: Working stress load combinations

Combination	Allowable Tensile Stress (ksi)
a1 = DC + DIFF + CLL + (CE + IE)	$0.19\sqrt{f'_c}$
a2 = DC + DIFF + CLL + (CE + IE) + OTHER LOADS	$0.22\sqrt{f'_c}$
b1 = DC + U + CLL + (CE + IE)	$0.19\sqrt{f'_c}$
b2 = DC + U + CLL + (CE + IE) + OTHER LOADS	$0.22\sqrt{f'_c}$
c1 = DC + DIFF + 0.7WS + 0.7WUP	$0.19\sqrt{f'_c}$
c2 = DC + DIFF + 0.7WS + 0.7WUP + OTHER LOADS	$0.22\sqrt{f'_c}$
d1 = DC + DIFF + CLL + CE + 0.7WS + WUP + 0.7WE	$0.19\sqrt{f'_c}$
d2 = DC + DIFF + CLL + CE + 0.7WS + WUP + 0.7WE + OTHER LOADS	$0.22\sqrt{f'_c}$
e1 = DC + U + CLL + (CE + IE) + 0.3WS + 0.3WE	$0.19\sqrt{f'_c}$
e2 = DC + U + CLL + (CE + IE) + 0.3WS + 0.3WE + OTHER LOADS	$0.22\sqrt{f'_c}$
f1 = DC + CLL + (CE +IE) + CLE + 0.3WS + 0.3WE	$0.19\sqrt{f'_c}$
f2 = DC + CLL + (CE +IE) + CLE + 0.3WS + 0.3WE + OTHER LOADS	$0.22\sqrt{f'_c}$

Notes:
1. OTHER LOADS = CR + SH + TU + TG + EH + EV + ES + WA
2. Allowable compressive stress in concrete where f'c is the compressive strength at the time of load application.
3. d: equipment not working
   e: normal erection
   f: moving equipment

Strength Limit State Load Combinations

1. For maximum force effects:

$\Sigma\phi F_u$ = 1.1(DC + DIFF) + 1.3CE + A + AI (LRFD 5.14.2.3..4-1)

2. For minimum force effects:

$\Sigma\phi F_u$ = DC + CE + A + AI (LRFD 5.14.2.3.4-2)

WS, WE and other loads were ignored in this analysis.

Allowable stress:

Compressive stress $= -0.5f'_c$

$= -0.5 \times 6$ ksi

= -3 ksi

Tensile stress $= 0.19\sqrt{f'_c}$

$= 0.19\sqrt{6}$

$= 0.465\,ksi$

Since the design example has a 200'-0" typical span, only one balanced cantilever structure will be considered in the stability analysis during construction.

The load combinations "a" to "f" as specified in the AASHTO LRFD Bridge Design Spec. Table 5.14.2.3.3-1, were computed.

The following construction loads were applied in the stability analysis.

$$CLL1 = 0.005ksf \times 43 = 0.215klf.$$

$$CLL2 = 0.01ksf \times 43 = 0.43klf.$$

CE = construction equipment such as stressing jack and stressing platform

= 5 Kips.

$$CE + IE = 5 \times 1.1 = 5.5kips.$$

$$W_{up} = 0.005ksf \times 43 = 0.215klf\ .$$

$$A = 78 \times 12 \times 0.155 = 145kips.$$

1. For maximum force effects:

$$\sum \phi F_u = 1.1 \times (DC + DIFF) + 1.3 \times CE + A + AI$$

2. For minimum force effects:

$$\sum \phi F_u = DC + CE + A + AI$$

where:

A = static load of typical segment

= 145 kips.

CE = 5 kips.

Although calculations have not been shown in this example, of load cases "a" to "f", strength limit state load combination "e" controls.

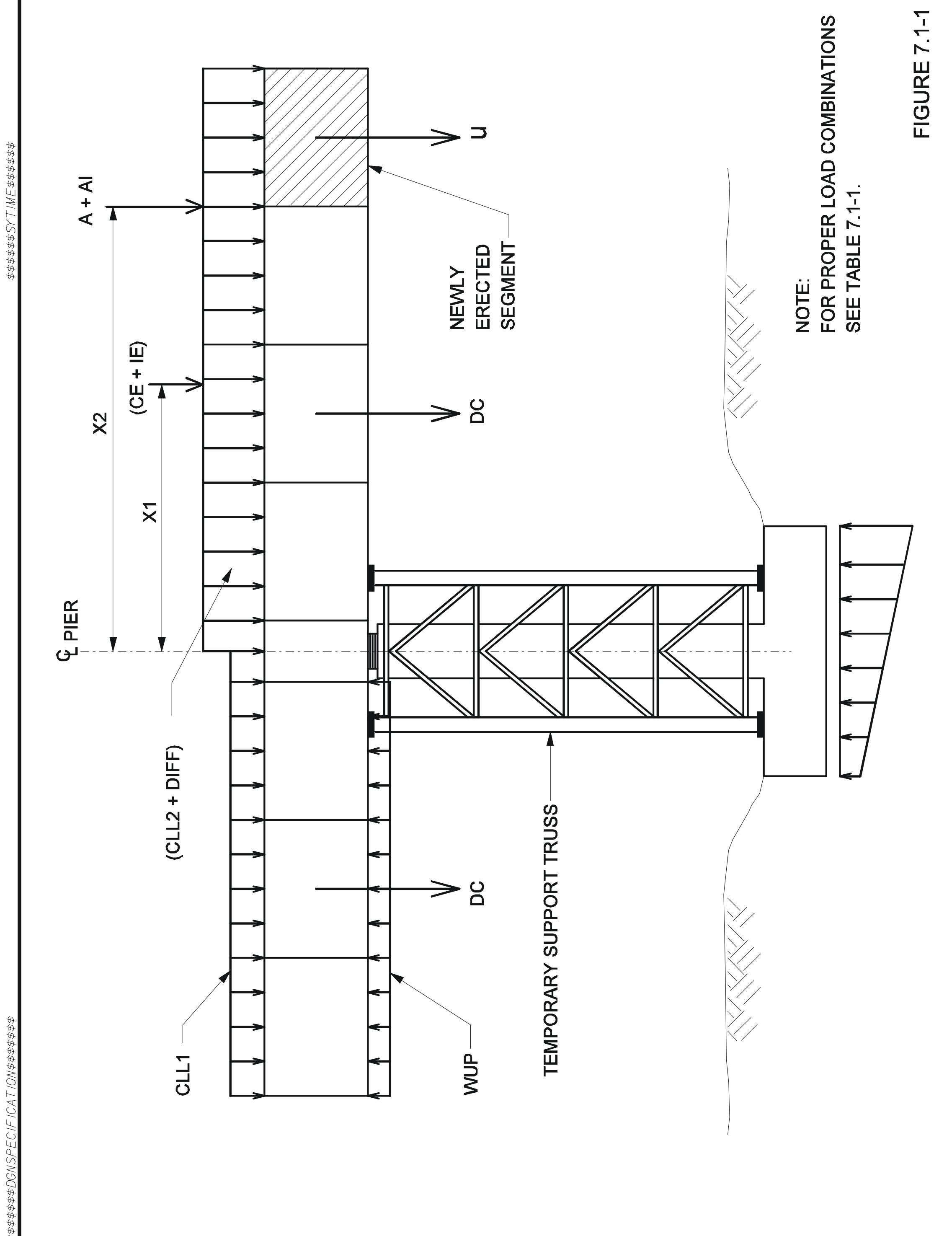
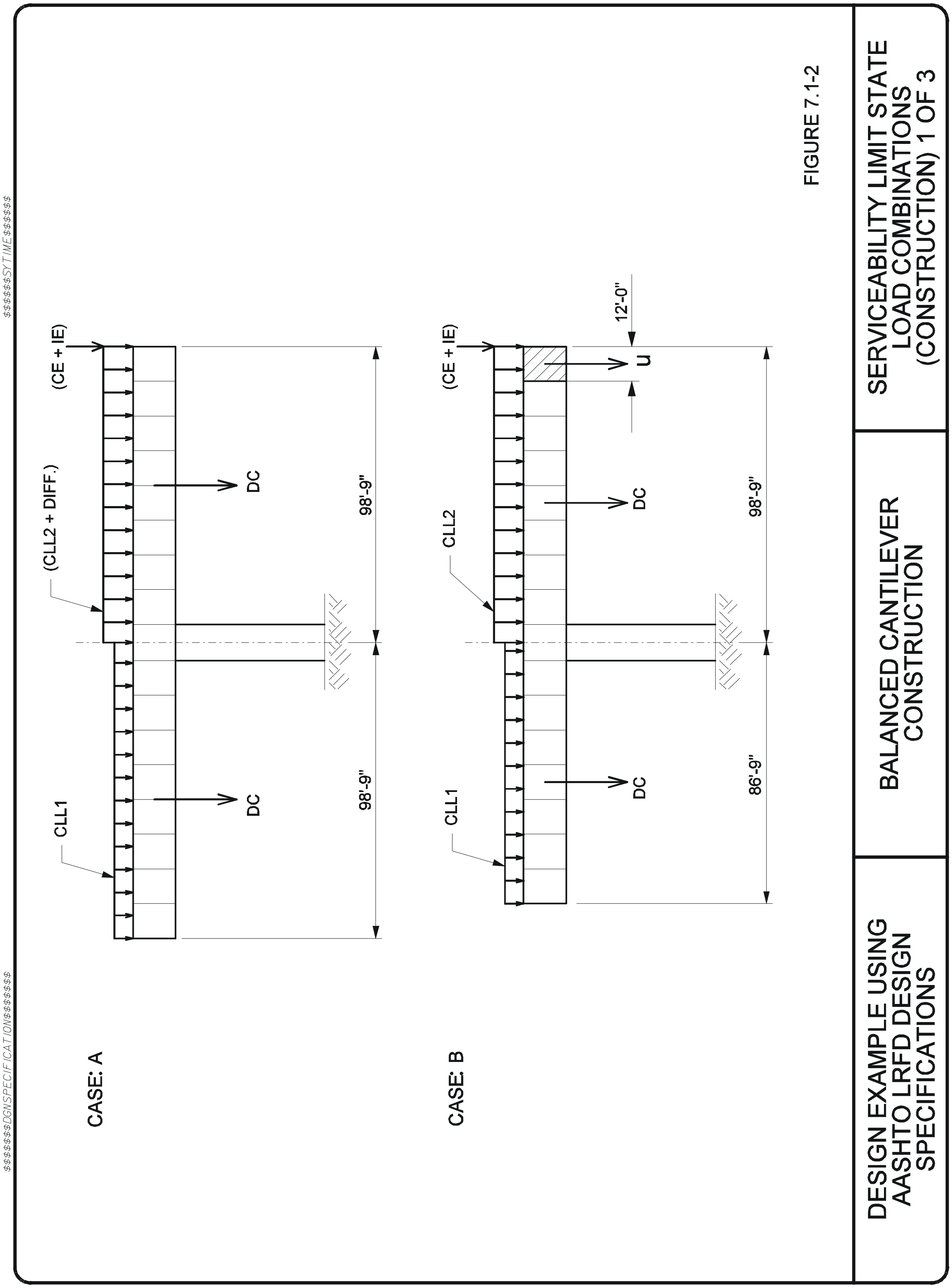


FIGURE 7.1-1

DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS

BALANCED CANTILEVER CONSTRUCTION

UNBALANCED CONSTRUCTION LOADS


$$$$$$$DGNSPECIFICATION$$$$$$$$
$$$$$$$SYTIME$$$$$$$
CASE: A
CLL1
(CLL2 + DIFF.)
(CE + IE)
DC
DC
98'-9"
98'-9"
CASE: B
CLL1
CLL2
(CE + IE)
DC
DC
U
12'-0"
86'-9"
98'-9"
FIGURE 7.1-2
DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS
BALANCED CANTILEVER CONSTRUCTION
SERVICEABILITY LIMIT STATE LOAD COMBINATIONS (CONSTRUCTION) 1 OF 3

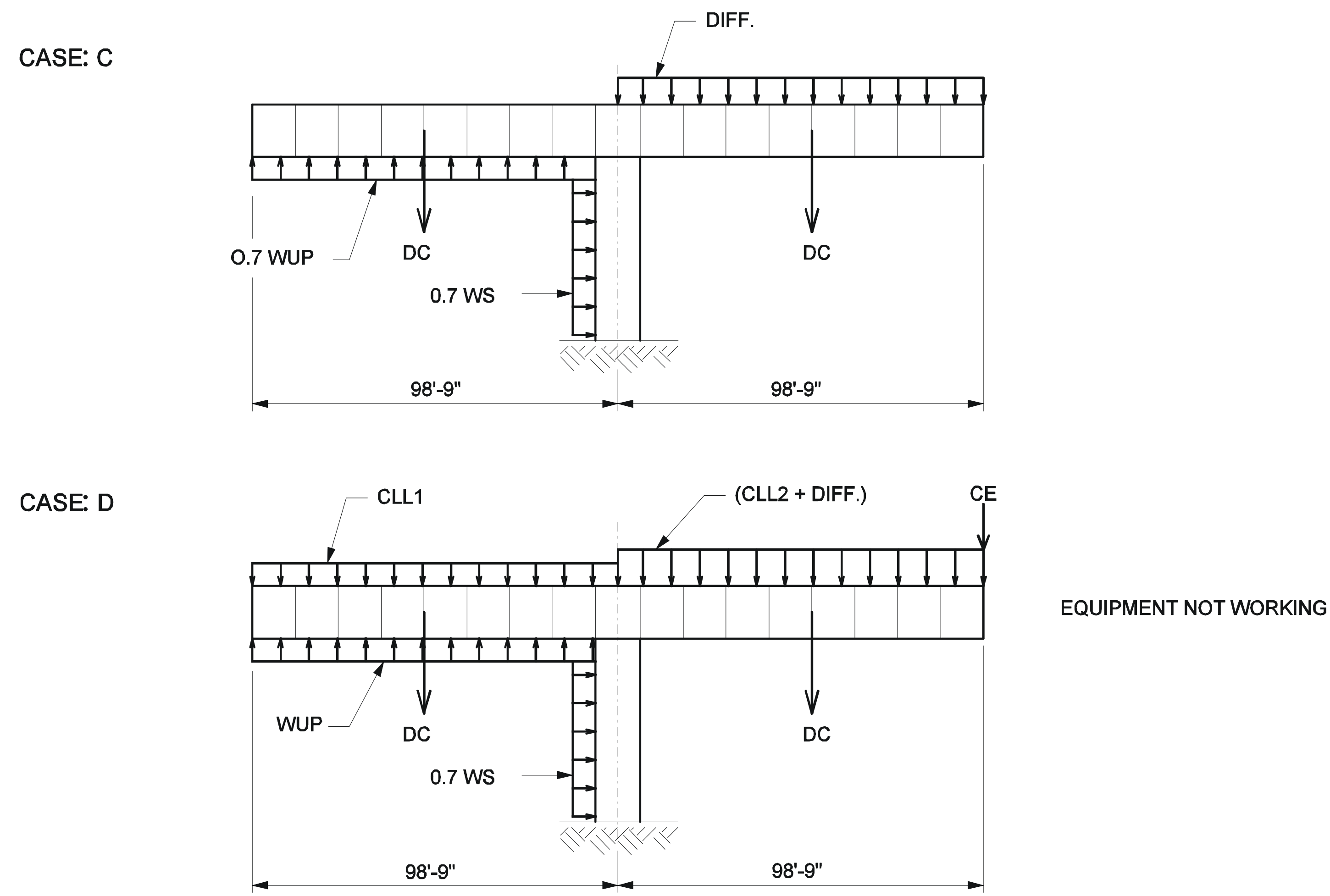


FIGURE 7.1-3

DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS	BALANCED CANTILEVER CONSTRUCTION	SERVICEABILITY LIMIT STATE LOAD COMBINATIONS (CONSTRUCTION) 2 OF 3

$$$$$$$DGNSPECIFICATION$$$$$$$ $$$$$$SYTIME$$$$$$

CASE: E

CASE: F

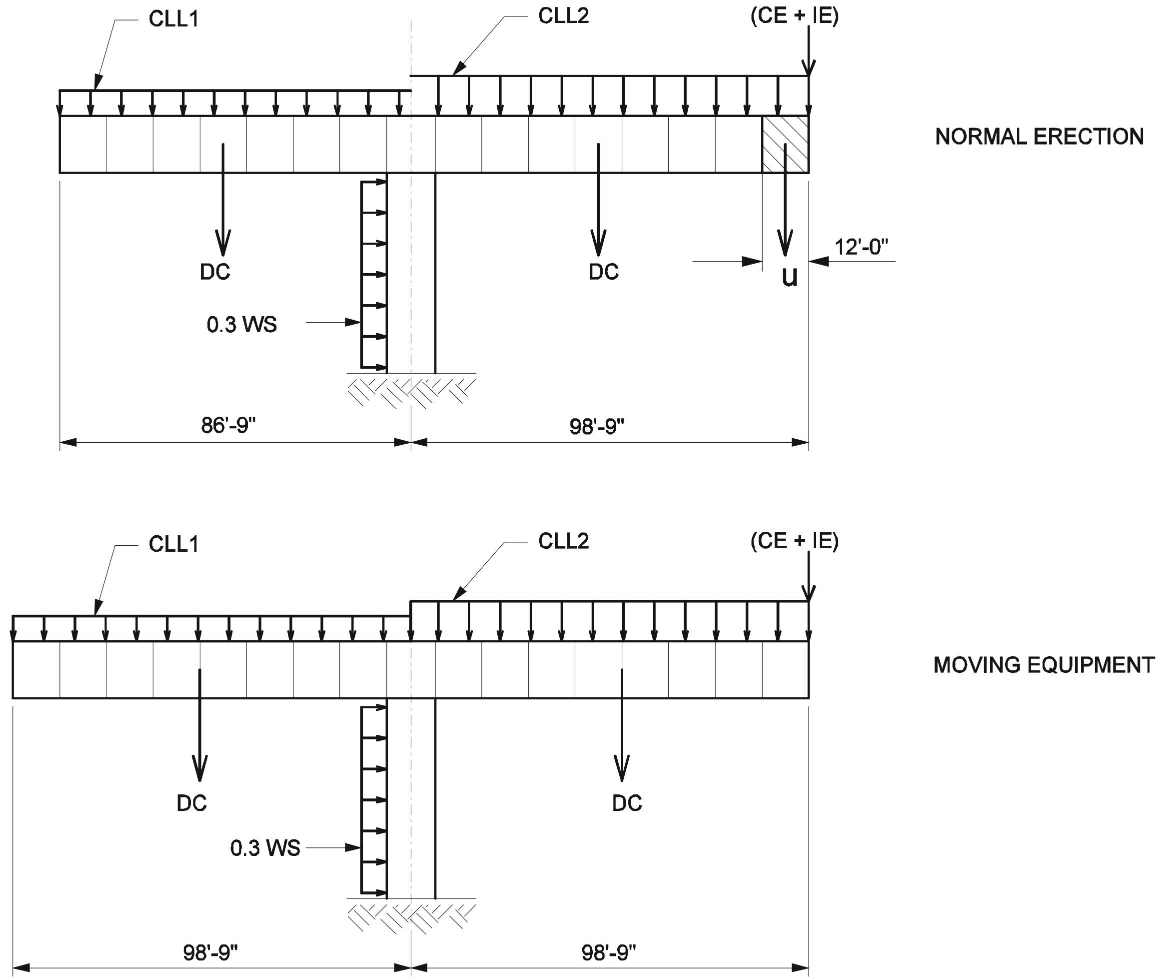


FIGURE 7.1-4

DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS	BALANCED CANTILEVER CONSTRUCTION	SERVICEABILITY LIMIT STATE LOAD COMBINATIONS (CONSTRUCTION) 3 OF 3

$$$$$$$DGNSPECIFICATION$$$$$$$ $$$$$$SYTIME$$$$$$

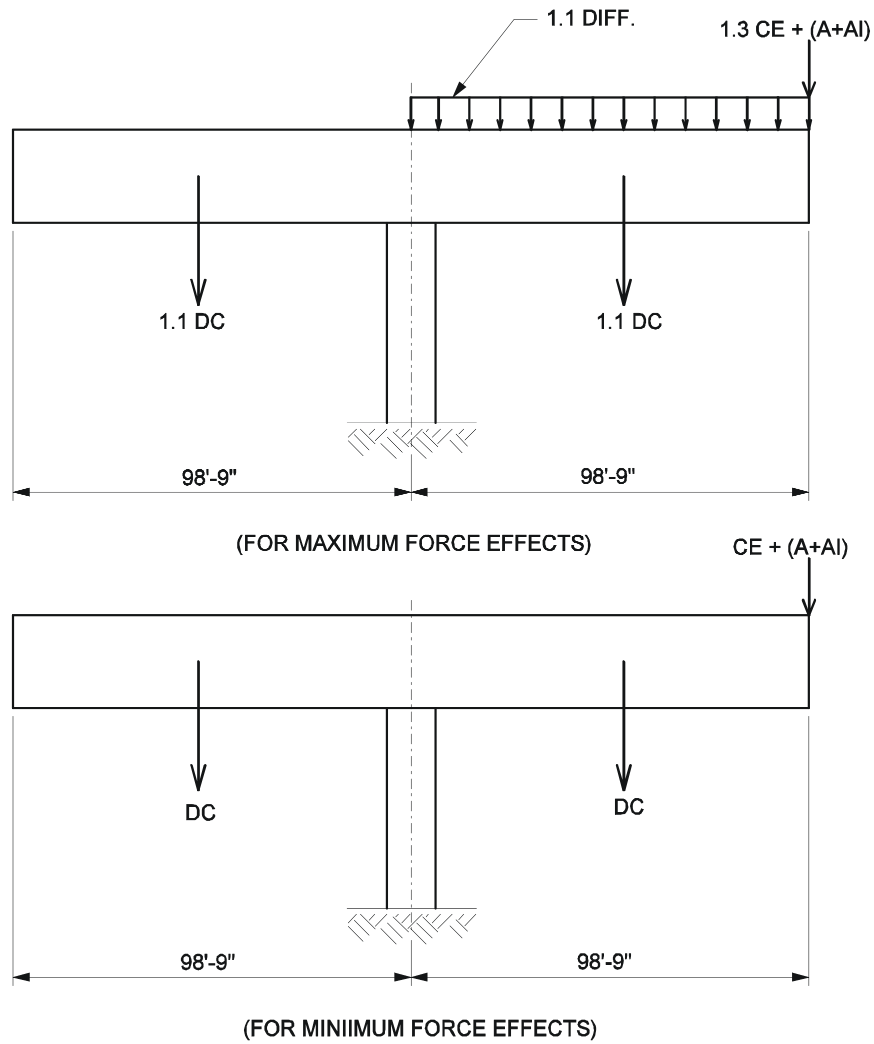


FIGURE 7.1-5

DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS	BALANCED CANTILEVER CONSTRUCTION	STRENGTH LIMIT STATE LOAD COMBINATIONS

7.2 Erection Tendons

It is common practice in precast balance cantilever segmental bridges to use temporary or permanent post-tensioning bars to attach the segment being erected to the previously erected segment. In case of permanent erection PT bars, the post-tensioned bars could be designed as part of the permanent cantilever tendons and stressed to full allowable jacking force. However, if reusable temporary post-tensioned bars are utilized, the jacking force should be limited to approximately 50 percent of G.U.T.S. of the bars.

The epoxy resin is applied to the match cast faces of the joint between two segments before post-tensioning bars are stressed. Purposes of the epoxy resin are as follows:

1. Lubrication to facilitate the proper alignment between segments.
2. Hardened epoxy provides a water-tight joint, preventing moisture, water and chlorides from reaching the tendons.
3. Hardened epoxy helps distribute compressive stresses and shear stresses more uniformly.
4. Hardened epoxy prevents cementitious grout in the tendon duct from leaking out.

The application of epoxy is normally 1/16" thick applied on both faces of match cast joints.

In accordance with the Article 5.14.2.4.2 of the LRFD Specifications for a Type A joint, the temporary post-tensioning bars should be designed to provide a minimum stress of 0.03 ksi and an average stress of 0.04 ksi across the joint until the epoxy has cured. The intention of the stress limitation is to prevent uneven epoxy thickness across the match-cast joint which could lead to systematic error in geometry control.

Essentially, there are two load cases that need to be considered when designing temporary post-tensioning bars:

1. Dead load of the segment plus construction loads and temporary post-tensioning bars. The erection PT bars should be stressed during the open time of the epoxy (approximately 45 to 60 minutes). The allowable joint stresses for this load case should conform to Article 5.14.2.4.2 of the LRFD specifications.
2. Case 1. plus permanent cantilever tendons. Normally, one or two hours after the open time of the epoxy is completed, the allowable joint stress is zero tension, preferably some compression.

DESIGN OF ERECTION PT BARS

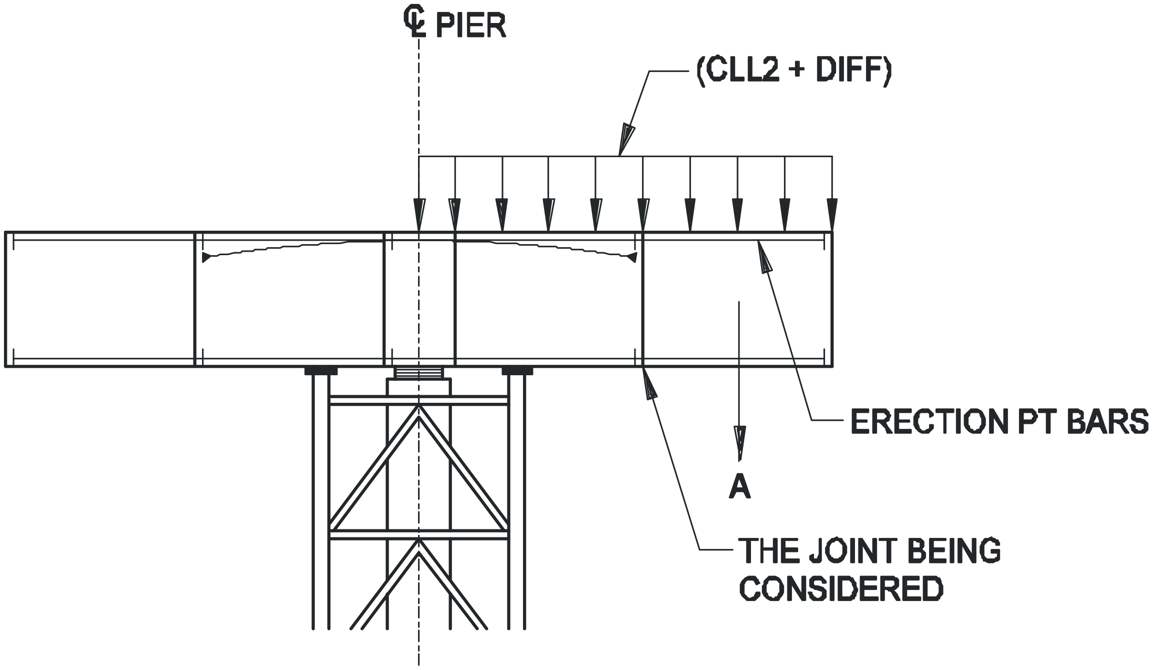


Figure 7.2-1 During segment erection

Section Properties (use + typical section: including shear lag effect)

$$A_c = 78\ sf$$

A_c *eff = 70.38 sf*

I = 791.892 ft^4

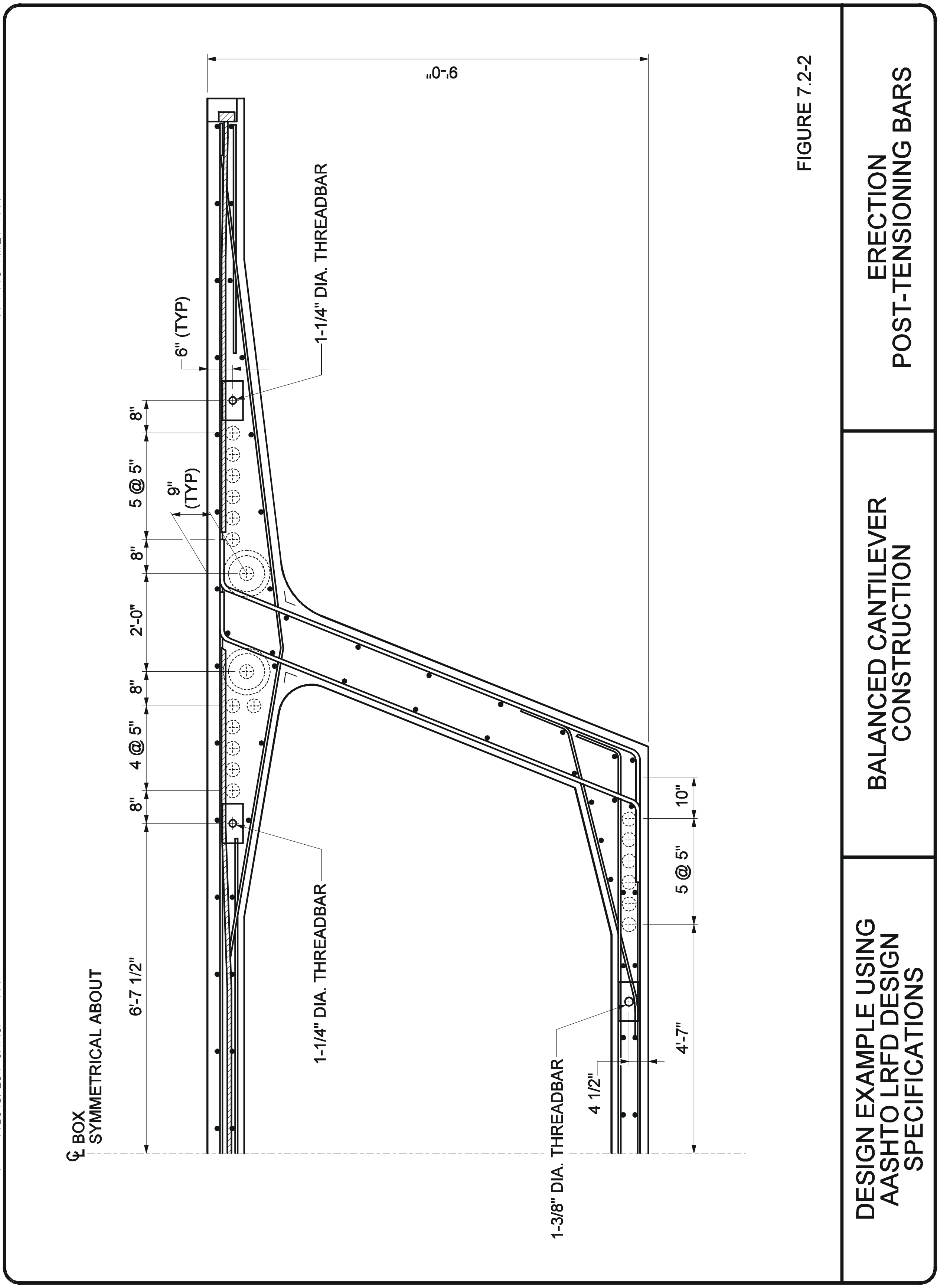
$$Y_t = 3.4 \rightarrow S_t = 232.89\ ft^3$$

$$Y_b = 5.6\,Ft \rightarrow S_b = 141.40\ ft^3$$

$$CLL2 = 0.01x43 = 0.43\,Plf$$

$$\begin{aligned} Segment\ weight + DIFF &= 1.02 \times 78 \times 12 \times 0.155 \\ &= 148\,kips \end{aligned}$$

$$\begin{aligned} M_{\max}\,at\ the\,joint &= -148 \times 12 \times \frac{1}{2} - \frac{1}{2} \times 0.43 \times 12^2 \\ &= -918.96\ \ k - \text{ft} \end{aligned}$$


9'-0"
6" (TYP)
1-1/4" DIA. THREADBAR
8"
5 @ 5"
9" (TYP)
8"
2'-0"
8"
4 @ 5"
8"
6'-7 1/2"
1-1/4" DIA. THREADBAR
℄ BOX SYMMETRICAL ABOUT
10"
5 @ 5"
4'-7"
4 1/2"
1-3/8" DIA. THREADBAR
$$$$$$$DGNSPECIFICATION$$$$$$$
$$$$$$SYTIME$$$$$$
DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS
BALANCED CANTILEVER CONSTRUCTION
ERECTION POST-TENSIONING BARS
FIGURE 7.2-2

Design Assumptions

Permanent erection bars were selected in this design example.

$$f_{pu}\ for\, PT\, bars = 150\, ksi$$

$$P_u(1\ 3/8''dia.bar) = 1.58x150 = 237\, kips$$

$$P_u(1\,1/4''dia.bar) = 1.25x150 = 187.5\, kips$$

$$P_u(1''dia.bar) = 0.85x150 = 127.5\, kips$$

Jacking force: 75% of G.U.T.S.

Check anchoring forces after anchor set for 1 1/4" dia. PT bars.

Losses due to friction:

$$\Delta F_{PF} = F_{pj}\left(1 - e^{-(\kappa x + \mu\alpha)}\right)$$ (LRFD5.9.5.2.2b-1)

where:

F_{pj} = Force in the prestressing steel at jacking, (kips)

x = length of a prestressing tendon from the jacking end to any point under consideration, (ft)

κ = wobble coefficient, (ft^{-1})

n = coefficient of friction (1/rad);

$\prec$ = sum of the absolute values of angular change of prestressing steel path from jacking end, (rad)

e = base of the Napierian logarithm

Jacking force: $P_j = 0.75 \times 187.5 = 140.625\ kips$

$$L = 12\ \ ft\ \text{(segment length)}$$

$$\kappa = 0.0002\ \text{per ft}$$

$$\mu = 0.3$$

$\alpha = 0.0$

Anchor set $\delta = 1/16" = 0.0052 \ ft.$

$$\Delta P_F = 140.625 \times \left(1 - e^{-(0.0002 \times 12)}\right)$$
$$= 0337\, kips$$

$$\therefore P_{(L)} = 140.625 - 0337 = 140.29\, kips$$

Friction loss is negligible.

Loss of stress due to anchor set = $E_{s.}$ ε

= 30,000(0.0052/12)

= 13 ksi

Pi = 140.625 – 1.25x13 = 124.375 kips (66% G.U.T.S.)

Therefore, anchoring forces, immediately after seating equal to 66% of G.U.T.S.

Try: 4 – 1 ¼" dia. top bars and

2 – 1 3/8" dia. bottom bars as shown in Figure 7.2-2

$$\because \quad P_i\, top = 4 \times 0.66 \times 187.5 = 495\, kips$$

$$P_i\, bottom = 2 \times 0.66 \times 237 = 312.84\, kips$$

$$\sum P_i = 807.84\, kips$$

Compute C.G.S. location relative to the top fiber

$$807.84 \times Y_s = 495 \times 0.5 + 312.84 \times (9 - 0.375)$$

$$Y_s = 3.65\, ft$$

PT bars eccentricity $= 3.65 - 3.4 = 0.25 \ ft\, (below\, C.G.C)$

a) CHECK JOINT STRESSES DUE TO DEAD LOADS AND PT BARS

$$f = -\frac{\sum P_i}{A_c} + \frac{\sum P_i e}{S_t} + \frac{M_{DL}}{S_t}$$
$$= -\frac{807.84}{70.38} + \frac{807.84 \times 0.25}{232.89} + \frac{918.96}{232.89}$$
$$= -11.478 + 0.867 + 3.95$$
$$= -6.66 ksf = -0.046\, ksi$$

$$\left|-0.046ksi\right| > 0.03ksi\,O.K.(LRFD\ 5.14.2.4.2)$$

$$\begin{aligned} f_b &= -\frac{\sum P_i}{A_c} - \frac{\sum P_i e}{S_b} - \frac{M_{DL}}{S_b} \\ &= -11.478 - \frac{807.84 \times 0.25}{141.40} - \frac{918.96}{141.40} \\ &= -11.478 - 1.428 - 6.450 \\ &= -19.406\,ksf\ = -0.134\,ksi \end{aligned}$$

$$\left|f_b\right| = \left|-0.134ksi\right| > 0.03ksi \quad O.K.$$

$$\begin{aligned} Average\ stress &= \frac{0.046 + 0.134}{2} \\ &= 0.09ksi \\ &> 0.04ksi\ (LRFD\ 5.14.2.4.2) \end{aligned}$$

CHECK STRESSES AT THE JOINT DUE TO DEAD LOADS, PT BARS

AND CANTILEVER TENDONS

(A) Tendon size: 4 - 12∅0.6” strands.

$$P_u\ perstrand\ = 58.6\,kips$$

$$P_i\ = 0.7 \times 50.6 \times 48 = 1968.96\,kips$$

$$Tendon\ eccentricity = 3.4\,ft - 0.5Ft = 2.9\,ft$$

Stress due to cantilever tendons:

$$\begin{aligned} f_t &= -\frac{\sum P_i}{A_c} - \frac{\sum P_i e}{S_t} \\ &= -\frac{1968.96}{70.38} - \frac{1968.96 \times 2.9}{232.89} \\ &= -27.98 - 24.52 \\ &= -52.5\,ksf\ = -0.3646\,ksi \end{aligned}$$

$$f_b = -\frac{\sum P_i}{A_c} + \frac{\sum P_i e}{S_b}$$
$$= -27.98 + \frac{1968.96 \times 2.9}{232.89}$$
$$= -27.98 + 24.52$$
$$= -3.46 ksf \ = -0.024 ksi$$

(b) Tendon size: 2 - 12∅0.6” strands. (50% less P.T.)

$$ft = 0.5(-0.3646) = -0.1823 \ ksi$$

$$f_b = 0.5(-0.024) = -0.012 \ ksi$$

SUMMATION OF STRESSES

For segments with 4 - 12∅0.6” tendons

$$\sum f_t = -0.046 - 0.3646$$
$$= -0.4106 ksi \qquad O.K.$$

$$\sum f_b = -0.134 - 0.024$$
$$= 0.158 ksi \ \ O.K.$$

For segments with 2 - 12∅0.6” tendons

$$\sum f_t = -0.046 - 0.1823$$
$$= -0.2283 ksi \quad O.K.$$

$$\sum f_b = -0.134 - 0.012$$
$$= -0.146 ksi \ \ O.K.$$

Conclusion:

The proposed permanent PT bars satisfy the allowable joint stresses.

8. DETAILING

8.1 Combined Transverse Bending and Longitudinal Shear

Based on previously determined shear reinforcing and previously determined web reinforcement required for flexure, the standard practice has been to use the worst case of adding 50% of shear steel to 100% of the flexural steel, or 100% of the shear steel to 50% of the flexural steel.

A rational approach can also be used, where the compression strut in an equivalent truss model would be shifted to the extreme edge of the web. This compression would then be eccentric to a section through the web which would counteract an applied moment. If the applied moment were to exceed the amount that could be resisted in this manner, additional reinforcing could be added. This approach has not been shown at this time, but may be included in the future.

8.2 Shear Key Design

There are two types of shear keys in match-cast joints between precast segments:

- Web shear keys - Located on the faces of the webs of precast box girders. Corrugated multiple shear keys are preferred due to their superior performance.
- Alignment keys - Located in the top and bottom slabs. Alignment keys are not expected to transfer the major shear forces; rather they facilitate the correct alignment of the two match-cast segments being erected in vertical and horizontal directions. For a single-cell box, normally a minimum of three alignment keys are required on the top slab and one on the bottom slab.

Both shear and alignment keys should not be located in the tendon duct zones.

The design of web shear keys should satisfy two design criteria:

1. Geometric Design: As per LRFD Fig. 5.14.2.4.2-1, the total depth of shear keys shall extend approximately 75% of the section depth and at least 75% of the web thickness.
2. Shear Strength Design: As per *AASHTO Standards Specifications, 17th Edition, 2002*, Article 9.20.1.5, reverse shearing stresses should be considered in shear key design. At the time of erection, shear stress carried by the shear key should not exceed $2\sqrt{f'_c}$ (psi). Alternatively, strength of the shear key could also be computed in accordance with article 12.2.21 of *AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges, Second Edition, 1999*. However, the AASHTO Guide Specificiation Shear Key Provision was developed for dry joints only.

When designing shear keys, only web shear keys are considered in transferring the shear forces. However, alignment shear keys help in preventing local relative vertical displacement on the deck slab between two adjacent precast segments due to concentrated load on one side of the match cast joint. Therefore, in longer slabs spanning between two webs or longer cantilevers wings, it is necessary to provide more than one alignment shear key.

1. Geometric consideration.

h = 9 ft

shear key depth = 0.75 × 9 ft = 6.75 ft.

b_w = 16 inches

Shear key width = 0.75 × 16 = 12 inches

2. Shear strength design of the shear keys

AASHTO LRFD Bridge Design Spec. does not specify any guideline on the strength design of shear keys. Use AASHTO Standard Specifications, article 9.20.1.5.

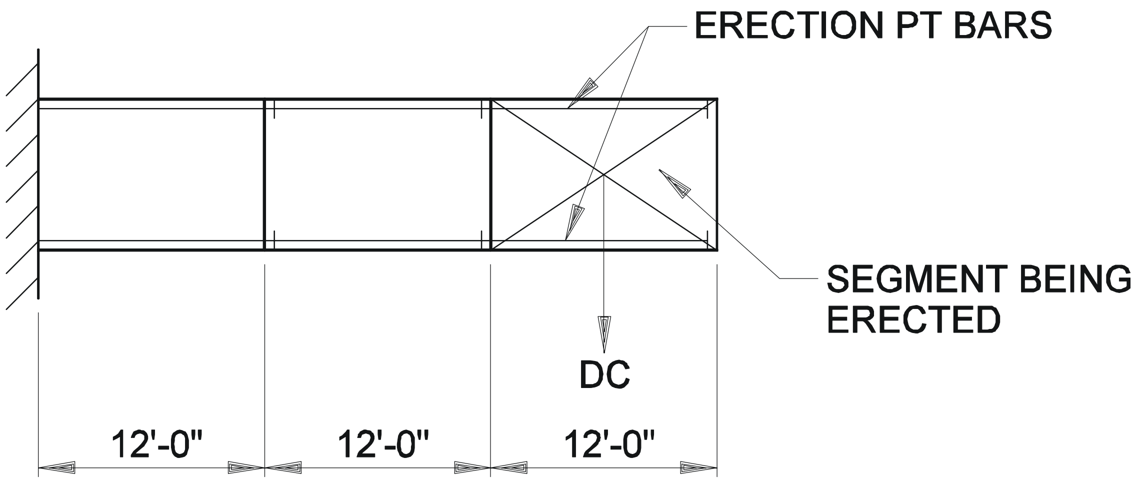


Figure 8.2-1: Precast Segment Being Erected

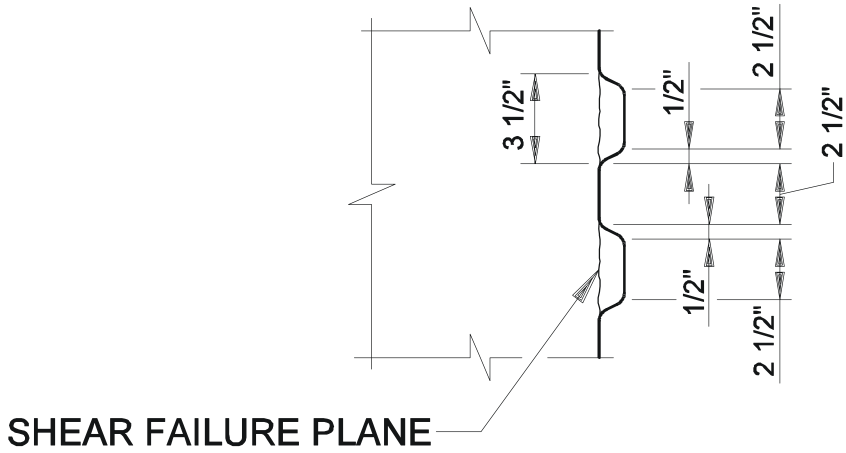


Figure 8.2-2: Details of Shear Keys

a) AASHTO Standard Specifications, article 9.20.1.5

$V_u = 1.1(V_{DC} + DIFF)$

where: V_{DC} = shear force due to self weight of one typical segment (kips)

= 78 x 12 x 0.155 = 145 kips

DIFF = 2% of V_{DC}

$V_u = 1.1 \times 145 \times 1.02 = 162.8\, kips$

$V_n = V_c$

$V_u / \phi = V_c$

Consider one web only,

$V_c = 0.5\, V_u / \phi$, per web,

$V_c = A_k \cdot v$, per key,

where: $\phi = 0.9$ article 9.14 of AASHTO Standard Specifications.

$v =$ allowable shear stress

$v = 0.2\sqrt{f_c'}\,(psi)$

$A_k =$ shear area of one key

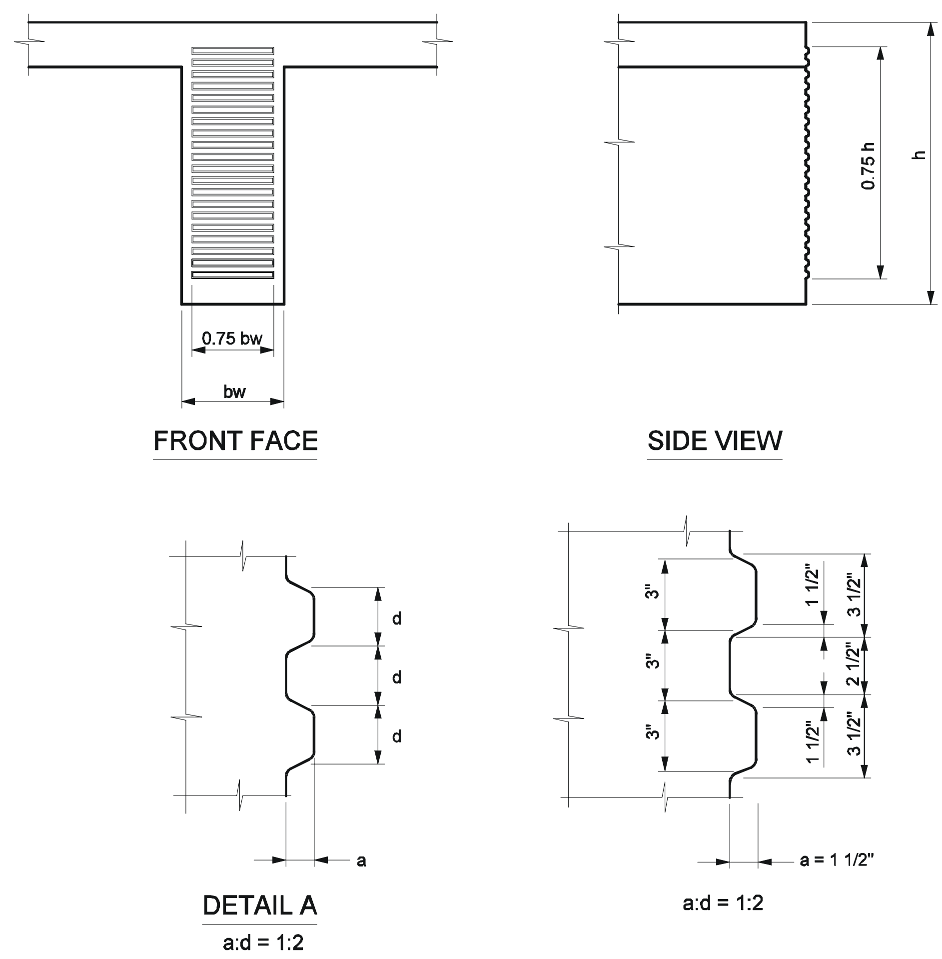
$$A_k = 3.5 \times (12) = 42\, inch^2$$

$$V_c\ perweb = (0.5 \times 162.8)/0.9 = 90.44\, kips$$

$$V_c\ perkey = 42 \times 2\sqrt{6000} = 6506.6\ lbs = 6.5\, kips$$

Number of male keys required per web $= \dfrac{90.44}{6.5} = 13.9 \quad say\, 14\, keys$.

$$$$$$$DGNSPECIFICATION$$$$$$$ $$$$$$SYTIME$$$$$$

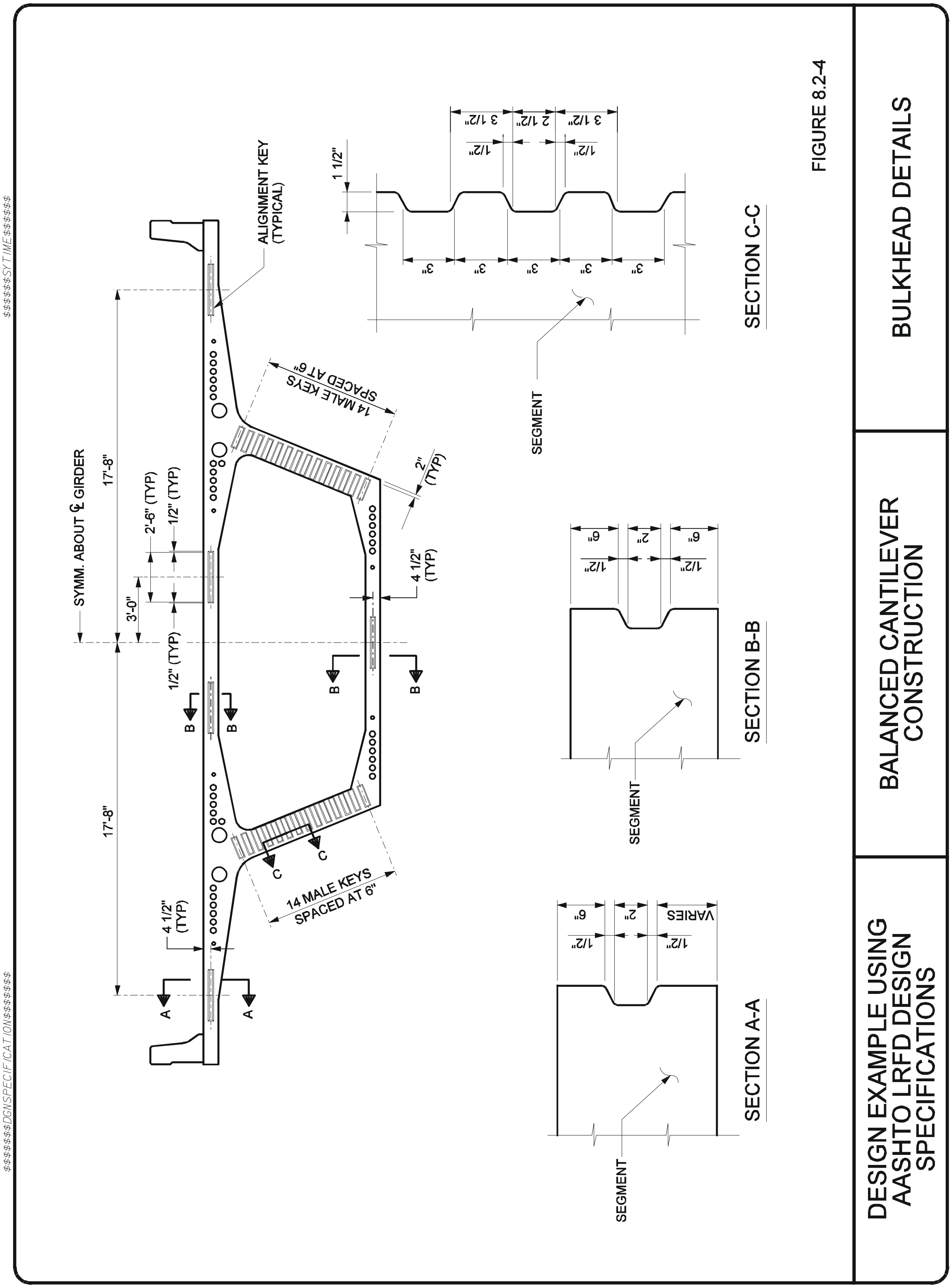


NOTES:

1. 1 1/4"≤ a ≥TWICE THE DIAMETER OF THE TOP SIZE AGGRGATE.
2. AS PER AASHTO LRFD SPECIFICATIONS FIG. 5.14.2.4.2-1.

FIGURE 8.2-3

DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS	BALANCED CANTILEVER CONSTRUCTION	SHEAR KEY DETAILS


SYMM. ABOUT ℄ GIRDER
17'-8"
17'-8"
2'-6" (TYP)
1/2" (TYP)
3'-0"
1/2" (TYP)
4 1/2" (TYP)
ALIGNMENT KEY (TYPICAL)
14 MALE KEYS SPACED AT 6"
2" (TYP)
A
B
C
SECTION A-A
6"
2"
VARIES
1/2"
SEGMENT
SECTION B-B
SECTION C-C
3"
3 1/2"
2 1/2"
1 1/2"
FIGURE 8.2-4
DESIGN EXAMPLE USING AASHTO LRFD DESIGN SPECIFICATIONS
BALANCED CANTILEVER CONSTRUCTION
BULKHEAD DETAILS
$$$$$$DGNSPECIFICATION$$$$$$
$$$$$$SYTIME$$$$$$

9. DISCUSSION AND RECOMMENDATIONS FOR IMPROVEMENT

9.1 Discussion

This design example provides an excellent opportunity to review and apply *AASHTO LRFD Design Specifications, Third Edition, 2004* (LRFD) to segmental concrete bridges. At the same time, comparative studies were made with other current design specifications, namely *AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges, Second Edition, 1999* (Segmental Specification) and *AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002* (Standard Specification). It was also interesting to study the difference using the Standard Specification HS20-44 and HS25-44 live loads.

It is interesting to note that in general, the ultimate limit state load combinations from LRFD verses the Standard Specification produce forces similar for the HL-93 loading and HS25-44 loading.

Perhaps one of more notable comparisons is shear design using LRFD Modified Compression Field Theory verses the Segmental Specification/AASHTO T-10 proposal. The V_c contribution for both codes proved to be somewhat similar. However, when comparing amounts of shear steel required at critical shear locations, the Segmental Specification required approximately 50% more shear steel than the Modified Compression Field Theory. This is due to the Segmental Specification assuming compression diagonals at a 45 degree angle of inclination to determine V_s, while the Modified Compression Field Theory utilizes an angle based on equilibrium which can be much less than 45 degrees in prestressed components.

9.2 Longitudinal Design

Listed below are recommendations for improvement:

1) Locked-in forces contained in the “EL” loading according to the LRFD code should be lumped with “DC”. Therefore, locked-in forces will receive an identical load factor as “DC”.

2) Post-tensioning secondary forces should be separated from “EL”. These effects shall be designated as “PS” and given a γ factor of 1.0 for all strength limit states.

3) Revise the limit of V_n from $10\sqrt{f'_c}$ to $12\sqrt{f'_c}$ in the shear design proposal for AASHTO T-10 Committee.

4) Specify if the minimum reinforcing check is required for segmental construction.

5) Consider specifying an allowable tension of $3\sqrt{f'_c}$ for unreinforced epoxied joints outside the precompressed tensile zone for the segmental bridge special load case.

6) Add shear key design provisions into the LRFD code.

9.3 Transverse Design

The author of this example supports recent AASHTO T-5 and T-10 Committee proposed changes in reference to transverse design. For transverse design, it seems rational not to superimpose axle loads with uniform loads, since they cannot occupy the same area coincidentally. Due to these proposed modifications, this design example indicates a transverse analysis similar to that produced by the Standard Specifications for service limit states. Under ultimate limit states, the exclusion of lane load results in moments smaller than that of the Standard Specification (HS20 loading). This is due to small dead load influences in transverse analysis and live load factors of 1.75 verses 2.17. For this reason, it is recommended that a higher load factor be entertained for live load.

10. ACKNOWLEDGEMENT

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Last but not least, I would like to thank my wife Maria Theryo for her patience and understanding due to loss of quality family time of many weekends and nights.

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