Attachment 2

AECOM Transportation

November 12, 2010

GOLDEN GATE BRIDGE
HIGHWAY & TRANSPORTATION DISTRICT

Draft Structural Analysis Report

Golden Gate Bridge Moveable Median Barrier Study

Prepared for the Golden Gate Bridge Highway and Transportation District

Federal Project Number STPL-6003(037)

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Table of Contents

List of Figures

List of Tables

List of Appendices

2.1.19.4.3
0117

Section One
Executive Summary

The Golden Gate Bridge Highway and Transportation District (District) is planning to install a moveable median barrier (MMB) system with all of its components, including the barrier and the Barrier Transfer Machine (BTM), on the top deck of the Golden Gate Bridge (Bridge). This system will provide a semi-rigid barrier between opposing traffic lanes on the Bridge while, at the same time, allowing the District the flexibility to reconfigure the lanes on the Bridge to meet the peak traffic capacity demands.

The purpose of this study is to determine:

(1) The additional load on the Bridge due to the MMB system;

(2) The adequacy of the Bridge structural components in the load path to carry the additional load of the barrier system, namely the orthotropic deck, the sub floor beam, and the steel plate girder, as shown in **Figure 1**; and

(3) The change of global deflection of the suspended structure of the Bridge due to the additional load of the barrier system.

Barrier Systems Inc.'s Steel Reactive Tension System-Quickchange® Moveable Barrier (SRTS-12" QMB) is currently being proposed for implementation on the Bridge and is analyzed in this study. The system is found to be structurally feasible.

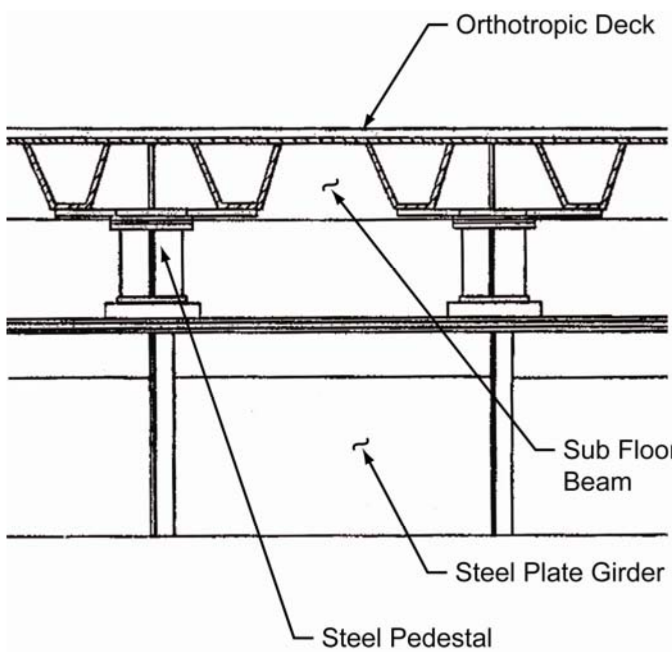
The Orthotropic Deck meets the stress check requirement per AASHTO LRFD 2007 live load conditions; thus, the MMB system will have an insignificant impact to the structural integrity of the existing structural components.

An evaluation of the local deflection of the orthotropic deck determined that premature deterioration of the wearing surface due to the addition of the MMB system is unlikely. Although the deck exceeds the recommended deflection limit per AASHTO LRFD in some cases, the deck does meet all the deflection provisions as stipulated in Caltrans Bridge Design Specifications 2004.

Additionally, the Sub Floor Beam and the Steel Plate Girder are adequate to carry the additional load from the barrier system.

The change in the deflected shape of the Suspended Bridge due to the addition of the MMB system is considered to be insignificant. This is because the change in dead load of the deck is less than three percent with the addition of the MMB.

Figure 1: Structural Components



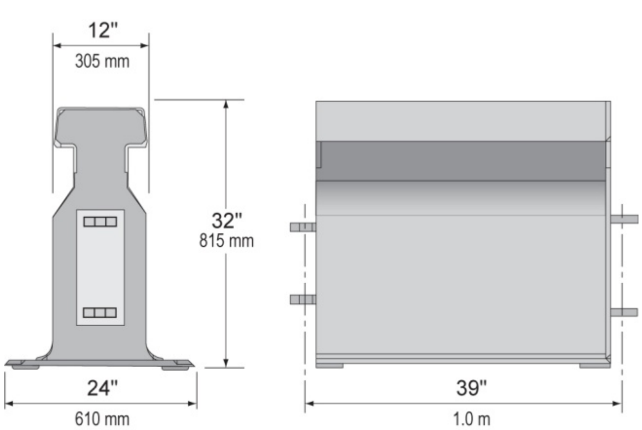
Section Two
General Information

MMB System Description

The Barrier Systems Inc. (BSI) Steel Reactive Tension System Quickchange® Moveable Barrier (SRTS-12" QMB) consists of steel encased concrete barriers linked together by steel pins that can be laterally transferred using a Barrier Transfer Machine. A typical BTM is shown in **Figure 2**. The BTM can shift the QMB system laterally between 8 and 24 feet. The SRTS-12" QMB barrier segments, as shown in **Figure 3**, are each 32 inches high, 37 inches long (39 inches pin to pin), 12 inches wide at the top, and 24 inches wide at the base to increase stability. The individual barrier units each weigh 1,500 pounds.[i] The weight of an unloaded BTM is 60,000 pounds, and the weight of a loaded BTM is 86,000 pounds.[ii]

Figure 2: Barrier Transfer Machine

Figure 3: Steel Reactive Tension System 12" Quickchange Moveable Barrier



Structural System Description

Original construction of the Golden Gate Bridge was completed in 1937 and opened to vehicular traffic. In 1983, the entire original reinforced concrete deck of the roadway was replaced with a steel orthotropic deck.

The orthotropic bridge deck frames into the Sub Floor Beams, which are spaced at about 25 feet on center. The Sub Floor Beams are supported on Steel Pedestals. The pedestals rest on top of the transverse Steel Plate Girders which are framed into the longitudinal stiffening girders.

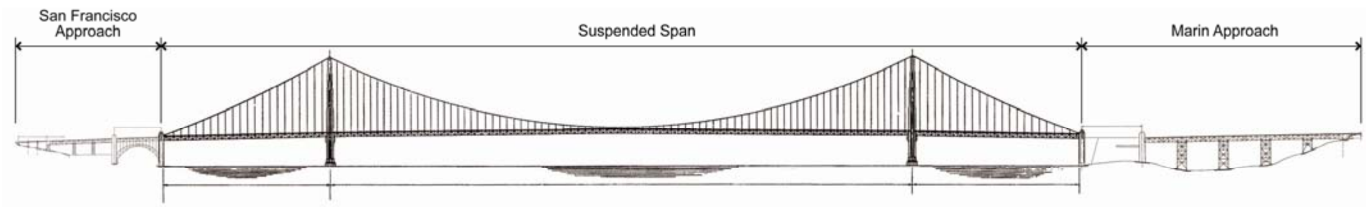
The Bridge consists of three major sections: the San Francisco Approach, the Suspended Span, and the Marin Approach. The San Francisco Approach is further sectioned into the Girder Span, the Truss Span, and the Arch Span. The Suspended Span is segmented into the San Francisco Side span, the Center Span, and the Marin Side Span. The Marin Approach primarily consists of the Truss Span. (see **Figure 4**).

The Suspended Span is symmetrical about the center line of the Center Span. The general layout of the three primary deck structural components under the load path of direct support to the moveable median barrier is similar throughout the entire Suspended Span and varies slightly at the Approach Spans.

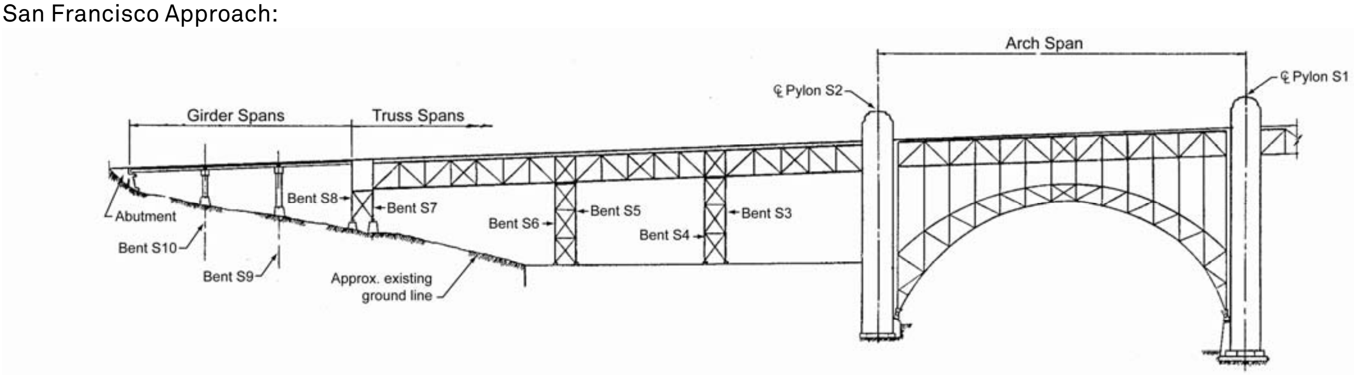
The dimensions of each element for the San Francisco Approach, the Suspended Span, and the Marin Approach are summarized in **Table 1**.

A typical section within the Suspended Span is examined in this study. This section represents majority of the deck and has elements with the longest unsupported length; therefore, the Suspended Span is considered to be the critical section.

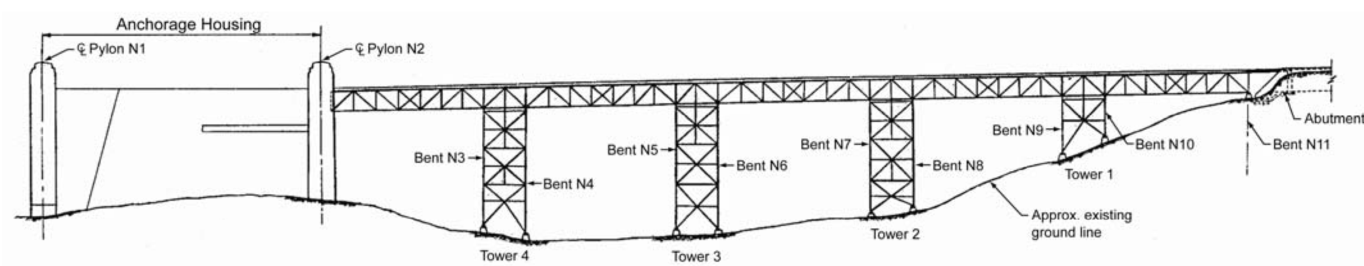
Figure 4: Golden Gate Bridge Layout



San Francisco Approach:



Marin Approach:



Source: As-Built Plans from "Golden Gate Bridge Deck and Sidewalk Replacement" (contract # 82-B-2, dated July 26, 1982)
As-Built Sheet Numbers: 25, 92, 173

Table 1: Dimensional Layout of Structural Components

Structural Component	SF Approach			Suspended Span	Marin Approach
	Girder Span	Truss Span	Arch Span		
Orthotropic Deck Layout					
Top Steel Plate Span Length (same as rib center-to-center spacing)	± 1'-2"	± 1'-2"	± 1'-2"	± 1'-2"	± 1'-2"
Rib Span Length	± 23'-6"	± 25'-0"	± 25'-0"	± 25'-0"	± 25'-0"
Sub Floor Beam Layout					
Center-to-Center Spacing	± 23'-6"	± 25'-0"	± 25'-0"	± 25'-0"	± 25'-0"
Pedestal Support Spacing	± 4'-6"	± 4'-9"	± 4'-3"	± 4'-9"	± 4'-9"
Steel Plate Girder Layout					
Center-to-Center Spacing	± 23'-6"	± 25'-0"	± 25'-0"	± 25'-0"	± 25'-0"

Source: As-Built Plans from "Golden Gate Bridge Deck and Sidewalk Replacement" (contract # 82-B-2, dated July 26, 1982),
As built sheet numbers:
SF Approach – 178-186
Suspended Span – 37 (end floor beam), 39 (intermediate floor beam)
Marin Approach – 92, 102, 112

Traffic Lane Configuration

About 105,000 vehicles utilize the Bridge daily. To provide an optimal number of lanes in each direction to accommodate the travel demand, Bridge operations crews change the lane configurations throughout the day. The three configurations are shown in **Figure 5**. These three configurations consist of the following:

- Four northbound lanes / Two southbound lanes;
- Three northbound lanes / Three southbound lanes; and
- Two northbound lanes / Four southbound lanes.

Currently, opposing traffic lanes are separated by plastic-tubular pylons, which do not prohibit vehicles from potentially crossing over into oncoming traffic.

To decrease the likelihood of head-on collisions on the Bridge while meeting daily traffic demands, the District proposes installing a MMB system.

Two lane configuration alternatives are discussed in the Draft Traffic Engineering and Analysis Report (AECOM, July 2010) for implementation of the MMB system. These configurations are shown in **Figure 6** and **Figure 7**. The structural evaluation was conducted based on the Preferred Lane Configuration Alternative (as shown in Figure 6). The locations of the BTMs during operations are shown in **Figure 8**.

Figure 5: Existing Lane Configuration Cross Sections

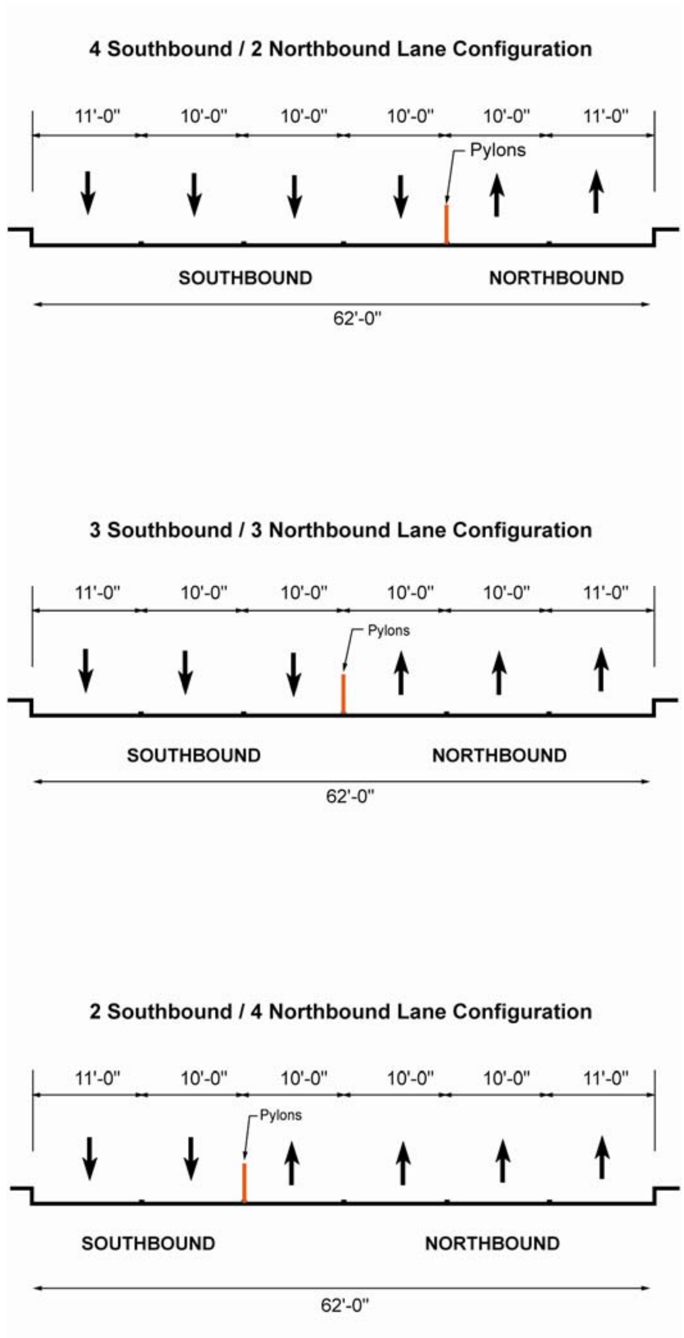


Figure 6: Preferred Alternative Lane Configuration Cross Sections

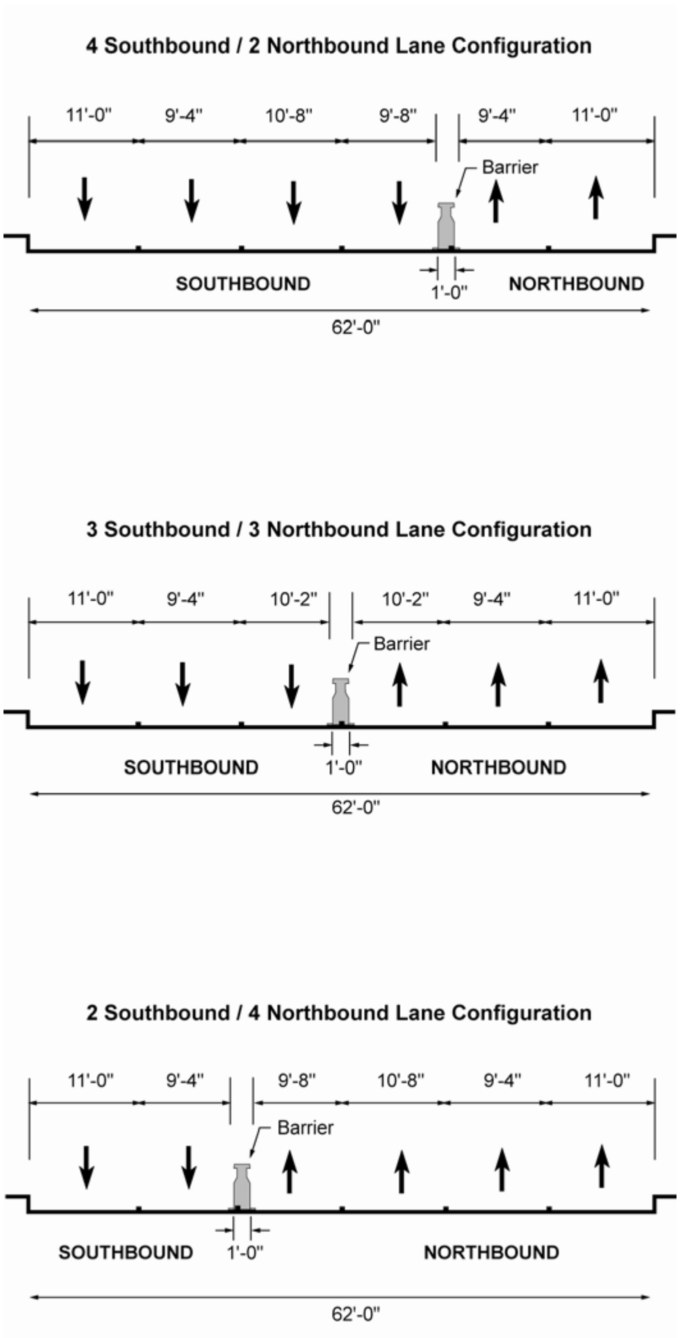
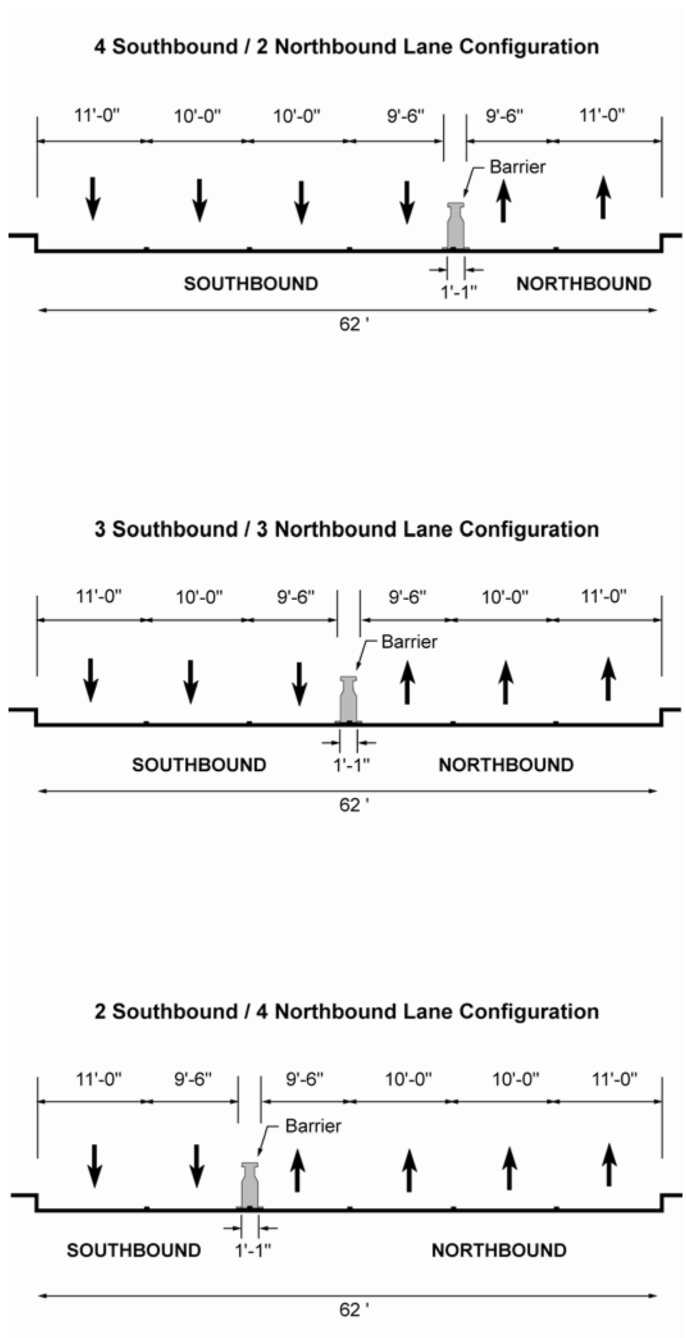


Figure 7: Distributed Lane Width Alternative Lane Configuration Cross Sections



2.1.19.4.3
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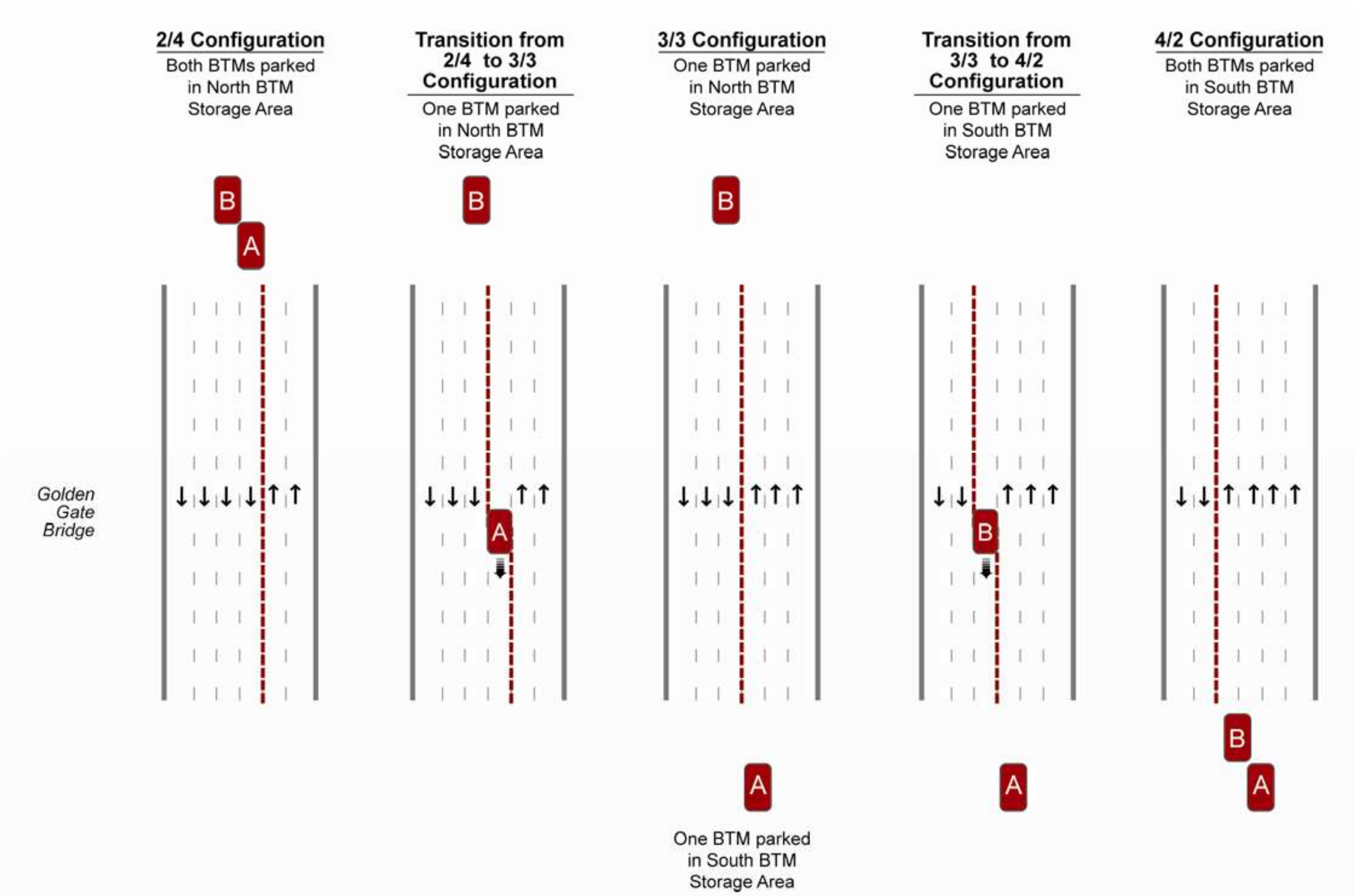
Figure 8: Barrier Transfer Machine Locations



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Section Three
Analysis Methodology

Approach and Assumptions

Depending on the structural elements being examined, the assessment of the impact of the MMB system on the Bridge is performed using direct and indirect analysis methods. The added MMB and BTM load demands on the Bridge structural components are compared by utilizing:

- **Method 1:** the load carrying capacity of the structural elements;
- **Method 2:** the live load demand used in the design for the 1983 deck replacement; or
- **Method 3:** the loads relieved from reduced dead load during the 1983 deck replacement.

For information on which method is used for each structural component, see the "Structural Component Evaluation" section.

Analysis Basis

The following summarizes the criteria used:

- AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007
    - current live load demand, and
    - live load deflection limit
- Caltrans Bridge Design Specifications, 2004, deflection requirements per Section 10.41.4.8.1.
- AISC Steel Construction Manual, 13th Edition, 2005 and AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007 to assess the structural capacity as needed.
- As-Built Plans from "Golden Gate Bridge Deck and Sidewalk Replacement" (contract # 82-B-2, dated July 26, 1982) provided by the District for structure configuration.

Loading Condition

The Bridge structural components (Orthotropic Deck, steel Sub Floor Beams, and transverse Steel Plate Girders) in relationship with vehicular loads and the MMB are shown in **Figure 9**. The vehicular loads and the MMB system loads are discussed below.

The current design vehicular live load on the roadways of bridges, (per AASHTO LRFD 2007), designated HL-93, consists of a combination of a design truck or design tandem and design lane load. The assumptions include:

- The HL-93 truck is shown in **Figure 10**.
- The tandem is a pair of 25-kip axles spaced 4-ft apart, with the wheels spaced at 6-ft apart transversely.
- The lane load is 0.64 klf uniform load in the longitudinal direction and is uniformly distributed over a 10-ft width.

The MMB (BSI SRTS-12" QMB) load is treated as dead load since it is a stationary fixture once a lane shift is completed.

The BTM is treated as live load, similar to the highway truck. Per BTM specifications, the wheel load of a BTM shall not exceed 17 kips when unloaded and 25 kips when loaded with the moveable median barrier. [ii] However, email correspondence with Jack Mazer, QMB Application Manager, indicated that the wheel loads can be controlled to 22 kips with proper shimming of the machine axles and hydraulic controls. Refer to **Appendix G** for more details. Thus, 22 kips per wheel (or 44 kips per axle) is used for the structural evaluation as shown in **Figure 10**.

The BTM makes limited number of trips over the Bridge at a maximum load and at a maximum speed of 10 mph. It is expected that the dynamic, vibratory and impact effect of the BTM is smaller than of a typical design truck. Therefore, a lower impact factor of 10 percent is assumed for the preliminary evaluation of BTM effects on the Bridge.

Figure 9: Cross Section View of the Fully Loaded Deck - MMB and Truck Wheel Loads Across Six Lanes

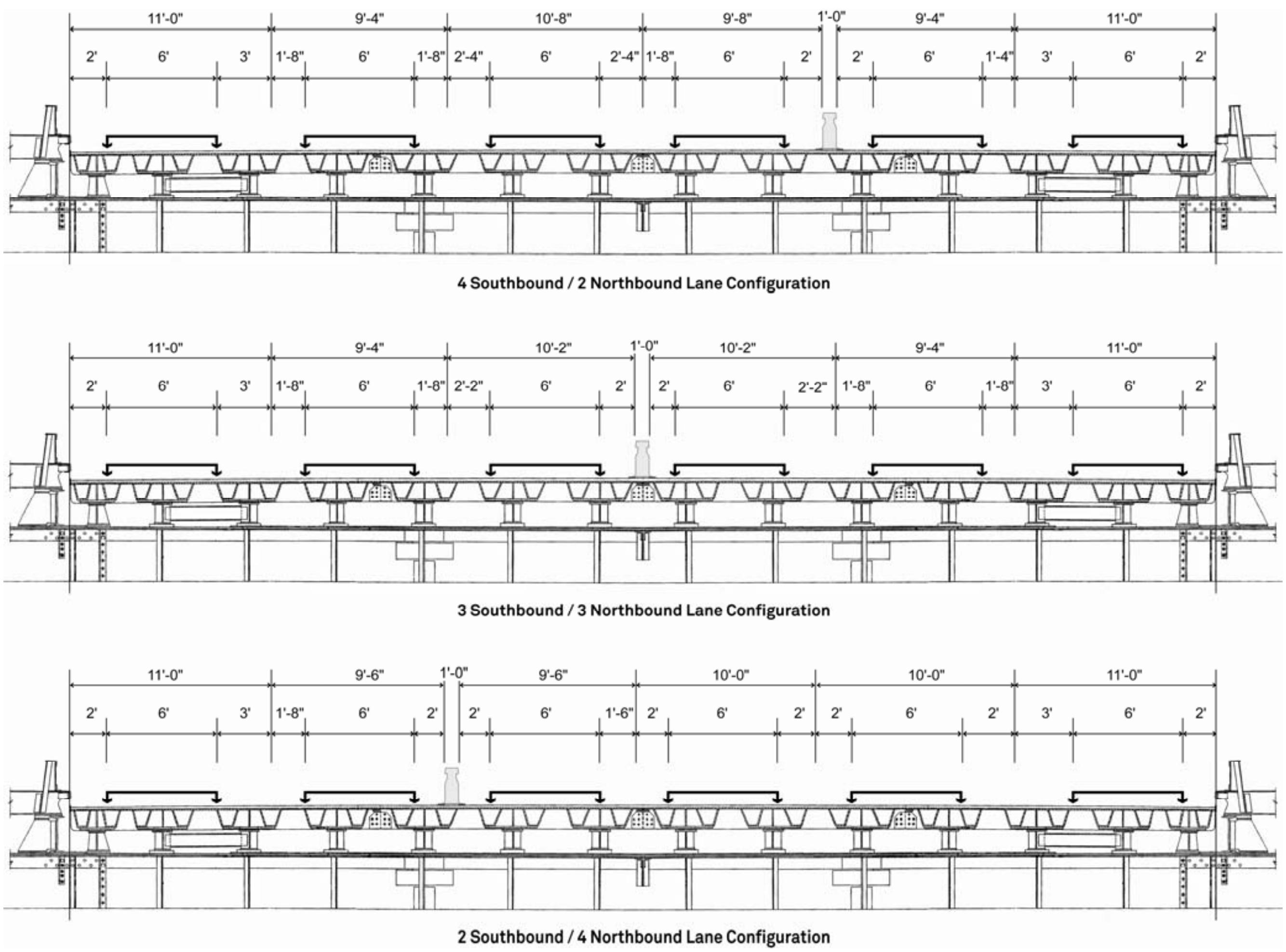
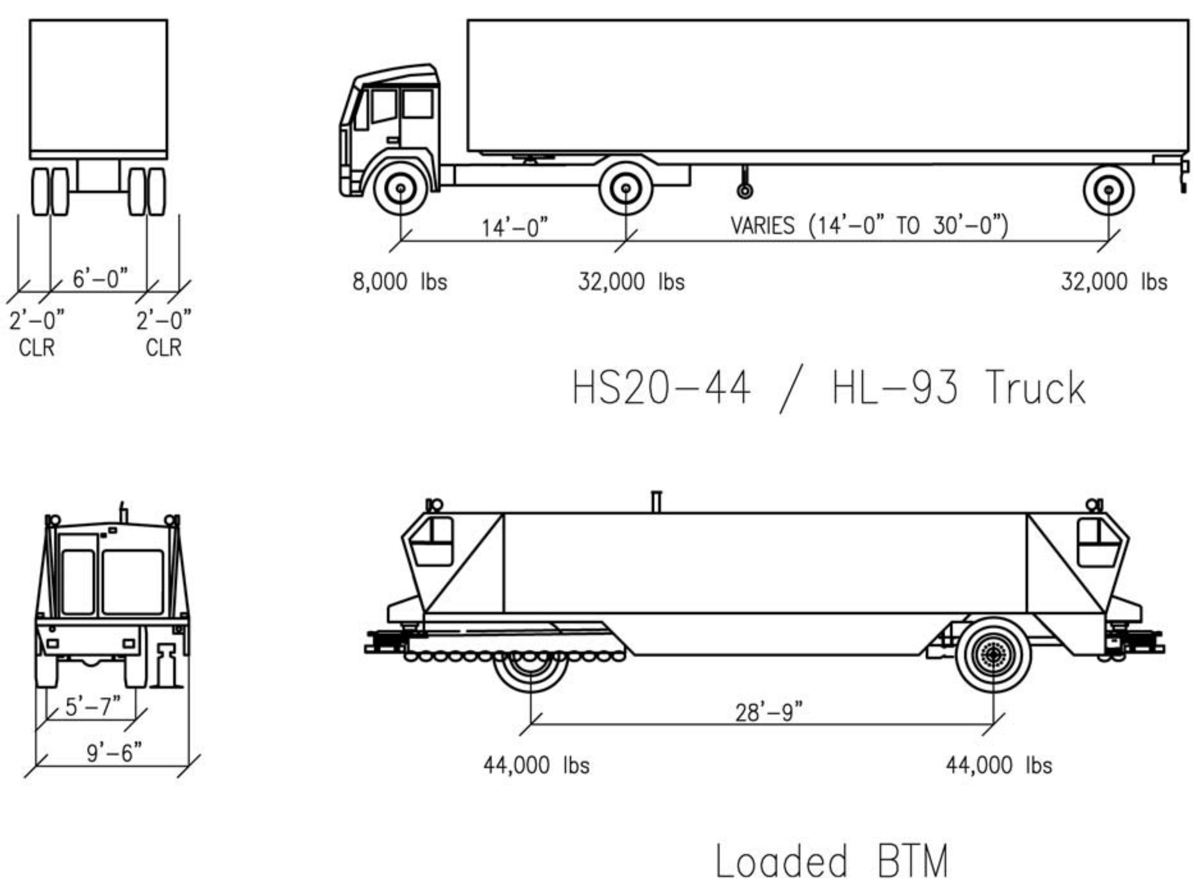


Figure 10: HS20-44 Truck (same as HL-93 Truck) Compared to Loaded BTM



Structural Component Evaluation

The demand and capacity of each structure component are determined using the methods described in the "Approach and Assumptions" section, and are further discussed below.

Orthotropic Deck

The stresses of the deck top plate and of the rib bottom plate are evaluated using Method 1 described in the "Approach and Assumptions" section. Finite element analysis is used to determine the maximum stresses due to dead plus live load (BTM, HL-93, Tandem and Lane Loads) per AASHTO LRFD Bridge Design Specification, 2007. The SAP finite element analysis model is shown in **Figure 11**.

1. **Stress Check**

   Demand: Stresses on the top deck plate and the bottom plate of the rib per finite element analysis.

   Capacity: 20 ksi allowable stress per As-Built Sheet G-3, General Notes.

2. **Local Deflection Check**

   Demand: Displacement per finite element analysis.

   (a) **Displacement Limit per AASHTO LRFD 2007 Section 2.5.2.6.2**

   The following provisions shall apply to orthotropic plate decks:

Vehicular Load Location	Provision
Vehicular load on deck plate:	Span/300
Vehicular load on ribs of orthotropic metal decks:	Span/1000
Vehicular load on ribs of orthotropic metal decks (extreme relative deflection between adjacent ribs):	0.10"

   (b) **Displacement Limit per Caltrans BDS 2004**

Vehicular Load Location	Provision
Vehicular load on deck plate:	Span/300
Vehicular load on ribs of orthotropic metal decks:	Span/500

Sub Floor Beams

Demand-1: Maximum moment and shear demand due to six lanes of HS20-44 loading plus the MMB load.

Assumed Minimum Capacity-1: Moment and shear demand from six lanes of HS20-44 loading.

Demand-2: Maximum moment and shear demand due to six lanes of HL-93 loading plus the MMB load.

Assumed Minimum Capacity-2: Moment and shear demand from six lanes of HL-93 loading.

This check is to determine if the BTM will stress the rib more than the existing condition. The parametric study shows this approach is applicable. See **Appendix A** for calculation details.

Steel Plate Girder

Demand: Moment and shear demand due to the MMB system.

Assumed Capacity: Moment and shear due to change in the deck dead load from deck replacement.

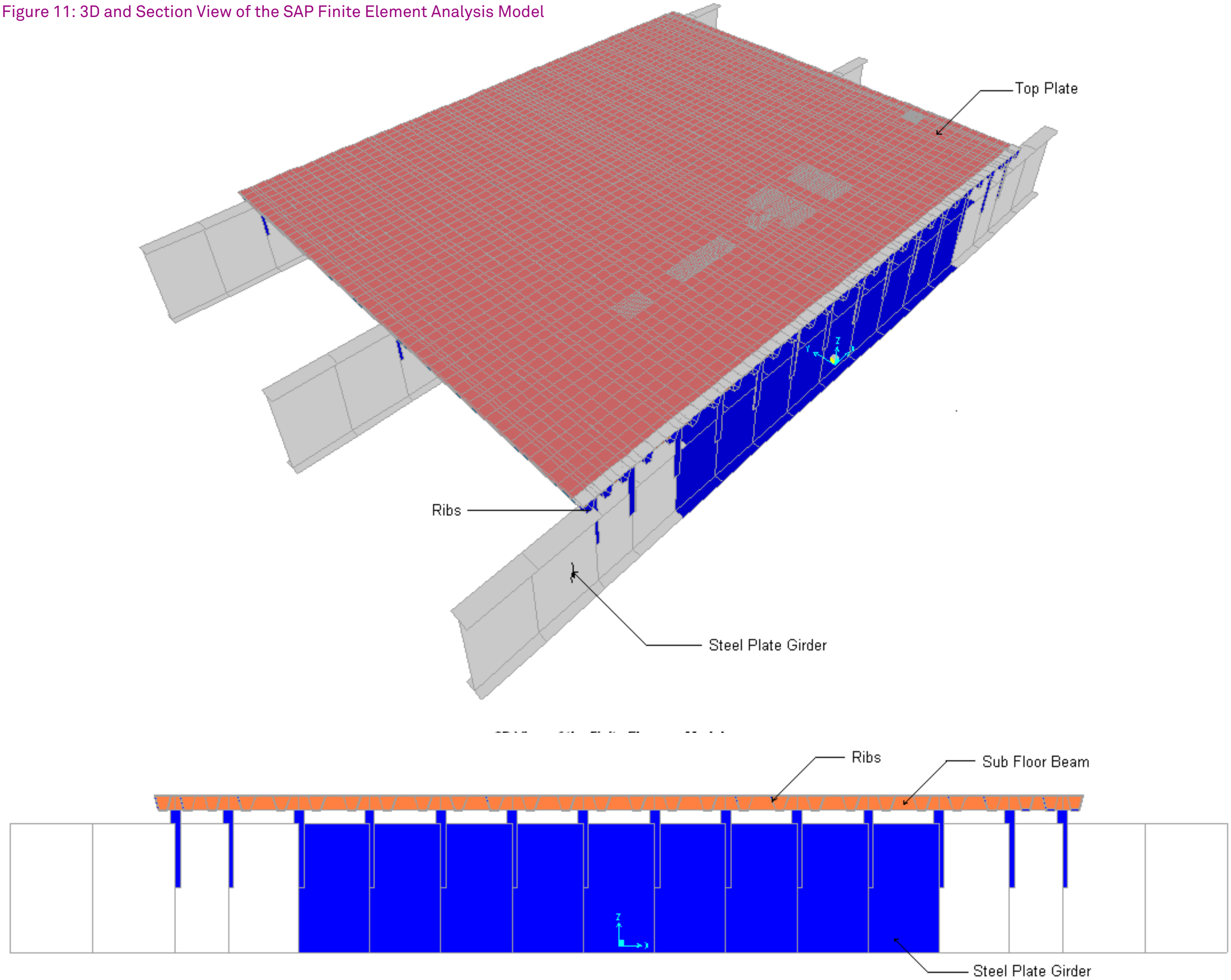
The Steel Plate Girder is evaluated using the dead load relieved during the 1983 deck replacement. This assumes the design live load demand before and after the deck replacement is the same, and the Steel Plate Girder is alleviated of some load under the service condition of dead load plus live load.

The live load before and after the deck replacement are assumed to be the same because the design lane load for the original deck is likely to be H-20-35 of the 1935 Specifications of AASHTO. The H-20-35 lane load is the same as the HS20-44 lane load used for the deck replacement. Since the lane loads are the same, the truck loads are likely the same. (The As-Built drawings from the 1930s can be used to verify this assumption).

Global Deflection Evaluation

The added dead load, due to the MMB, is compared to the current dead load of the Bridge. The purpose of this is to evaluate the percentage change the deflected shape of the Suspension Bridge due to the MMB system. The current dead load of the Bridge is determined by subtracting the dead load reduction during the 1983 deck replacement from the original Bridge dead load, as shown in the 1930's As-Built plans.

Figure 11: 3D and Section View of the SAP Finite Element Analysis Model



Section View of the Finite Element Model

Section Four
Analysis Results

The orthotropic deck meets the stress check. The finite element analysis shows the maximum deck stress is 0.8ksi (less than five percent) greater than the allowable stress and may be considered adequate. The orthotropic deck also meets the local deflection limit per Caltrans BDS 2004 even though it exceeds the deflection limit per AASHTO LRFD. It is anticipated that no increase in roadway maintenance will be needed due to the addition of the MMB system, especially if the Bridge has been tested or traveled with the maximum live load effect per AASHTO LRFD.

The sub floor beams and the steel plate girders are expected to perform adequately with the addition of the MMB system. This will result in a maximum D/C ratio of 1.03.

The analysis results are shown below using the following abbreviations:

- Md = factored moment demand
- Mc = moment capacity
- Vd = factored shear demand
- Vc = shear capacity
- D/C = demand-capacity ratio

Structural Component Evaluation

Orthotropic Deck Stress Check

a. Deck Top Plate
Max. Stress = 15ksi
Allowable Stress = 20ksi
D/C: 0.75, OK

b. Rib Bottom Plate
Max. Stress = 20.8ksi
Allowable Stress = 20.0ksi
D/C: 1.04, considered OK

Local Deflection Check

a. Deck plate:
Maximum Vertical Displacement = 0.028in
Deflection Limit = 0.047in

b. Ribs:
Maximum Vertical Displacement = 0.50in
Deflection Limit = 0.30in (per AASHTO LRFD)
Deflection Limit = 0.60 in (per Caltrans BDS)

The live load combination meets the Caltrans BDS requirement. The service load case without the BTM governs:

HL-93 + Tandem (with lane load) = 0.5in
BTM + Tandem (with lane load) = 0.49in

c. Extreme relative deflection between adjacent ribs:
Maximum Relative Vertical Displacement = 0.19in
Displacement Limit = 0.10in (per AASHTO LRFD)

No live load combination meets the AASHTO LRFD requirement, but this deflection limit check is not required by Caltrans BDS. The service load case without the BTM governs:

HL-93 + Tandem (with lane load) = 0.187in
BTM + Tandem (with lane load) = 0.186in

Sub Floor Beams

Md-1 (HS20-44 + MMB): 26.6 k-ft
Assumed Minimum Mc-1(HS20-44): 26.2 k-ft
D/C: 1.02

Vd-1 (HS20-44 + MMB): 43.3 k
Assumed Minimum Vc-1(HS20-44): 43 k
D/C: 1.01

Md-2 (HL-93 + MMB): 30.04 k-ft
Assumed Minimum Mc-1(HL-93): 29.04 k-ft
D/C: 1.03

Vd-2 (HL-93 + MMB): 48.5 k
Assumed Minimum Vc-1(HL-93): 48.3 k
D/C: 1.00

Steel Plate Girder

Md: 312 k-ft
Assumed Mc: 789 k-ft
D/C: 0.40

Vd: 7.7 k
Assumed Vc: 31.3 k
D/C: 0.24

Global Deflection Evaluation

- Original Suspended Span dead load = 77,200 tons
- Assumed maximum dead load reduction of the Suspended Span during deck replacement = 12,300 tons
- Assumed current Suspended Span dead load = 77,200-12,300 = 64,900 tons
- Additional load due to MMB = 1,580 tons
- Increase in dead load due to MMB = 1,580/ 64,900 = 2.4%
- Thus, assumed change in the deflected shape of the Suspended Span = 2.4%

Section Five
Further Investigation

The following may need to be investigated in the future work phase to fully address the adequacy of the Bridge deck.

Steel Pedestals
The Steel Pedestal is in the direct load path. Although it is a small and rigid element, axial, shear, and possibly bending effect may need to be evaluated.

Connection Details
The connection details of the orthotropic deck may need to be evaluated for strength, fatigue, and general good practice per AASHTO requirement. Bridge Inspection Report(s) will be needed to determine the condition of the existing connection details.

End Notes

[i] Barrier Systems Inc Technical Brief 980204 Rev. 4, "Steel Reactive Tension System Quickchange Moveable Barrier (SRTS-12" QMB)

[ii] Barrier Systems, Inc. MS 060126 Rev. 1, "Specifications for "Products", Barrier Transfer Machine Specifications (Mid Sized Permanent Machines)

2.1.19.4.3
0117

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GOLDEN GATE BRIDGE
HIGHWAY & TRANSPORTATION DISTRICT

APPENDIX
Draft Structural Analysis Report
Golden Gate Bridge Moveable Median Barrier Study
Prepared for the Golden Gate Bridge Highway and Transportation District

Federal Project Number STPL-6003(037)

Appendix Table of Contents

Appendix A:
Detailed Calculations

EVALUATION OF ORTHOTROPIC DECK

AECOM	Job Title:	GGB Moveable Median Barrier			Subject:	Orthotropic Deck Check
	Job No:	60097852				
	Prepared By:	LL	Date:	9/24/2010		
	Checked By:	HSL	Date:	10/15/2010		

Orthotropic Deck - Finite Element Analysis

1. STRESS CHECK

Purpose:
To determine the maximum stresses on the top deck plate and rib bottom plate of rib of orthotropic deck due to dead load plus live load (BTM, HL-93, Tandem and Lane Loads) per current AASHTO LRFD Bridge Design Specification, 4th ed., 2007.

Procedure: Use SAP2000 to perform a finite element analysis:

- Model the deck as 2 continuous 25-ft spans by 62ft wide.
- Model the deck (which includes the top plate and the ribs) with 4-node thin-shell elements.
- Model the sub floor beams with 4-node thin-shell elements.
- Model the pedestals and the steel plate girders with 2-node frame (beam) elements.
- Top deck plate thickness = 5/8”
- Rib plate thickness = 3/8”
- Sub floor beam thickness = ½”
- Apply dead load from the 2" wearing surface as area loads and self weight
- Apply live load from BMT, HL-93, Tandem and Lane Loads
- Examine different service load combinations effects.

Service Load Combination Per AASHTO LRFD Bridge Design Specification, 4th ed., 2007,

1. DL + BTM
2. DL + HL-93
3. DL + Tandem (with lane load)
4. DL + BTM + HL-93
5. DL + BTM + Tandem (with lane load)

Check the stresses on top of plate and bottom plate of rib against allowable stress of 20ksi per As Built Drawing G-3 General Note. Conclude the deck is adequate if its maximum total stress (Dead Load + Live Load) under one and two-lane load configurations is smaller than the allowable stress of 20ksi.

Assumption:
Since BTM moves in a lower speed (10mph during operation per correspondence from Barrier System Inc.) than a typical highway truck, the impact factor for BTM is assumed to be 10%, which is the impact factor for crane (pendant-operated bridge cranes) per ASCE7-05 Section 4.11.

Reference:
1. AASHTO LRFD Bridge Design Specification, 4th ed., 2007, with CA Amendments
2. Golden Gate Bridge As-Built Drawing Dated 10/5/1981
3. ASCE 7-05 Section 4.11
4. Technical Brief 980204 Rev. 4, Steel Reactive Tension System Quickchange Moveable Barrier (SRTS-12" QMB), Barrier Systems, Inc.

Orthotropic Deck - Finite Element Analysis (con't)

2. DEFLECTION CHECK

Purpose:
To determine the maximum deflection per AASHTO LRFD 2007 and Caltrans BDS 2004, and to conclude if the MMB system is feasible.

Procedure:
Use SAP2000 to perform a finite element analysis (same as the procedure shown in stress check)
For deflection check only live load combinations are used.

Live Load Combinations Per AASHTO LRFD Bridge Design Specifications 2007

1. BTM
2. HL-93
3. Tandem (with lane load)
4. BTM + HL-93
5. BTM + Tandem (with lane load)

1. Live Load Deflection limit of the orthotropic deck per AASHTO LRFD Section 2.5.2.6.2:
   - a) Vehicular load on deck plate Span/300
   - b) Vehicular load on ribs of orthotropic metal decks Span/1000
   - c) Vehicular load on ribs of orthotropic metal decks (extreme relative deflection between adjacent ribs)..0.10"

2. Live Load Deflection limit of the orthotropic deck per Caltrans Bridge Specifications 2004
   Stiffness Requirements (10.41.4.8)
   - The deflections of ribs, beams, and girders due to live load impact may exceed the limitations in Article 10.6 but preferably shall not exceed 1/500 of their span.
   - To prevent excessive deterioration of the wearing surface, the specified wheel load plus 30-percent impact preferably shall be less than 1/300 of the distance between webs of rib.

Assumption:
Same as stress check

Reference:
Same as stress check

SAP MODEL LAYOUT

AECOM	Job Title:	GGB Moveable Median Barrier			Subject: SAP Model Layout
	Job No:	60097852			
	Prepared By:	LL	Date:	9/24/2010	
	Checked By:	HSL	Date:	10/15/2010	

2.1.19.4.3
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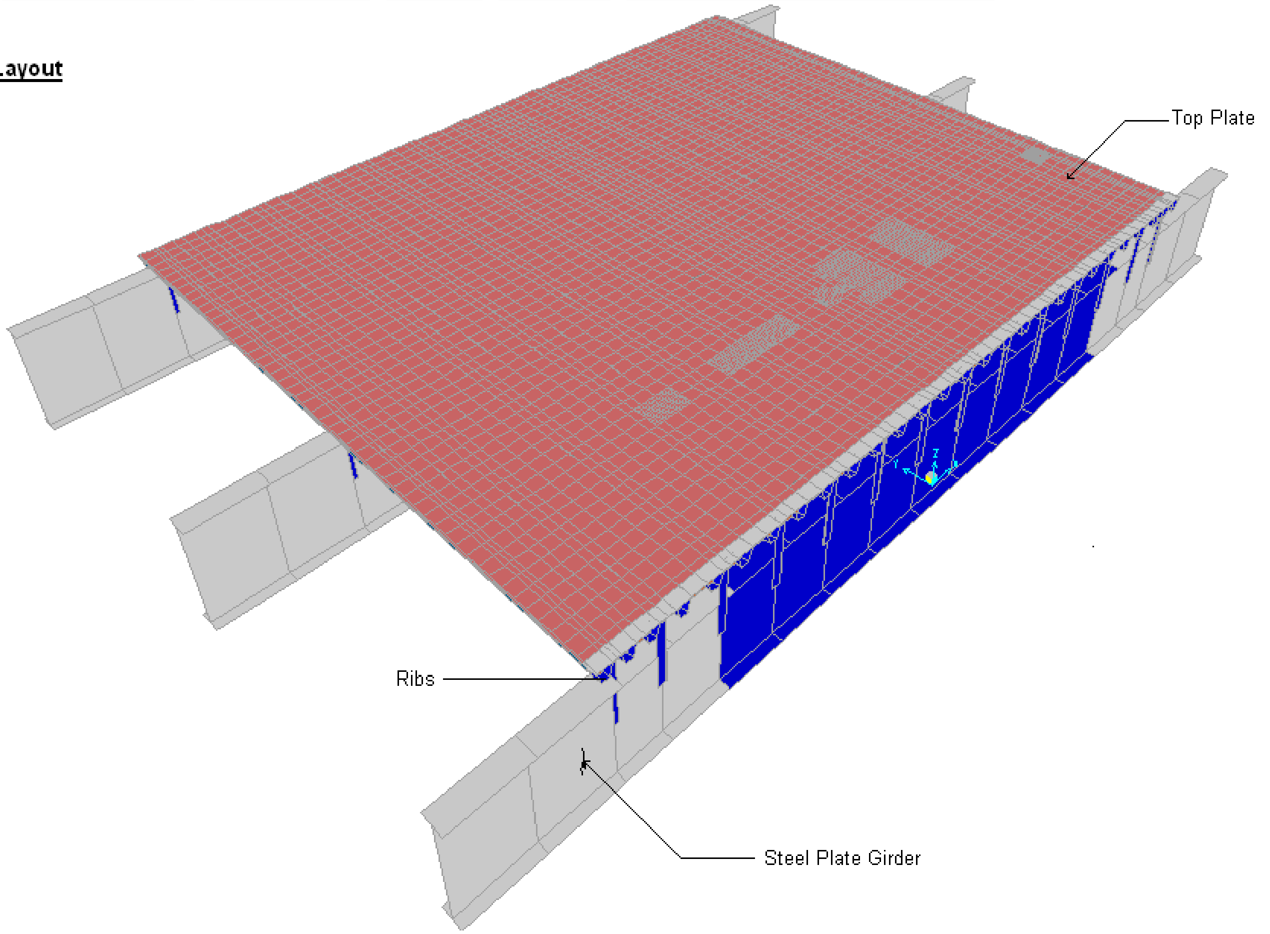
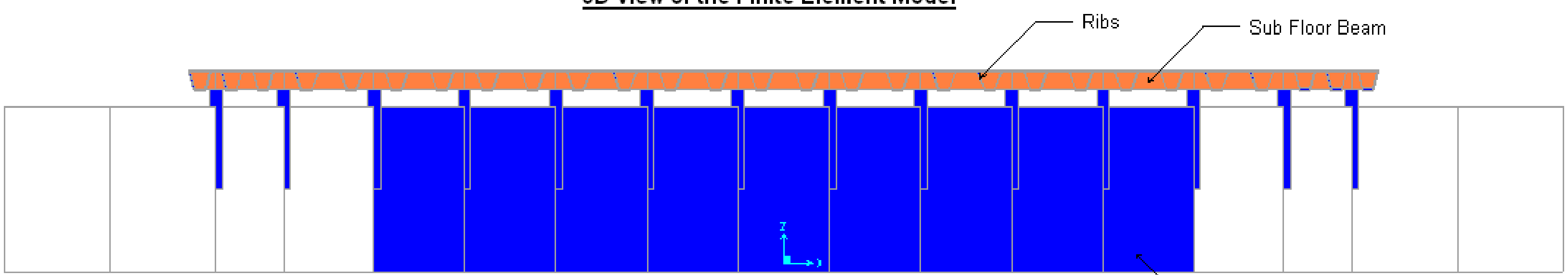
SAP Model Layout



3D View of the Finite Element Model



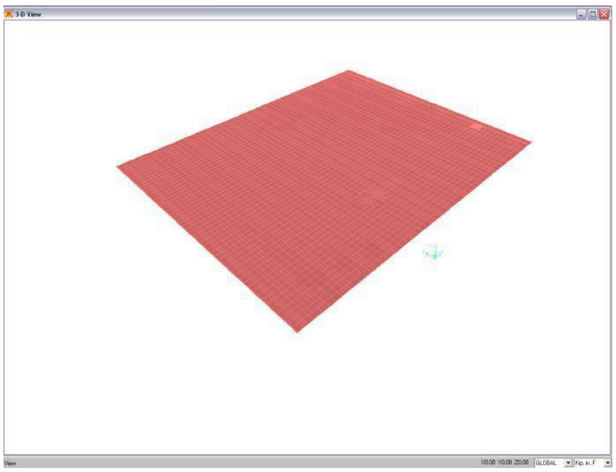
Section View of the Finite Element Model

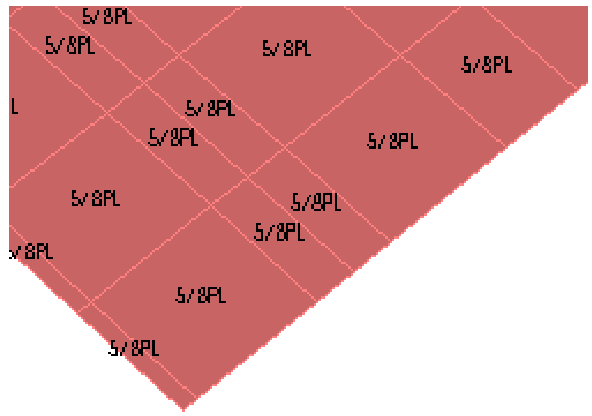
SECTION PROPERTIES
FOR THE COMPONENTS

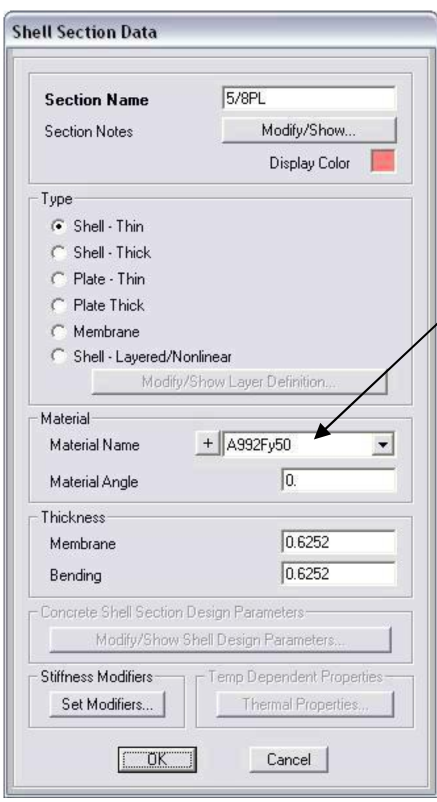
Section Properties for Different Components

– All units are in inches

1. Top Plate







Note: Per Golden Gate Bridge As-Built drawings dated 10/5/1981, material property is ASTM A709, Grade 36T, which has the same modulus of elasticity, E = 29,000ksi as A992 Fy so elastic analysis results from the model are O.K.

2.1.19.4.3
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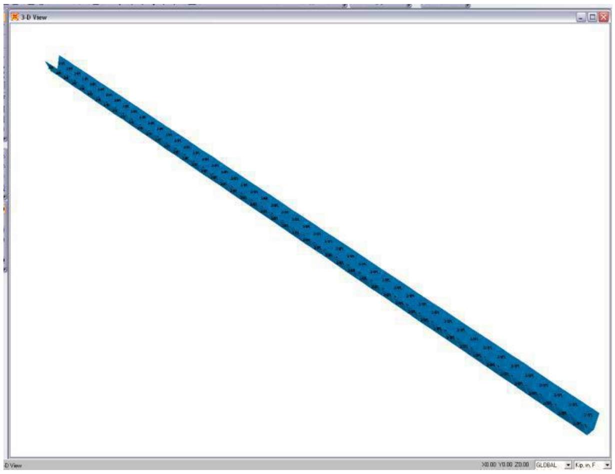
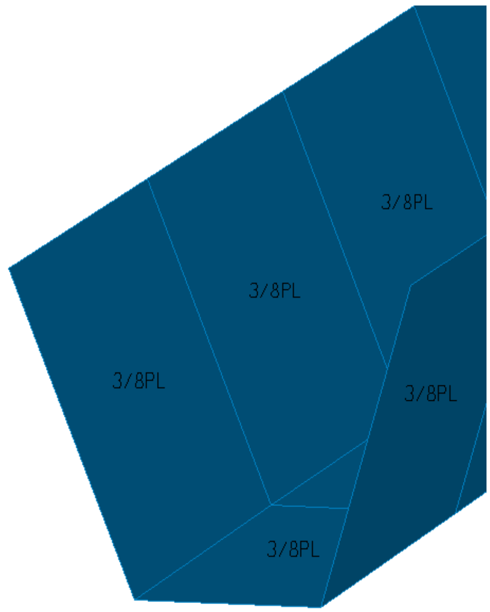
JOB TITLE	Golden Gate Bridge Movable Barrier	ORIGINATOR	LL	DATE	8/25/2010
JOB No.	60097852	CALCU No.		REVIEWER	HSL
				DATE	10/15/2010

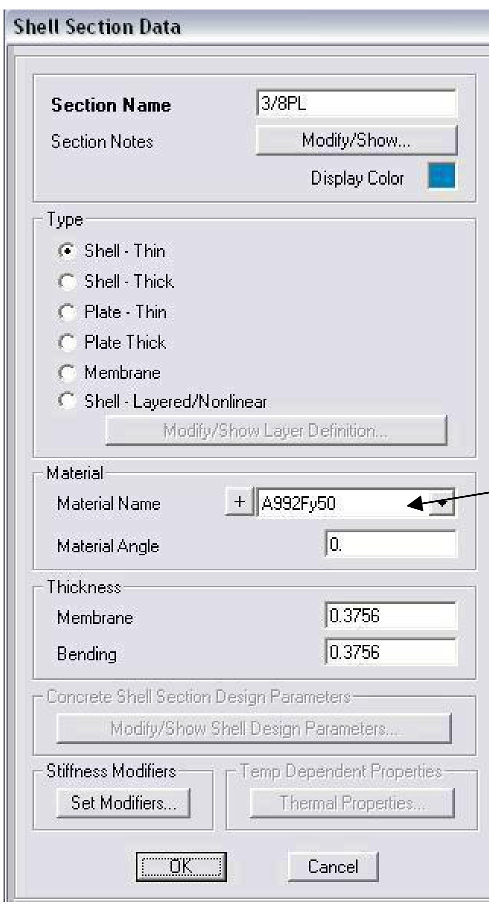
Section Properties for Different Components (con't)

- All units are in inches

2. Ribs





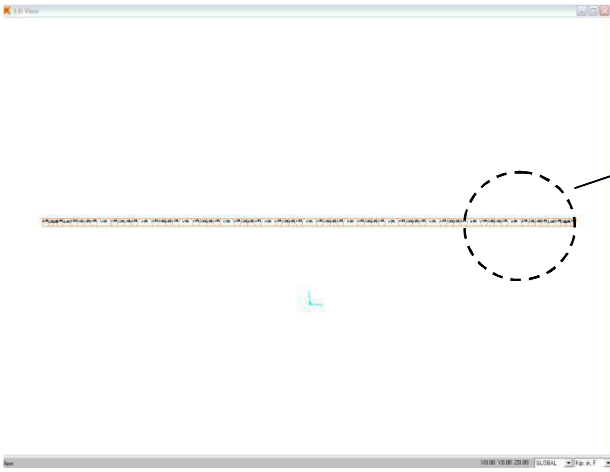


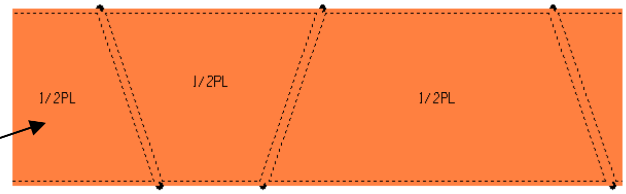
Same E as A709, OK

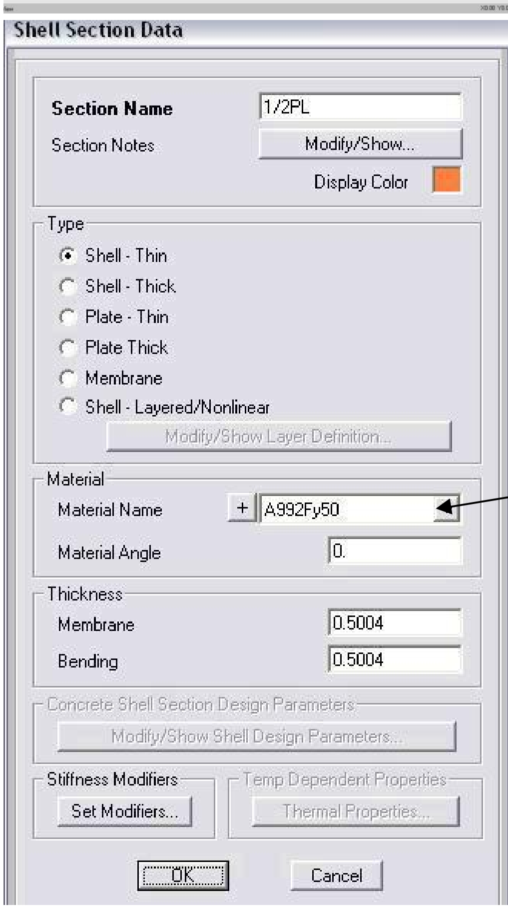
Section Properties for Different Components (con't)

- All units are in inches

3. Sub Floor Beam





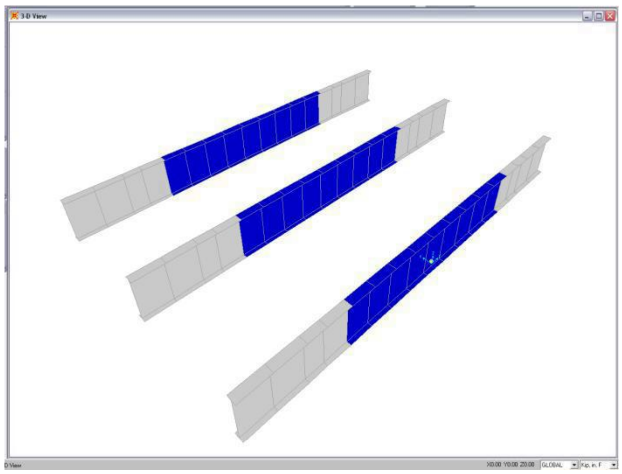


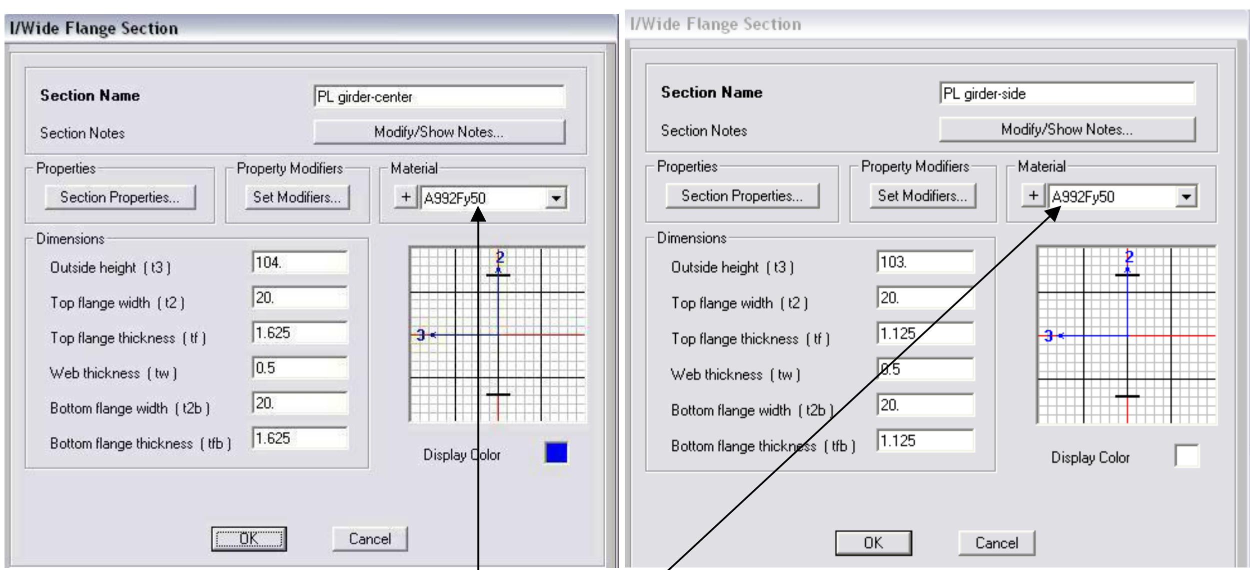
Same E as A709, OK

Section Properties for Different Components (con't)

- All units are in inches

4. Steel Plate Girder





Same E as A709, OK

LOADING

DEAD LOAD

AECOM	Job Title:	GGB Moveable Median Barrier Study			Subject:	Orthotropic Deck Dead Load
	Job No:	60097852				
	Prepared By:	MG/PC	Date:	8/3/2010		
	Supervised By:	HSL	Date:	10/15/2010		

Dead Load: Othrotropic Deck

Constants:

Width of Bridge Deck $W := 62ft$

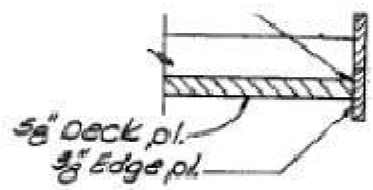
Unit Weight of Steel $\gamma_{steel} := 490pcf$

Unit Weight of Concrete $\gamma_{conc} := 150pcf$

Orthotropic Deck:

Edge Plate Area (2 total) $A_{pl\_ed} := 2 \cdot \frac{3}{8} in \cdot 4.5ft = 0.28\,ft^2$

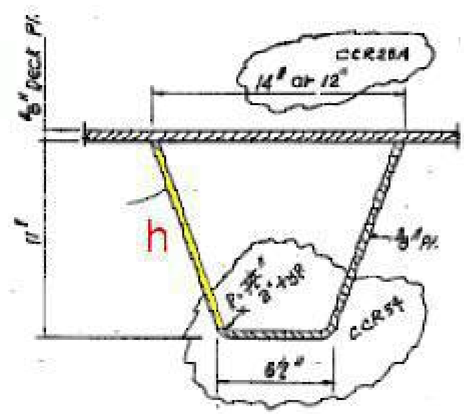
Deck Plate Area $A_{pl\_t} := \frac{5}{8} in \cdot 62ft = 3.23\,ft^2$



Trapezoidal Tub Area
(28 total)

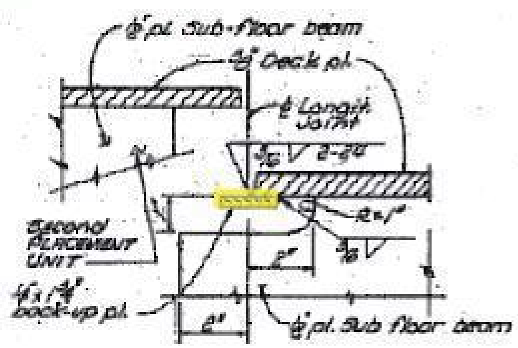
$h := \sqrt{11^2 + 3.75^2} = 11.62$

$A_{tub} := \frac{3}{8} in \cdot (2 \cdot 11.62in) \cdot 28 = 1.69\,ft^2$



TYPICAL RIB DETAIL

Longitudinal Joint
Backup Plate Area (3 total) $A_{jt} := \frac{1}{4} in \cdot 1.75in \cdot 3 = 0.01\,ft^2$



AECOM	Job Title:	GGB Moveable Median Barrier Study			Subject:	Orthotropic Deck Dead Load
	Job No:	60097852				
	Prepared By:	MG/PC	Date:	8/3/2010		
	Supervised By:	HSL	Date:	10/15/2010		

Total Area of
Steel Components

$$A_{tot} := A_{pl\_ed} + A_{pl\_t} + A_{tub} + A_{jt} = 5.21\,ft^2$$

Self Weight of
Steel Components
(Add 10% for welding and
miscellaneous components)

$$w_{steel} := A_{tot} \cdot \gamma_{steel} \cdot 1.1 = 2810.41\,\frac{lb}{ft}$$

Wearing Surface Area

$$A_{ws} := 2in \cdot 62ft = 10.33\,ft^2$$

Self Weight of Wearing Surface

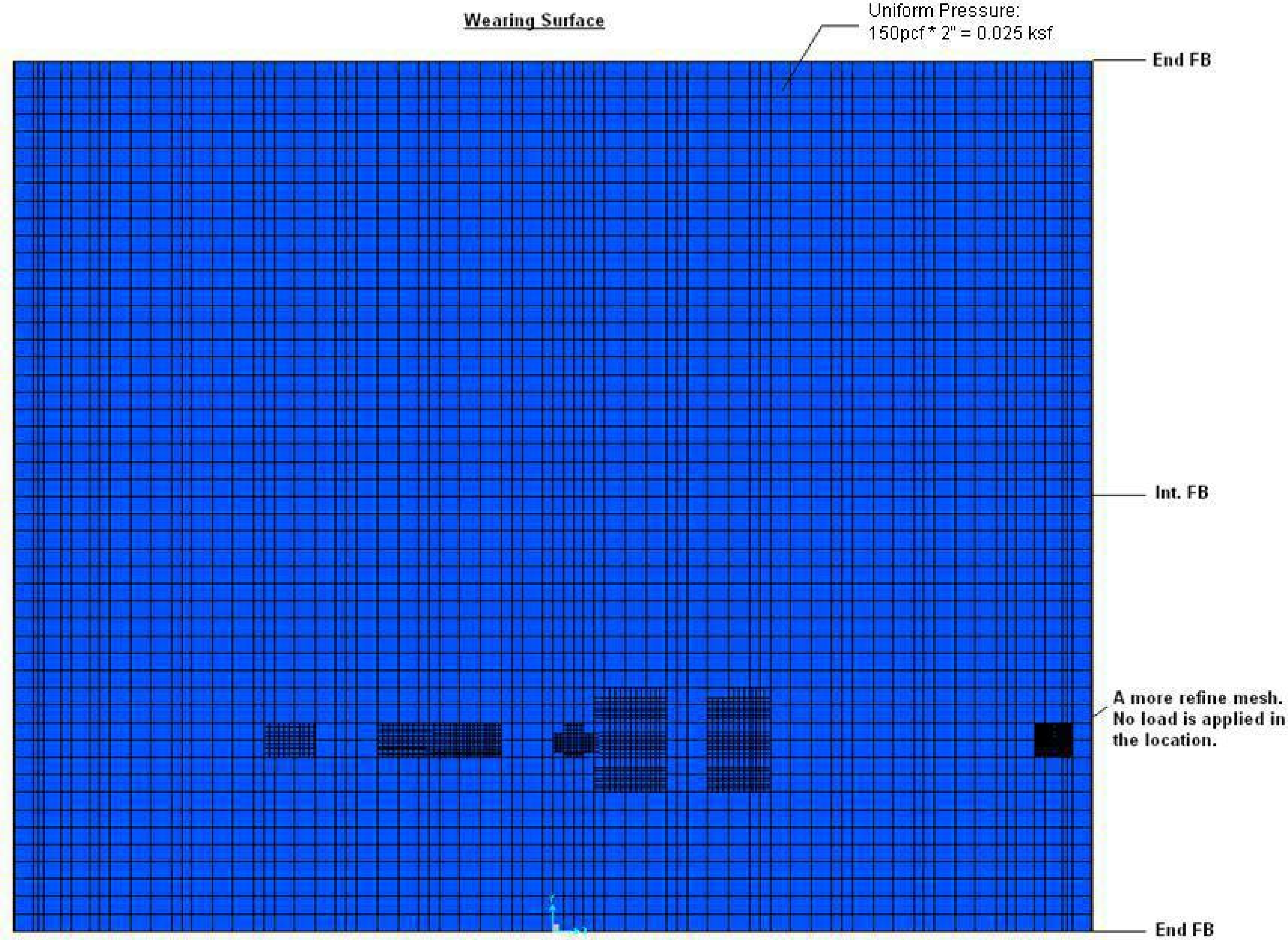
$$w_{ws} := A_{ws} \cdot \gamma_{conc} = 1550\,\frac{lb}{ft}$$

Self Weight of Orthotropic
Deck, DL

$$w_{DL} := w_{steel} + w_{ws} \qquad w_{DL} = 4.36 \cdot \frac{k}{ft}$$

Reference As Built Sheets C-13, C-14,
C-15, C-16, C-18

Wearing Surface



LIVE LOAD

Governing Live Load Case Analysis

Purpose:
To determine which live load cases of BTM wheel load, MMB lane load, AASHTO 2007 HL-93 design truck, design tandems, and design lane load produce maximum shear demand, maximum positive moment demand, and maximum negative moment demand.

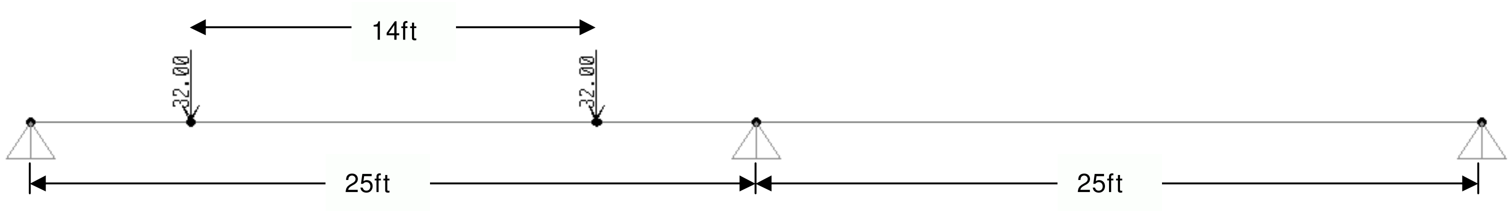
Conclusion:
Comparing maximum positive moment demand and maximum negative moment demand, we found that the maximum positive demand controlled. Thus the load cases that produce the positive moment are analyzed in SAP finite element analysis.

AECOM	Job Title:	GGB Moveable Median Barrier			Subject:	Live Load
	Job No:	60097852				
	Prepared By:	LL /MG	Date:	9/24/2010		
	Checked By:	HSL	Date:	10/15/2010		

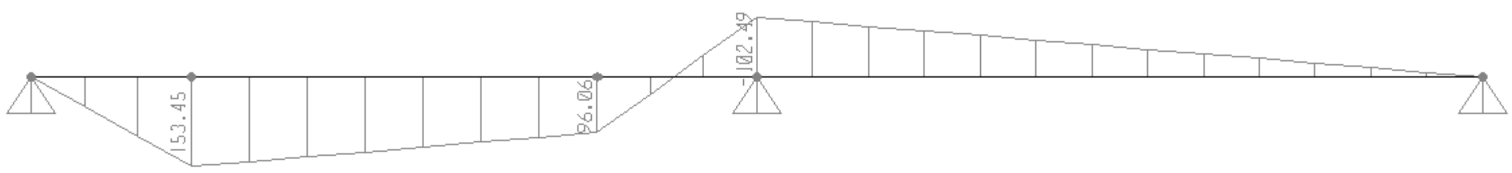
Positive Moment – Case 1: HL93 Truck

Load Layout

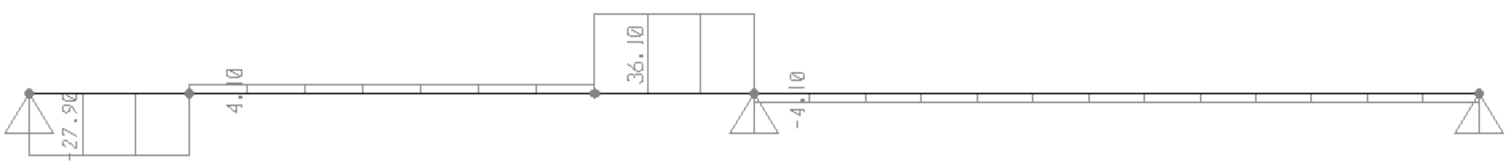
Truck Load = 32k / axle / lane



Moment Diagram (k-ft)



Shear Diagram (k)

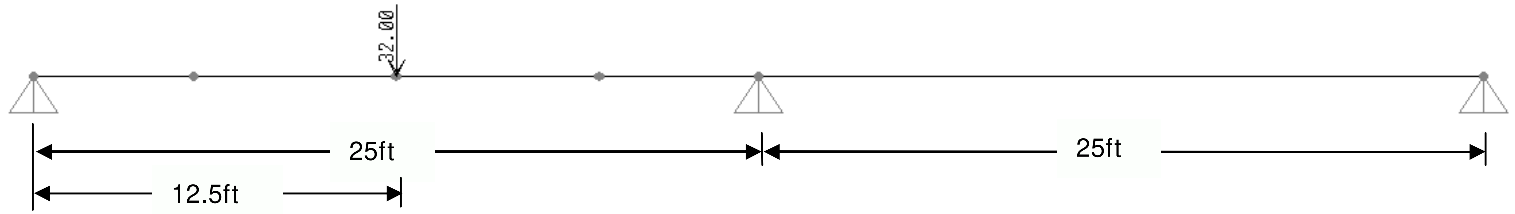


AECOM	Job Title:	GGB Moveable Median Barrier			Subject: Live Load
	Job No:	60097852			
	Prepared By:	LL /MG	Date:	9/24/2010	
	Checked By:	HSL	Date:	10/15/2010	

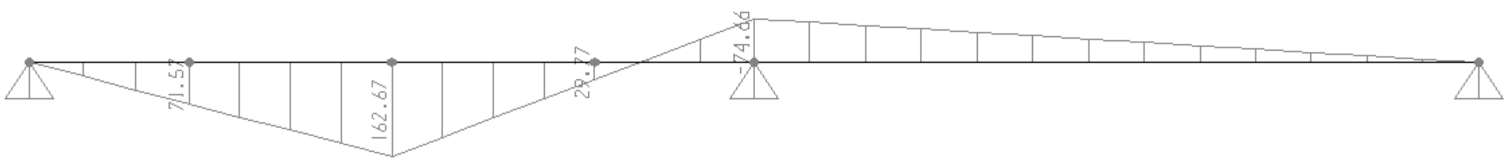
Positive Moment – Case 2: HL93 Truck (shifted)

Load Layout

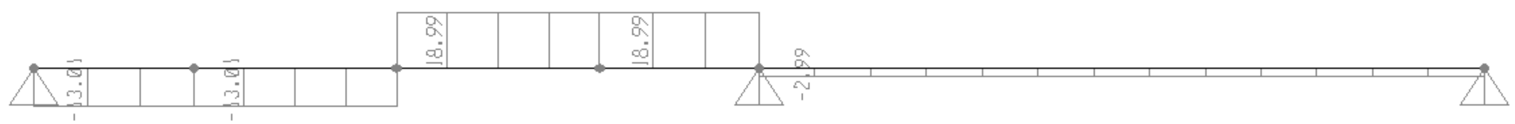
Truck Load = 32k / axle / lane



Moment Diagram (k-ft)



Shear Diagram (k)

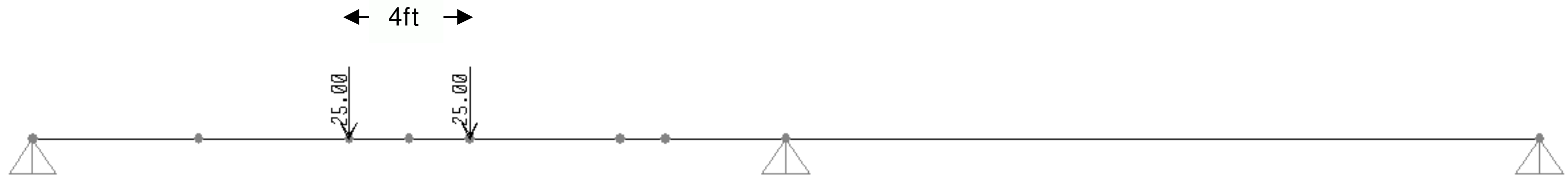


Job Title:	GGB Moveable Median Barrier			Subject: Live Load
Job No:	60097852			
Prepared By:	LL /MG	Date:	9/24/2010	
Checked By:	HSL	Date:	10/15/2010	

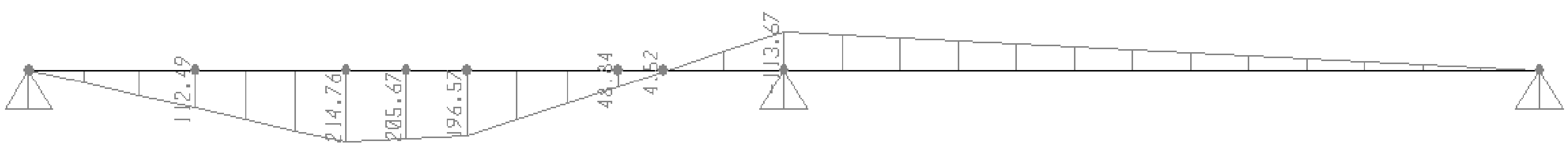
Positive Moment – Case 3: Design Tandem

Load Layout

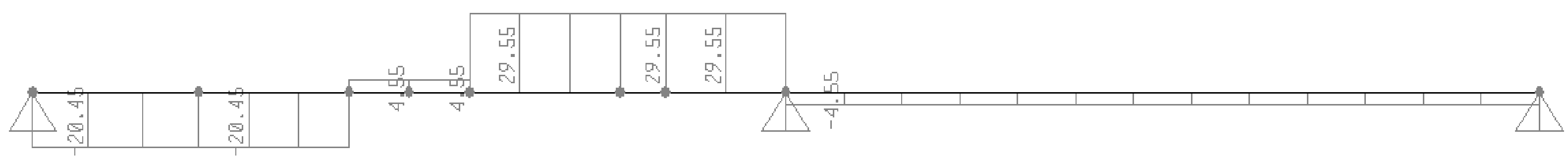
Truck Load = 25k/ axle / lane



Moment Diagram (k-ft)



Shear Diagram (k)

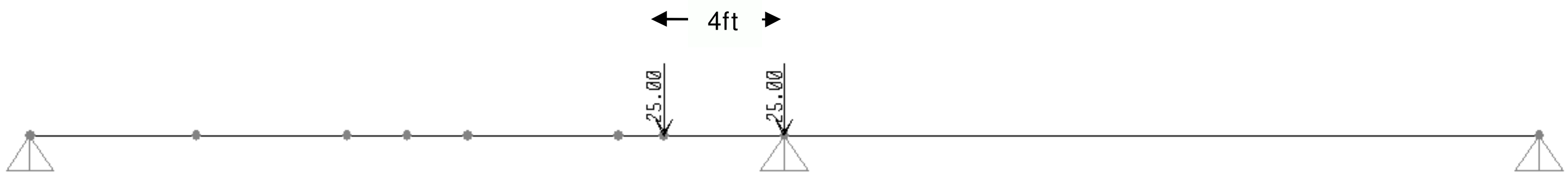


AECOM	Job Title:	GGB Moveable Median Barrier			Subject: Live Load
	Job No:	60097852			
	Prepared By:	LL /MG	Date:	9/24/2010	
	Checked By:	HSL	Date:	10/15/2010	

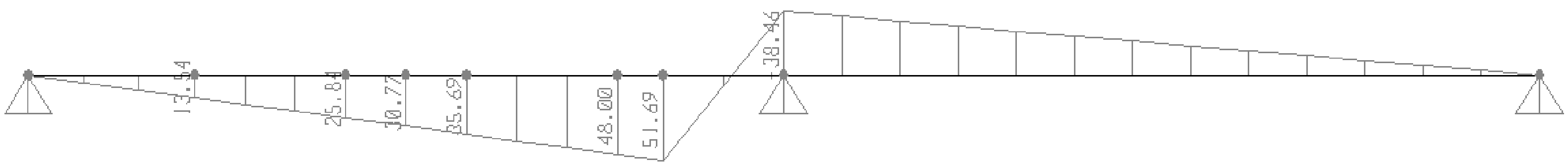
Positive Moment – Case 4: Design Tandem (shifted)

Load Layout

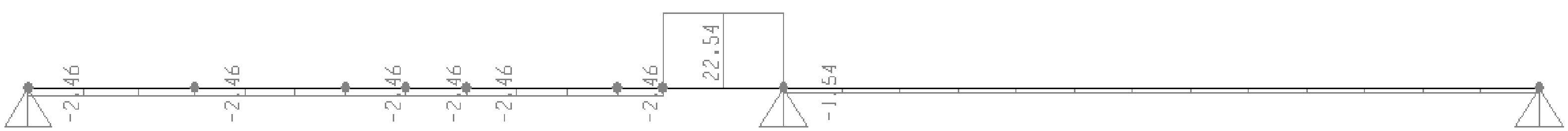
Truck Load = 25k / axle / lane



Moment Diagram (k-ft)



Shear Diagram (k)

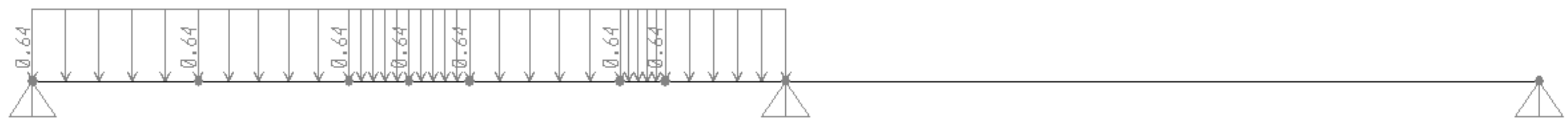


AECOM	Job Title:	GGB Moveable Median Barrier			Subject: Live Load
	Job No:	60097852			
	Prepared By:	LL /MG	Date:	9/24/2010	
	Checked By:	HSL	Date:	10/15/2010	

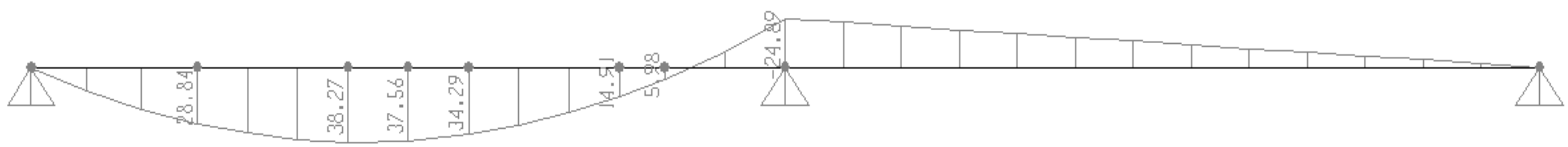
Positive Moment – Case 5: Design Lane Load

Load Layout

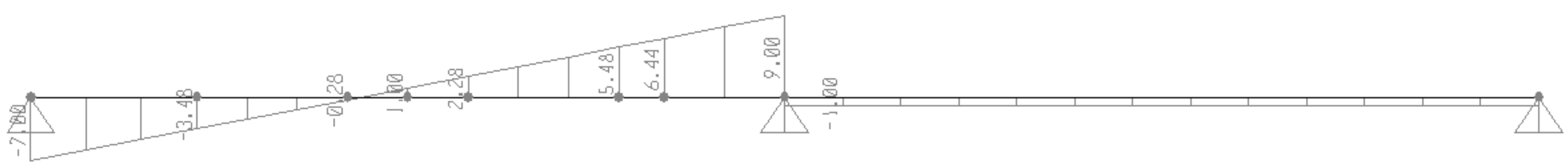
Lane Load = 0.64k/ft/ lane



Moment Diagram (k-ft)



Shear Diagram (k)

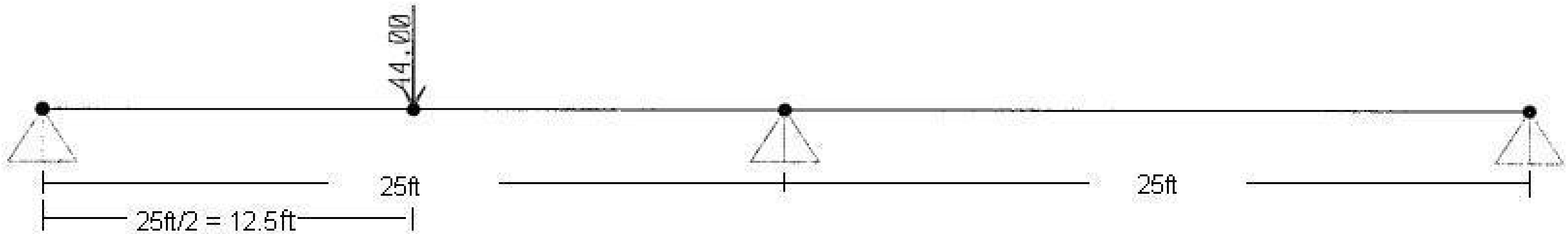


AECOM	Job Title:	GGB Moveable Median Barrier			Subject:	Live Load
	Job No:	60097852				
	Prepared By:	LL /MG	Date:	9/24/2010		
	Checked By:	HSL	Date:	10/15/2010		

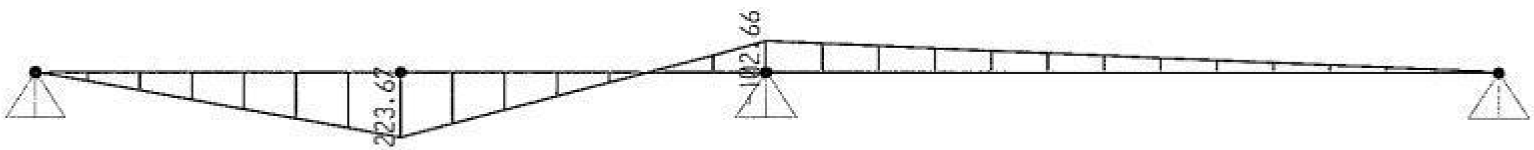
BSI System Positive Moment - Case 1a: BTM

Load Layout

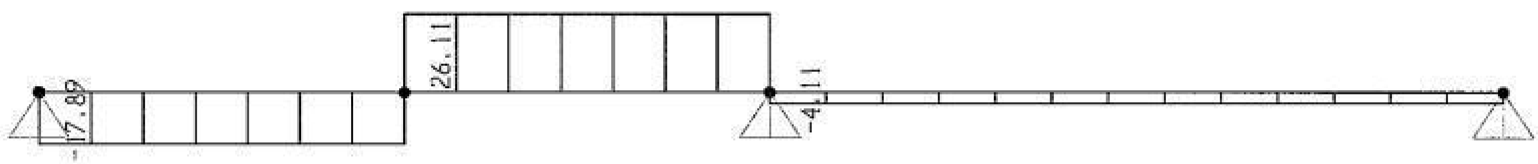
BTM Load = 44k / axle / lane



Moment Diagram (k-ft)



Shear Diagram (k)

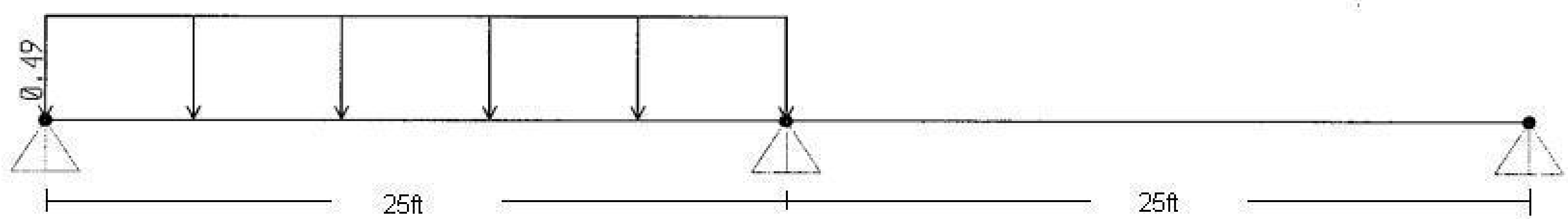


AECOM	Job Title:	GGB Moveable Median Barrier			Subject: Live Load
	Job No:	60097852			
	Prepared By:	LL /MG	Date:	9/24/2010	
	Checked By:	HSL	Date:	10/15/2010	

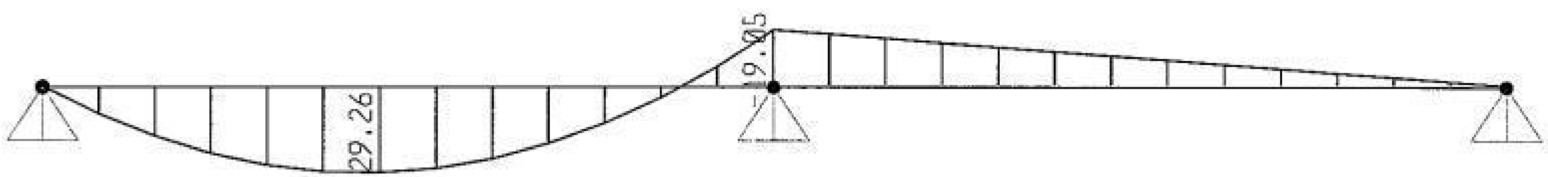
BSI System Positive Moment – Case 1b: MMB

Load Layout

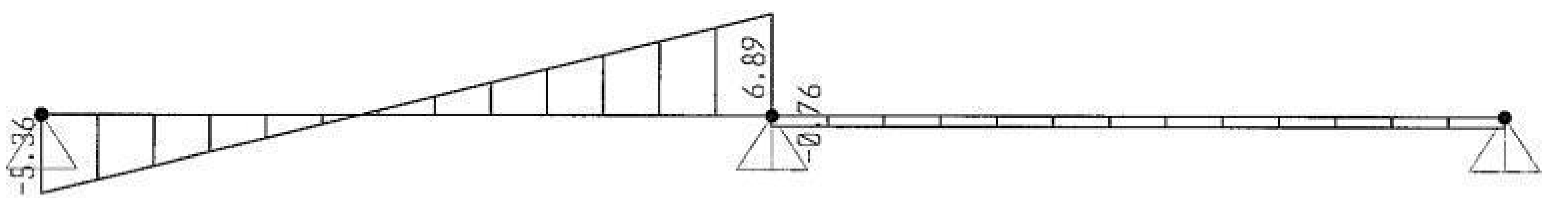
MMB = 0.49 k/ft



Moment Diagram (k-ft)



Shear Diagram (k)

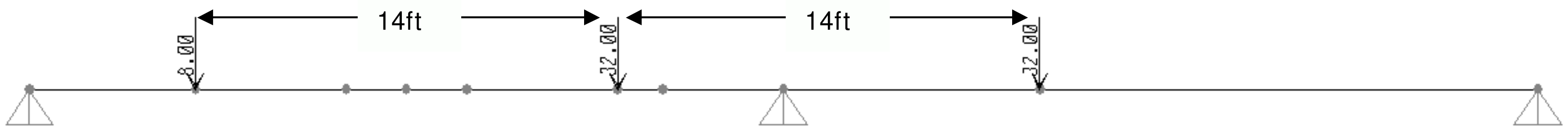


2.1.19.4.3
0117

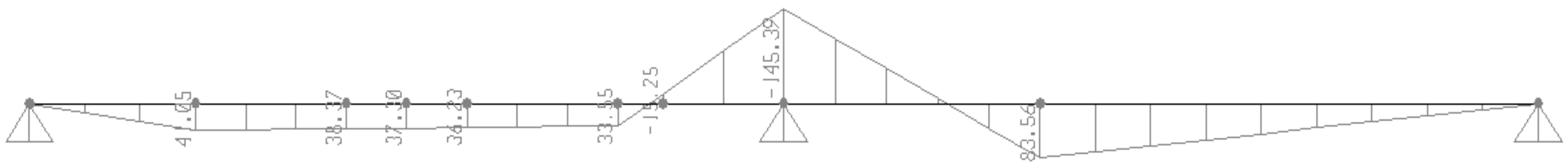
AECOM	Job Title:	GGB Moveable Median Barrier			Subject: Live Load
	Job No:	60097852			
	Prepared By:	LL /MG	Date:	9/24/2010	
	Checked By:	HSL	Date:	10/15/2010	

Negative Moment – Case 1: HL93

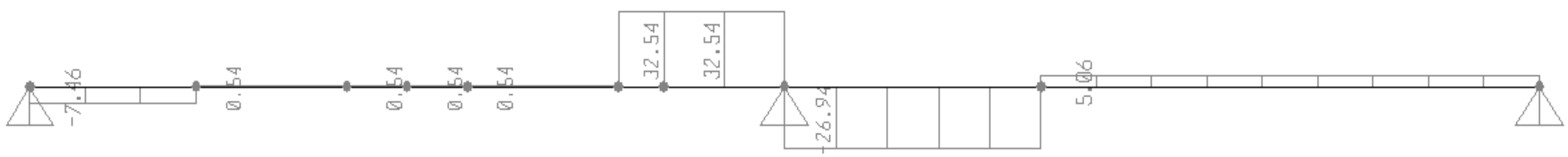
Load Layout



Moment Diagram (k-ft)



Shear Diagram (k)

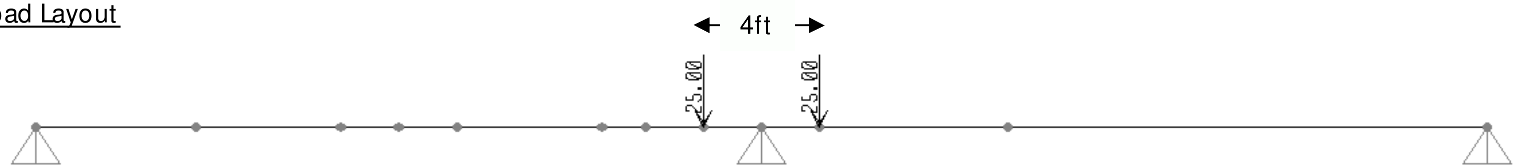


2.1.19.4.3
0117

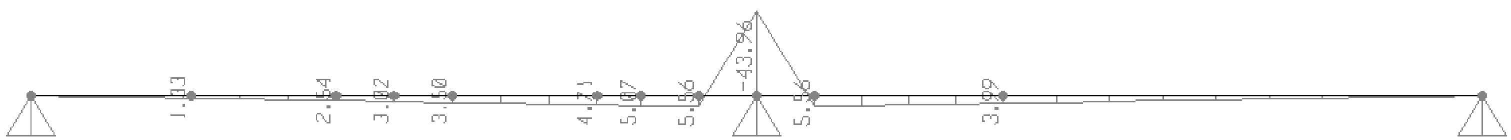
AECOM	Job Title:	GGB Moveable Median Barrier			Subject:	Live Load
	Job No:	60097852				
	Prepared By:	LL /MG	Date:	9/24/2010		
	Checked By:	HSL	Date:	10/15/2010		

Negative Moment – Case 2: One Tandem

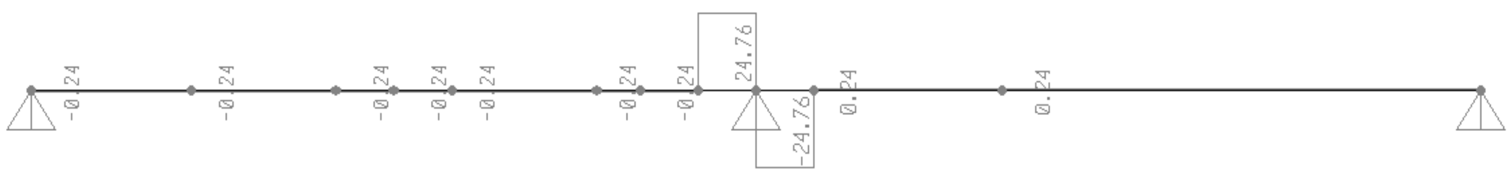
Load Layout



Moment Diagram (k-ft)



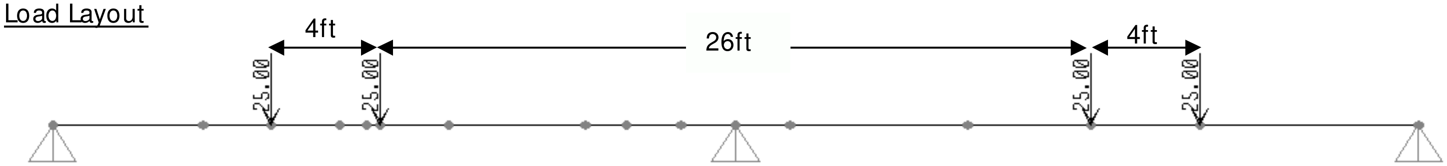
Shear Diagram (k)



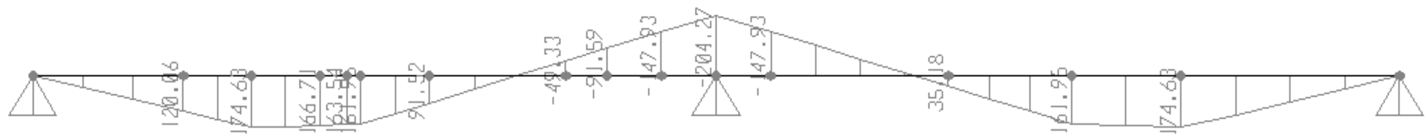
AECOM	Job Title:	GGB Moveable Median Barrier			Subject: Live Load
	Job No:	60097852			
	Prepared By:	LL /MG	Date:	9/24/2010	
	Checked By:	HSL	Date:	10/15/2010	

Negative Moment – Case 3: Two Design Tandems

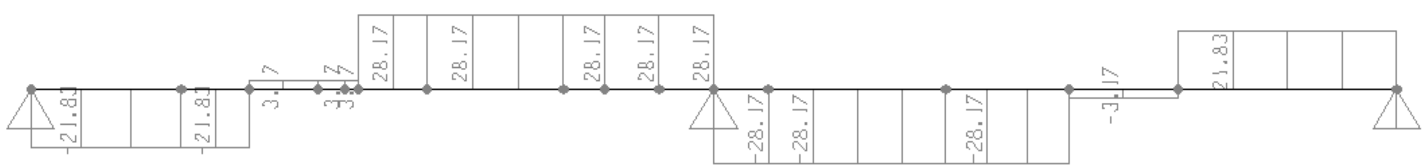
Load Layout



Moment Diagram (k-ft)



Shear Diagram (k)

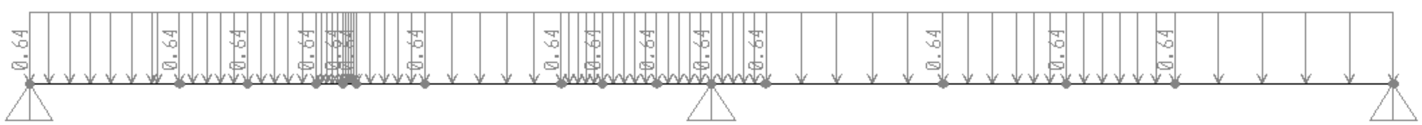


2.1.19.4.3
0117

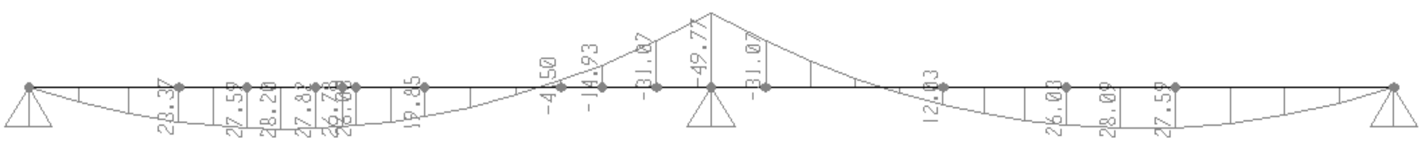
AECOM	Job Title:	GGB Moveable Median Barrier			Subject:	Live Load
	Job No:	60097852				
	Prepared By:	LL /MG	Date:	9/24/2010		
	Checked By:	HSL	Date:	10/15/2010		

Negative Moment – Case 4: Design Lane Load

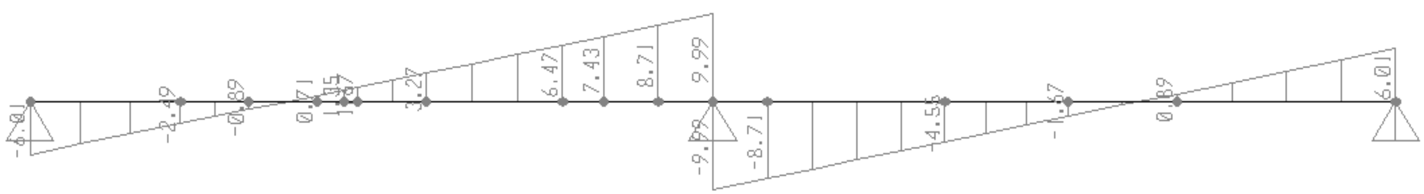
Load Layout



Moment Diagram (k-ft)



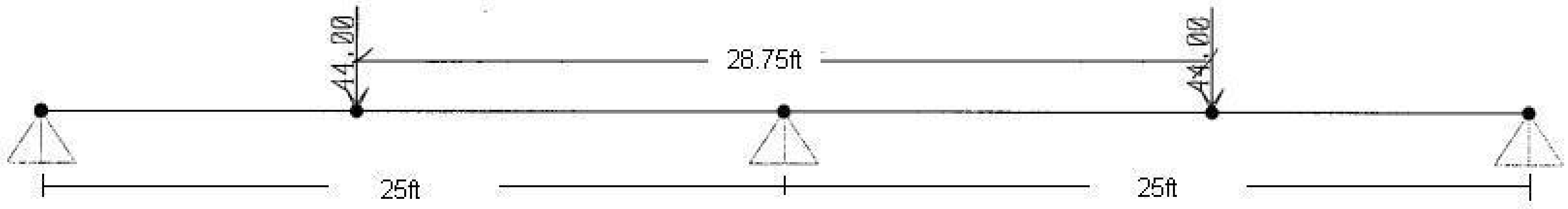
Shear Diagram (k)



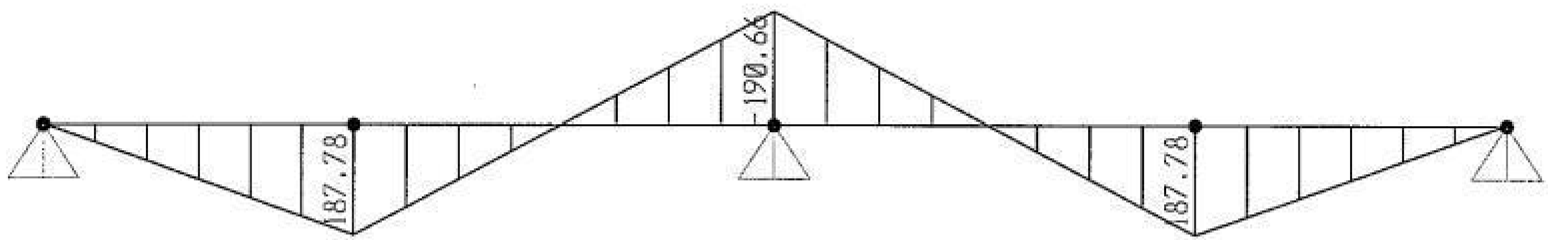
	Job Title:	GGB Moveable Median Barrier			Subject:	Live Load
AECOM	Job No:	60097852				
	Prepared By:	LL /MG	Date:	9/24/2010		
	Checked By:	HSL	Date:	10/15/2010		

BSI System Negative Moment – Case 1a: BTM

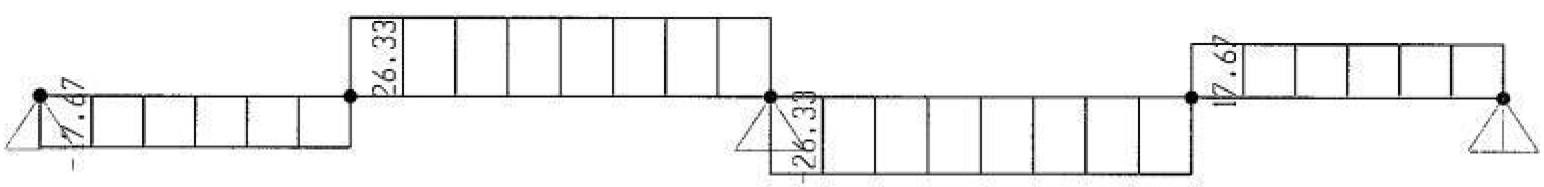
Load Layout



Moment Diagram (k-ft)



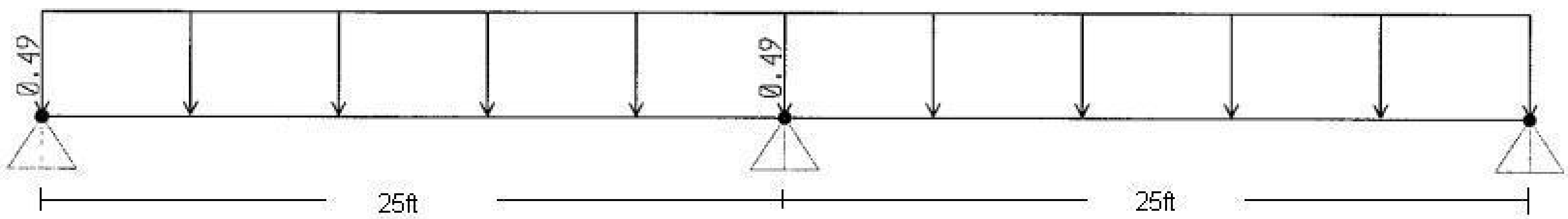
Shear Diagram (k)



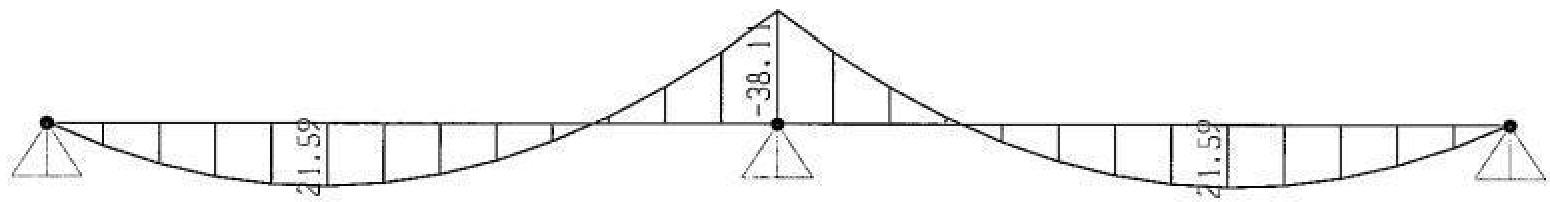
AECOM	Job Title:	GGB Moveable Median Barrier			Subject: Live Load
	Job No:	60097852			
	Prepared By:	LL /MG	Date:	9/24/2010	
	Checked By:	HSL	Date:	10/15/2010	

BSI System Negative Moment – Case 1b: MMB

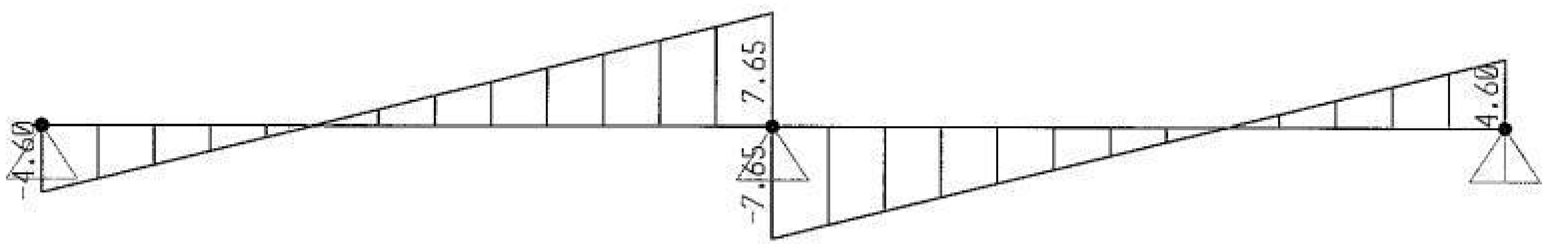
Load Layout



Moment Diagram (k-ft)



Shear Diagram (k)



AECOM	Job Title:	GGB Moveable Median Barrier			Subject:	Orthotropic Deck Check
	Job No:	60097852				
	Prepared By:	LL	Date:	9/27/2010		Live Load
	Checked By:	HSL	Date:	10/15/2010		

Summary:

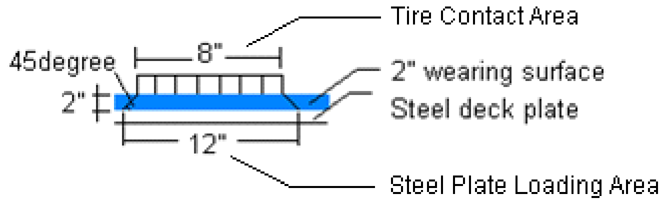
Positive Moment				
		Moment	IM Factor	Moment (with IM)
AASHTO Live Load		k-ft		k-ft
Case 1:	HL93 Truck over one span	153.5	1.33	204.1
Case 2:	HL93 Truck (Truck load shifted)	162.7	1.33	216.4
Case 3:	Design Tandem over one span	214.8	1.33	285.6
Case 4:	Design Tandem (Tandem load shifted)	51.7	1.33	68.7
Case 5:	Lane Load	38.3	1.00	38.3
	Control Case: Case 3			285.6
BSI MMB System				
Case 1a:	One BTM axle loads at the center of one span	223.7	1.10	246.0
Case 1b:	MMB line load applied across one span	29.3	1.10	32.2
	Control Case: Case 1a			246.0

Nagetive Moment				
		Moment	IM Factor	Moment (with IM)
AASHTO Live Load		k-ft		k-ft
Case 1:	HL93 Truck over two spans	145.4	1.33	193.4
Case 2:	One Design Tandem over two spans	44.0	1.33	58.5
Case 3:	Two Design Tandem over two spans	204.3	1.33	271.7
Case 4:	Lane Load	49.8	1.00	49.8
	Control Case: Case 3			271.7
BSI MMB System				
Case 1a:	Two BTM axle loads across two spans	190.7	1.10	209.7
Case 1b:	MMB line load applied across two spans	38.1	1.10	41.9
	Control Case: Case 1a			209.7

Conclusion:
Comparing maximum positive moment demand and maximum negative moment demand, positive moment demand controlled. Therefore, positive moment is used in SAP finite element model.

AECOM	Job Title:	GGB Moveable Median Barrier			Subject:	Orthotropic Deck Check
	Job No:	60097852				
	Prepared By:	LL	Date:	9/27/2010		Live Load
	Checked By:	HSL	Date:	10/15/2010		

Live Load Input in SAP Finite Element Model



1. **For BTM Wheel Load:**
   Tire Contact Area = 8” X 18.6”
   Loading Area = 12”x 22.6”
   Stress = 22k/ (12" x 22.6") = **0.081 ksi**

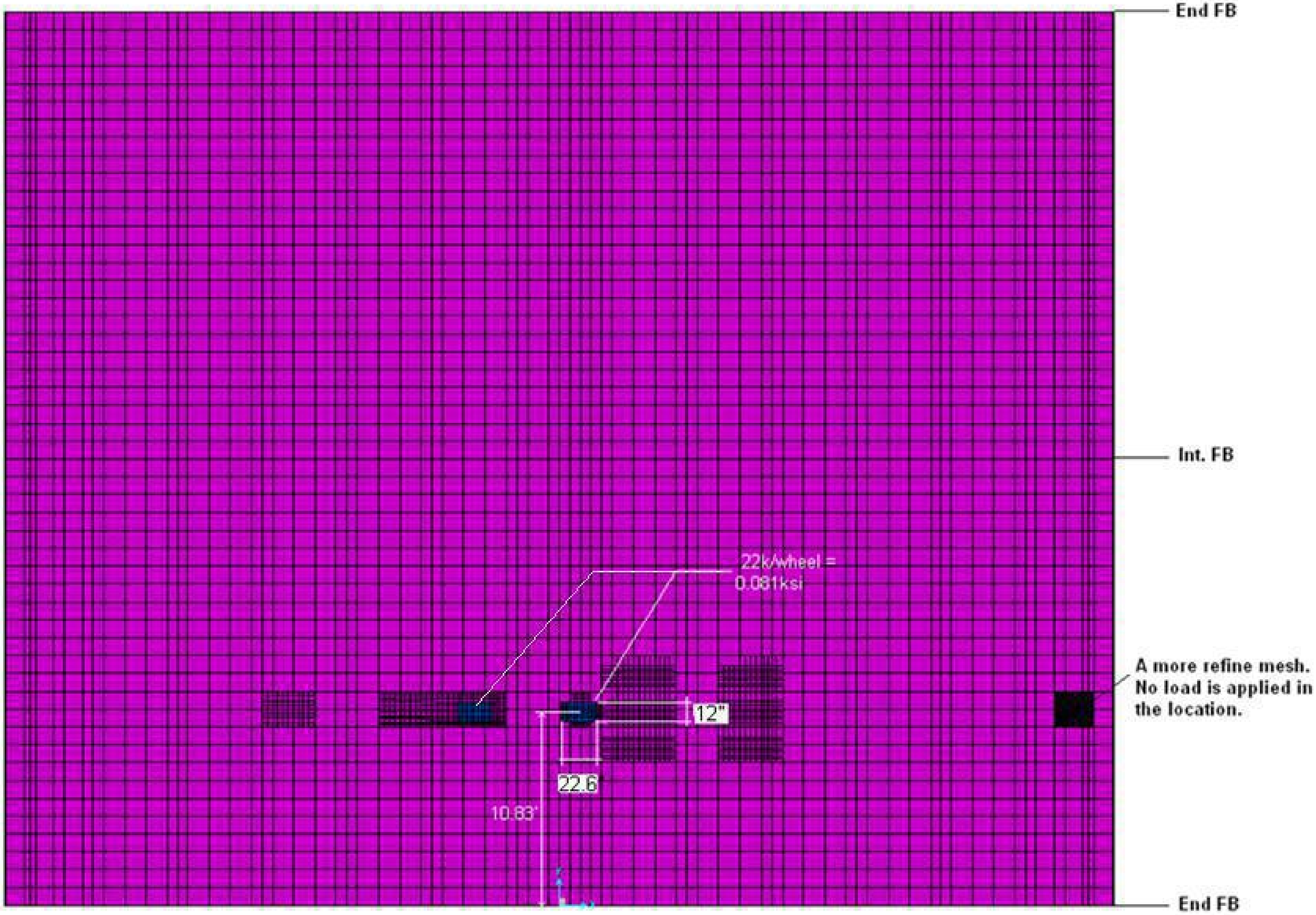
2. **For HL-93 Wheel Load:**
   Tire Contact Area = 10” X 20”
   Loading Area = 14”x 24”
   Stress = 16k/ (14" x 24") = **0.0476 ksi**

3. **For Tandem Wheel Load:**
   Tire Contact Area = 10” X 20”
   Loading Area = 14”x 24”
   Stress = 12.5k/ (14" x 24") = **0.0372 ksi**

4. **For Lane Load:**
   9.5ft is used in the model, instead of 10ft; therefore,
   640 plf/ 9.5ft = **0.000468 ksi**

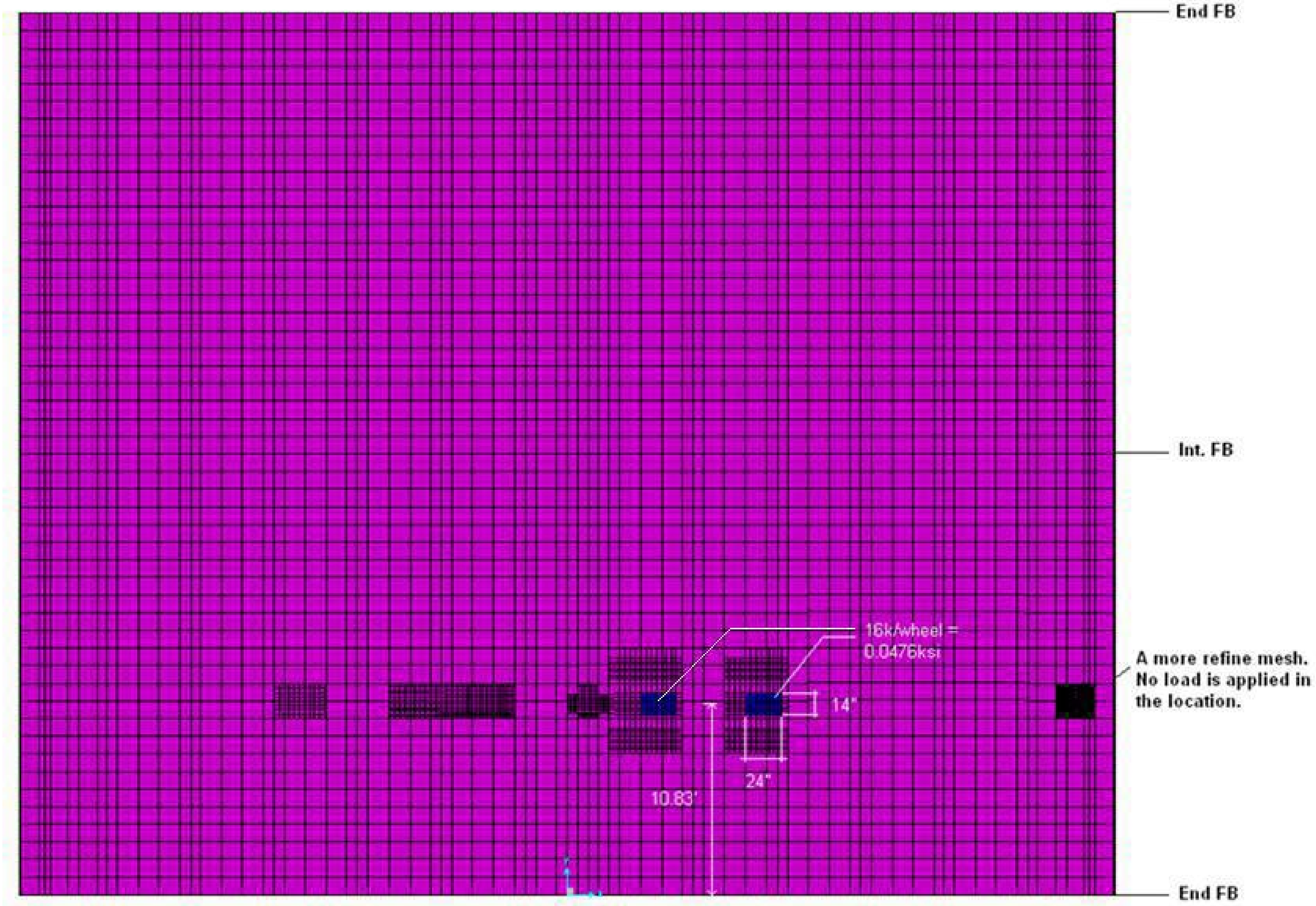
AECOM	Job Title:	GGB Moveable Median Barrier			Subject:	Orthotropic Deck Check
	Job No:	60097852				
	Prepared By:	LL	Date:	9/27/2010	Live Load	
	Checked By:	HSL	Date:	10/15/2010		

BTM Wheel Load @ deck plate



HL-93 Wheel Load

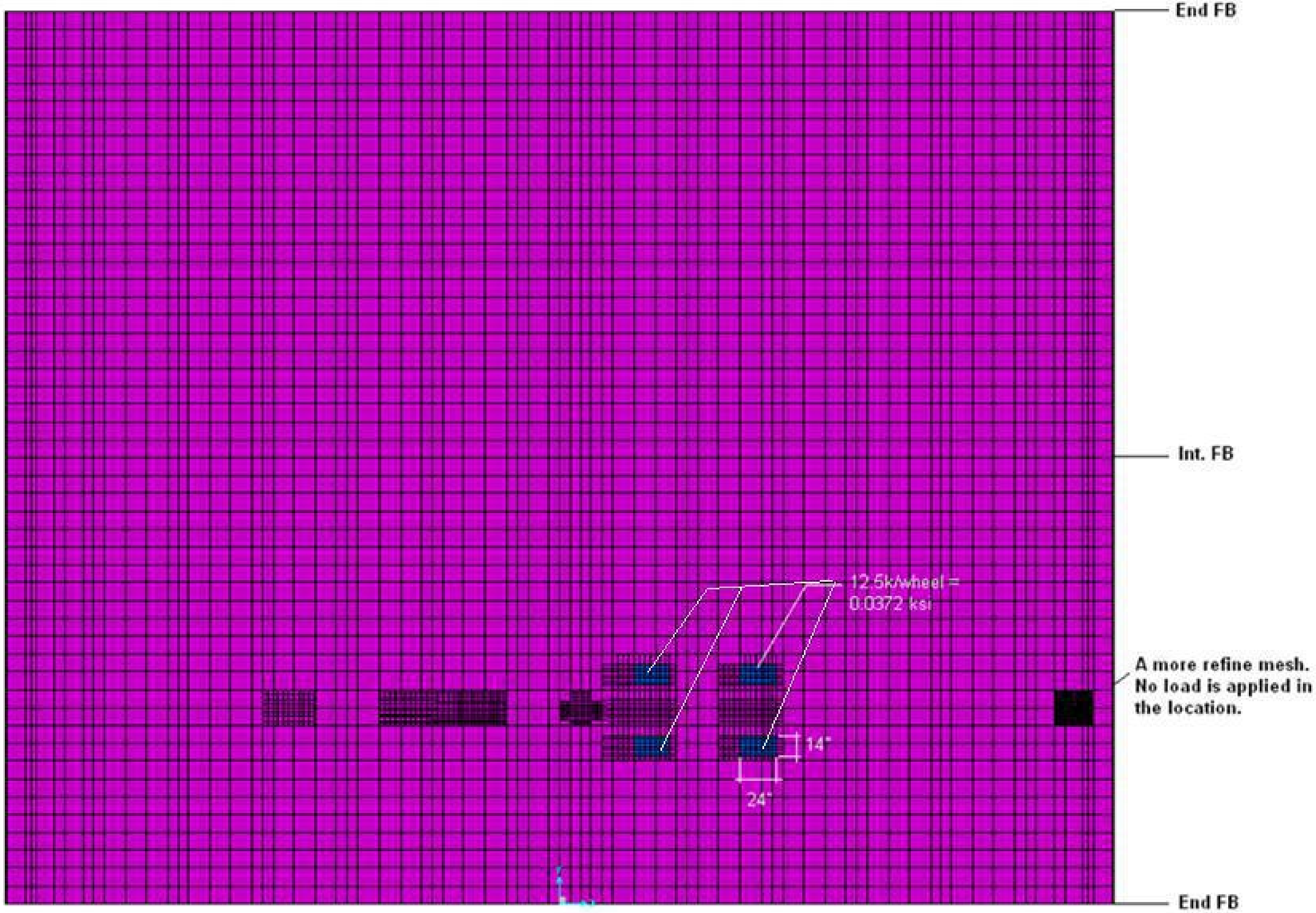
Per AASHTO 2007



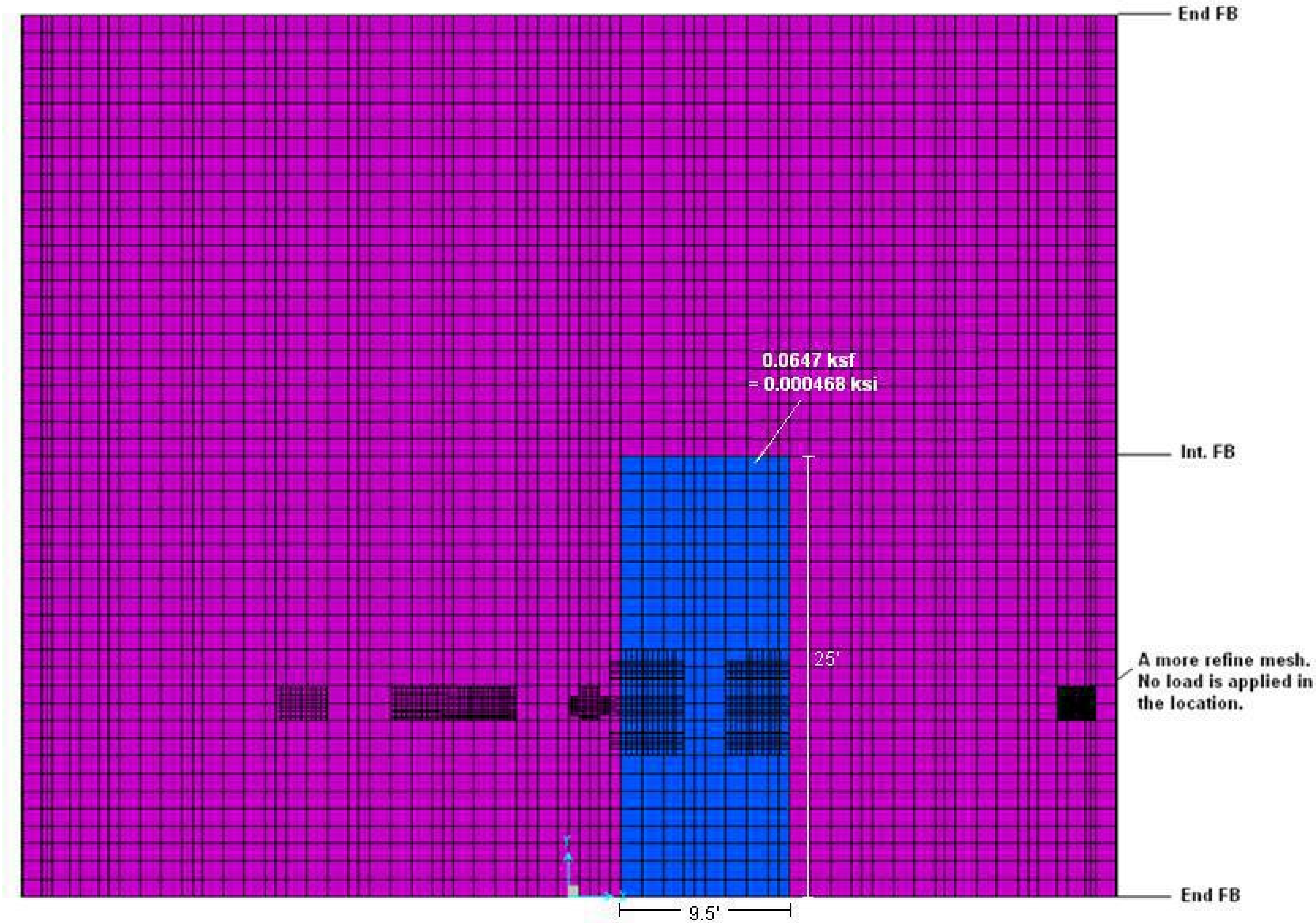
AECOM	Job Title:	GGB Moveable Median Barrier			Subject:	Orthotropic Deck Check
	Job No:	60097852				
	Prepared By:	LL	Date:	9/27/2010	Live Load	
	Checked By:	HSL	Date:	10/15/2010		

Tandem Wheel Loads

Per AASHTO 2007



One-Span Lane Load



AECOM

JOB TITLE GGB Moveable Median Barrier

JOB NO. 60097852 CALCULATION NO. ORIGINATOR PC DATE 8/4/2010

SHEET OF REVIEWER HSL DATE 10/15/2010

SAP Model Verification:

1) Stress @ bott. of the rib due to DL:

M_{max} @ ≈ Mid Span = 193.06 k' (WS + Self wt + 10% of Self wt for Misc. metal, see whole deck check Calc.).

Section Modulus:

$S_{top} = 3462.2 \text{ in}^3$

$S_{bott} = 1600.34 \text{ in}^3$

See whole deck section modulus calc. attached after model verification

$\sigma_{bott} = M/S_{bott} = (193.06/1600.34) \times 12 = 1.45$ ksi

2) Stress @ bott of rib due to DL from SAP: (SAP Model <deck-c.sdb> w/ spring supports.
<deck.sdb> from file above. Steel Pl girder moved to c.g. location

DL, Self Wt = 1.16 ksi

Wearing Surface = 0.72 ksi

Tot. = 1.16 + 0.72 = 1.88 ksi > 1.45 ksi as expected. good.

3) SAP Model <deck-c-deck only.sdb> w/o springs, w/o Steel Plate Girders. w/ Massless sub F.B.

Self wt = 0.96 ksi

WS = 0.58 ksi

Tot. = 0.96 + 0.58 = 1.54 ksi > 1.45 ksi. How come?

4) It's possible that the overlapping of the geometry due to mesh intersection results in larger self wt. Calc. than that 10% increase by hand calc. Verify →.

AECOM

JOB TITLE GGB Moveable Median Barrier

JOB NO. 60097852 CALCULATION NO. ORIGINATOR PC DATE 8/4/2010

SHEET OF REVIEWER HSL DATE 10/15/2010

Self wt of Deck in SAP: <deck-c-deck only.sdb>.

Base Reaction DEAD = 132.6 k.

Self wt. of Deck by hand: (w/10% increase).

2.810 k/ft × 50' = 140.5 k.

No. Self wt in SAP is smaller than by hand calc. Thus the stress @ bott rib. due to DL in SAP should be smaller than by hand calc.

5) It may be that the top plate is not stiff enough, thus in the finite element model, the bending rigidity is actually less than what the hand calc. assumes (that plane-remain-plane assumption is not valid).

→ Verify: Change Model's top PL stiffness to 100 times larger.
<deck-c-deck only2.sdb>.

Bott. of rib stress due to:

Self wt. = 0.81 ksi

WS = 0.48 ksi

Tot. = 0.81 + 0.48 = 1.29 ksi

$1.29 \times (140.5^k / 132.6^k) = 1.38$ ksi

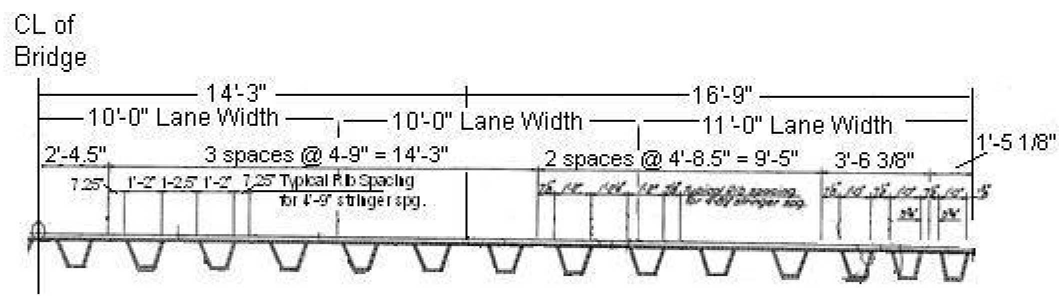
Yes. Stress in the rib ↓ when the Top Plate Stiff is increased.
∴ Say o.k. The Model is valid.

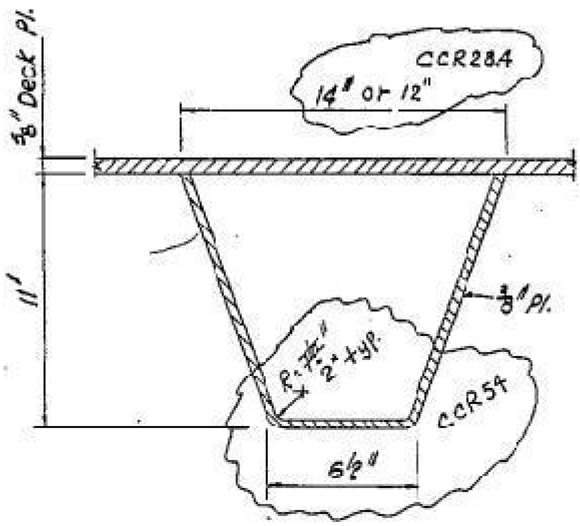
AECOM	Job Title:	GGB Moveable Median Barrier			Subject: Capacity of Orthotropic Deck
	Job No:	60097852			
	Prepared By:	LL	Date:	8/4/2010	
	Checked By:	HSL	Date:	10/15/2010	

Determine Capacity of Orthographic Deck

Per AASHTO LRFD Bridge Design Specification, 4th ed., 2007
Golden Gate Bridge As-Built Drawing C-13 & C-15 Dated 10/5/1981.

Orthographic Deck Section Properties





TYPICAL RIB DETAIL
Scale: 3"=1'-0"
STANDARD DECK

Total Number of Ribs $N := 14 \cdot 2 = 28$

Length of the Deck $L := 62ft$

Thickness of the Deck $t := \frac{5}{8} in$

Determine Neutral Axial of the Section

Area of the Deck $A_d := L \cdot t$ $A_d = 465 \cdot in^2$

Area of Single Rib $A_r := 11in \cdot \frac{3}{8} in \cdot 2 + \frac{3}{8} in \cdot 6.5in$ $A_r = 10.69 \cdot in^2$

Total Area of the Ribs (without Deck) $A_{rt} := A_r \cdot N = 299.25 \cdot in^2$

Total Area of the Section $A := A_{rt} + A_d$ $A = 764.25 \cdot in^2$

N.A. measured from the top $y := \frac{0.5 \cdot A}{L}$ $y = 0.51 \cdot in$

AECOM	Job Title:	GGB Moveable Median Barrier			Subject:	Evaluation of Orthotropic Deck
	Job No:	60097852				
	Prepared By:	LL	Date:	8/12/2010		
	Checked By:	HSL	Date:	10/15/2010		

$L = 62\,ft$ $t = 0.625 \cdot in$ $y = 0.51 \cdot in$ $N = 28$

Determine Moment of Inertia, I

$$I_1 := \frac{t^3 \cdot L}{12} + (t \cdot L) \cdot (y - t \cdot 0.5)^2 \qquad I_1 = 33.94 \cdot in^4$$

$$I_2 := \frac{\left(\frac{3}{8} in\right) \cdot (11 in)^3}{12} + \left(\frac{3}{8} in \cdot 11 in\right) \cdot \left[\frac{11 in}{2} + (t - y)\right]^2 \qquad I_2 = 171.48 \cdot in^4$$

$$I_2 := 2N \cdot I_2 \qquad I_2 = 9602.91 \cdot in^4$$

$$I_3 := \frac{\left(\frac{3}{8} in\right)^3 \cdot 6.5 in}{12} + \left(6.5 in \cdot \frac{3}{8} in\right) \cdot \left(11 in - \frac{3}{8} in \cdot 0.5 + \frac{5}{8} in - y\right)^2 \qquad I_3 = 290.9 \cdot in^4$$

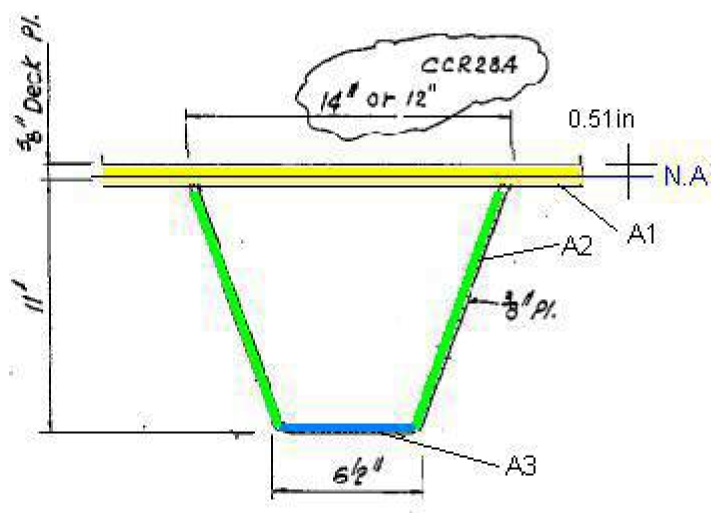
$$I_3 := N \cdot I_3 \qquad I_3 = 8145.17 \cdot in^4$$

$$I := I_1 + I_2 + I_3 \qquad I = 17782.02 \cdot in^4$$

Determine Section Modulus

$$S_{top} := \frac{I}{y} \qquad S_{top} = 34621.72 \cdot in^3$$

$$S_{bot} := \frac{I}{\left(\frac{5}{8} in + 11 in - y\right)} \qquad S_{bot} = 1600.34 \cdot in^3$$



TYPICAL RIB DETAIL
Scale: 3"=1'-0"
STANDARD DECK

RESULTS

I. DECK PLATE CHECK

AECOM	Job Title:	GGB Moveable Median Barrier			Subject:	Deck Top Plate Check
	Job No:	60097852				
	Prepared By:	LL /PC	Date:	9/23/2010		
	Checked By:	HSL	Date:	10/4/2010		

Deck Top Plate Check

Reference:
Per AASHTO LRFD Bridge Design Specification, 4th ed., 2007
Golden Gate Bridge As-built Drawing C-15 Dated 10/5/1981, and
Correspondence from Barrier System Inc.dated June 28, 2010
(See Appendix at the end of the calculation)

Assumption:
Since BTM moves in a lower speed (10mph during operation per correspondence from Barrier System Inc.) than a typical highway truck, the impact factor for BTM is assumed to be 10%, which is the impact factor for crane (pendant-operated bridge cranes) per ASCE 7-05 Section 4.11.

Approach:
The deck plate is checked using:
1. the criteria by Kloeppels formula for minimum deck thickness
2. the Finite Element Analysis for stresses

1. Minimum Deck Plate Thickness Check (Kloeppels formula):

Minimum Deck Plate Thickness t.p_min = 0.004 a * P ^ (1/3)

where a = max. spacing of the walls of the closed ribs (in mm)
P = wheel load and wearing surface pressure (in kPa)

Max. spacing of the walls of the closed ribs (in mm) $a := 14\text{in}$ $a = 355.6\cdot\text{mm}$

$a := \dfrac{a}{\text{mm}}$ $a = 355.6$

BTM Wheel load unit pressure
including the specified dynamic load allowance
(10% is assumed for BTM)
and wearing surface

$$P := \frac{\left(\dfrac{22\text{k}\cdot 1.10}{8\text{in}\cdot 18.6\text{in}} + 2\text{in}\cdot 150\text{pcf}\right)}{\text{ksi}}$$

$P = 0.1628$ ksi

$P = 1122.52$ kPa

Minimum Deck Plate Thickness $t_{p\_min} := 0.004\cdot a\cdot P^{\frac{1}{3}}\cdot\text{mm}$ $t_{p\_min} = 0.58\cdot\text{in}$

Current Deck Plate Thickness $t_p := \dfrac{5}{8}\text{in}$ $t_p = 0.63\cdot\text{in}$

Check $\text{Thickness} := \left|\begin{array}{ll}\text{"OK"} & \text{if } t_{p\_min} < t_p \\ \text{"NG"} & \text{otherwise}\end{array}\right.$ $\boxed{\text{Thickness} = \text{"OK"}}$

AECOM	Job Title:	GGB Moveable Median Barrier			Subject:	Deck Top Plate Check
	Job No:	60097852				
	Prepared By:	LL /PC	Date:	9/23/2010		
	Checked By:	HSL	Date:	10/4/2010		

2. Stress Check (Finite Element Analysis)

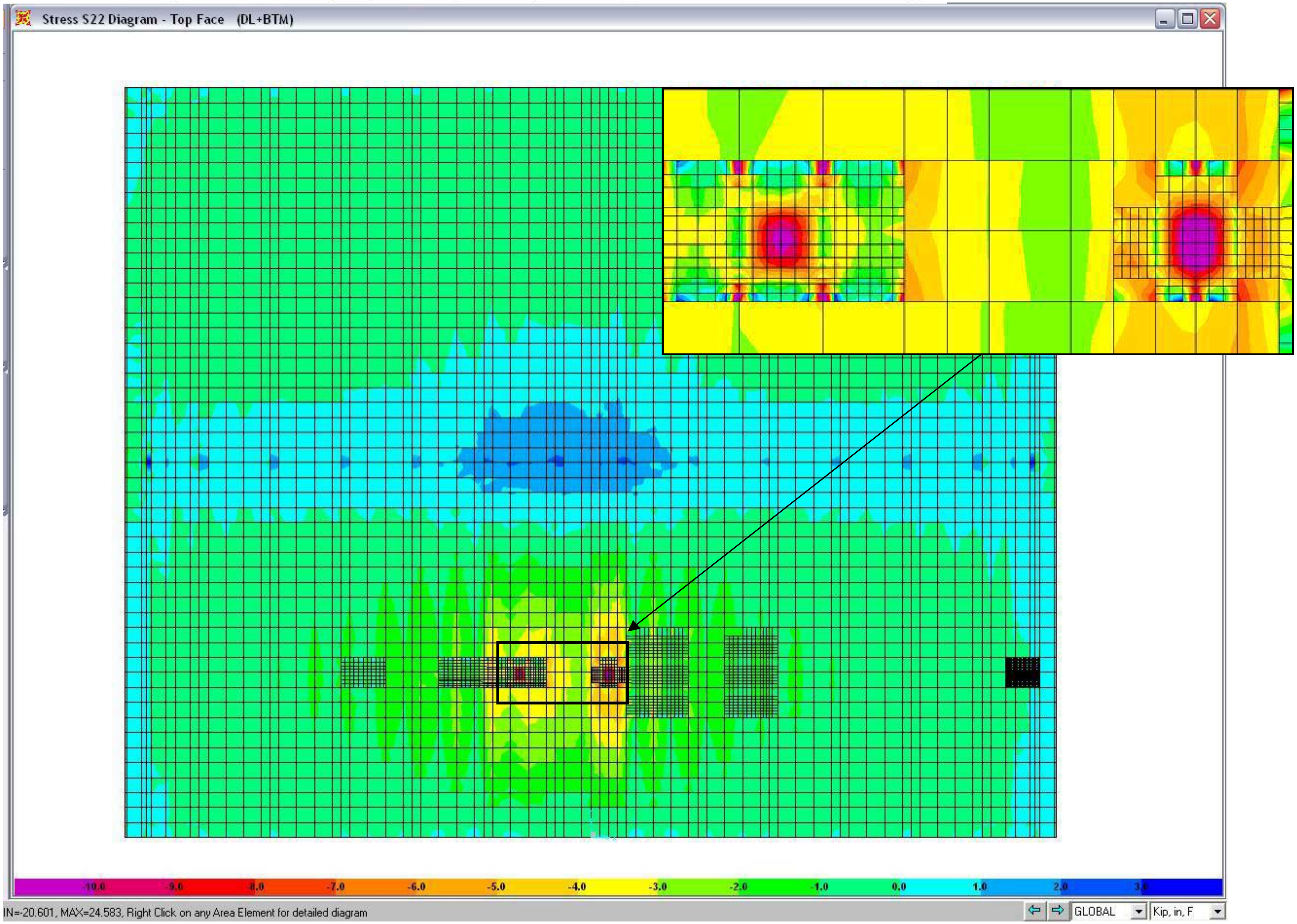
Load Case	Top Face (ksi)	Bottom Face (ksi)		Multiple Presence Factor	Top Face (ksi)	Bottom Face (ksi)
DL + BTM	-12.5	6.2		1.2	-15.0	7.4
DL + HL-93	-9.9	3.4		1.2	-11.9	4.0
DL + Tandem (with lane load)	-9.5	4.4		1.2	-11.4	5.3
DL + BTM + HL93 (with lane load)	-13.4	6.0		1.0	-13.4	6.0
DL + BTM + Tandem (with lane load)	-13.8	5.9		1.0	-13.8	5.9
Control Stress	-13.8	6.2			-15.0	7.4

Discussion:

Per Golden Gate Bridge As-built Drawing C-15 Dated 10/5/1981, the allowable stress for top plate is 20ksi. Results shown that the control case is dead load with BTM with stress of 15ksi (D/C = 0.75 <1.0). The top plate satisfied both criteria by Kloeppels formula and the stress check.

2.1.19.4.3
0117

AECOM	Job Title:	GGB Moveable Median Barrier			Subject: Stress at Top Face of Deck Top Plate
	Job No:	60097852			
	Prepared By:	LL /PC	Date:	9/23/2010	Load Case: DL + BTM
	Checked By:	HSL	Date:	10/4/2010	(Without multiple presence factor)



II. DECK RIB BOTTOM PLATE CHECK

AECOM	Job Title:	GGB Moveable Median Barrier			Subject:	Rib Bottom Plate Check
	Job No:	60097852				
	Prepared By:	LL/PC	Date:	9/27/2010		
	Checked By:	HSL	Date:	10/15/2010		

Deck Rib Bottom Plate Stress Check

Reference:
Per AASHTO LRFD Bridge Design Specification, 4th ed., 2007,
Golden Gate Bridge As-built Drawing C-15 Dated 10/5/1981, and
Correspondence from Barrier System Inc.dated June 28, 2010
(See Appendix at the end of the calculation)

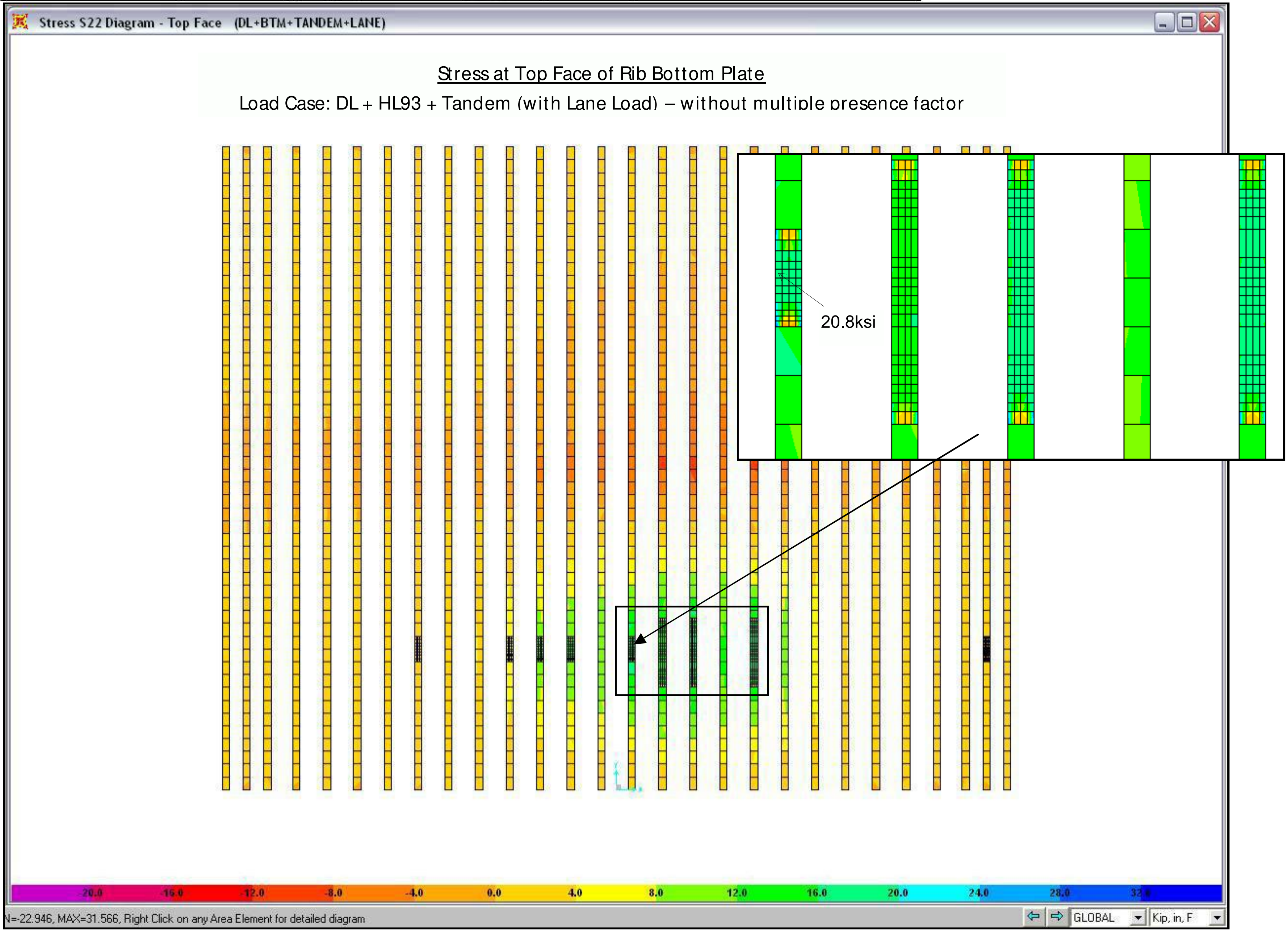
Rib Bottom Plate Stresses (Finite Element Analysis)

Load Case	Bottom Face (ksi)	Top Face (ksi)	Multiple Presence Factor	Bottom Face (ksi)	Top Face (ksi)
DL + BTM	16.0	16.4	1.2	19.2	19.7
DL + HL-93	15.3	15.2	1.2	18.4	18.2
DL + Tandem (with lane load)	16.7	16.7	1.2	20.0	20.0
DL + BTM + HL-93	18.6	19.3	1.0	18.6	19.3
DL + BTM + Tandem (with lane load)	20.2	20.8	1.0	20.2	20.8
DL + HL93+ Tandem (with lane load)	20.2	19.8	1.0	20.2	19.8
Control Case				20.2	20.8

Discussion:

Per Golden Gate Bridge As-built Drawing C-15 Dated 10/5/1981, the allowable stress for bottom plate is 20ksi. Maximum stress occurs with BTM + Tandem + lane load combination at 20.8ksi, which is less than 5% over the allowable stress limit; therefore, it is considered OK.

AECOM	Job Title:	GGB Moveable Median Barrier			Subject: Rib Bottom Plate Check
	Job No:	60097852			
	Prepared By:	LL /PC	Date:	9/27/2010	
	Checked By:	HSL	Date:	10/15/2010	



III. Local Deflection Check

	Job Title:	GGB Moveable Median Barrier			Subject:	Deflection Check
AECOM	Job No:	60097852				
	Prepared By:	LL/PC	Date:	9/28/2010		
	Checked By:	HSL	Date:	10/15/2010		

Summary of the Evaluation of Orthotropic Deck

	1. Stress Check	
	Deck Top Plate	Results show that the top plate is adequate under all load cases.
	Deck Rib Bottom Plate	Max. stresses at bottom plates are within limit or within 5% overstress of the allowable stress (20ksi).
	2. Local Deflection Check	
a.	Deflection due to vehicular load on deck plate	All load cases satisfy the requirement.
b.	Deflection due to vehicular load on ribs of orthotropic metal decks	Some load cases exceed the AASHTO LRFD requirement; however, all load cases meet the Caltrans BDS 2004 requirement.
c.	Deflection due to vehicular load on ribs of orthotropic metal decks (extreme relative deflection between adjacent ribs)	Some load cases exceed the AASHTO LRFD requirement; however, this deflection check is not required by Caltrans BDS.

AECOM	Job Title:	GGB Moveable Median Barrier			Subject:	Deflection Check
	Job No:	60097852				
	Prepared By:	LL/PC	Date:	9/28/2010		
	Checked By:	HSL	Date:	10/15/2010		

Local Deflection Check

1. *Per AASHTO LRFD Bridge Design Specification, 4th ed., 2007*

Check if local deflection meets AASHTO Section 2.5.2.6.2 for the orthotropic plate decks:
a. Vehicular load on deck plate Span/300
b. Vehicular load on ribs of orthotropic metal decks Span/1000
c. Vehicular load on ribs of orthotropic metal decks (extreme relative deflection between adjacent ribs).....0.10"

2. *Per Caltrans Bridge Design Specifications 2004*

Stiffness Requirements (10.41.4.8)
- The deflections of ribs, beams, and girders due to live load impact may exceed the limitations in Article 10.6 but preferably shall not exceed 1/500 of their span.

- To prevent excessive deterioration of the wearing surface, the specified wheel load plus 30-percent impact preferably shall be less than 1/300 of the distance between webs of rib.

AECOM	Job Title:	GGB Moveable Median Barrier			Subject: Deflection Check
	Job No:	60097852			
	Prepared By:	LL/PC	Date:	9/27/2010	
	Checked By:	HSL	Date:	10/15/2010	

a. Deflection due to Vehicular Load on Deck Plate

Span between ribs $\quad Span := 14in$

Deflection Limit due to vehicular load on deck plate $\quad limitation_1 := \frac{Span}{300} \quad limitation_1 = 0.0467 \cdot in$

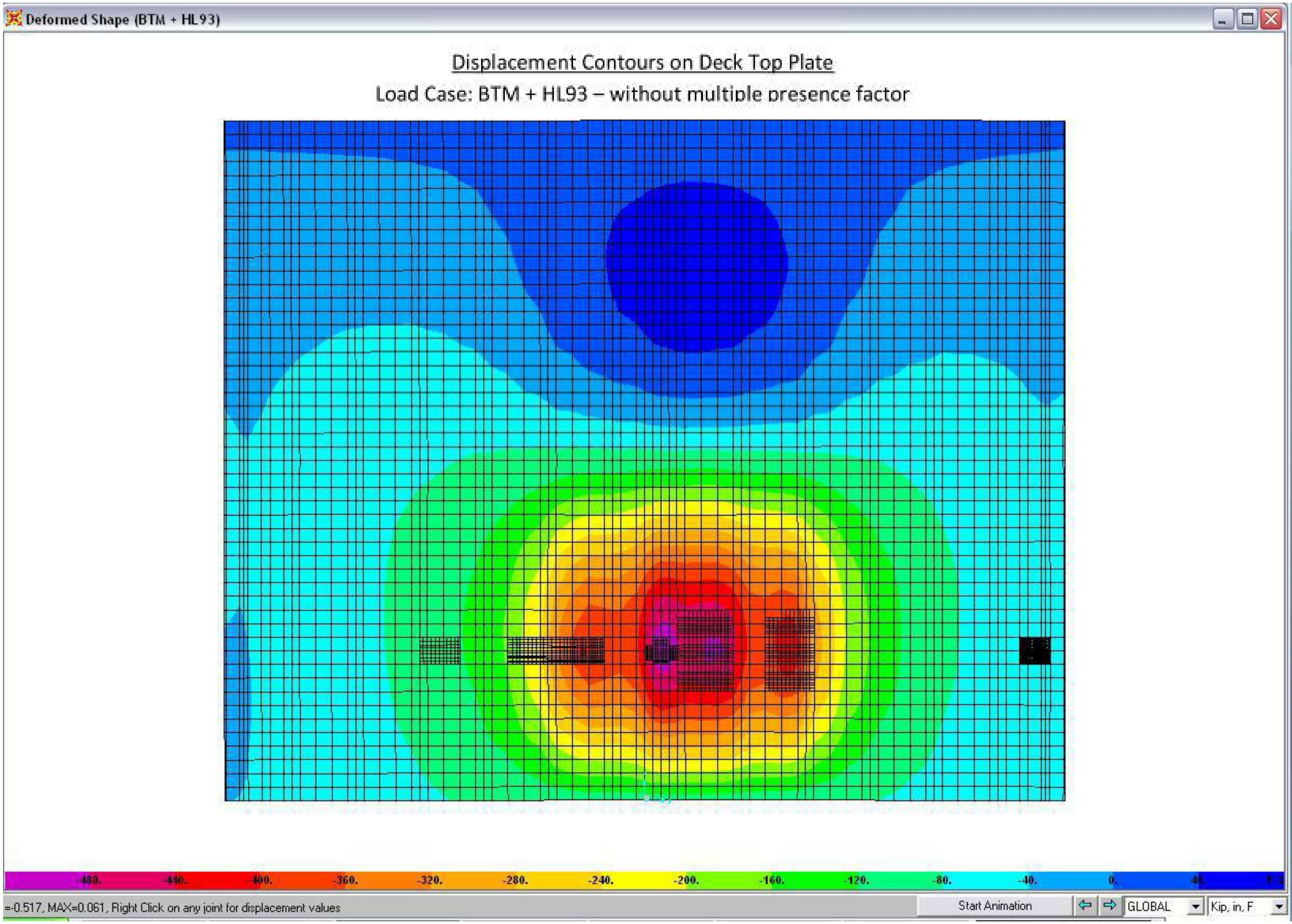
Summary of Deflection due to Vehicular Load on Deck Plate

Results from Finite Element Analysis

Load Case	Lane Load	Deflection at Point 1 (in)	Deflection at Point 2 (in)	Relative Displacement (in)	Check
BTM	No	0.342	0.354	0.012	OK
HL-93	Yes	0.346	0.366	0.020	OK
Tandem + Lane Load	Yes	0.485	0.497	0.012	OK
BTM + HL-93	Yes	0.483	0.455	0.028	OK
BTM + Tandem (w/ lane load)	Yes	0.615	0.608	0.008	OK
Control Case: BTM + HL-93				0.028	OK

Discussion:
The deflection limitation due to vehicular load on deck plate is calculated to be 0.047in. Per shown results, the maximum relative displacement is 0.028in, indicating the existing deck meets the span/300 deflection limit requirement.

AECOM	Job Title:	GGB Moveable Median Barrier			Subject: Rib Bottom Plate Check
	Job No:	60097852			
	Prepared By:	LL /PC	Date:	9/27/2010	
	Checked By:	HSL	Date:	10/15/2010	



AECOM	Job Title:	GGB Moveable Median Barrier			Subject:	Deflection Check
	Job No:	60097852				
	Prepared By:	LL/PC	Date:	9/28/2010		
	Checked By:	HSL	Date:	10/15/2010		

b. Deflection due to Vehicular Load on Ribs of Orthotropic Metal Decks

Longitudinal Span between rib supports $\quad Span := 25ft$

Deflection Limit on ribs of orthotropic metal decks (AASHTO) $\quad limitation_{2a} := \frac{Span}{1000} \quad limitation_{2a} = 0.3 \cdot in$

(Caltrans) $\quad limitation_{2b} := \frac{Span}{500} \quad limitation_{2b} = 0.6 \cdot in$

Summary of Deflection at Bottom Plate of Rib *Results from Finite Element Analysis*

Load Case	Local Displ. (in)	Global Displ. (in)	Result Displ (in)	Multiple Presence Factor	Result Displ (in)
BTM	0.33	0.05	0.28	1.2	0.34
HL-93	0.35	0.06	0.29	1.2	0.35
Tandem (with Lane Load)	0.48	0.08	0.40	1.2	0.48
HL-93 + HL-93	0.50	0.09	0.41	1.0	0.41
HL-93 + Tandem (with lane load)	0.62	0.13	0.49	1.0	0.49
BTM + HL-93	0.49	0.10	0.39	1.0	0.39
BTM + Tandem (with lane load)	0.61	0.14	0.47	1.0	0.47

Discussion:

The deflection limit due to vehicular load on ribs of orthotropic metal decks is 0.3in per AASHTO LRFD but 0.6in per Caltrans BDS 2004. None of the load cases meets the AASHTO LRFD requirement, but all meet the Caltrans BDS 2004 requirement. Note that the deflection limits per current AASHTO were not required in AASHTO 1977, the code used to design deck replacement in 1986.

Analysis shows that the addition of BTM is not likely to exceed maximum deflection occurred on the bridge per current ASHTO LRFD.

Deflection due to Load Combination with BTM:

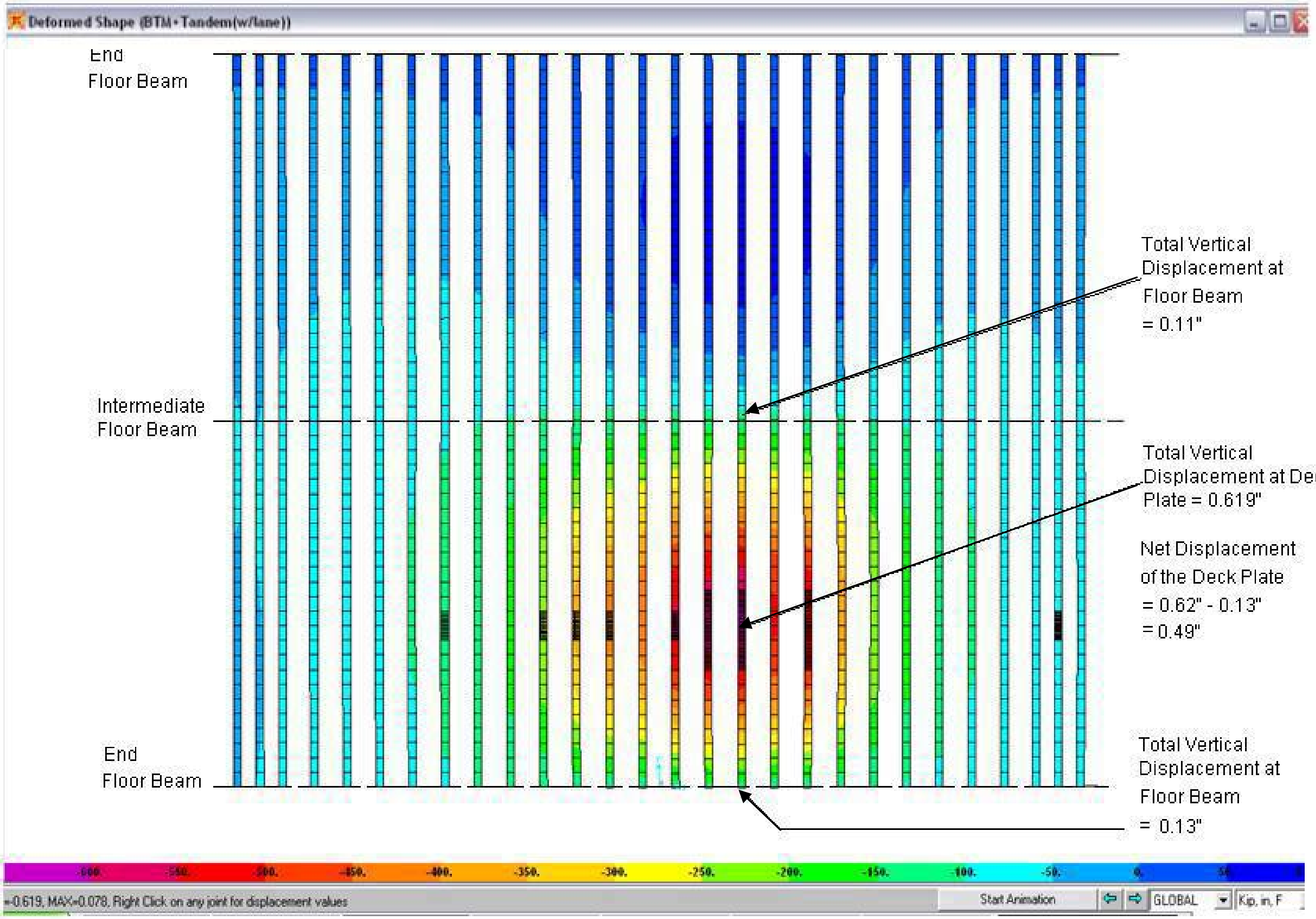
BTM = 0.34in
BTM + HL-93 = 0.39in
BTM + Tandem (with lane load) = 0.49in

Control Case Deflection:

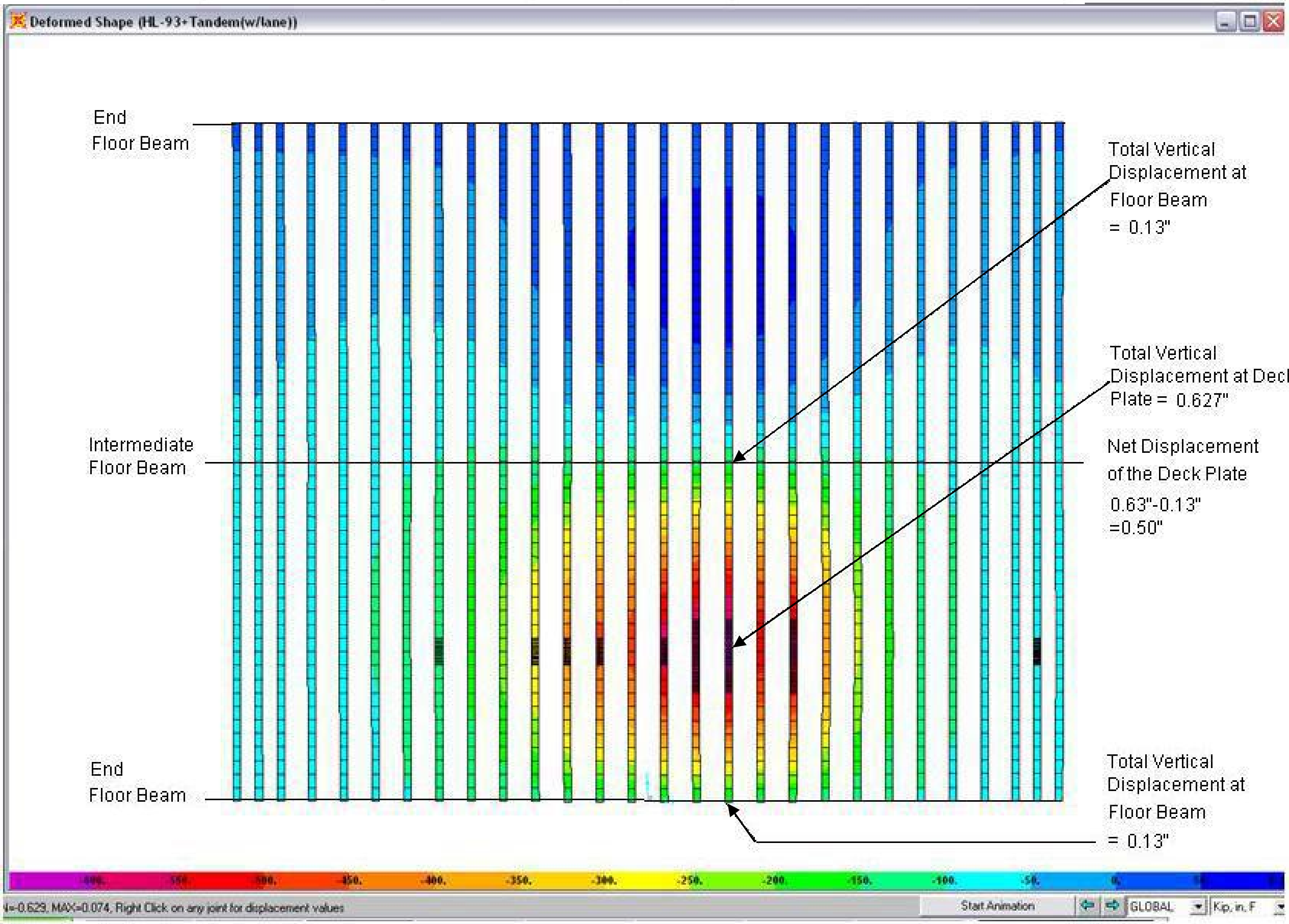
HL-93 + Tandem (with lane load) = 0.50in

The control case, HL-93 + Tandem (with lane load), possibly occurred on the bridge, has deflection higher than all the load combinations with BTM. Also, the steel deck meets deflection provisions as stipulated in Caltrans Bridge Design Specifications 2004. Therefore, no increase in roadway maintance is anticipated due to the addition of the MMB system, especially the bridge has been tested or traveled by HL-93 plus Tandem load.

AECOM	Job Title:	GGB Moveable Median Barrier			Displacement Contours on Rib Bottom Plate
	Job No:	60097852			
	Prepared By:	LL /PC	Date:	9/28/2010	Load Case: BTM + Tandem (w/ lane) without multiple presence factor
	Checked By:	HSL	Date:	10/15/2010	



AECOM					
Job Title:	GGB Moveable Median Barrier			Displacement Contours on Rib Bottom Plate	
Job No:	60097852				
Prepared By:	LL /PC	Date:	9/28/2010	Load Case: HL-93 + Tandem (w/ lane) without multiple presence factor	
Checked By:	HSL	Date:	10/15/2010		



AECOM	Job Title:	GGB Moveable Median Barrier		Subject:	Deflection Check
	Job No:	60097852			
	Prepared By:	LL/PC	Date: 9/28/2010		
	Checked By:	HSL	Date: 10/15/2010		

c. Deflection due to Vehicular Load on ribs of orthotropic metal decks (extreme relative deflection between adjacent ribs)

Limitation $limitation_3 := 0.1in$

Summary of Relative Deflection between ribs

Results are generated from finite element analysis, joints are selected in the critical area, where loads are applied. Please see the following pages for detail results of the following load cases:

For Single Lane:
1. BTM
2. HL-93
3. Tandem (w/ lane load)

For Two Lanes:
1. BTM + HL-93
2. HL-93 + HL-93
3. BTM + Tandem (w/ lane load)
4. HL-93 + Tandem (w/ lane load)

Summary		Max. Exceeded (%)	
Load Case	Relative Displacement (in)	Per AASHTO LRFD	Per Caltrans BDS
BTM	0.130	30%	Not Required
HL-93	0.128	28%	
Tandem (w/ lane load)	0.174	74%	
BTM + HL-93	0.140	40%	
HL-93 + HL-93	0.141	41%	
BTM + Tandem (w/ lane load)	0.186	86%	
HL-93 + Tandem (w/ lane load)	0.187	87%	

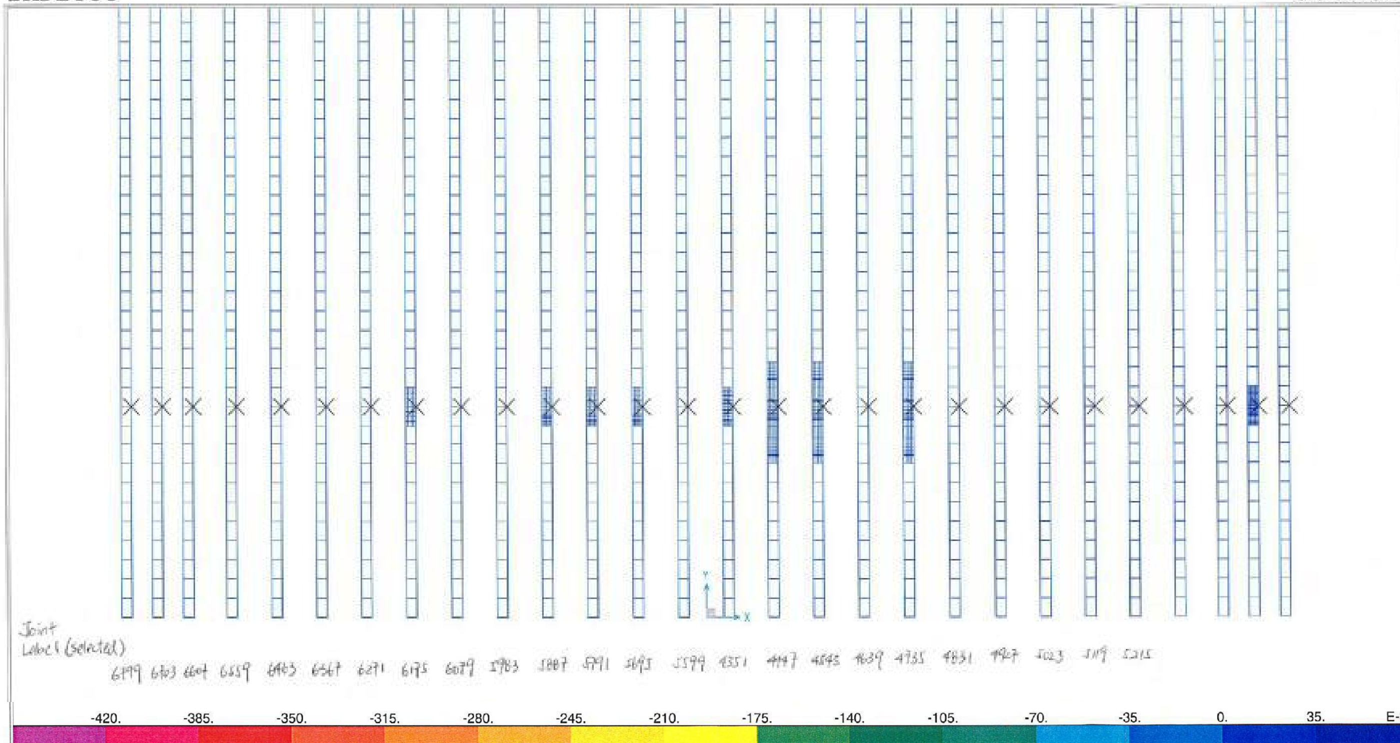
Discussion:

Displacements from all load cases exceed the deflection criteria per AASHTO LRFD. The bridge may not have been designed to meet this criteria, for this deflection requirement was not available in AASHTO 1977, the code used in deck replacement design in 1986. Note this criteria is not required by the Caltrans BDS 2004.

We compared the result between BTM and the live load requirements per AASHTO LRFD 2007, and found that HL-93 plus tandem load has the larger relative displacement than all load cases combined with BTM. Since the bridge is currently adequate under the existing load condition, the MMB is considered feasible.

AECOM	Job Title:	GGB Moveable Median Barrier			Subject:	Deflection Check
	Job No:	60097852				
	Prepared By:	LL	Date:	9/28/2010	Joint Label	
	Checked By:	HSL	Date:	10/15/2010		

SAP2000 9/28/10 14:13:13



SAP2000 v14.2.0 - File:Deck_analysis_0921_old_code - 3-D View - Kip, in, F Units

AECOM	Job Title:	GGB Moveable Median Barrier		
	Job No.:	60097852		
	Calculation No.:			
Subject: *Displacement Check*	Prepared By:	LL	Date:	9/22/2010
	Checked By:	HSL	Date:	10/15/2010

Per AASHTO LRFD 2007

Displacement due to vehicular load on ribs of orthotropic metal decks

(extreme relative deflection between relatives deflection between adjacent ribs)

Load Combination: BTM

Joint Label	Total Displacement (in)	Joint Label	Total Displacement (in)	Relative Displacement (in)	Check	
6799	-0.019	6703	-0.020	0.001	OK	
6703	-0.020	6607	-0.023	0.003	OK	
6607	-0.023	6559	-0.029	0.006	OK	
6559	-0.029	6463	-0.034	0.006	OK	
6463	-0.034	6367	-0.040	0.006	OK	
6367	-0.040	6271	-0.048	0.008	OK	
6271	-0.048	6175	-0.059	0.011	OK	
6175	-0.059	6079	-0.077	0.018	OK	
6079	-0.077	5983	-0.108	0.031	OK	
5983	-0.108	5887	-0.169	0.061	OK	
5887	-0.169	5791	-0.286	0.116	Exceed by	16%
5791	-0.286	5695	-0.294	0.008	OK	
5695	-0.294	5599	-0.272	0.022	OK	
5599	-0.272	4351	-0.321	0.049	OK	
4351	-0.321	4447	-0.191	0.130	Exceed by	30%
4447	-0.191	4543	-0.123	0.068	OK	
4543	-0.123	4639	-0.085	0.038	OK	
4639	-0.085	4735	-0.064	0.021	OK	
4735	-0.064	4831	-0.052	0.012	OK	
4831	-0.052	4927	-0.044	0.008	OK	
4927	-0.044	5023	-0.039	0.005	OK	
5023	-0.039	5119	-0.034	0.004	OK	
5119	-0.034	5215	-0.030	0.004	OK	
5215	-0.030	5359	0.000			

Note: This deflection limit check is not required by Caltrans BDS 2004.

AECOM	Job Title:	GGB Moveable Median Barrier		
	Job No.:	60097852		
	Calculation No.:			
Subject: *Displacement Check*	Prepared By:	LL	Date:	9/22/2010
	Checked By:	HSL	Date:	10/15/2010

Per AASHTO LRFD 2007

Displacement due to vehicular load on ribs of orthotropic metal decks
(extreme relative deflection between relatives deflection between adjacent ribs)

Load Combination: HL-93

Joint Label	Total Displacement (in)	Joint Label	Total Displacement (in)	Relative Displacement (in)	Check
6799	-0.020	6703	-0.021	0.001	OK
6703	-0.021	6607	-0.024	0.003	OK
6607	-0.024	6559	-0.029	0.005	OK
6559	-0.029	6463	-0.033	0.004	OK
6463	-0.033	6367	-0.037	0.004	OK
6367	-0.037	6271	-0.040	0.004	OK
6271	-0.040	6175	-0.044	0.004	OK
6175	-0.044	6079	-0.048	0.004	OK
6079	-0.048	5983	-0.053	0.005	OK
5983	-0.053	5887	-0.060	0.007	OK
5887	-0.060	5791	-0.070	0.010	OK
5791	-0.070	5695	-0.087	0.016	OK
5695	-0.087	5599	-0.117	0.030	OK
5599	-0.117	4351	-0.173	0.056	OK
4351	-0.173	4447	-0.281	0.108	Exceed by 8%
4447	-0.281	4543	-0.335	0.054	OK
4543	-0.335	4639	-0.300	0.035	OK
4639	-0.300	4735	-0.345	0.045	OK
4735	-0.345	4831	-0.217	0.128	Exceed by 28%
4831	-0.217	4927	-0.135	0.082	OK
4927	-0.135	5023	-0.092	0.043	OK
5023	-0.092	5119	-0.068	0.024	OK
5119	-0.068	5215	-0.052	0.016	OK
5215	-0.052	5359	0.000		

Note: This deflection limit check is not required by Caltrans BDS 2004.

AECOM	Job Title:	GGB Moveable Median Barrier		
	Job No.:	60097852		
	Calculation No.:			
Subject: *Displacement Check*	Prepared By:	LL	Date:	9/22/2010
	Checked By:	HSL	Date:	10/15/2010

Per AASHTO LRFD 2007

Displacement due to vehicular load on ribs of orthotropic metal decks
(extreme relative deflection between relatives deflection between adjacent ribs)

Load Combination: Tandem (with lane load)

Joint Label	Total Displacement (in)	Joint Label	Total Displacement (in)	Relative Displacement (in)	Check
6799	-0.028	6703	-0.030	0.002	OK
6703	-0.030	6607	-0.034	0.004	OK
6607	-0.034	6559	-0.040	0.007	OK
6559	-0.040	6463	-0.046	0.006	OK
6463	-0.046	6367	-0.052	0.005	OK
6367	-0.052	6271	-0.057	0.005	OK
6271	-0.057	6175	-0.062	0.005	OK
6175	-0.062	6079	-0.068	0.006	OK
6079	-0.068	5983	-0.075	0.007	OK
5983	-0.075	5887	-0.085	0.010	OK
5887	-0.085	5791	-0.099	0.015	OK
5791	-0.099	5695	-0.122	0.023	OK
5695	-0.122	5599	-0.164	0.042	OK
5599	-0.164	4351	-0.244	0.079	OK
4351	-0.244	4447	-0.392	0.149	Exceed by 49%
4447	-0.392	4543	-0.471	0.078	OK
4543	-0.471	4639	-0.420	0.051	OK
4639	-0.420	4735	-0.482	0.062	OK
4735	-0.482	4831	-0.308	0.174	Exceed by 74%
4831	-0.308	4927	-0.192	0.116	Exceed by 16%
4927	-0.192	5023	-0.131	0.061	OK
5023	-0.131	5119	-0.096	0.035	OK
5119	-0.096	5215	-0.074	0.022	OK
5215	-0.074	5359	0.000		

Note: This deflection limit check is not required by Caltrans BDS 2004.

AECOM		Job Title:	GGB Moveable Median Barrier		
		Job No.:	60097852		
		Calculation No.:			
Subject:	*Displacement Check*	Prepared By:	LL	Date:	9/22/2010
		Checked By:	HSL	Date:	10/15/2010

Per AASHTO LRFD 2007

Displacement due to vehicular load on ribs of orthotropic metal decks
(extreme relative deflection between relatives deflection between adjacent ribs)

Load Combination: BTM + HL-93

Joint Label	Total Displacement (in)	Joint Label	Total Displacement (in)	Relative Displacement (in)	Check	
6799	-0.039	6703	-0.042	0.003	OK	
6703	-0.042	6607	-0.047	0.005	OK	
6607	-0.047	6559	-0.057	0.010	OK	
6559	-0.057	6463	-0.067	0.010	OK	
6463	-0.067	6367	-0.077	0.010	OK	
6367	-0.077	6271	-0.088	0.011	OK	
6271	-0.088	6175	-0.103	0.015	OK	
6175	-0.103	6079	-0.125	0.022	OK	
6079	-0.125	5983	-0.161	0.036	OK	
5983	-0.161	5887	-0.229	0.068	OK	
5887	-0.229	5791	-0.356	0.127	Exceed by	27%
5791	-0.356	5695	-0.381	0.025	OK	
5695	-0.381	5599	-0.389	0.008	OK	
5599	-0.389	4351	-0.494	0.105	Exceed by	5%
4351	-0.494	4447	-0.472	0.022	OK	
4447	-0.472	4543	-0.458	0.014	OK	
4543	-0.458	4639	-0.385	0.073	OK	
4639	-0.385	4735	-0.409	0.024	OK	
4735	-0.409	4831	-0.269	0.140	Exceed by	40%
4831	-0.269	4927	-0.180	0.089	OK	
4927	-0.180	5023	-0.131	0.049	OK	
5023	-0.131	5119	-0.102	0.029	OK	
5119	-0.102	5215	-0.082	0.020	OK	
5215	-0.082	5359	0.000			

Note: This deflection limit check is not required by Caltrans BDS 2004.

AECOM	Job Title:	GGB Moveable Median Barrier		
	Job No.:	60097852		
	Calculation No.:			
Subject: *Displacement Check*	Prepared By:	LL	Date:	9/22/2010
	Checked By:	HSL	Date:	10/15/2010

Per AASHTO LRFD 2007

Displacement due to vehicular load on ribs of orthotropic metal decks

(extreme relative deflection between relatives deflection between adjacent ribs)

Load Combination: HL-93 + HL-93

Joint Label	Total Displacement (in)	Joint Label	Total Displacement (in)	Relative Displacement (in)	Check
6799	-0.044	6703	-0.047	0.003	OK
6703	-0.047	6607	-0.053	0.006	OK
6607	-0.053	6559	-0.065	0.012	OK
6559	-0.065	6463	-0.076	0.011	OK
6463	-0.076	6367	-0.088	0.012	OK
6367	-0.088	6271	-0.102	0.014	OK
6271	-0.102	6175	-0.120	0.019	OK
6175	-0.120	6079	-0.149	0.029	OK
6079	-0.149	5983	-0.198	0.049	OK
5983	-0.198	5887	-0.277	0.079	OK
5887	-0.277	5791	-0.398	0.120	Exceed by 20%
5791	-0.398	5695	-0.420	0.023	OK
5695	-0.420	5599	-0.421	0.001	OK
5599	-0.421	4351	-0.502	0.081	OK
4351	-0.502	4447	-0.481	0.021	OK
4447	-0.481	4543	-0.468	0.013	OK
4543	-0.468	4639	-0.395	0.073	OK
4639	-0.395	4735	-0.419	0.024	OK
4735	-0.419	4831	-0.278	0.141	Exceed by 41%
4831	-0.278	4927	-0.189	0.090	OK
4927	-0.189	5023	-0.139	0.049	OK
5023	-0.139	5119	-0.110	0.030	OK
5119	-0.110	5215	-0.089	0.021	OK
5215	-0.089	5359	0.000		

Note: This deflection limit check is not required by Caltrans BDS 2004.

AECOM		Job Title:	GGB Moveable Median Barrier		
		Job No.:	60097852		
		Calculation No.:			
Subject:	*Displacement Check*	Prepared By:	LL	Date:	9/22/2010
		Checked By:	HSL	Date:	10/15/2010

Per AASHTO LRFD 2007

Displacement due to vehicular load on ribs of orthotropic metal decks

(extreme relative deflection between relatives deflection between adjacent ribs)

Load Combination: BTM + Tandem (with lane load)

Joint Label	Total Displacement (in)	Joint Label	Total Displacement (in)	Relative Displacement (in)	Check
6799	-0.047	6703	-0.050	0.003	OK
6703	-0.050	6607	-0.057	0.006	OK
6607	-0.057	6559	-0.069	0.012	OK
6559	-0.069	6463	-0.081	0.012	OK
6463	-0.081	6367	-0.092	0.011	OK
6367	-0.092	6271	-0.105	0.013	OK
6271	-0.105	6175	-0.121	0.016	OK
6175	-0.121	6079	-0.145	0.024	OK
6079	-0.145	5983	-0.183	0.038	OK
5983	-0.183	5887	-0.254	0.071	OK
5887	-0.254	5791	-0.385	0.131	Exceed by 31%
5791	-0.385	5695	-0.416	0.031	OK
5695	-0.416	5599	-0.436	0.020	OK
5599	-0.436	4351	-0.565	0.128	Exceed by 28%
4351	-0.565	4447	-0.583	0.019	OK
4447	-0.583	4543	-0.594	0.010	OK
4543	-0.594	4639	-0.505	0.089	OK
4639	-0.505	4735	-0.546	0.042	OK
4735	-0.546	4831	-0.360	0.186	Exceed by 86%
4831	-0.360	4927	-0.236	0.124	Exceed by 24%
4927	-0.236	5023	-0.170	0.067	OK
5023	-0.170	5119	-0.130	0.039	OK
5119	-0.130	5215	-0.104	0.027	OK
5215	-0.104	5359	0.000		

Note: This deflection limit check is not required by Caltrans BDS 2004.

AECOM	Job Title:	GGB Moveable Median Barrier		
	Job No.:	60097852		
	Calculation No.:			
Subject: *Displacement Check*	Prepared By:	LL	Date:	9/22/2010
	Checked By:	HSL	Date:	10/15/2010

Per AASHTO LRFD 2007

Displacement due to vehicular load on ribs of orthotropic metal decks
(extreme relative deflection between relatives deflection between adjacent ribs)

Load Combination: HL-93 + Tandem (with lane load)

Joint Label	Total Displacement (in)	Joint Label	Total Displacement (in)	Relative Displacement (in)	Check
6799	-0.052	6703	-0.056	0.003	OK
6703	-0.056	6607	-0.063	0.007	OK
6607	-0.063	6559	-0.077	0.014	OK
6559	-0.077	6463	-0.090	0.013	OK
6463	-0.090	6367	-0.103	0.013	OK
6367	-0.103	6271	-0.118	0.015	OK
6271	-0.118	6175	-0.138	0.020	OK
6175	-0.138	6079	-0.169	0.031	OK
6079	-0.169	5983	-0.219	0.051	OK
5983	-0.219	5887	-0.302	0.082	OK
5887	-0.302	5791	-0.426	0.125	Exceed by 25%
5791	-0.426	5695	-0.456	0.029	OK
5695	-0.456	5599	-0.469	0.013	OK
5599	-0.469	4351	-0.573	0.104	Exceed by 4%
4351	-0.573	4447	-0.593	0.020	OK
4447	-0.593	4543	-0.604	0.011	OK
4543	-0.604	4639	-0.515	0.089	OK
4639	-0.515	4735	-0.556	0.041	OK
4735	-0.556	4831	-0.369	0.187	Exceed by 87%
4831	-0.369	4927	-0.245	0.124	Exceed by 24%
4927	-0.245	5023	-0.178	0.067	OK
5023	-0.178	5119	-0.138	0.040	OK
5119	-0.138	5215	-0.110	0.028	OK
5215	-0.110	5359	0.000		

Note: This deflection limit check is not required by Caltrans BDS 2004.

2.1.19.4.3
0117

EVALUATION OF SUB FLOOR BEAM

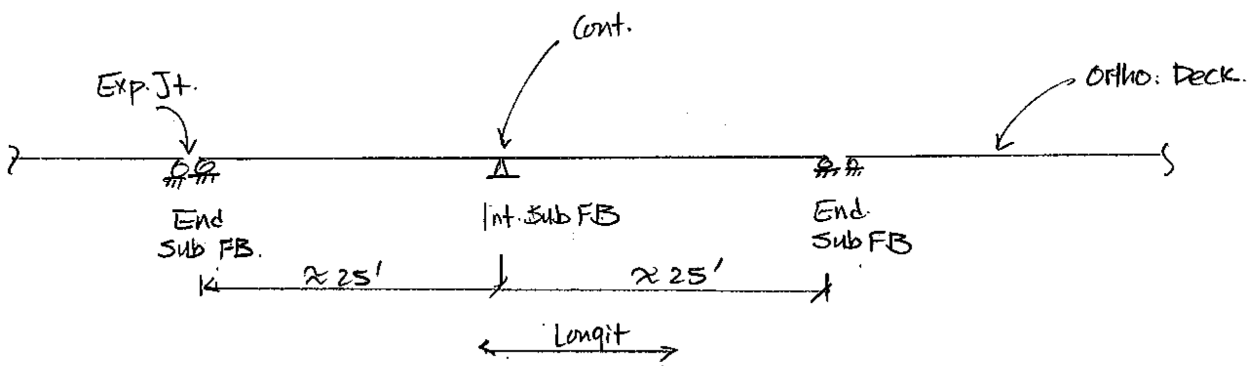
AECOM

JOB TITLE GGB MMB Study

JOB NO. 60097852 CALCULATION NO. ORIGINATOR PC DATE 8/2010

SHEET OF REVIEWER HSL DATE 10/15/2010

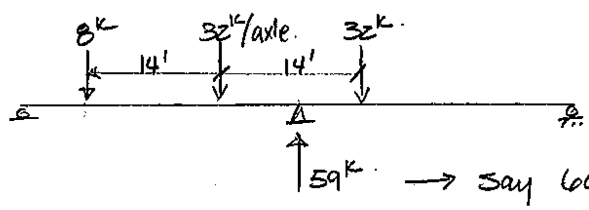
Determine: Live load on Sub F.B.



(Deck Config. in Longit. Direction).

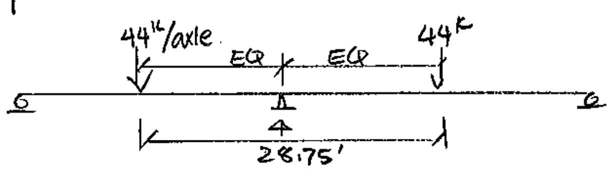
Forces on Int. Sub FB governs the design:

1) Max. load from HL-93 Truck:



→ say 60k/truck, or 30k/wheel line.

2) Max. load from BTM:



R = 58k < 59k ∴ BTM will not govern over HL93

O.K. No analysis needed.

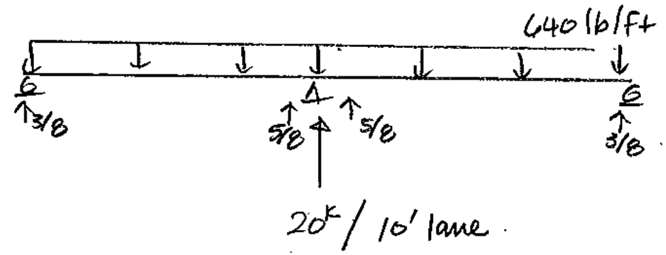
AECOM

JOB TITLE GGB MMB Study

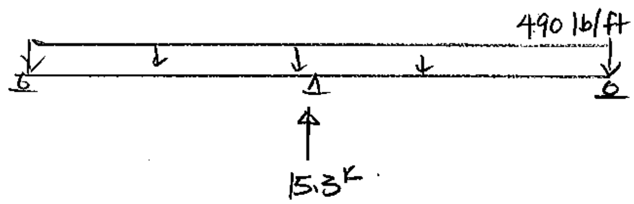
JOB NO. 60097852 CALCULATION NO. ORIGINATOR PC DATE 8/2010

SHEET OF REVIEWER HSL DATE 10/15/2010

3). Max. load from 1 Lane load.



4) Max. load from MMB:



2.1.19.4.3
0117

AECOM

JOB TITLE GGB Moveable Median Barrier

JOB NO. 60097852 CALCULATION NO. ORIGINATOR PC DATE 8/20/2010

SHEET OF REVIEWER HSL DATE 10/15/2010

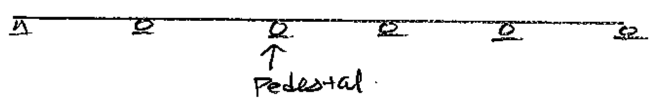
Parametric Study of the Struct. Resp. of the Sub F.B.

Determine:

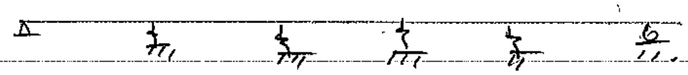
1) Influence of closely spaced pedestal support

2) Influence of the flexibility of the transv. Floor Beam below.

3) Will the sub F.B. behave more like continous beam on roller supports,



or as beam on flexible fdn?

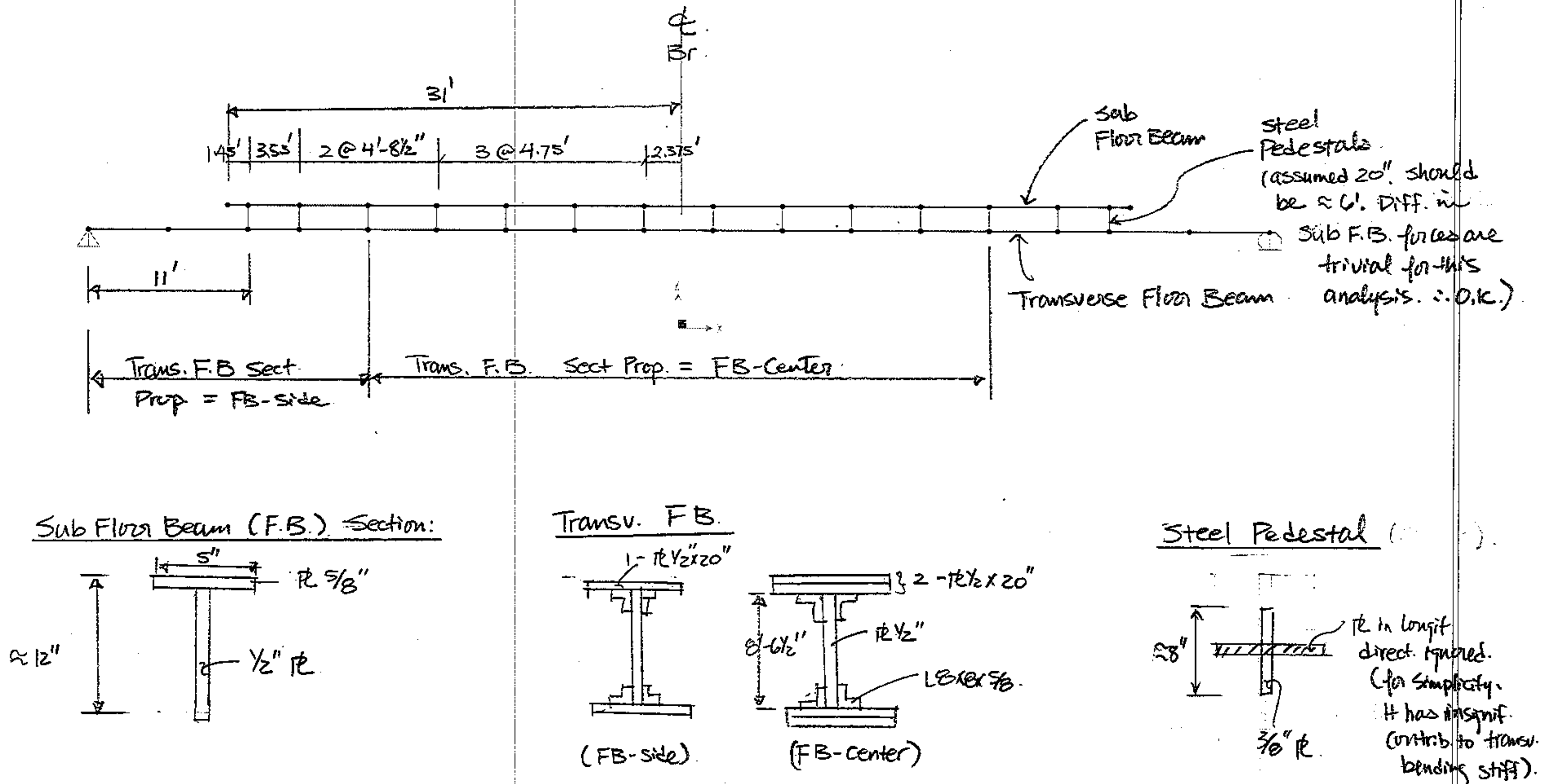


Conclusion: Sub F.B. behaves like beam on flexible fdn. However, the transv. F.B. and the Pedestals are much stiffer than the sub F.B. Majority of the loads on the sub F.B. gets transfered down to the transv. F.B., and thus forces on the adjacent spans are < 20% of the forces on the span where load is applied.

(See Attached SAP analysis for the parametric study).

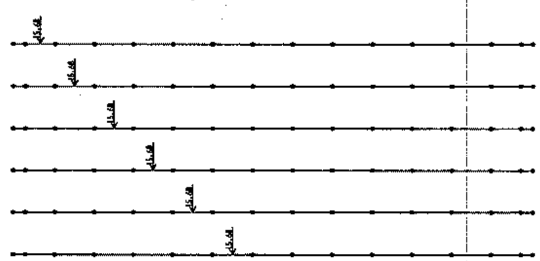
Model Definition

Mat'l Prop: Steel. E = 29,000 ksi



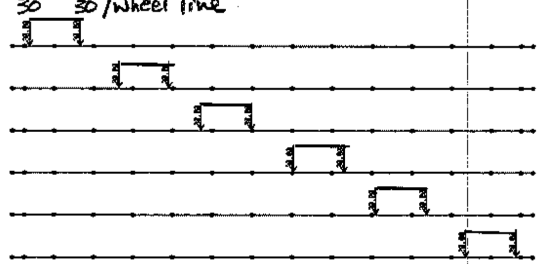
Parametric study: Apply loads @ varies locations on the Sub F.B. to determine the forces on the adjacent spans of the Sub FB (Load Case: LL-truck)

Case 1: MMB load applied @ varies locations of the Sub F.B:



30^k 30^k/wheel line

Case 2: Truck load.

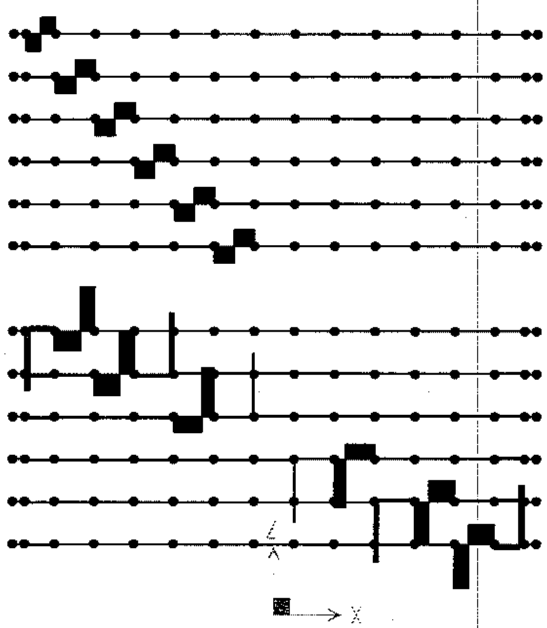


Case 3: Lane load = 2 k/ft

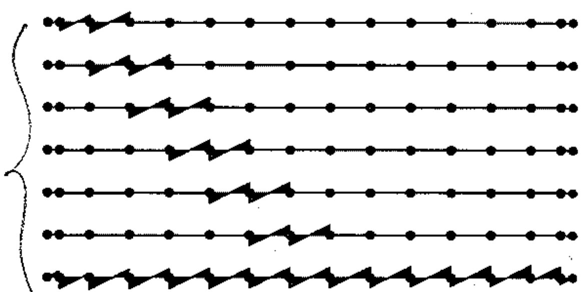
Parametric Study: Shear in Sub F.B. due to LL-truck load case:

case 1

case 2.



case 3



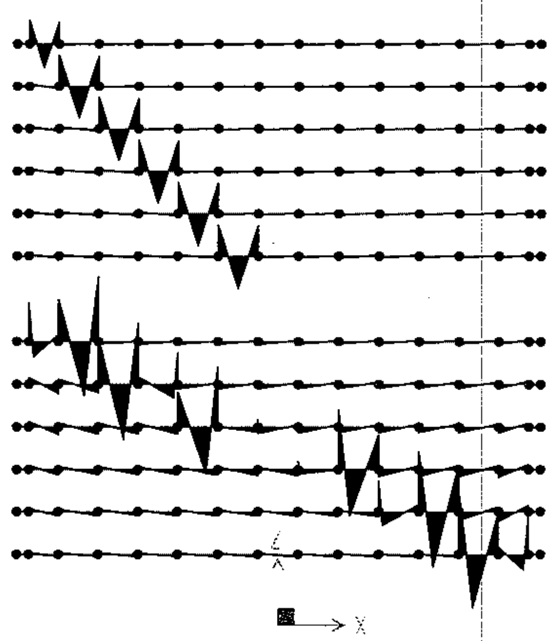
Note: the loads applied @ the spans gets transferred to the stiff. transv. Floor beam (F.B.) below. Forces in adjacent spans are less than 20% of the span where the load is applied.

→ May apply the live loads & MMB load on 1 or 2 spans only to evaluate element forces.

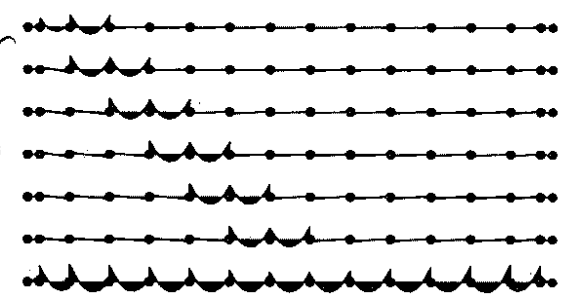
Parametric Study: Moment in Sub F.B. due to LL-truck Load Case.

Case 1

Case 2



Case 3



Note: Moments in adj. spans are less than 20% of the span where load is applied.

∴ May apply the live loads & MMB load on 1 or 2 spans only to eval. element forces. The forces will be slightly smaller than if all lanes are loaded, but o.k. since we're only using them to compare w/ and w/o the MMB effect.

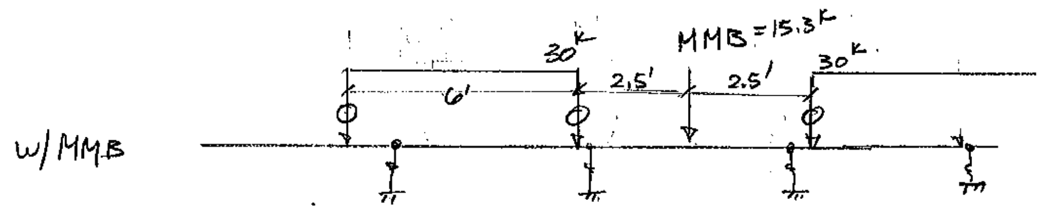
AECOM

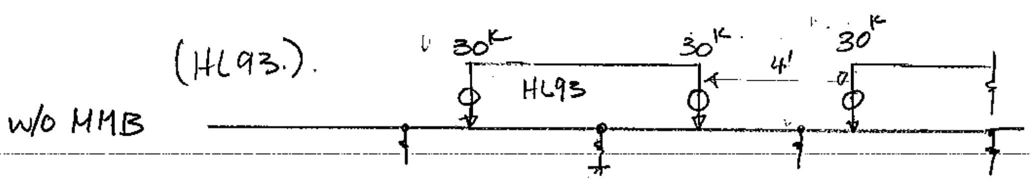
JOB TITLE GGB MMB Study

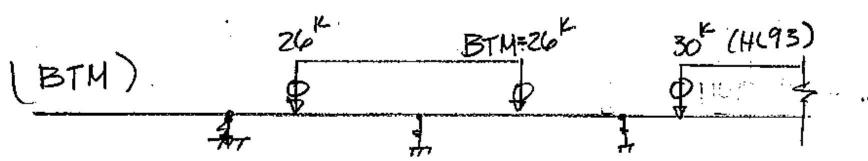
JOB NO. 60097852 CALCULATION NO. ORIGINATOR PC DATE

SHEET OF REVIEWER HSL DATE

From the parametric Study, we see the loads on a span have small impact on the adjacent span. Also, because the span Length (Max = 4'-9") is less than the spacing between the 2 truck wheel lines (when there's an MMB), MMB & the truck will not be loaded on the same span for Max. Moment effect.







By Inspection, existing highway load governs. Check the whole width with all 6-lanes loaded for additional assurance.

AECOM

JOB TITLE GGB MMB Study

JOB NO. 60097852 CALCULATION NO. ORIGINATOR PC DATE 8/2010

SHEET OF REVIEWER HSL DATE

Purpose: Determine the 4 in Sub F.B. forces due to MMB system. Analyze the sub F.B. with all 6-lanes loaded.

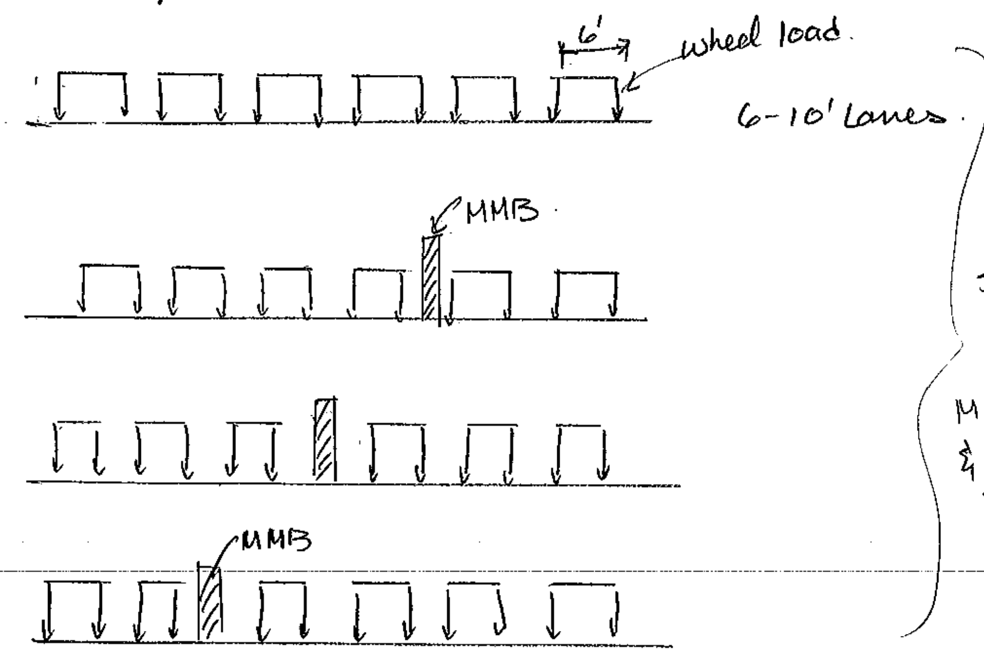
Approach: load the Sub F.B. as follows:

Case 1: (Existing Condition).

Case 2: (4-2 Config).

Case 3. (3-3 Config)

Case 4 (2-4 Config).



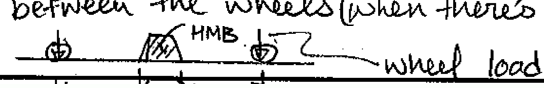
See Report for MMB location & wheel spacings.

Conclusion: Effect with adding MMB is insignificant. This is because:

1) Max. forces occur @ the ends of the beam

2) The effect of any load on beam forces quickly dissipates when passing the support of the beam. At the Pedestal Supports, the load gets transferred to the stiff Transv. F.B. below.

3) Where MMB is located, wheel loads of the truck can't be applied. The spacing between the supports is ≤ 4'-9", where as the spacing between the wheels (when there's MMB) is 5'.



AECOM

JOB TITLE GGB MMB Study

JOB NO. 60097852 CALCULATION NO. ORIGINATOR PC DATE 8/20/2010

SHEET OF REVIEWER HSL DATE 10/15/2010

Summary (Note: 33% Impact Incl., multi-lane Reduction Factor Not Incl.)

Max. sub FB. Forces. (HL93 Loading).

	Shear	Moment.
Existing lane Config.	48.3^k	29.04 k'
w/ MMB, 4-2 Config.	48.5^k	30.04 k'
w/ MMB, 3-3 Config	48.5^k	29.3k'
w/ MMB, 2-4 Config	48.5^k	29.3k'

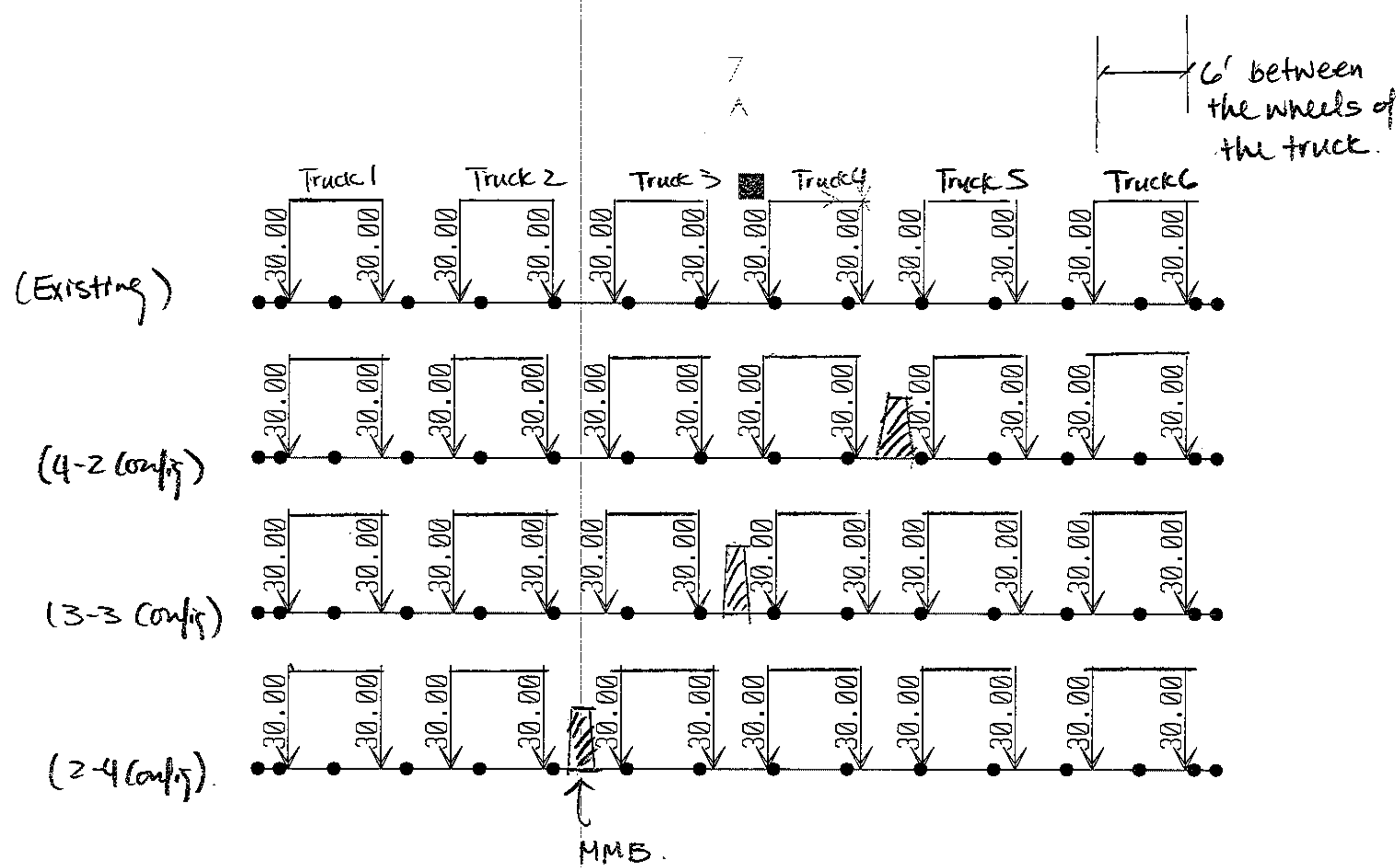
Max. sub FB. Forces. (HS20-44 Loading).

	Shear	Moment
Existing lane Config.	43^k	26.2k'
w/ MMB, 4-2 Config	43.3^k	26.6k'
w/ MMB, 3-3 Config.	43.2^k	26.4k'
w/ MMB, 2-4 Config.	43.3^k	26.6k'

Change in Sub F.B. is insignificant w/ MMB addition.

Applied Loads:

1) HS93 / HS-20 Truck Loads. (LL-Truck)

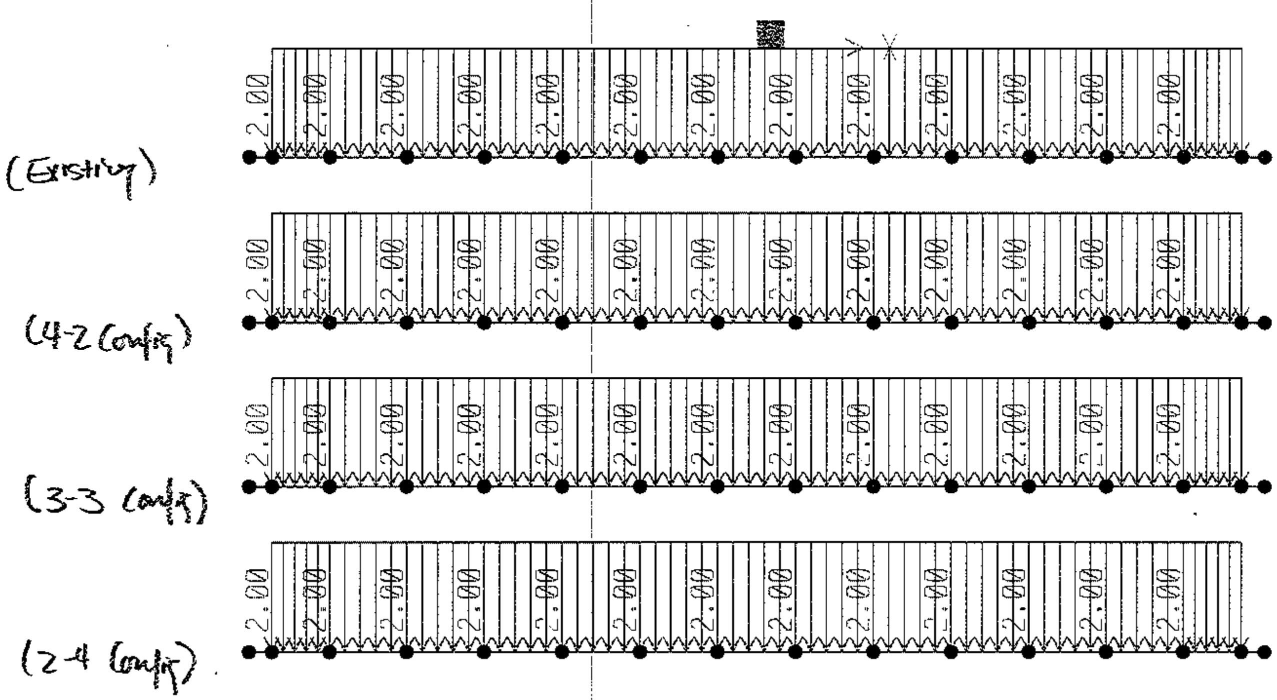


(Sub. F.B. shown. Pedestal & Transv. F.B. not show for clarity. Same Model as the one used for the Parametric Study)

SAP2000 8/5/10 10:15:35

Applied Loads:

2) 6-Lanes Fully Loaded. (LL-lane).
(640 lb/ft. Longit.)

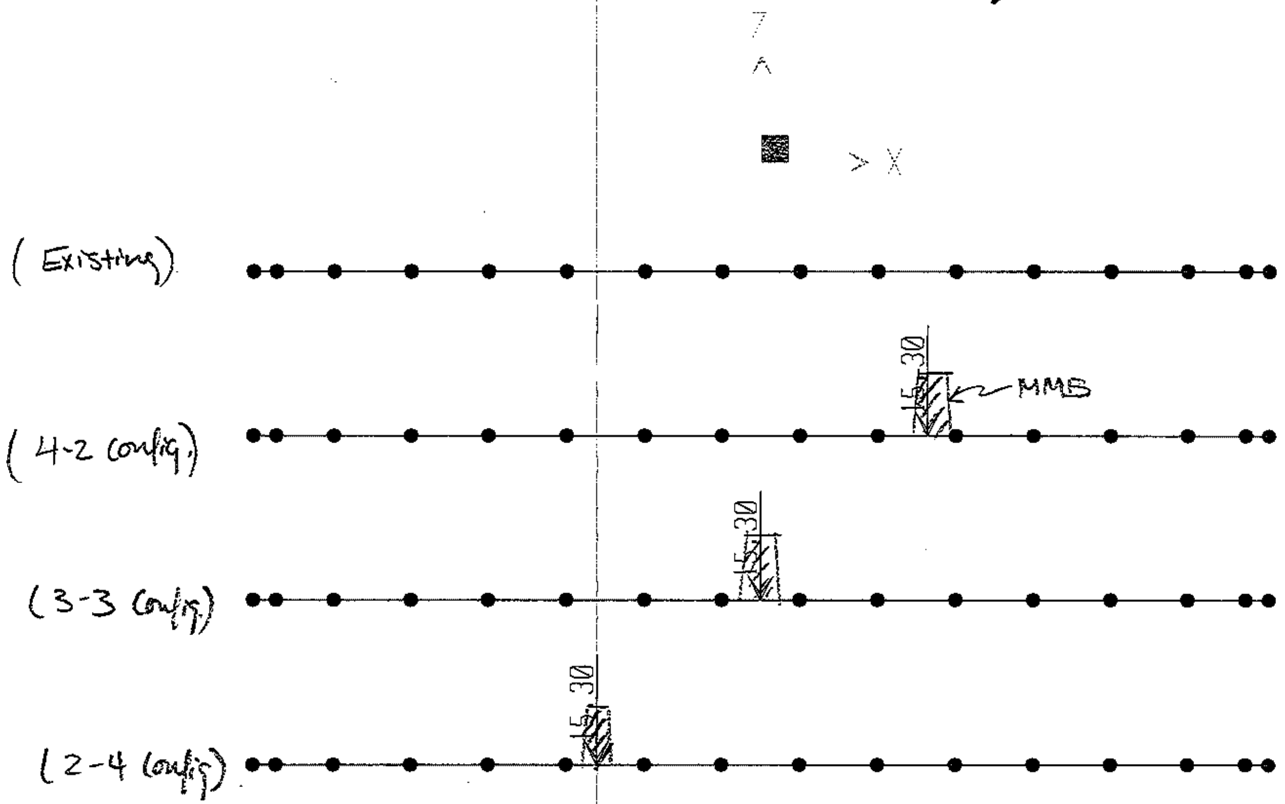


(Sub F.B. shown only).

SAP2000 v14.0.0 - File:Sub FB2 - Frame Span Loads (LL-lane) (As Defined) - Kip, ft, F Units

Applied Loads:

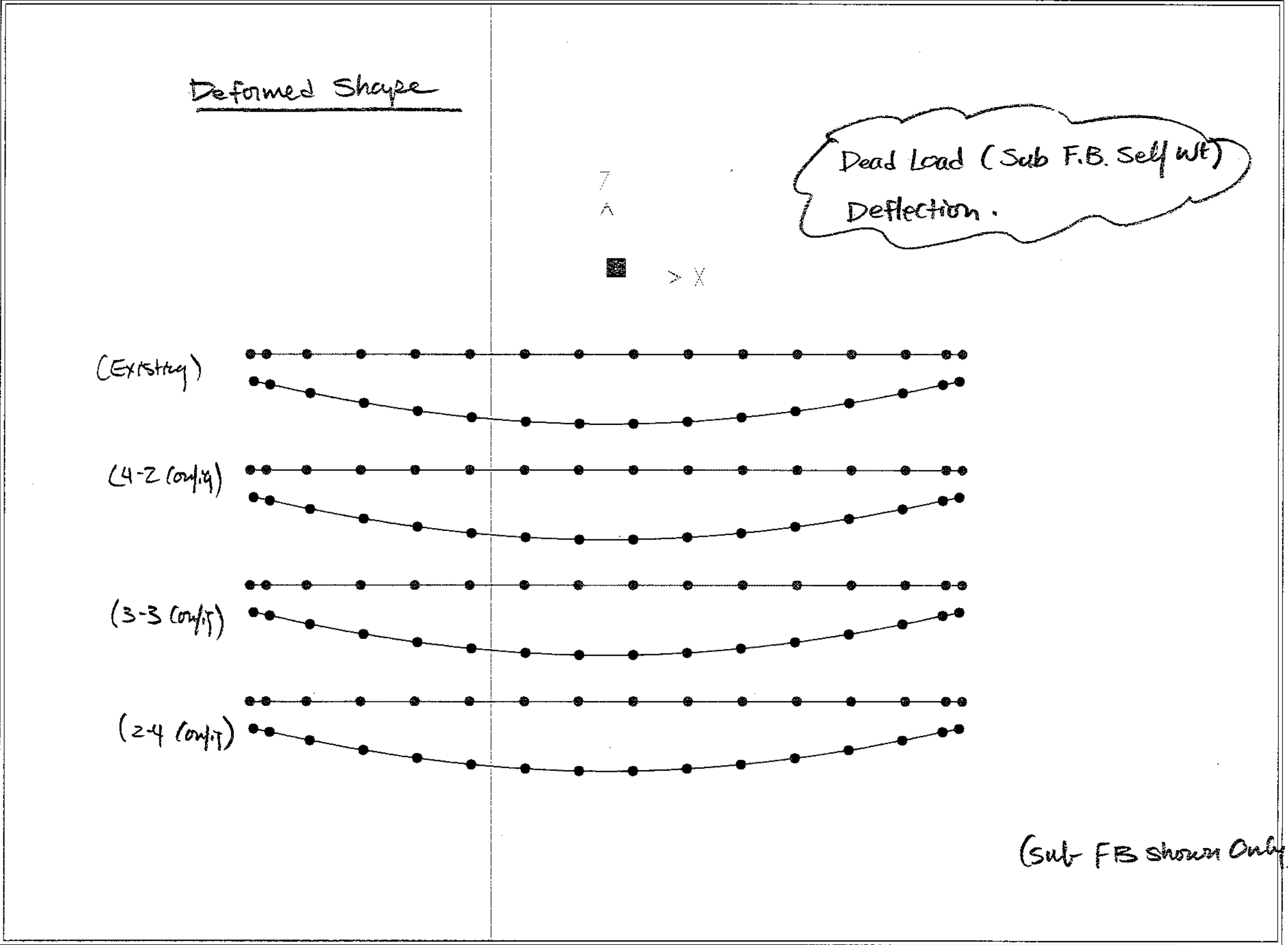
3) MMB Load / location. (MMB).



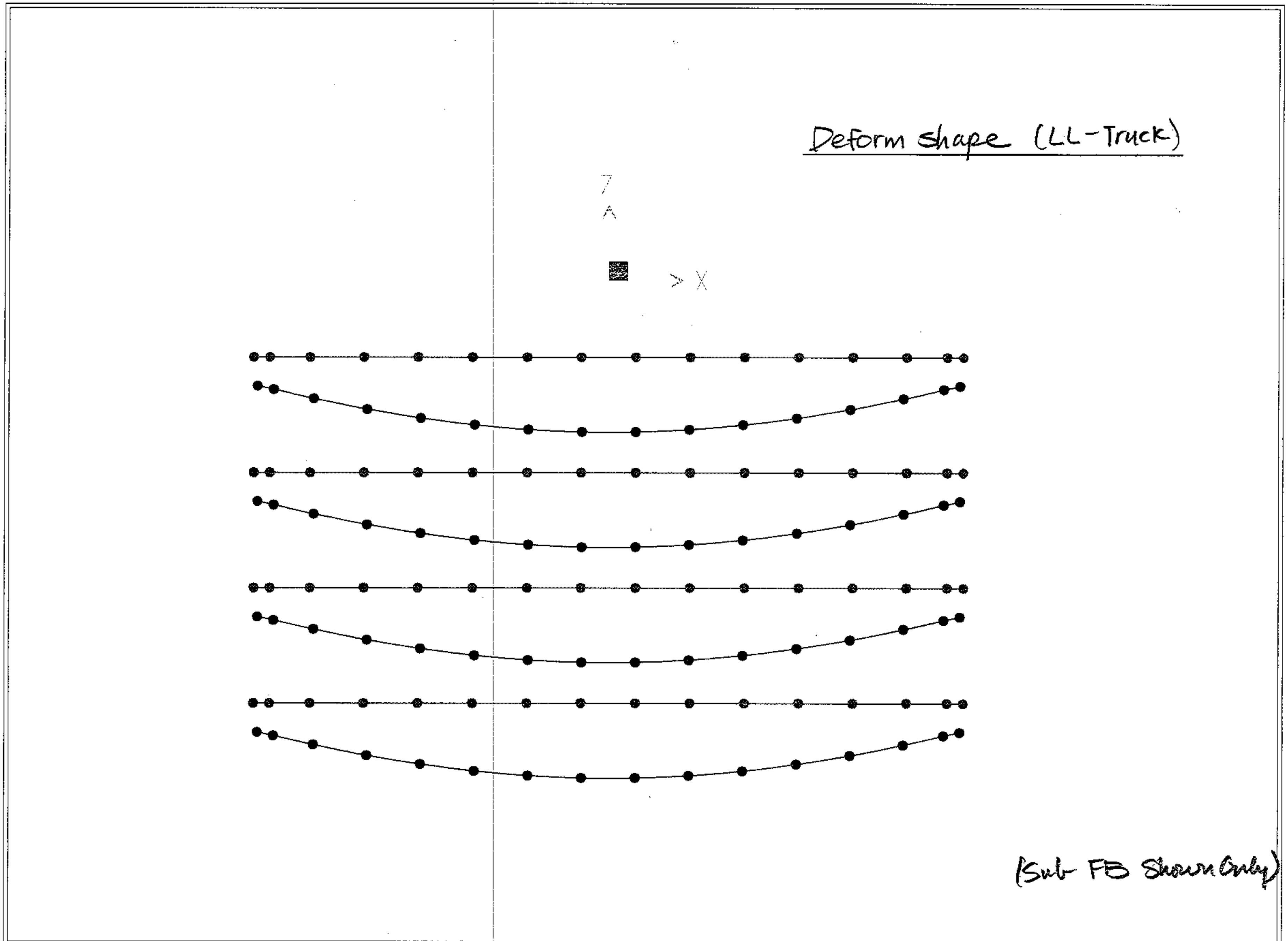
(Sub F.B. Shown Only)

SAP2000 v14.0.0 - File:Sub FB2 - Frame Span Loads (MMB) (As Defined) - Kip, ft, F Units

SAP2000 8/5/10 10:52:19



SAP2000 v14.0.0 - File:Sub FB2 - Deformed Shape (DEAD) - Kip, ft, F Units

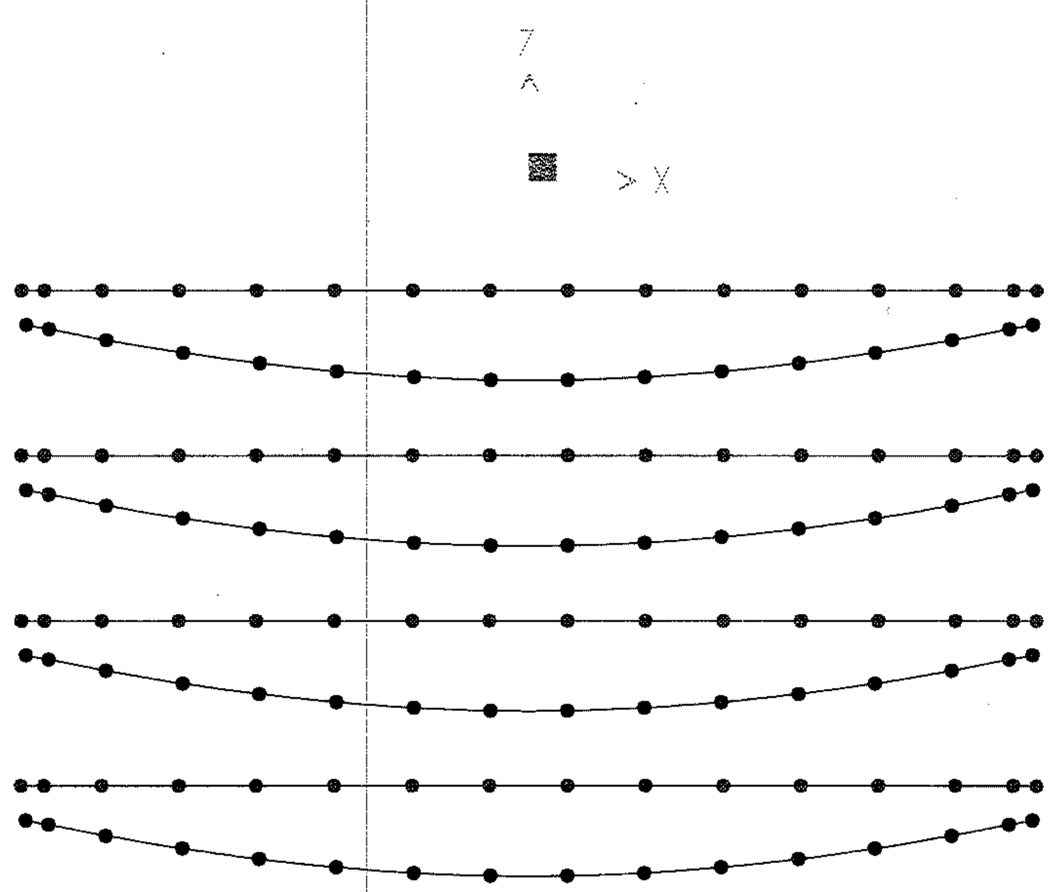


SAP2000 v14.0.0 - File:Sub FB2 - Deformed Shape (LL-truck) - Kip, ft, F Units

SAP2000

8/5/10 10:53:05

Deformed Shape (Lane load)

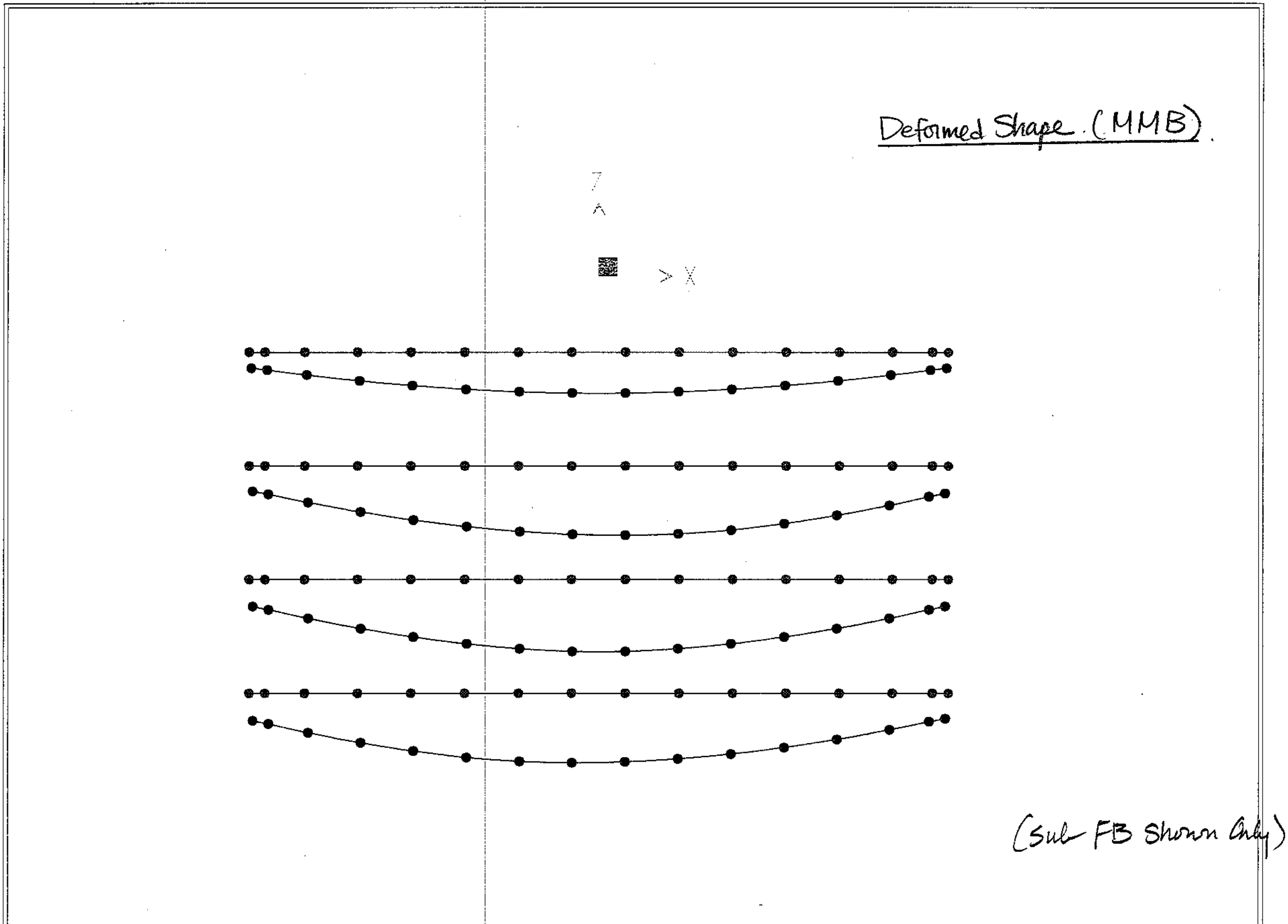


(Sub FB Shown Only)

SAP2000 v14.0.0 - File:Sub FB2 - Deformed Shape (LL-lane) - Kip, ft, F Units

SAP2000

8/5/10 10:52:55

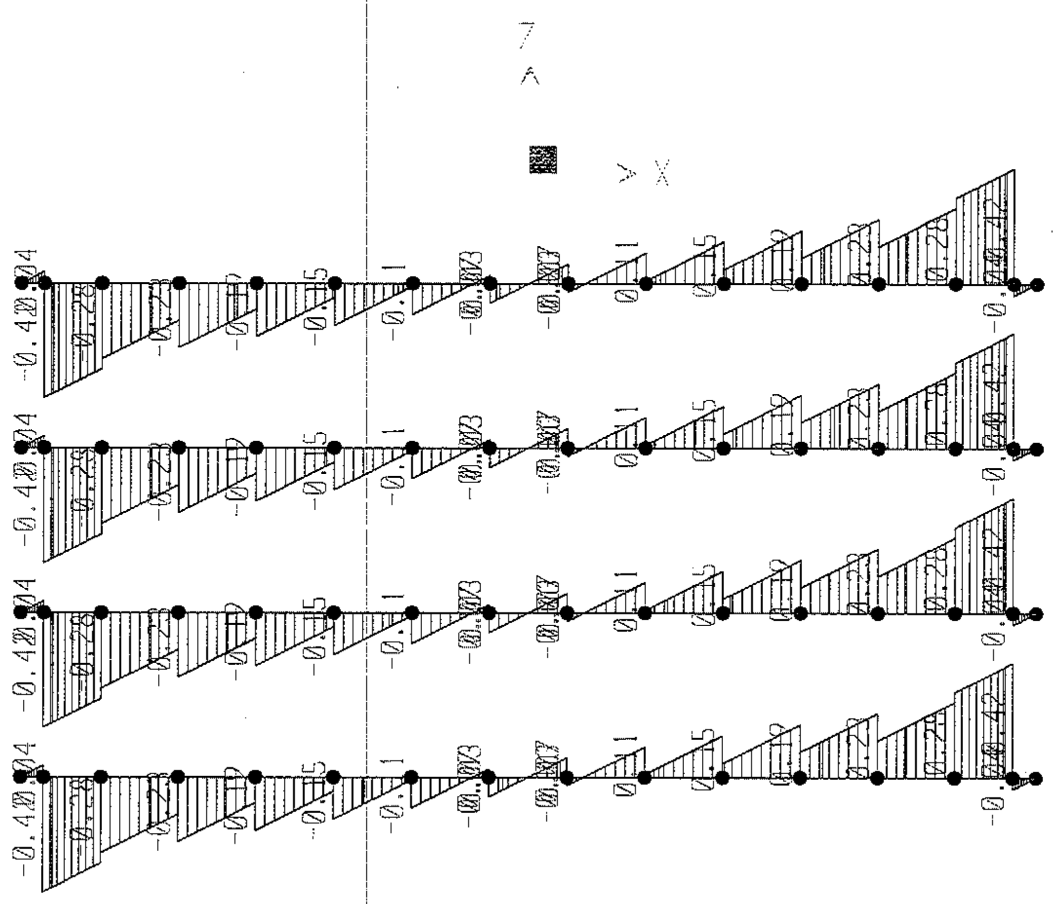


SAP2000 v14.0.0 - File:Sub FB2 - Deformed Shape (MMB) - Kip, ft, F Units

SAP2000

8/5/10 10:54:01

Sub F.B. Self Wt. Shear (K)



(Sub FB Shown Only)

SAP2000 v14.0.0 - File:Sub FB2 - Shear Force 2-2 Diagram (DEAD) - Kip, ft, F Units

SAP2000

8/5/10 10:54:38

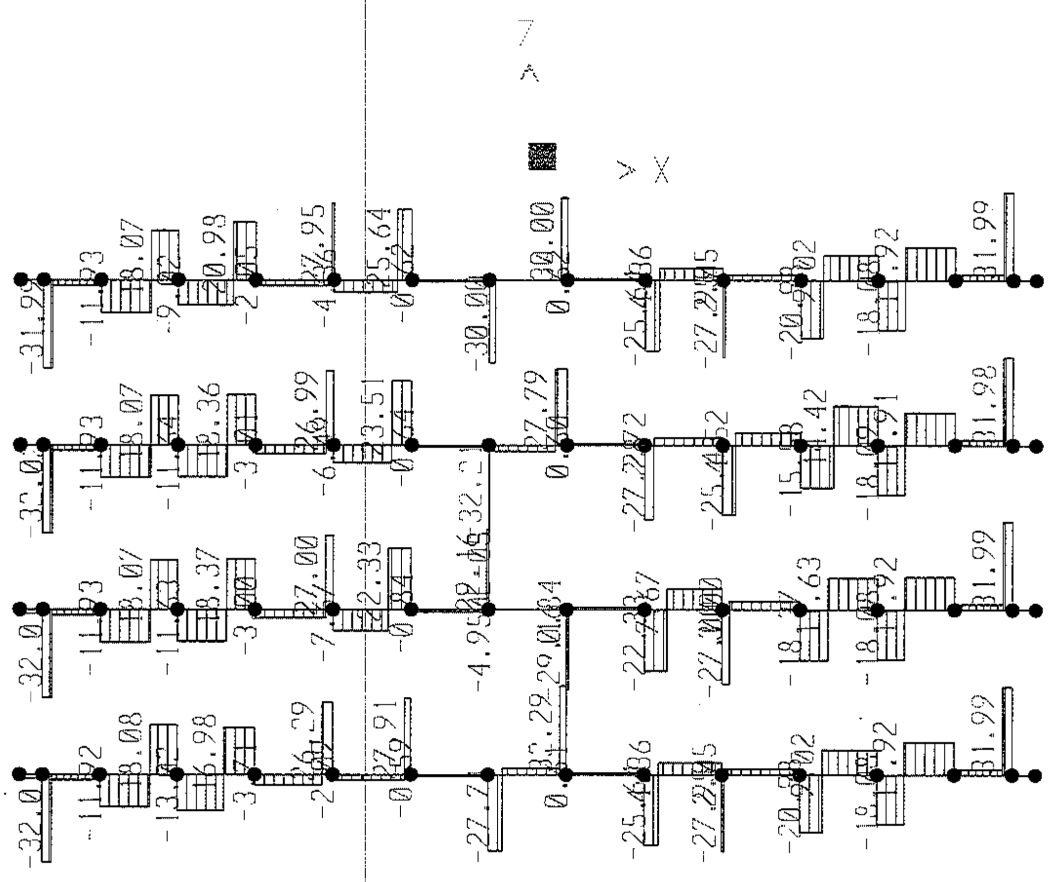
Shear Due to Truck load. (LL-Truck)

(Existing) Max. = 31.99^K.

(4-2 Config) Max = 32^K

(3-3 Config) Max = 32^K

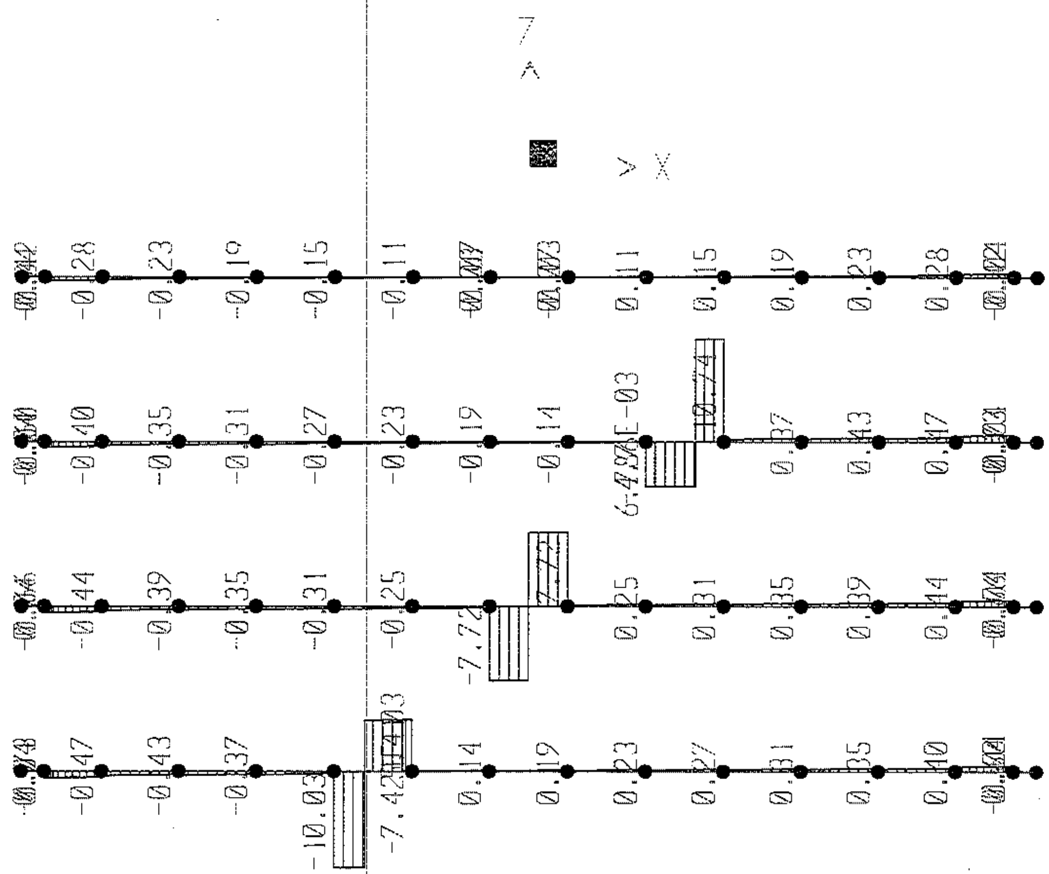
(2-4 Config) Max = 32^K.



(Sub FB shown Only)

SAP2000 v14.0.0 - File:Sub FB2 - Shear Force 2-2 Diagram (LL-truck) - Kip, ft, F Units

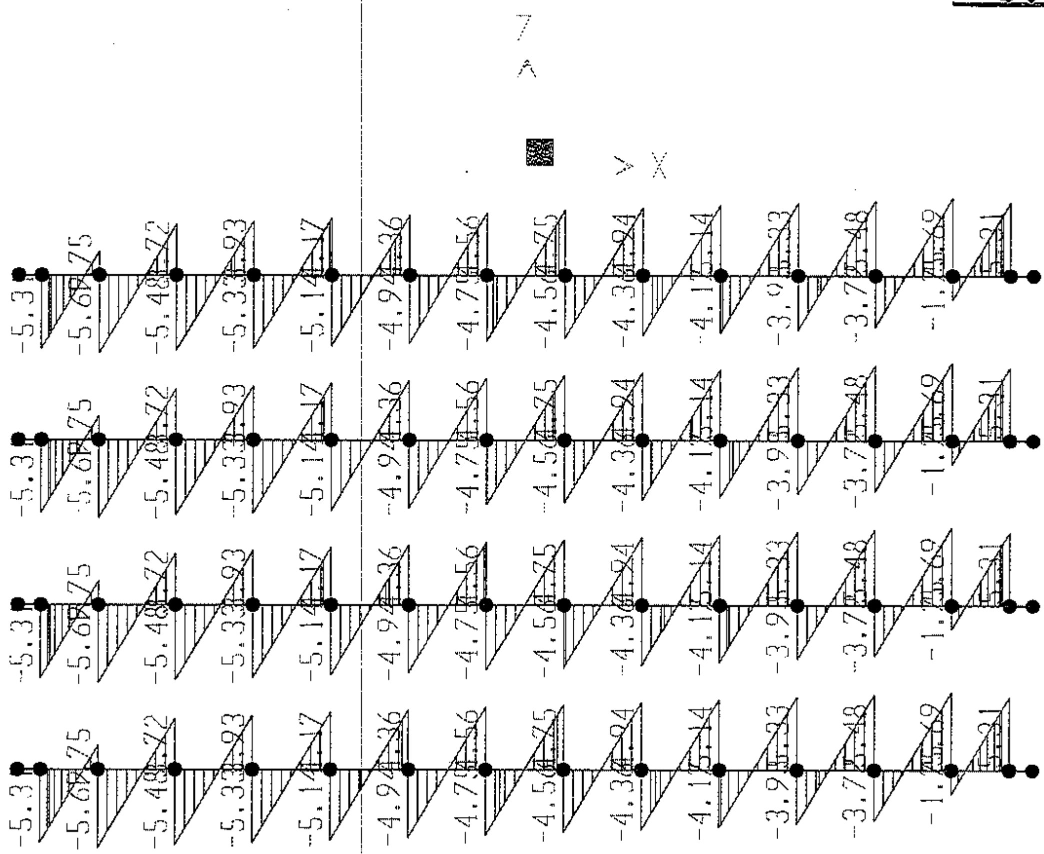
Shear Due to MMB Wt.'



(Sub FB shown only)

SAP2000 v14.0.0 - File:Sub FB2 - Shear Force 2-2 Diagram (MMB) - Kip, ft, F Units

Shear Due to Lane Load.



Max= 5.7^{K}.

SAP2000 v14.0.0 - File:Sub FB2 - Shear Force 2-2 Diagram (LL-lane) - Kip, ft, F Units

Shear due to 6-lane Loads + MMB.

(Per AASHTO LRFD 4th Ed., HL93).

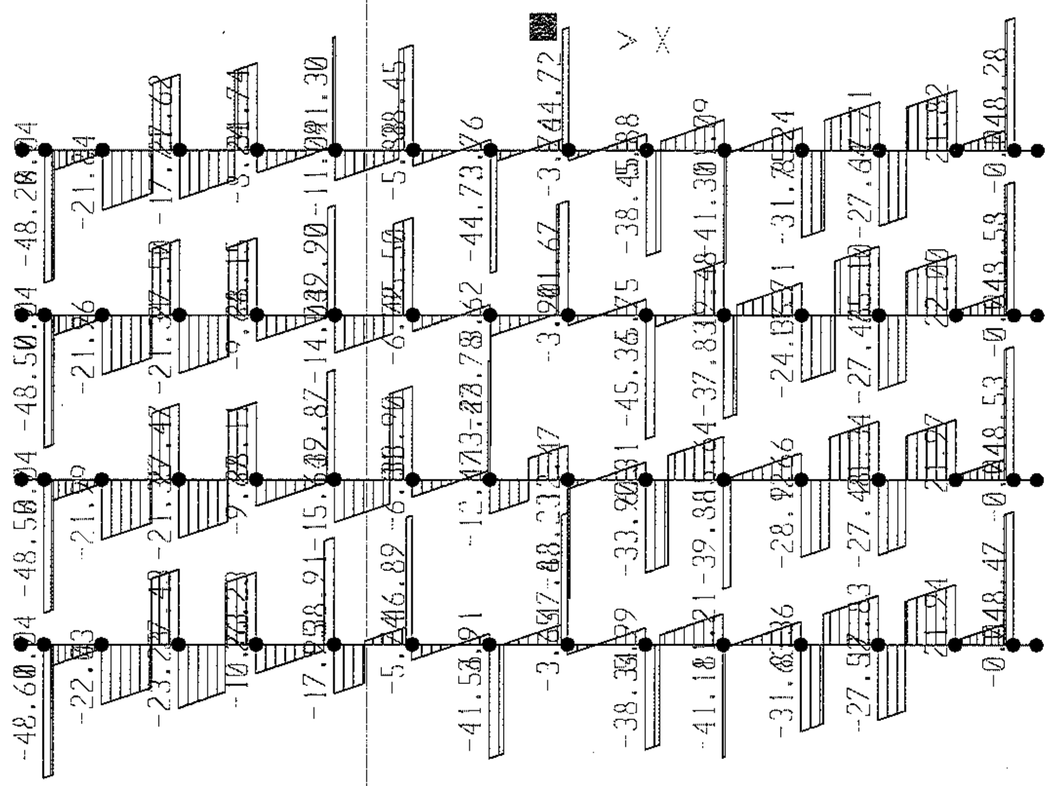
Note: No MMB for existing.

(Existing) $V_{max} = 48.3^k$

(4-2 Config) $= 48.5^k$

(3-3 Config) $= 48.5^k$

(2-4 Config) $= 48.5^k$



Impact

Load Case = 1.33 x (LL-Truck) + 1.0 (LL-lane) + 1.0 MMB.

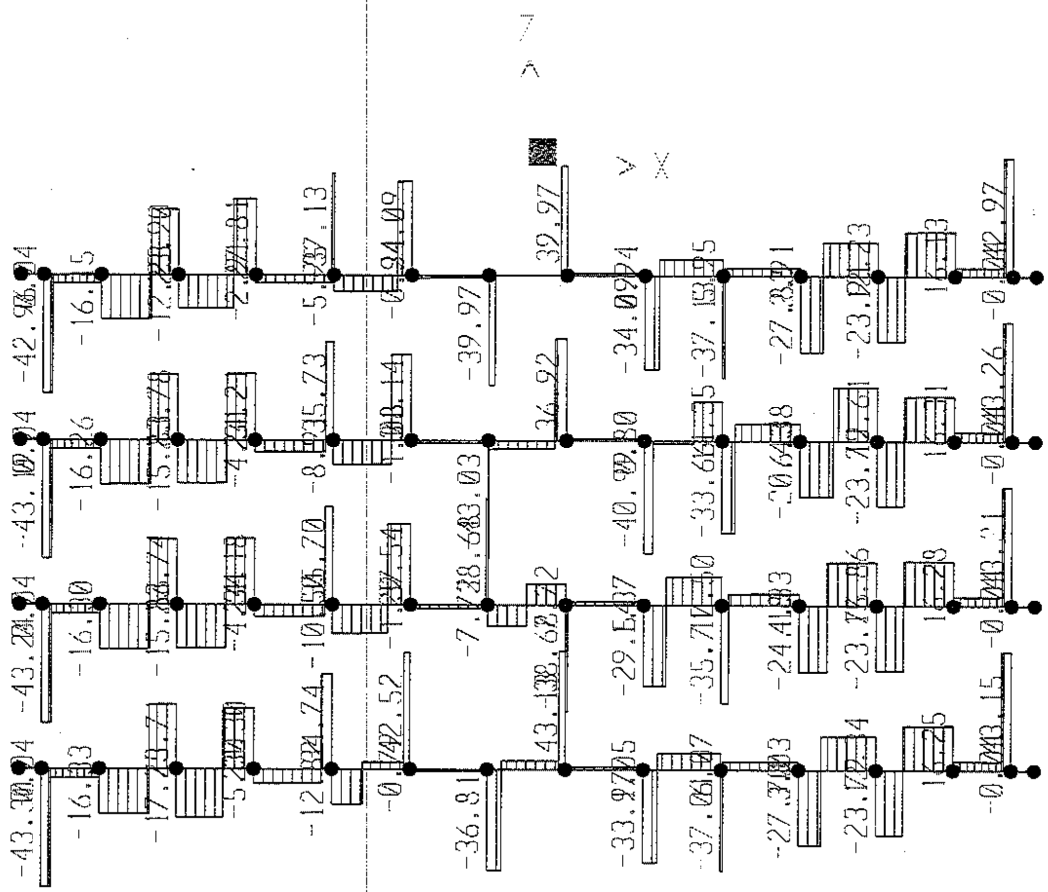
Shear due to 6-lane Loads + MMB.
(per AASHTO 1977, HS20-44).

(Existing). $V_{max} = 43^k$

(4-2 config) $= 43.3^k$

(3-3 config) $= 43.2^k$

(2-4 config) $= 43.3^k$



Impact
Load Case = 1.33 (LL-Truck) + 1.0 (MMB).

SAP2000 v14.0.0 - File:Sub FB2 - Shear Force 2-2 Diagram (HS20+MMB) - Kip, ft, F Units

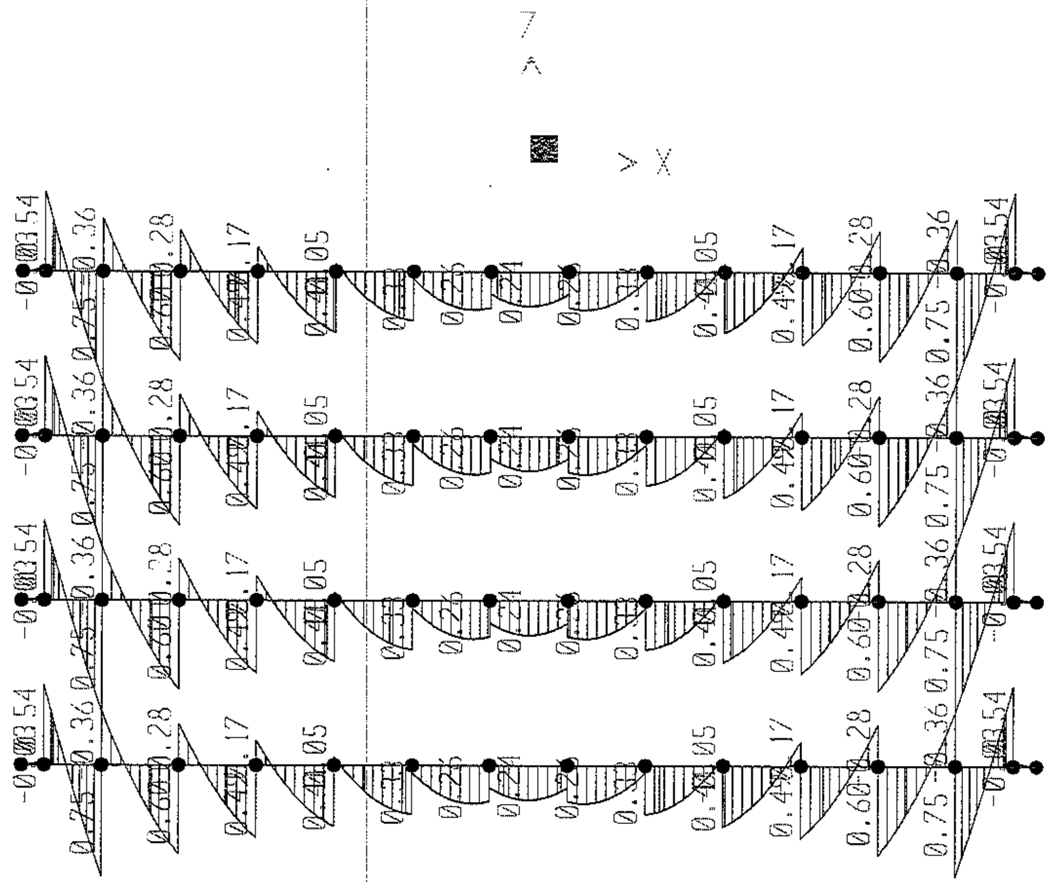
Self wt Moment (k-ft)

(E)

(4-2)

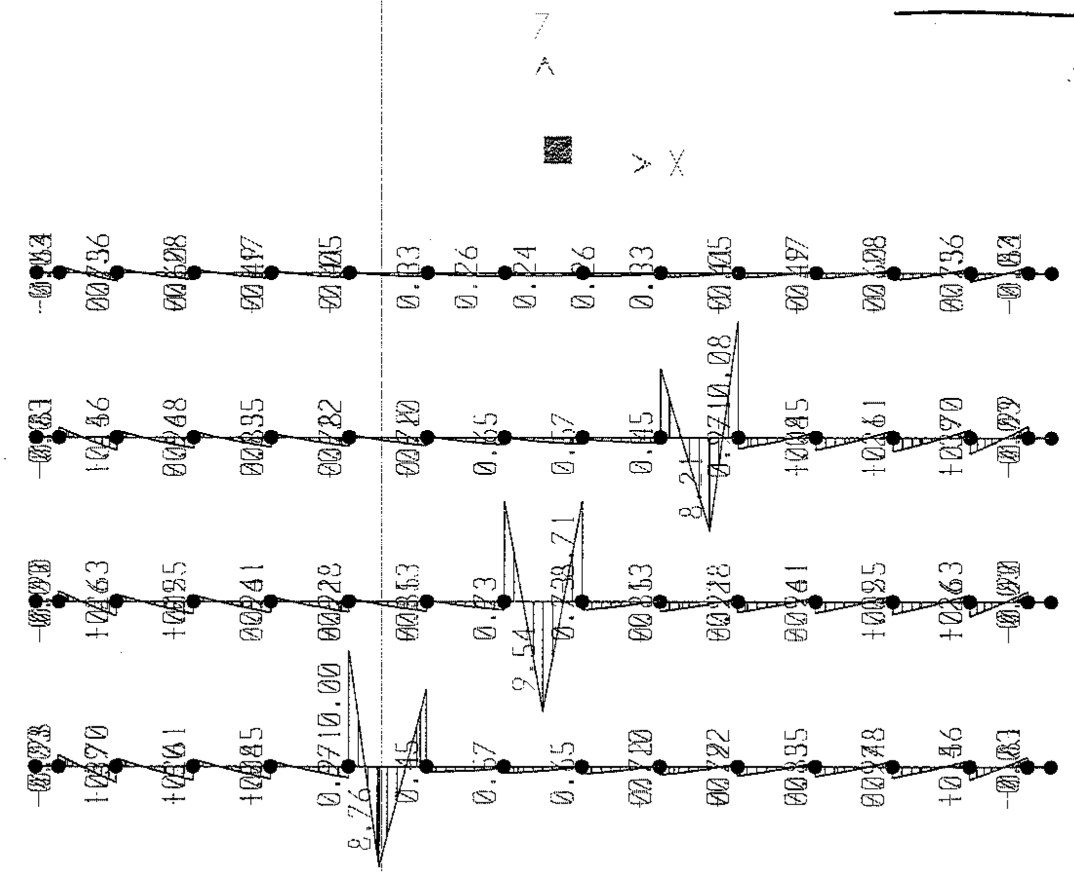
(3-3)

(2-4)



SAP2000 v14.0.0 - File:Sub FB2 - Moment 3-3 Diagram (DEAD) - Kip, ft, F Units

Moment : (MMB)

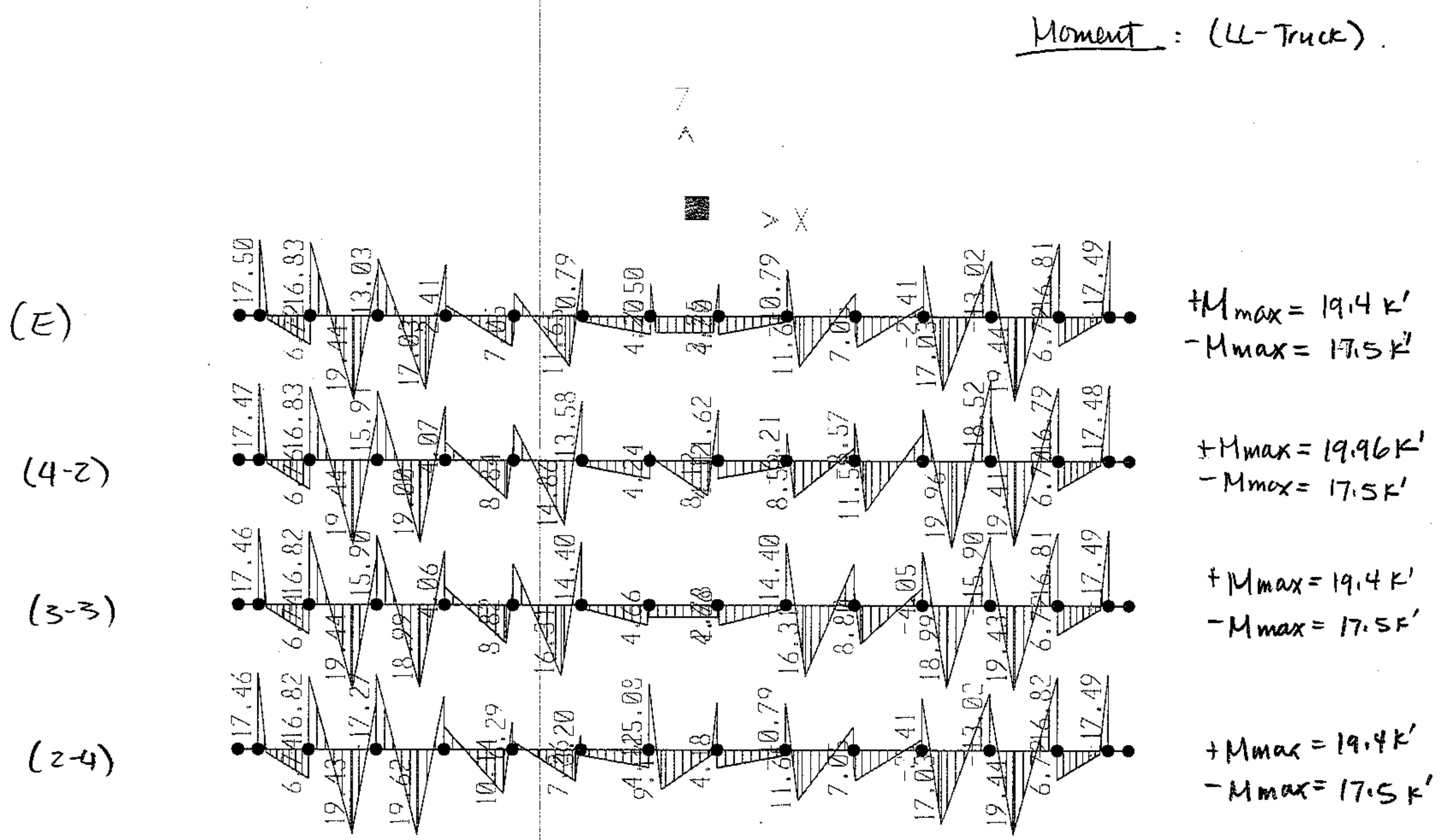


+M = 8.2 k'
−M = 10.1 k'

+M = 9.5 k'
−M = 8.7 k'

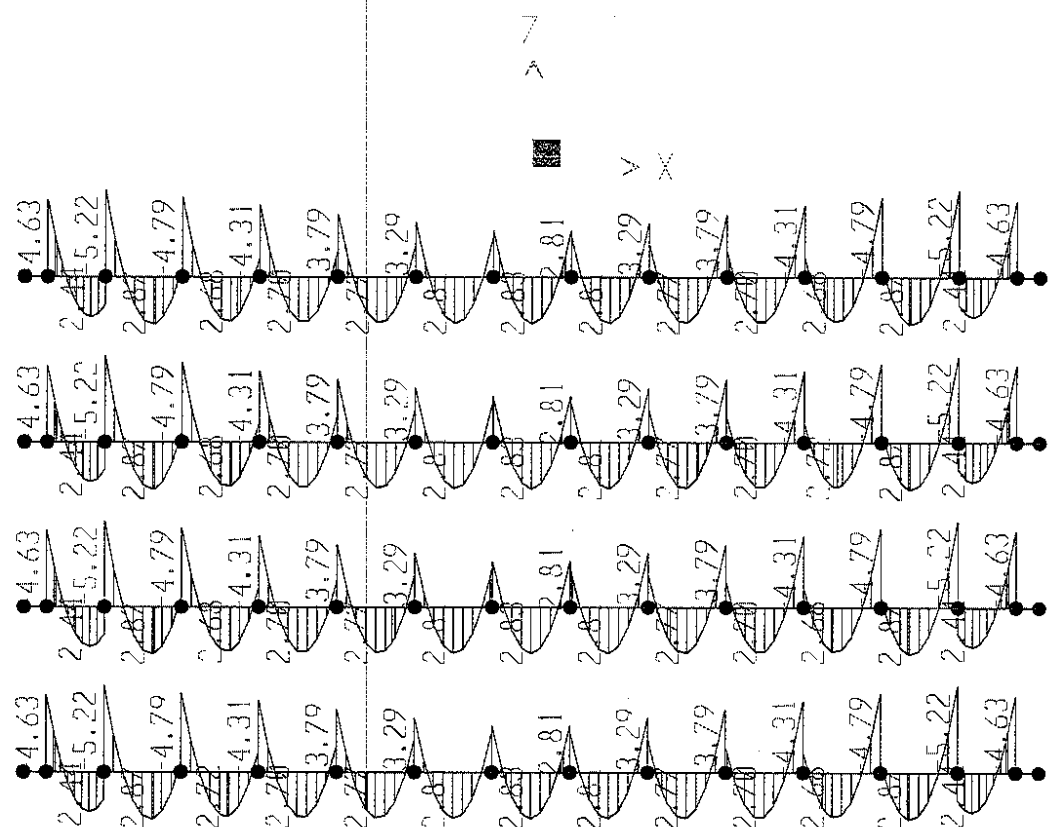
+M = 8.8 k'
−M = 10 k'

SAP2000 v14.0.0 - File:Sub FB2 - Moment 3-3 Diagram (MMB) - Kip, ft, F Units



SAP2000 v14.0.0 - File:Sub FB2 - Moment 3-3 Diagram (LL-truck) - Kip, ft, F Units

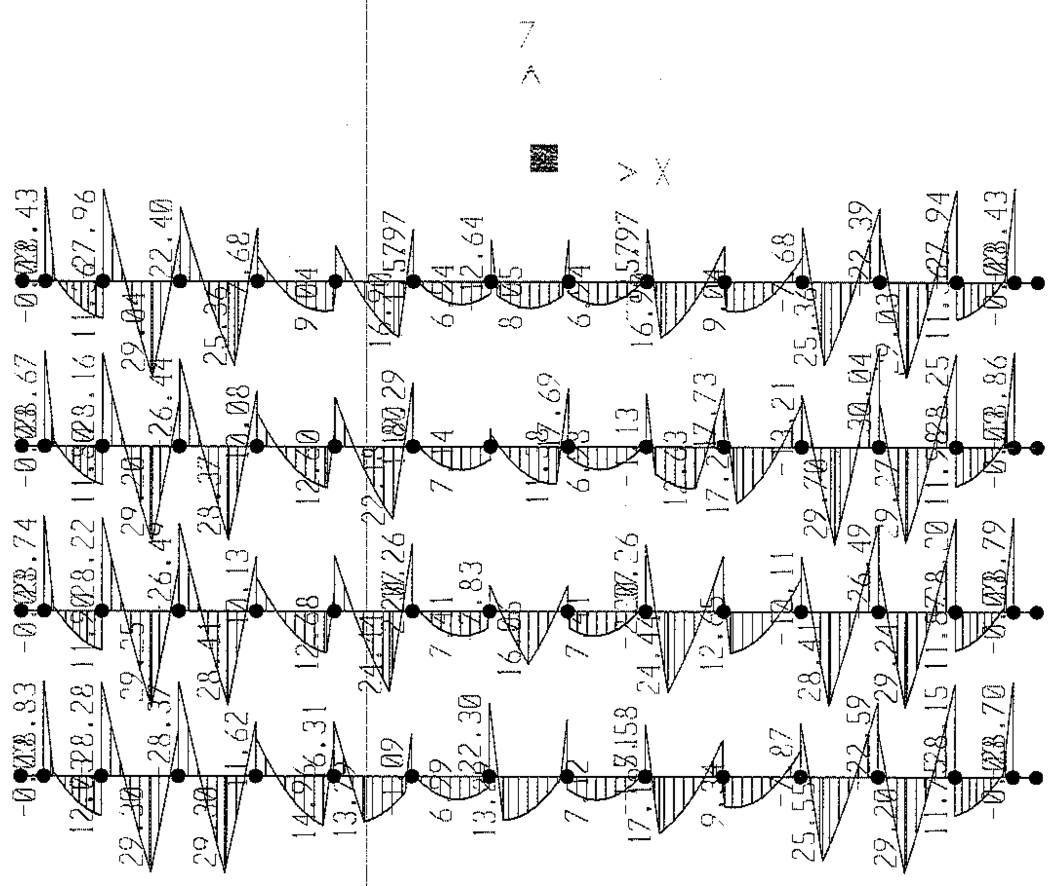
Moment : (LL-Lane).



+Mmax = 2.8 K'
−Mmax = 4.63 K'

SAP2000 v14.0.0 - File:Sub FB2 - Moment 3-3 Diagram (LL-lane) - Kip, ft, F Units

Moment (HL93 + MMB)



(Existing, no MMB)

+Mmax = 29.04 K'
−Mmax = 28.43 K'

(4-2 Config)

+Mmax = 29.7 K'
−Mmax = 30.04 K'

(3-3 Config)

+Mmax = 29.3 K'
−Mmax = 28.8 K'

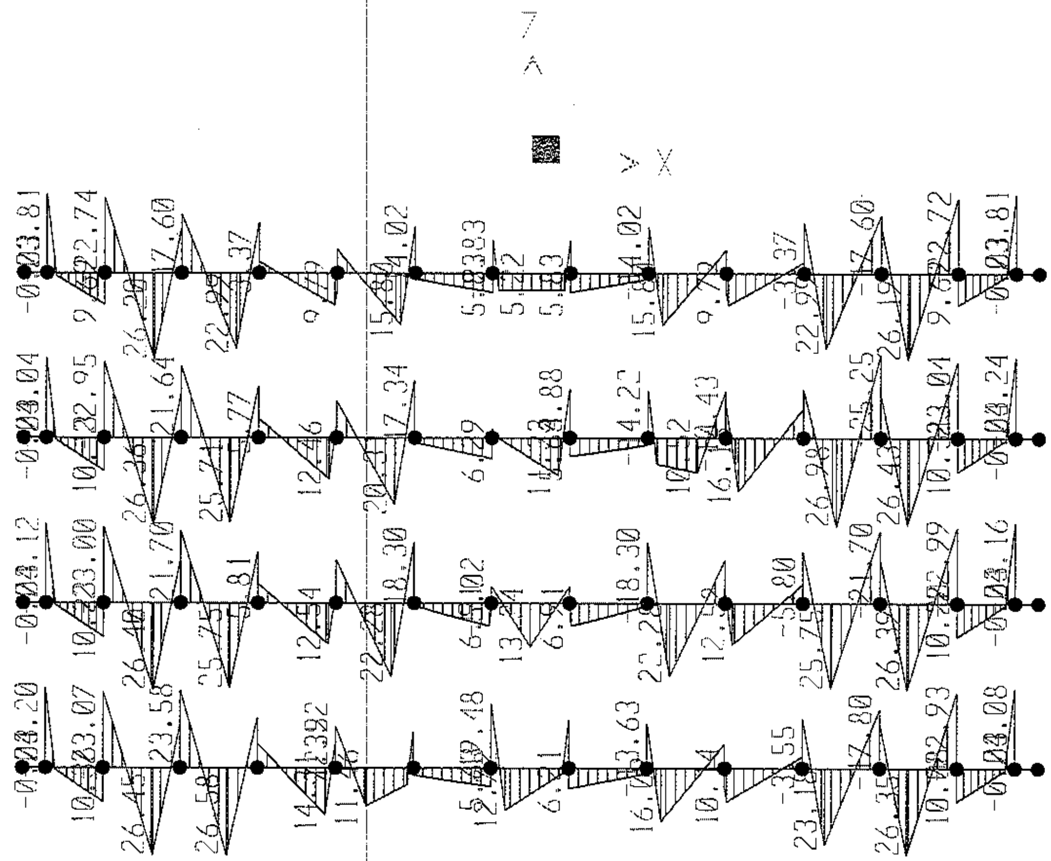
(2-4 Config)

+Mmax = 29.3 K'
−Mmax = 28.8 K'

Note: Effect with adding MMB is insignificant: 1) Max. forces occur @ the end of the beam; 2) The effect of any load quickly dissipates passing the supports of the beam — the load gets transfered to the stiff. Transv. F.B below, 3) Where MMB is located, wheel loads of the truck can't be applied between the 2 supports — c-c support spacing = 4'-9", & wheel-wheel spacing w/ MMB = 2' + 1' ℄ MMB + 2' = 5'

SAP2000 v14.0.0 - File:Sub FB2 - Moment 3-3 Diagram (HL93+MMB) - Kip, ft, F Units

Moment (HS20 + MMB)



(Existing, no MMB)
+Mmax = 26.2K'
−Mmax = 24.04K'

(4-2 Config)
+Mmax = 26.6K'
−Mmax = 25.3K'

(3-3 Config)
+Mmax = 26.4K'
−Mmax = 24.1K'

(2-4 Config)
+Mmax = 26.6K'
−Mmax = 24.2K'

SAP2000 v14.0.0 - File:Sub FB2 - Moment 3-3 Diagram (HS20+MMB) - Kip, ft, F Units

EVALUATION OF STEEL PLATE GIRDER

AECOM	Job Title:	GGB Moveable Median Barrier Study		Subject:	Steel Plate Girder Analysis
	Job No:	60097852			
	Prepared By:	MG/ PC	Date:	8/20/2010	
	Checked By:	HSL	Date:	10/15/2010	

Steel Plate Girder Analysis

Analysis Approach:

The steel plate girder is evaluated by using the dead load relieved during the 1983 deck replacement. This assumes that the design live load demand before and after the deck replacement is the same. Because of this, the Bridge is alleviated of some load under the service condition of dead plus live.

Dead Load "Before"

Dead load of original reinforced concrete deck, before 1983 orthotropic deck replacement

Original Reinforced Concrete Deck $w_{DL\_1} := 7in \cdot 62ft \cdot 150pcf$ $w_{DL\_1} = 5.42 \cdot \frac{k}{ft}$

Original Steel Stringer, 24 B 74 (12 total, Add 10% for Miscellaneous Components) $w_{DL\_2} := 12 \cdot 0.074 \frac{k}{ft} \cdot 1.1$ $w_{DL\_2} = 0.98 \cdot \frac{k}{ft}$

Reference As-Built Sheet No. 27

Total Dead Load $w_{DL\_old} := (w_{DL\_1} + w_{DL\_2})$ $w_{DL\_old} = 6.4 \cdot \frac{k}{ft}$

Dead Load "After"

Dead load after 1983 othortropic deck replacement

New Orthotropic Deck $w_{DL} := 4.36 \frac{k}{ft}$ $w_{DL} = 4.36 \cdot \frac{k}{ft}$

See Orthotropic Deck Dead Load Calculation

Note: The weight contribution of the steel stringers and sub floor beams is trivial as it will induce a minimal change on the moment demand, thus is it not included in this analysis.

Total Dead Load $w_{DL\_new} := w_{DL}$ $w_{DL\_new} = 4.36 \cdot \frac{k}{ft}$

Job Title:	GGB Moveable Median Barrier Study			Subject:	Steel Plate Girder Analysis
Job No:	60097852				
Prepared By:	MG/ PC	Date:	8/20/2010		
Supervised By:	HSL	Date:	10/15/2010		

Dead Load Reduction

The difference in dead load of original reinforced concrete deck and the replacement orthotropic deck

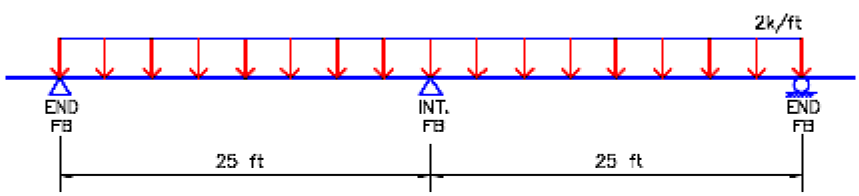
Dead Load Before $\quad w_{DL\_old} = 6.4 \cdot \frac{k}{ft}$

Dead Load After $\quad w_{DL\_new} = 4.36 \cdot \frac{k}{ft}$

Dead Load Reduction $\quad w_{red} := w_{DL\_old} - w_{DL\_new} \qquad w_{red} = 2.04 \cdot \frac{k}{ft}$

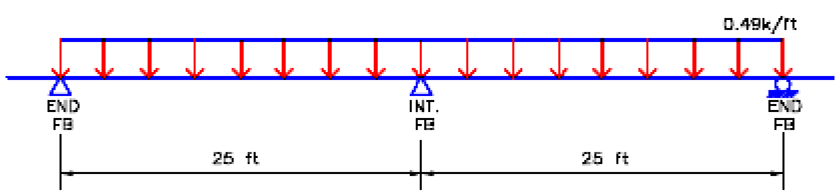
Reaction at Intermediate Floor Beam

Reaction at Intermediate Floor Beam due to DL Reduction



$$R_{Int\_DL} := \frac{10}{8} \cdot \left(2 \frac{k}{ft} \cdot 25ft\right) \qquad R_{Int\_DL} = 62.5 \cdot k$$

Reaction at Intermediate Floor Beam due to MMB



$$R_{Int\_MMB} := \frac{10}{8} \cdot \left(.49 \frac{k}{ft} \cdot 25ft\right) \qquad R_{Int\_MMB} = 15.31 \cdot k$$

	Job Title:	GGB Moveable Median Barrier Study			Subject:	Steel Plate Girder Analysis
AECOM	Job No:	60097852				
	Prepared By:	MG/ PC	Date:	8/20/2010		
	Supervised By:	HSL	Date:	10/15/2010		

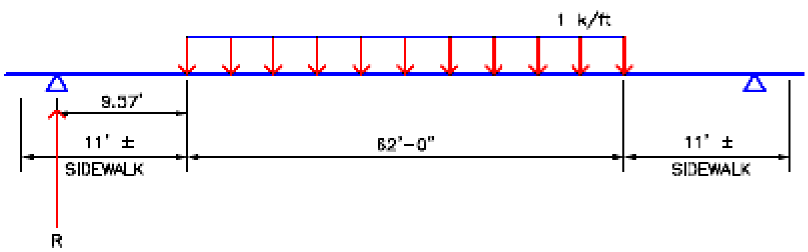
Dead Load Reduction over Deck Span of Transverse Floor Beam (Capacity)

Note: Load reduction from sidewalk deck replacement is ignored for simplicity and is more conservative

Roadway Deck Span $S := 62ft$

Reaction due to DL $R_{Int\_DL} = 62.5 \cdot k$

DL Reduction over Deck Span $DL_{red} := \frac{R_{Int\_DL}}{S}$ $DL_{red} = 1.01 \cdot \frac{k}{ft}$



Reaction at Support $R := DL_{red} \cdot \frac{1}{2} \cdot S$ $R = 31.25 \cdot k$

Maximum Shear $V_{max} := R$ $V_{max} = 31.25 \cdot k$

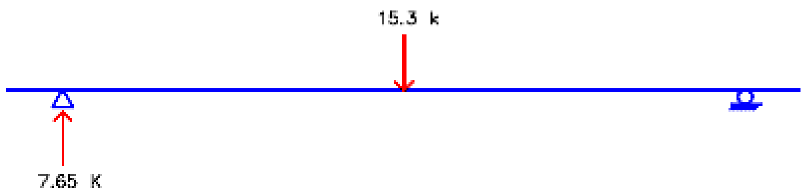
Distance, a $a := 9.75ft$

Load, w $w := DL_{red}$ $w = 1.01 \cdot \frac{k}{ft}$

Maximum Moment $M_{max} := R \cdot \left[a + \frac{R}{(2 \cdot w)} \right]$ $M_{max} = 789.06 \cdot k \cdot ft$

AECOM	Job Title:	GGB Moveable Median Barrier Study			Subject: Steel Plate Girder Analysis
	Job No:	60097852			
	Prepared By:	MG/ PC	Date:	8/20/2010	
	Supervised By:	HSL	Date:	10/15/2010	

Add MMB Load onto Deck Span of Transverse Floor Beam (Demand)



Three - Three Lane Configuration

Concentrated MMB Load, P $P := 15.3k$

Maximum Shear $V_{max} := \frac{P}{2}$ $V_{max} = 7.65 \cdot k$

Compare with DL $V_{max} = 7.65k < 31k$ OK

Maximum Moment $M_{max} := 7.65k \cdot (31ft + 9.75ft)$ $M_{max} = 311.74 \cdot k \cdot ft$

Four - Two/ Two-Four Lane Configuration

Maximum Shear By observation, $V_{max} < 15.3k < 31k$

Maximum Moment By observation $M_{max} < 312k \cdot ft < 789k \cdot ft$

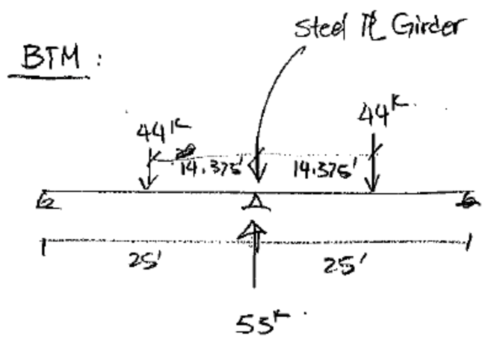
2.1.19.4.3
0117

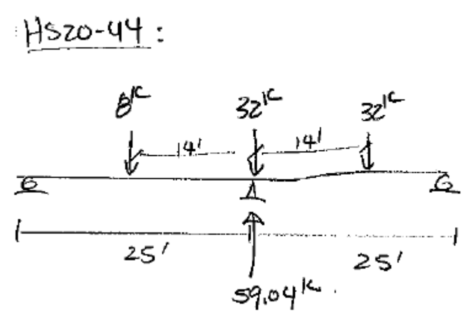
AECOM	Job Title:	GGB Moveable Median Barrier Study		Subject:	Steel Plate Girder Analysis
	Job No:	60097852			
	Prepared By:	MG/ PC	Date:	8/20/2010	
	Supervised By:	HSL	Date:	10/15/2010	

Conclusion for the Steel Plate Girder Analysis

The increase in the transverse floor beam moment and shear demand due to the addition of the MMB is less than the moment and shear load relieved from the bridge deck replacement. The original transverse floor beam remained after the deck replacement, and thus should be able to carry the additional MMB load.

Note: The effect of the BTM need not be analyzed because the BTM takes up one traffic lane, and a highway truck reaction is larger than the BTM reaction on the Transverse Floor Beam.





GLOBAL DEFLECTION CHECK

2.1.19.4.3
0117

AECOM	Job Title:	GGB Moveable Median Barrier	
	Job No.:	60097582	
	Calculation No.:		
Subject: *Global Deflection Check*	Prepared By:	LL /PC	Date: 9/28/2010
	Checked By:	HSL	Date: 10/15/2010

Reference:
- Golden Gate Bridge As-built Sheet B51A & B51A dated June 11, 1935 (Attached)
- Golden Gate Bridge, Highway and Transportation District, "Design and Construction Statisitcs."
<http://goldengatebridge.org/research/factsGGBDesign.php> (28 September 2010).

Dead Load in 1935 before Deck Replacement in 1986
- based on Golden Gate Bridge As-built Sheet B51A & B51A

Label	Dead Load Concentration per Cable (kips)	Label	Dead Load Concentration per Cable (kips)
1	152.8	67	542.5
3	533.0	69	540.0
5	562.8	71	541.0
7	512.9	73	536.7
9	514.7	75	537.3
11	516.7	77	536.3
13	517.3	79	537.7
15	518.1	81	535.1
17	520.8	83	534.4
19	522.8	85	531.7
21	524.6	87	531.0
23	528.2	89	528.4
25	5828.4	91	529.0
27	528.6	93	525.6
29	531.7	95	527.3
31	534.5	97	524.0
33	536.1	99	523.9
35	537.4	101	520.7
37	537.7	103	521.0
39	540.4	105	517.8
41	549.2	107	518.5
43	518.3	109	515.5
47	549.9	111	514.2
49	569.4	113	512.5
51	556.2	115	513.5
53	552.8	117	512.1
55	555.2	119	514.4
57	550.0	121	511.1
59	550.7	123	511
61	544.4	125	511
63	545.8	127	513.3
65	541.5	129	264.2

Per side of cable	Subtotal DL	38616 kips
For entire bridge	**Total DL = subtotal X 4**	**154462 kips**

AECOM	Job Title:	GGB Moveable Median Barrier			Subject: Global Deflection Check
	Job No:	60097852			
	Prepared By:	LL /PC	Date:	9/28/2010	
	Checked By:	HSL	Date:	10/15/2010	

Global Deflection Check

Total length of suspension span including main span and side spans

$L := 4200ft + 2 \cdot 1125ft$

Bridge Dead Load including main span and side spans before Deck Replacement
(Per GGB As-built Sheet B51A & B52B)

$DL_{original} := 154462k$ $\quad DL_{original} = 77231 \cdot ton$

Dead Weight reduced due to Deck Replacement in 1986
(Per Golden Gate Bridge, Highway and Transportation District Website)

$DL_{reduced} := 12300ton$ $\quad DL_{reduced} = 12300 \cdot ton$

Dead Weight reduced due to Deck Replacement in 1986
(Per calculation in steel plate girder analysis)

$DL_{reduced2} := (6.4 - 4.36) \cdot \frac{k}{ft} \cdot L$

$DL_{reduced2} = 6579 \cdot ton$

Assumed the dead weight reduced for deck replacement is higher number of the two above, i.e. DL reduced = 12300ton, which is a conservative assumption.

Existing Bridge Dead Load

$DL_{current} := DL_{original} - DL_{reduced}$

$DL_{current} = 64931 \cdot ton$

Additional Load due to MMB

$MMB := 0.49 \frac{k}{ft} \cdot L$ $\quad MMB = 1580 \cdot ton$

Percentage of Dead Load Increase Due to MMB

$Percentage\_Change := \frac{MMB}{DL_{current}} \cdot 100\%$

$\boxed{Percentage\_Change = 2.43 \cdot \%}$

Conclusion

Assuming the dead load reduction due to deck replacement on the Suspended Span is no more than 12300 tons, the change in the Suspended Bridge deflect shape due to the addition of the moveable median barrier is less than 3%, and may be considered as insignificant.

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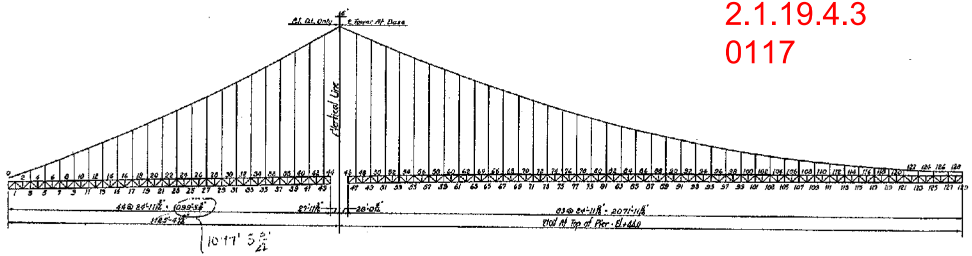
0117



CENTER SPAN DEAD LOAD CONCENTRATIONS — KIPS PER CABLE

ITEM	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65
Cable & Handropes		157.2		156.6		155.9		155.3		154.7		154.2		153.6		153.0		152.5		152.0
Cable Bands		7.0		7.0		7.0		7.0		7.0		7.0		7.0		7.0		7.0		6.3
½ Suspenders + Tie at Cable		12.1		11.5		11.0		10.5		10.0		9.5		9.0		8.5		8.1		7.6
Concentrations at Cable		177.1		175.9		174.7		173.6		172.5		171.5		170.4		169.3		168.4		165.9
½ Suspenders + Split Collars Sockets & Shims		12.9		12.3		11.8		11.2		10.7		10.2		9.7		9.2		8.7		8.3
Truss	24.5	41.9	40.4	41.0	38.3	40.6	38.1	40.1	38.1	40.9	38.1	40.1	38.1	40.6	38.1	40.1	38.1	40.6	38.1	40.1
Main Laterals, Center Struts & Portals	42.4	17.6	22.4	7.5	7.5	9.5	7.1	7.3	7.1	9.4	6.7	6.8	6.7	6.7	6.1	6.2	6.1	6.5	6.1	6.1
Floorbeams & Knee Braces		25.3	30.4	25.0	24.9	25.1	25.0	25.0	25.0	25.1	25.0	25.0	25.0	25.1	25.0	25.0	25.0	25.1	25.0	25.0
Roadway Stringers & Scuppers		20.2	14.4	15.3	13.7	15.2	13.6	15.3	13.7	15.2	13.6	15.3	13.7	15.2	13.6	15.3	13.7	15.2	13.6	15.3
Sidewalk Stringers, Laterals & Curb		9.5	8.0	7.8	7.6	7.6	7.6	7.0	8.0	7.8	8.2	7.6	8.2	7.6	8.0	7.8	7.6	7.6	7.6	7.8
Roadway Slab		72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6
Sidewalk Slab		13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	14.0	14.9	14.1	14.9	14.0	13.9	13.9	13.9	13.9	13.9	13.9
Handrailing & Electroliers		2.0	3.7	2.0	2.0	2.0	2.0	2.0	3.7	2.1	2.5	2.2	2.5	2.1	3.7	2.0	2.0	2.0	2.0	2.0
Conduits, Pipe Lines, Diaphones etc.		0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
Ladders & Platforms																			2.9	
Paint on Structural Steel	.6	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
Total Suspended Structure at Panel Points	67.7	217.4	202.3	198.9	182.0	199.0	181.4	196.7	183.6	199.3	183.1	195.4	183.2	197.5	182.5	193.6	180.5	195.5	183.3	192.6
Total Suspended Structure at Suspender Points	15.9	372.8		393.5		381.5		372.2		382.7		378.5		380.3		375.1		377.4		375.6
Total Concentrations	*15.9	549.9		562.4		556.2		558.0		555.2		550.0		550.7		544.4		545.8		541.5

ITEM	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80	81	82	83	84	85
Cable & Handropes		151.5		151.0		150.6		150.1		149.7		149.2		148.8		148.4		148.1		147.7
Cable Bands		6.3		6.3		6.3		6.3		6.3		6.3		6.3		6.3		5.3		5.3
½ Suspenders + Tie at Cable		7.2		6.8		6.4		6.0		5.6		5.3		4.9		4.6		4.3		4.0
Concentrations at Cable		165.0		164.1		163.3		162.4		161.6		160.8		160.0		159.3		157.7		157.0
½ Suspenders + Split Collars Sockets & Shims		7.9		7.5		7.1		6.7		6.3		5.9		5.6		5.3		5.0		4.7
Truss	38.1	41.2	39.6	42.2	40.1	42.6	40.1	42.2	40.5	43.7	41.6	43.8	41.6	44.2	41.6	43.8	41.6	44.2	41.5	43.8
Main Laterals, Center Struts & Portals	6.1	6.0	6.1	6.1	6.1	7.1	4.7	5.0	4.7	6.0	4.3	4.5	4.3	5.5	4.1	4.3	4.1	5.3	3.8	4.0
Floorbeams & Knee Braces	25.0	25.1	25.0	25.0	25.0	25.1	25.0	25.0	25.0	25.1	25.0	25.0	25.0	25.1	25.0	25.0	25.0	25.1	25.0	25.0
Roadway Stringers & Scuppers	13.7	15.2	13.6	15.3	13.7	15.2	13.6	15.3	13.7	15.2	13.6	15.3	13.7	15.2	13.6	15.3	13.7	15.2	13.6	15.3
Sidewalk Stringers, Laterals & Curb	8.0	7.8	7.6	7.6	7.6	7.8	8.0	7.8	7.6	7.6	7.6	7.8	8.0	7.8	8.2	7.6	8.2	7.6	8.0	7.8
Roadway Slab	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6
Sidewalk Slab	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	14.0	14.9	14.1	14.9	14.0	13.9	13.9
Handrailing & Electroliers	3.7	2.0	2.0	2.0	2.0	2.0	3.7	2.0	2.0	2.0	2.0	2.0	3.7	2.1	2.5	2.2	2.5	2.1	3.7	2.0
Conduits, Pipe Lines, Diaphones etc.	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
Ladders & Platforms																				
Paint on Structural Steel	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
Total Suspended Structure at Panel Points	182.6	195.2	181.9	193.7	182.5	194.9	183.1	192.0	181.5	193.9	182.1	192.5	184.3	193.6	184.0	191.7	184.1	192.6	183.6	190.6
Total Suspended Structure at Suspender Points		377.5		375.9		377.7		374.3		375.7		375.5		377.7		375.8		376.7		374.7
Total Concentrations		542.5		540.0		541.0		536.7		537.3		536.3		537.7		535.1		534.4		531.7

ITEM	86	87	88	89	90	91	92	93	94	95	96	97	98	99	100	101	102	103
Cable & Handropes		147.3		147.0		146.7		146.4		146.1		145.9		145.6		145.5		145.2
Cable Bands		5.3		5.3		5.3		5.3		5.3		5.3		5.3		3.6		3.6
½ Suspenders + Tie at Cable		3.7		3.4		3.1		2.9		2.7		2.4		2.2		2.0		1.9
Concentrations at Cable		156.3		155.7		155.1		154.6		154.1		153.6		153.1		151.1		150.7
½ Suspenders + Split Collars Sockets & Shims		4.4		4.1		3.8		3.6		3.3		3.1		2.9		2.7		2.5
Truss	41.6	44.2	41.6	43.8	41.6	44.2	41.6	43.8	41.6	44.2	41.2	43.1	40.8	42.9	40.1	42.2	40.1	42.4
Main Laterals, Center Struts & Portals	3.8	5.1	3.8	4.0	3.8	5.1	3.8	4.0	3.8	5.1	3.8	4.0	3.8	5.1	3.8	4.0	3.8	5.1
Floorbeams & Knee Braces	25.0	25.1	25.0	25.0	25.0	25.1	25.0	25.0	25.0	25.1	25.0	25.0	25.0	25.1	25.0	25.0	25.0	25.1
Roadway Stringers & Scuppers	12.7	15.2	13.6	15.3	13.7	15.2	13.6	15.3	13.7	15.2	13.6	15.3	13.7	15.2	13.6	15.3	12.7	15.2
Sidewalk Stringers, Laterals & Curb	7.6	7.6	7.6	7.8	8.0	7.8	7.6	7.6	7.6	7.8	8.0	7.8	7.6	7.6	7.6	7.8	8.0	7.8
Roadway Slab	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6
Sidewalk Slab	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	14.0
Handrailing & Electroliers	2.0	2.0	2.0	2.0	3.7	2.0	2.0	2.0	2.0	2.0	3.7	2.0	2.0	2.0	2.0	2.0	3.7	2.1
Conduits, Pipe Lines, Diaphones etc.	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
Ladders & Platforms	2.9														2.9			
Paint on Structural Steel	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
Total Suspended Structure at Panel Points	184.6	191.6	181.6	190.0	183.0	191.2	181.6	189.3	181.7	190.7	183.3	188.5	180.9	188.6	183.0	182.0	182.3	188.3
Total Suspended Structure at Suspender Points		374.7		372.7		373.9		371.0		373.2		370.4		370.0		369.9		370.9
Total Concentrations		531.0		528.4		529.0		525.6		527.3		524.0		523.9		520.7		521.0

*Carried by Link

ITEM	104	105	106	107	108	109	110	111	112	113	114	115	116
Cable & Handropes		145.0		144.9		144.7		144.6		144.4		144.3	
Cable Bands		3.6		3.6		3.6		3.6		3.6		3.6	
½ Suspenders + Tie at Cable		1.7		1.5		1.4		1.3		1.2		1.1	
Concentrations at Cable		150.3		150.0		149.7		149.5		149.2		149.0	
½ Suspenders + Split Collars, Sockets & Shims		2.4		2.2		2.1		2.0		1.9		1.8	
Truss	38.6	41.3	39.2	41.6	39.2	41.3	39.2	41.0	38.1	40.1	38.1	40.3	38.1
Main Laterals, Center Struts & Portals	3.8	4.0	3.8	5.1	3.8	4.0	3.8	5.1	3.8	4.0	3.8	5.1	3.8
Floorbeams & Knee Braces	25.0	25.0	25.0	25.1	25.0	25.0	25.0	25.1	25.0	25.0	25.0	25.1	25.0
Roadway Stringers & Scuppers	13.6	15.3	13.7	15.2	13.6	15.3	13.7	15.2	13.6	15.3	13.7	15.2	13.6
Sidewalk Stringers, Laterals & Curb	8.2	7.6	8.2	7.8	8.0	7.8	7.6	7.6	7.6	7.8	8.0	7.8	7.6
Roadway Slab	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6
Sidewalk Slab	14.9	14.1	14.9	14.0	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9
Handrailing & Electroliers	2.5	2.2	2.5	2.1	3.7	2.0	2.0	2.0	2.0	2.0	3.7	2.0	2.0
Conduits, Pipe Lines, Diaphones, etc.	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
Ladders & Platforms													
Paint on Structural Steel	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
Total Suspended Structure at Panel Points	181.7	186.0	181.4	182.2	181.3	185.5	179.3	186.0	178.1	184.1	180.3	185.9	178.1
Total Suspended Structure at Suspender Points		367.5		368.5		366.6		364.7		363.3		364.5	
Total Concentrations		517.8		518.5		515.5		514.2		512.5		513.5	

ITEM	117	118	119	120	121	122	123	124	125	126	127	128	℄
Cable & Handropes	144.2		144.1		144.1		144.0		144.0		143.9		72.0
Cable Bands	3.6		3.6		3.3		3.3		3.3		3.3		1.7
½ Suspenders + Tie at Cable	1.0		0.9		0.8		0.8		0.8		0.7		0.4
Concentrations at Cable	148.8		148.6		148.2		148.1		148.1		147.9		74.1
½ Suspenders + Split Collars, Sockets & Shims	1.7		1.6		1.5		1.5		1.5		1.4		0.7
Truss	40.1	38.1	40.9	38.1	40.1	38.1	40.3	38.1	40.1	38.1	40.3	38.1	20.1
Main Laterals, Center Struts & Portals	4.0	3.8	5.1	3.8	4.0	3.8	5.1	3.8	4.0	3.8	5.1	3.8	2.3
Floorbeams & Knee Braces	25.0	25.0	25.1	25.0	25.0	25.0	25.1	25.0	25.0	25.0	25.1	25.0	12.5
Roadway Stringers & Scuppers	15.3	13.7	15.2	13.6	15.3	13.7	15.2	13.6	15.3	13.7	15.2	13.7	7.7
Sidewalk Stringers, Laterals & Curb	7.6	7.6	7.8	8.0	7.8	7.6	7.6	7.6	7.8	8.0	7.8	8.2	3.8
Roadway Slab	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	36.3
Sidewalk Slab	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	14.0	14.9	7.1
Handrailing & Electroliers	2.0	2.0	2.0	3.7	2.0	2.0	2.0	2.0	2.0	3.7	2.1	2.5	1.1
Conduits, Pipe Lines, Diaphones, etc.	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	1.4
Ladders & Platforms		2.9											6.3
Paint on Structural Steel	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	.6
Total Suspended Structure at Panel Points	183.7	181.1	185.1	180.2	183.7	178.2	184.6	178.1	183.7	180.3	185.1	180.3	99.9
Total Suspended Structure at Suspender Points	363.3		365.8		362.9		362.9		362.9		365.4		150.1
Total Concentrations	512.1		514.4		511.1		511.0		511.0		513.3		264.2

NOTE:-
Assumed weight of painted wrapped cable = 2880 lbs. per lin. ft.
Assumed weight of painted suspenders - 12.2 lbs. per lin. ft. per rope.

THE GOLDEN GATE BRIDGE
AT
SAN FRANCISCO, CAL.
FOR THE
GOLDEN GATE BRIDGE AND HIGHWAY DISTRICT
OF CALIFORNIA

D.L. CONCENTRATIONS — CENTER SPAN

DATE JUNE 11 1935 | SHEET No. 851 A

CORRECT: Chief Engineer

PLANS PREPARED BY STRAUSS ENGINEERING CORPORATION. Vice-President.

APPROVED: Consulting Engineer. Consulting Engineer. Consulting Engineer.

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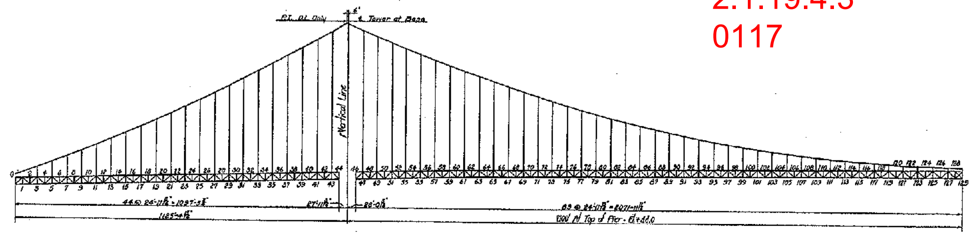

SIDE SPAN DEAD LOAD CONCENTRATIONS – KIPS PER CABLE

ITEM	0	1	2	3	4	5	6	7	8	9	10	11	12
Cable & Handropes		152.0		152.9		153.4		153.9		154.5		155.0	
Cable Bands				7.8		7.8		7.8		7.8		7.8	
½ Suspenders + Tie at Cable				1.2		1.6		2.0		2.5		2.9	
Concentrations at Cable		152.0		161.9		162.8		163.7		164.8		165.7	
½ Suspenders + Split Sockets & Shims				1.9		2.3		2.7		3.2		3.6	
Truss	20.9	30.5	29.8	30.7	26.9	30.5	22.4	30.5	20.9	30.8	30.2	31.8	30.1
Main Laterals, Center Struts & Portals	9.2	10.8	14.1	6.1	6.1	6.1	6.9	4.7	5.0	4.7	6.0	4.1	4.3
Floor Beams & Knee Braces	26.3	24.9	24.8	25.0	24.9	25.0	25.0	25.0	24.9	25.0	25.0	25.0	24.9
Roadway Stringers, Break in Floor & Scuppers	16.2	13.6	13.6	15.3	13.7	15.2	13.6	15.3	13.7	15.2	13.6	15.9	13.7
Sidewalk Stringers, Laterals Curb & Break in Floor	4.4	7.9	7.6	7.6	7.6	7.8	8.0	7.8	8.2	7.6	8.2	7.8	8.0
Roadway Slab	39.3	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6
Sidewalk Slab	7.4	13.9	13.9	13.9	13.9	13.9	13.9	14.0	14.9	14.1	14.9	14.0	13.9
Handrailing & Electroliers	1.0	2.0	2.0	2.0	2.0	2.0	3.7	2.1	2.5	2.2	2.5	2.1	3.7
Conduits, Pipe Lines, Diaphones etc	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
Ladders & Platforms													
Paint on Structural Steel	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
Total Suspended Structure at Panel Points	127.6	177.7	173.7	176.4	171.0	176.7	174.6	176.0	172.0	176.7	174.3	177.6	172.5
Total Suspended Structure at Suspender Points	325.4			371.1		400.0		349.2		349.9		351.0	
Total Concentrations	*325.4	152.8		533.0		562.8		512.9		516.7		516.7	

ITEM	13	14	15	16	17	18	19	20	21	22	23	24	25
Cable & Handropes	155.6		156.1		156.7		157.3		157.9		158.5		159.2
Cable Bands	7.8		7.8		7.8		7.8		7.8		8.4		8.4
½ Suspenders + Tie at Cable	3.4		3.9		4.4		4.9		5.4		5.9		6.5
Concentrations at Cable	166.8		167.8		168.9		170.0		171.1		172.8		174.1
½ Suspenders + Split Sockets & Shims	4.1		4.6		5.1		5.6		6.1		6.6		7.2
Truss	32.1	31.3	32.6	30.9	32.6	31.5	32.8	31.2	32.8	31.6	32.8	31.2	32.6
Main Laterals, Center Struts & Portals	4.1	5.1	3.8	3.9	3.0	4.0	3.8	3.9	3.8	4.0	3.0	3.9	3.8
Floor Beams & Knee Braces	25.0	25.0	25.0	24.9	25.0	25.0	25.0	24.9	25.0	25.0	25.0	24.9	25.0
Roadway Stringers, Break in Floor & Scuppers	15.2	13.6	16.3	13.7	15.2	13.6	15.3	13.7	15.2	13.6	15.3	13.7	15.2
Sidewalk Stringers, Laterals Curb & Break in Floor	7.8	7.6	7.6	7.6	7.8	8.0	7.6	7.6	7.6	7.6	7.6	8.0	7.6
Roadway Slab	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6
Sidewalk Slab	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9	13.9
Handrailing & Electroliers	2.0	2.0	2.0	2.0	2.0	3.7	2.0	2.0	2.0	2.0	2.0	3.7	2.0
Conduits, Pipe Lines, Diaphones etc	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
Ladders & Platforms										2.9			
Paint on Structural Steel	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
Total Suspended Structure at Panel Points	178.1	172.6	178.7	170.8	179.3	174.4	180.1	171.1	180.3	175.3	181.1	173.2	181.6
Total Suspended Structure at Suspender Points	350.5		350.9		351.9		352.8		355.5		355.4		354.3
Total Concentrations	517.3		518.1		520.8		522.8		526.6		528.2		528.4

ITEM	26	27	28	29	30	31	32	33	34	35	36	37	38
Cable & Handropes		159.8		160.5		161.2		161.9		162.6		163.3	
Cable Bands		8.4		8.4		8.4		8.4		8.4		8.4	
½ Suspenders + Tie at Cable		7.0		7.6		8.2		8.8		9.4		10.0	
Concentrations at Cable		175.2		176.5		177.8		179.1		180.4		181.7	
½ Suspenders + Split Sockets & Shims		7.7		8.3		8.9		9.6		10.2		10.8	
Truss	31.5	32.6	30.9	32.6	31.3	32.1	30.2	31.8	30.3	30.8	26.9	30.5	29.4
Main Laterals, Center Struts & Portals	4.0	3.8	3.9	3.0	5.1	4.1	4.3	4.1	6.0	4.7	5.0	4.7	8.9
Floor Beams & Knee Braces	25.0	25.0	24.9	25.0	25.0	25.0	24.9	25.0	25.0	25.0	24.9	25.0	25.0
Roadway Stringers, Break in Floor & Scuppers	13.6	15.3	13.7	15.2	13.6	15.3	13.7	15.2	13.6	15.3	13.7	15.2	13.6
Sidewalk Stringers, Laterals Curb & Break in Floor	7.6	7.6	7.6	7.6	8.0	7.8	8.2	7.6	8.2	7.8	8.0	7.6	7.6
Roadway Slab	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6	72.6
Sidewalk Slab	13.9	13.9	13.9	13.9	13.9	14.0	14.9	14.1	14.9	14.0	13.9	13.9	13.9
Handrailing & Electroliers	2.0	2.0	2.0	2.0	3.7	2.1	2.5	2.2	2.5	2.1	3.7	2.0	2.0
Conduits, Pipe Lines, Diaphones etc	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
Ladders & Platforms													
Paint on Structural Steel	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
Total Suspended Structure at Panel Points	172.3	181.8	170.8	182.5	174.5	182.8	172.6	183.5	174.4	183.8	172.0	183.8	172.9
Total Suspended Structure at Suspender Points		350.4		356.2		356.7		357.0		357.0		356.0	
Total Concentrations		528.6		531.7		534.5		536.1		537.4		537.7	

*Carried by Link

ITEM	39	40	41	42	43	44
Cable & Handropes	164.0		164.8		165.5	
Cable Bands	8.4		8.4		8.4	
½ Suspenders + Tie at Cable	10.7		11.3		12.0	
Concentrations at Cable	183.1		184.5		185.9	
½ Suspenders + Split Collars, Sockets and Shims	11.4		12.1		12.7	
Truss	30.5	28.9	30.7	29.9	32.1	20.9
Main Laterals, Center Struts & Portals	6.1	6.1	6.1	14.1	11.6	2.5
Floor Beams & Knee Braces	25.0	24.9	25.0	27.9	24.9	26.3
Roadway Stringers, Break in Floor & Scuppers	15.3	13.7	15.2	19.6	13.4	27.0
Sidewalk Stringers, Laterals, Curb & Break in Floor	7.6	7.8	7.8	8.0	3.2	5.5
Roadway Slab	72.6	72.6	72.6	72.6	72.6	36.3
Sidewalk Slab	13.9	13.9	13.9	13.9	13.9	6.5
Handrailing & Electroliers	2.0	2.0	2.0	3.7	2.0	1.0
Conduits, Pipe Lines, Diaphones, etc.	0.2	0.2	0.2	0.2	0.2	0.1
Ladders & Platforms						
Paint on Structural Steel	1.1	1.1	1.1	1.1	1.1	1.0
Total Suspended Structure at Panel Points	185.7	171.0	186.7	185.0	193.7	131.1
Total Suspended Structure at Suspender Points	357.3		356.7		332.4	84.9
Total Concentrations	540.4		549.2		518.3	*84.9

*Carried by Link

NOTE:-
Assumed weight of painted wrapped cable - 2800 lbs. per lin. ft.
Assumed weight of painted suspenders - 12.2 lbs. per lin. ft. per rope.

THE GOLDEN GATE BRIDGE
AT
SAN FRANCISCO, CAL.
FOR THE
GOLDEN GATE BRIDGE AND HIGHWAY DISTRICT
OF CALIFORNIA

D.L. CONCENTRATIONS – SIDE SPAN

DATE June 11, 1935 SHEET No. D52A

CORRECT: Chief Engineer.
Plans prepared by Strauss Engineering Corporation. Vice-President.
APPROVED: Consulting Engineer. Consulting Engineer. Consulting Engineer.

Golden Gate Bridge, Highway & Transportation District

Bridge Design and Construction Statistics

Length, Width, Height, Weight

Total length of Bridge including approaches from abutment to abutment: 1.7 miles = 8,981 ft = 2,737 m

Total length of Bridge including approaches from abutment to abutment, plus the distance to the Toll Plaza: 9,150 ft = 2,788 m

Length of suspension span including main span and side spans: 1.2 miles = 6,450 ft = 1,966 m

Length of main span portion of suspended structure (distance between towers): 4,200 ft = 1,280 m

Length of one side span: 1,125 ft = 343 m

Width of Bridge: 90 ft = 27 m

Width of roadway between curbs: 62 ft = 19 m

Width of sidewalk: 10 ft = 3 m

Clearance above mean higher high water: 220 ft = 67 m

Total weight of each anchorage: 60,000 tons = 54,400,000 kg

Original combined weight of Bridge, anchorages, and approaches: 894,500 tons = 811,500,000 kg

Total weight of Bridge, anchorages, and approaches (1937): 894,500 tons = 811,500,000 kg

Total weight of Bridge, anchorages, and approaches (1986)*: 887,000 tons = 804,700,00 kg*

Weight of Bridge, excluding anchorages and approaches, and including the suspended structure, main towers, piers and fenders, bottom lateral system and orthotropic redecking (1986): 419,800 tons = 380,800,000 kg*

> * The total bridge weight listed for 1986 includes the reduction in weight due to the redecking in 1986. The weight of the original reinforced concrete deck and its supporting stringers was 166,397 tons (150,952,000 kg). The weight of the new orthotropic steel plate deck, its two inches of epoxy asphalt surfacing, and its supporting pedestals is now 154,093 tons (139,790,700 kg). This is a total reduction in weight of the deck of 12,300 tons (11,158,400 kg), or 1.37 tons (1133 kg) per lineal foot of deck.

Bridge Deflection, Load Capacity

Watch this video to see how the Golden Gate Bridge can move up and down by as much as 16 feet depending on the temperature.

Maximum transverse deflection, at center span: 27.7 ft = 8.4 m

Maximum downward deflection, at center span: 10.8 ft = 3.3 m

Maximum upward deflection, at center span: 5.8 ft = 1.77 m

Live load capacity per lineal foot: 4,000 lbs. = 1,814.4 kg

As an example of how the Bridge is built to move, during the winter storms in 1982, the main span bowed approximately 6 to 7 feet.

The three maximum deflections noted above at the center of the suspension bridge are due to the following loading conditions:

1. The transverse deflection is due to a sustained transverse wind load. The maximum transverse movement of 27.7 ft is based on the maximum allowable longitudinal movement of the wind locks at the support towers;
2. The maximum downward deflection is due to a condition with maximum live load on the center span, no live load on the side spans and maximum design temperature to elongate the main cables; and
3. The maximum upward deflection is due to a condition opposite to condition 2 above, with maximum live load on side spans, no live load on center span and minimum design temperature to shorten the cable length.

Main Tower Stats

The Golden Gate Bridge has two main towers that support the two main cables.

Height of tower above water: 746 ft = 227 m

Height of tower above roadway: 500 ft = 152 m

Tower base dimension (each leg): 33 x 54 ft = 10 x 16 m

Load on each tower from main cables: 61,500 tons = 56,000,000 kg

Weight of both main towers: 44,000 tons = 40,200,000 kg

Transverse deflection of towers: 12.5 inches = 0.32 m

Longitudinal deflection of towers: shoreward: 22 in = 0.56 m and channelward: 18 in = 0.46 m

The south tower foundation depth below mean low water is: 110 ft = 34m

To build south tower pier to support the south tower, construction workers pumped 9.41 million gallons or 35.6 million liters of water out of the fender that was constructed first.

Main Cable Stats

The Golden Gate Bridge has two main cables which pass over the tops of the two main towers and are secured at either end in giant anchorages.

The main cables rest on top of the 746-foot main towers in huge steel castings called saddles.

Diameter of one main cable including the exterior wrapping: 36 3/8 in. = .92 m

Length of one main cable: 7,650 ft = 2,332 m

Total length of galvanized steel wire used in both main cables: 80,000 mi = 129,000 km

Number of galvanized steel wires in one main cable that are 0.192 inches in diameter: 27,572

Number of bundles or strands of galvanized steel wire in one main cable: 61

Weight of both main cables, suspender cables and accessories: 24,500 tons = 22,200,000 kg

The galvanized steel wire comprising each main cable was laid by spinning the wire using a loom-type shuttle that moved back and forth as it laid the wire in place to form the cables. The spinning of the main cable wires was completed in 6 months and 9 days.

The galvanized steel wire used for the main cables is carbon steel with the following average chemical composition and physical properties:

Ladle test results (specified)	
C:	0.81% (0.85)
Mn:	0.66% (---)
P:	0.026% (0.04)
S:	0.028% (0.04)
Si:	0.24% (---)
Tested properties (specified)	
Tensile Str,	Fu = 235,600 psi (220,000 psi min)
Yield Str,	Fy = 182,600 psi (160,000 psi min)
Elongation in 10" at rupture = 6.3% (4.0% min)	

Suspender Rope (vertical ones) Stats

The Golden Gate Bridge has 250 pairs of vertical suspender ropes that are spaced 50 feet apart across both sides of the Bridge. Each suspender rope is 2-11/16 inches in diameter. All of the ropes were replaced between 1972 and 1976, with the last rope replacement completed on May 4, 1976.

Concrete Quantities

	Cu. yd.	**Cu. m.**
San Francisco Pier and Fender	130,000	99,400
Marin Pier	23,500	18,000
Anchorages, Pylons, and Cable Housing	182,000	139,160
Approaches	28,500	21,800
Paving	25,000	19,115

Structural Steel Quantities

	Tons	**Kg.**
Main Towers	44,400	40,280,000
Suspended Structure	24,000	21,772,000

Anchorages	4,400	3,991,000
Approaches	10,200	9,250,000

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0117

Appendix B:

Excerpts from AASHTO Standard Specifications for Highway Bridges, 1977, Section 1.2, 1.7.51, and Appendix B.

1.1.16—ROADWAY WIDTH, CURBS AND CLEARANCES FOR DEPRESSED ROADWAYS

(A) Roadway Width

The clear width between curbs shall be not less than that specified for tunnels.

(B) Clearance Between Walls

The minimum width between walls for depressed roadways carrying two lanes of traffic shall be 30 feet (9.144 m).

(C) Curbs

The width of curbs shall be not less than 18 inches (.457 m). The height of curbs shall be as specified for bridges.

Section 2—LOADS

1.2.1—LOADS

Structures shall be proportioned for the following loads and forces when they exist:

Dead load.
Live load.
Impact or dynamic effect of the live load.
Wind loads.
Other forces, when they exist, as follows:
Longitudinal forces, centrifugal force, thermal forces, earth pressure, buoyancy, shrinkage stresses, rib shortening, erection stresses, ice and current pressure, and earthquake stresses.

Members shall be proportioned for the allowable stresses permitted by the design procedure and the limitations imposed by the appropriate material.

When stress sheets are required, a diagram or notation of the assumed loads shall be shown and the stresses due to the various loads shall be shown separately.

Where required by design conditions, concrete placing sequence shall be indicated on the plans or in the special provisions.

The loading combinations shall be in accordance with Article 1.2.22.

1.2.2—DEAD LOAD

The dead load shall consist of the weight of the structure complete, including the roadway, sidewalks, car tracks, pipes, conduits, cables and other public utility services.

The snow and ice load is considered to be offset by an accompanying decrease in live load and impact and shall not be included except under special conditions.

If a separate wearing surface is to be placed when the bridge is constructed, or if placement of a separate wearing surface is anticipated in the future by the department, adequate allowance shall be made for its weight in the design dead load. Otherwise provision for a future wearing surface is not required.

Special consideration shall be given to the necessity for a separate wearing surface for those regions where the use of chains on tires or studded snow tires, is anticipated.

Where the abrasion of concrete is not anticipated, the traffic may bear directly on the concrete slab. If considered desirable, 1/4 inch (6.4 mm) or more may be added to the slab for a wearing surface.

The following weights are to be used in computing the dead load:

	$\#/ft^3$	(kg/m^3)
Steel or cast steel	.490	(7849)
Cast iron	.450	(7208)
Aluminum alloys	.175	(2803)
Timber (treated or untreated)	50	(801)
Concrete, plain or reinforced	.150	(2403)
Compacted sand, earth, gravel or ballast	.120	(1922)
Loose sand, earth and gravel	.100	(1602)
Macadam or gravel, rolled	.140	(2243)
Cinder filling	60	(961)
Pavement, other than wood block	.150	(2403)
Railway rails, guard rails, and fastenings (per linear foot of track)	.200	(3204)
Stone masonry	.170	(2723)
Asphalt plank, 1 inch (25.4 mm) thick	9 lbs. per square foot ($44\ kg/m^2$)	

(A) Unit Load on Culverts

Earth pressures or loads on culverts may be computed ordinarily as the weight of earth directly above the structure. For box culverts, and culverts with cast-in-place inverts or footings, the weight of the earth may be taken at 70 percent of its actual weight. This will have the effect of increasing the allowable design dead load stresses 40 percent more than allowed for live load. For flexible and rigid pipes, not cast-in-place, the weight of the earth may be taken at 83 percent of its actual weight. This will have the effect of increasing the allowable design dead load stresses 20 percent more than allowed for live load.

For definite conditions of bedding and backfill, the principles of soil mechanics may be applied. The following are recommended formulas for these conditions:

(1) Culvert in trench on unyielding subgrade, or culvert untrenched on yielding foundation.

$$P=WH$$

(2) *Culvert untrenched on unyielding foundation (such as rock or piles).

$$P=W(1.92H-0.87B) \text{ for } H>1.7B \quad (1A)$$
$$P=2.59\,BW\,(e^{k}-1) \text{ for } H<1.7B \text{ where } k=\frac{0.385H}{B} \quad (1B)$$

where P=the unit pressure in pounds per square foot (Pa) due to earth backfill.
B =width in feet (m) of trench, or in case there is no trench, the overall width of the culvert.
H=depth in feet (m) of fill over culvert.
W =effective weight per cubic foot (N/m^3) of fill material, which may be taken as 70 percent, or 83 percent, of actual weight in accordance with above stated provisions.
e=2.7182818=base of natural logarithms, abstract number.

1.2.3.–LIVE LOAD

The live load shall consist of the weight of the applied moving load of vehicles, cars and pedestrians.

1.2.4–OVERLOAD PROVISION

The following provision for overload shall apply to all loadings except the H 20 (M 18) and HS 20 (MS 18) loadings.

Provision for infrequent heavy loads shall be made by applying in any single lane an H or HS (M or MS) truck as specified, increased 100 percent, and without concurrent loading of any other lanes. Combined dead, live and impact stresses resulting from such loading shall not be greater than 150 percent of the allowable stresses prescribed herein. The overload shall apply to all parts of the structure affected, except the roadway deck.**

1.2.5–HIGHWAY LOADINGS

(A) General

The highway live loadings on the roadways of bridges or incidental structures shall consist of standard trucks or of lane loads which are equivalent to truck trains. Two systems of loading are provided, the H (M) loadings and the HS (MS) loadings, the corresponding HS (MS) loadings being heavier than the H (M) loadings.

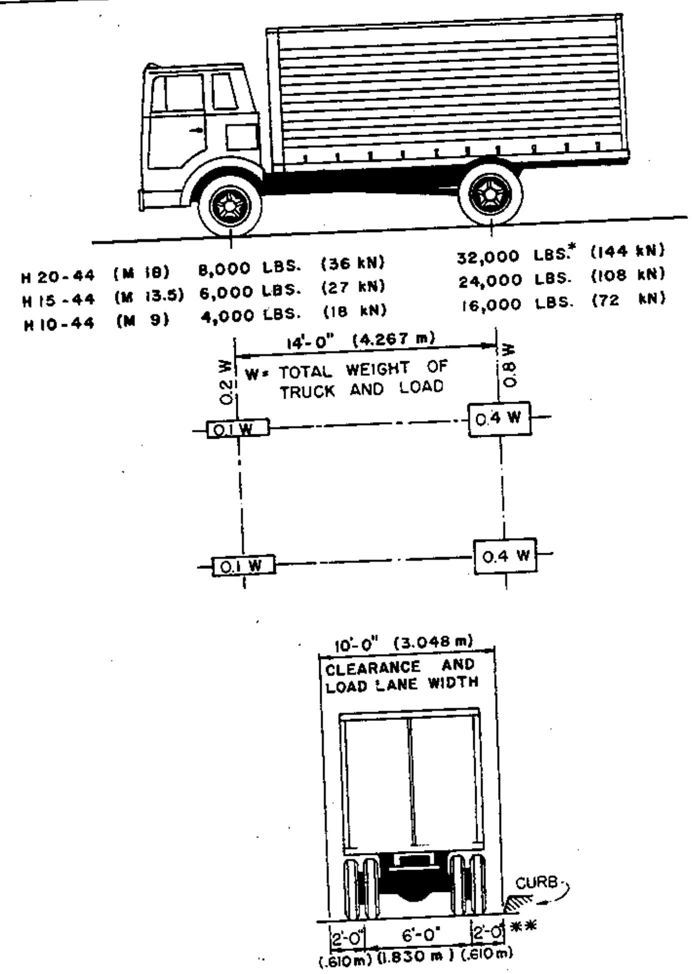
(B) H (M) Loadings

The H (M) loadings are illustrated in Figures 1.2.5A and 1.2.5B. They consist of a two-axle truck or the corresponding lane loading. The H (M)

*Note: Formulas 1A and 1B have been derived from Iowa Engineering Experiment Station Bulletin 96, "The Theory of External Loads on Closed Conduits in the Light of the Latest Experiments", by Anson Marston, Director, February 19, 1930.

**For orthotropic bridge superstructures, the roadway deck plate and stiffening ribs.

loadings are designated H (M) followed by a number indicating the gross weight in tons (metric tons) of the standard truck.



STANDARD H (M) TRUCKS

FIGURE 1.2.5A

*In the design of timberfloors and orthotropic steel decks (excluding transverse beams) for H20 (M 18) loading, one axle load of 24,000 pounds (108 kN) or two axle loads of 16,000 pounds (72 kN) each spaced 4 feet (1.219 m) apart may be used, whichever produces the greater stress, instead of the 32,000 pound (144 kN) axle shown.

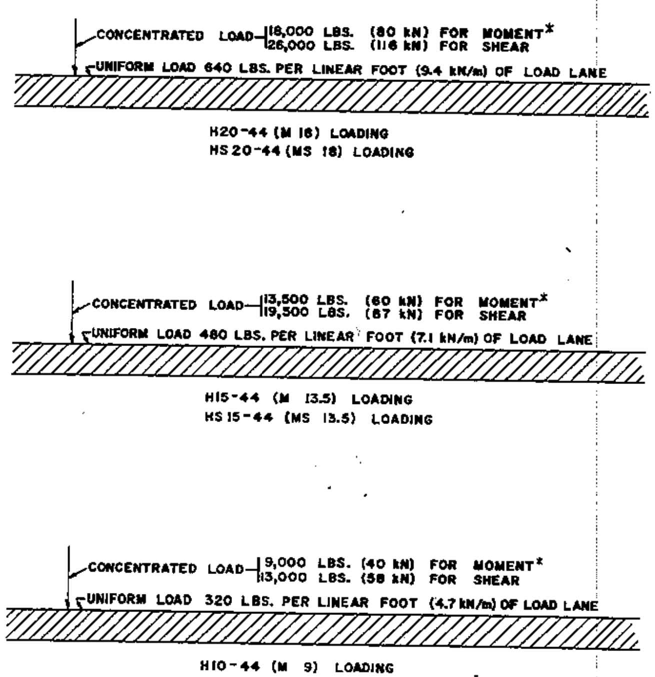
**For slab design, the center line of wheels shall be assumed to be 1 foot (.305m) from face of curb. (See Art. 1.3.2(B))

(C) HS (MS) Loadings

The HS (MS) loadings are illustrated in Figures 1.2.5B and 1.2.5C. They consist of a tractor truck with semi-trailer or of the corresponding lane loading. The HS (MS) loadings are designated by the letters HS (MS) followed by a number indicating the gross weight in tons (metric tons) of the tractor truck. The variable axle spacing has been introduced in order that the spacing of axles may approximate more closely the tractor trailers now in use. The variable spacing also provides a more satisfactory loading for continuous spans, in that heavy axle loads may be so placed on adjoining spans as to produce maximum negative moments.

(D) Classes of Loadings

Highway loadings shall be of five classes: H 20 (M 18), H 15 (M13.5), H 10 (M 9), HS 20 (MS 18), and HS 15 (MS 13.5). Loadings H 15 (M 13.5) and H 10 (M 9) are 75 percent and 50 percent, respectively, of loading H 20 (M 18). Loading HS 15 (MS 13.5) is 75 percent of loading HS 20 (MS (18). If

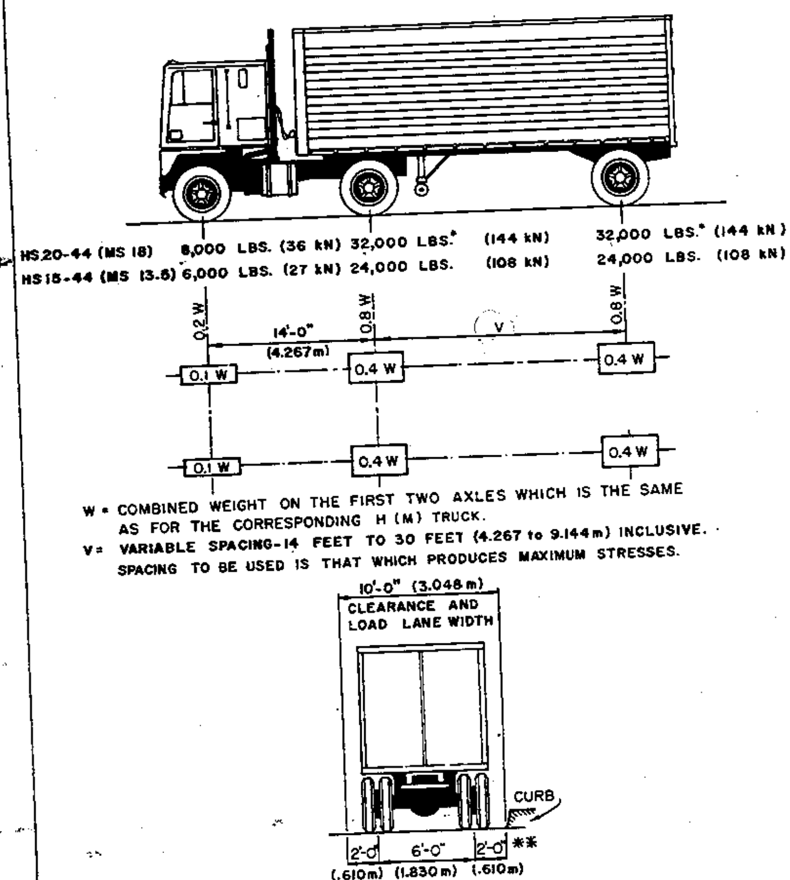


H (M) LANE AND HS (MS) LANE LOADINGS

FIGURE 1.2.5B

*For the loading of continuous spans involving lane loading refer to Article 1.2.8(C) which provides for an additional concentrated load.

loadings of weights other than those designated are desired, they shall be obtained by proportionately changing the weights shown for both the standard truck and the corresponding lane loads.



STANDARD HS (MS) TRUCKS

FIGURE 1.2.5C

*In the design of timber floors and orthotropic steel decks (excluding transverse beams) for H20 (M 18) loading, one axle load of 24,000 pounds (108 kN) or two axle loads of 16,000 pounds (72 kN) each, spaced 4 feet (1.219 m) apart may be used, whichever produces the greater stress, instead of the 32,000 pound (144 kN) axle shown.

**For slab design, the center line of wheels shall be assumed to be 1 foot (.305 m) from face of curb. (See Art. 1.3.2 (B))

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0117

(E) Designation of Loadings

The policy of affixing the year to loadings to identify them was instituted with the publication of the 1944 edition in the following manner:

H10 Loading, 1944 Edition shall be designated .... H10-44 (M 9)
H15 Loading, 1944 Edition shall be designated .... H15-44 (M 13.5)
H20 Loading, 1944 Edition shall be designated .... H20-44 (M 18)
H15-S12 Loading, 1944 Edition shall be designated .... HS15-44 (MS 13.5)
H20-S16 Loading, 1944 Edition shall be designated ..... HS20-44 (MS 18)

The affix remains unchanged until such time as the loading specification is revised. The same policy for identification shall be applied, for future reference, to loadings previously adopted by the American Association of State Highway and Transportation Officials.

(F) Minimum Loading

For trunk highways, or for other highways which carry, or which may carry, heavy truck traffic, the minimum live load shall be the HS 15 (MS 13.5) designated herein.

(G) Interstate Highway Bridge Loadings

Bridges supporting Interstate highways shall be designed for HS 20-44 (MS 18). Loading or an Alternate Military Loading of two axles four feet (1.219m) apart with each axle weighing 24,000 pounds (108kN), whichever produces the greatest stress.

1.2.6—TRAFFIC LANES

The lane loading or standard truck shall be assumed to occupy a width of ten feet (3.048m).

These loads shall be placed in 12-foot (3.658m) wide design traffic lanes spaced across the entire bridge roadway width, in numbers and positions required to produce the maximum stress in the member under consideration. Roadway width shall be the distance between curbs. Fractional parts of design lanes shall not be used. Roadway widths from 20 to 24 feet (6.096 to 7.315m) shall have two design lanes each equal to one-half the roadway width.

The lane loadings or standard trucks having a 10-foot (3.048m) width shall be assumed to occupy any position within their individual design traffic lane, which will produce the maximum stress.

1.2.7—STANDARD TRUCKS AND LANE LOADS

The wheel spacing, weight distribution, and clearance of the standard H and HS (M or MS) trucks shall be shown in Figures 1.2.5A and 1.2.5C and corresponding lane loads shall be as shown in Figure 1.2.5B.

Each lane loading shall consist of a uniform load per linear foot (m) of traffic lane combined with a single concentrated load (or two concentrated loads in the case of continuous spans—see Article 1.2.8(C), so placed on the span as to produce maximum stress. The concentrated load and uniform load shall be considered as uniformly distributed over a 10-foot (3.048m) width on a line normal to the center line of the lane.

For the computation of moments and shears, different concentrated loads shall be used as indicated in Figure 1.2.5B. The lighter concentrated loads shall be used when the stresses are primarily bending stresses and the heavier concentrated loads shall be used when the stresses are primarily shearing stresses.

Note: The system of lane loads here defined (and illustrated in Fig. 1.2.5B) was developed in order to give a simpler method of calculating moments and shears than that based on wheel loads of the truck.

Appendix B shows the truck train loadings of the 1935 Specifications of AASHO and the corresponding lane loadings.

In 1944 the HS series of trucks were developed. These approximate the effect of the corresponding 1935 truck preceded and followed by a train of trucks weighing ¾ as much as the basic truck.

1.2.8—APPLICATION OF LOADINGS

(A) Traffic Lane Units

In computing stresses, each 10-foot (3.048 m) lane loading or single standard truck shall be considered as a unit, and fractional load lane widths or fractional trucks shall not be used.

(B) Number and Position, Traffic Lane Units

The number and position of the lane loadings or truck loadings shall be as specified in Article 1.2.6 and, whether lane loading or truck loading, shall be such as to produce maximum stress, subject to the reduction specified in Article 1.2.9.

(C) Lane Loadings—Continuous Spans

The lane loadings shown in Figure 1.2.5B shall be modified as follows for the design of continuous spans. The lane loadings shall consist of the loads shown in Figure 1.2.5B and in addition thereto another concentrated load of equal weight shall be placed in one other span in the series in such position as to produce maximum negative moment. For maximum positive moment, only one concentrated load shall be used per lane, combined with as many spans loaded uniformly as required to produce maximum moment.

(D) Loading for Maximum Stress

The type of loading, whether lane loading or truck loading, to be used, and whether the spans be simple or continuous, shall be the loading which produces the maximum stress. The moment and shear tables given in Appendix A show which loading controls for simple spans. The axle spacing for HS (MS) trucks shall be varied between the specified limits to produce maximum stresses.

For continuous spans, the lane loading shall be continuous or discontinuous, as may be necessary to produce maximum stresses, and the concentrated load or loads as specified in paragraph (C) shall be placed in such position as to produce maximum stresses.

For continuous spans, only one Standard H (M) or HS (MS) truck per lane shall be considered on the structure and placed so as to produce maximum positive and negative moments.

2.1.19.4.3
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1.2.9—REDUCTION IN LOAD INTENSITY

Where maximum stresses are produced in any member by loading any number of traffic lanes simultaneously, the following percentages of the resultant live load stresses shall be used in view of improbable coincident maximum loading:

	Percent
One or two lanes	100
Three lanes	90
Four lanes or more	75

The reduction in intensity of floor beam loads shall be determined as in the case of main trusses or girders, using the width of roadway which must be loaded to produce maximum stresses in the floor beam.

1.2.10—ELECTRIC RAILWAY LOADING

If highway bridges carry electric railway traffic, the railway loading shall be determined on the basis of the class of traffic which the bridge may be expected to carry. The possibility that the bridge may be required to carry railroad freight cars shall be given consideration.

1.2.11—SIDEWALK, CURB, AND RAILING LOADING

(A) Sidewalk Loading

Sidewalk floors, stringers and their immediate supports, shall be designed for a live load of 85 pounds per square foot (4070 Pa) of sidewalk area. Girders, trusses, arches and other members shall be designed for the following sidewalk live loads

Spans 0 to 25 ft. (0 to 7.620 m) in length .... 85 lbs./ft^2 (4070 Pa)
Spans 26 to 100 ft. (7.925 to 30.480 m) in length ... 60 lbs./ft^2 (2873 Pa)
Spans over 100 ft. (30.480 m) in length according to the formula

$$P = \left(30 + \frac{3000}{L}\right)\left(\frac{55-W}{50}\right) \quad \text{or} \quad \left[\left(1435 + \frac{43800}{L}\right)\left(\frac{16.7-W}{15.2}\right)\right]$$

in which

P = live load per square foot (Pa). Max 60 lbs. per sq. ft. (2873 Pa)
L = loaded length of sidewalk in feet. (m)
W = width of sidewalk in feet. (m)

In calculating stresses in structures which support cantilevered sidewalks, the sidewalk shall be considered as fully loaded on only one side of the structure if this condition produces maximum stress.

Pedestrian bridges shall be designed for a live load of 85 pounds per square foot (4070 Pa) of walkway area.

(B) Curb Loading

Curbs shall be designed to resist a lateral force of not less than 500 pounds per linear foot (744 kg/m) of curb, applied at the top of the curb, or at an

elevation 10 inches (.254 m) above the floor if the curb is higher than 10 inches (.254 m).

Where sidewalk, curb and traffic rail form an integral system, the traffic railing loading shall apply and stresses in curbs computed accordingly.

(C) Railing Loading

For Railing Loads, see Article 1.1.8.

1.2.12—IMPACT

Live load stresses produced by H or HS (M or MS) loadings shall be increased for items in Group A by allowance as stated herein for dynamic, vibratory and impact effects. Impact shall not be applied to items in Group B.

(A) Group A

(1) Superstructure, including steel or concrete supporting columns, steel towers, legs of rigid frames and generally those portions of the structure which extend down to the main foundation.

(2) The portion above the ground line of concrete or steel piles which are rigidly connected to the superstructure as in rigid frame or continuous designs.

(B) Group B

(1) Abutments, retaining walls, piers, piles, except Group A(2).
(2) Foundation pressures and footings.
(3) Timber structures.
(4) Sidewalk loads.
(5) Culverts and structures having cover of 3 feet (.914 m) or more.

(C) Impact Formula

The amount of this allowance or increment is expressed as a fraction of live load stress, and shall be determined by the formula:

$$I = \frac{50}{L + 125} \quad \text{or} \quad \left(\frac{15.24}{L + 38}\right)$$

in which
I=impact fraction (maximum 30 percent).
L=length in feet (m) of the portion of the span which is loaded to produce the maximum stress in the member.

For uniformity of application the loaded length "L" shall be especially considered as follows:

For roadway floors, use the design span length.

For transverse members, such as floor beams, use the span length of member center to center of supports.

For computing truck load moments use the span length, except for cantilever arms use the length from moment center to the farthermost axle.

For shear due to truck loads use the length of the loaded portion of span from the point under consideration to the far reaction, except for cantilever arms use a 30 percent impact factor.

For continuous spans use the length of span under consideration for positive moment, and use the average of two adjacent loaded spans for negative moment.

For culverts with cover 0′ to 1′- 0″ inc. (0 to .305 m) I=30%
1′-1″ to 2′- 0″ inc. (.330 to .610 m) I=20%
2′-1″ to 2′-11″ inc. (.635 to .889 m) I=10%

1.2.13—LONGITUDINAL FORCES

Provision shall be made for the effect of a longitudinal force of five percent of the live load in all lanes carrying traffic headed in the same direction. All lanes shall be considered as loaded for bridges likely to become one directional in the future. The load used, without impact, shall be the lane load plus the concentrated load for moment specified in Article 1.2.8, with reduction for multiple-loaded lanes as specified in Article 1.2.9. The center of gravity of the longitudinal force shall be assumed to be located 6 feet (1.829 m) above the floor slab and transmitted to the substructure through the superstructure.

The longitudinal force due to friction at expansion bearings or shear resistance at elastomeric bearings shall also be provided for in the design.

1.2.14—WIND LOADS

The following wind load forces per square foot (m^2) of exposed area shall be applied to all structures (see Article 1.2.22 for percentage of basic unit stress to be used under various combinations of loads and forces). The exposed area considered shall be the sum of the areas of all members, including floor system and railing, as seen in elevation at 90 degrees to the longitudinal axis of the structure. The forces and loads given herein are for a wind velocity of 100 miles per hour (160.9 km/hr). For Group II loading, but not for Group III loading, they may be reduced or increased in the ratio of the square of the design wind velocity to the square of the base wind velocity provided the maximum probable wind velocity can be ascertained with reasonable accuracy, or there are permanent features of the terrain which make such changes safe and advisable. If change in the design wind velocity is made, the design wind velocity shall be shown on the plans.

(A) Superstructure Design

A moving uniformly distributed wind load of the following intensity shall be applied horizontally at right angles to the longitudinal axis of the structure in the design of the superstructure:

For trusses and arches . . . . 75 pounds per square foot (3591 Pa)
For girders and beams . . . . 50 pounds per square foot (2394 Pa)

The total force shall not be less than 300 pounds per linear foot (4380N/m) in the plane of the loaded chord and 150 pounds per linear foot (2190N/m) in the plane of the unloaded chord on truss spans, and not less than 300 pounds per linear foot (4380N/m) on girder spans.

The above forces shall be used for Group II loading. For Group III loading there shall be added thereto a load of 100 pounds per linear foot (1460N/m) applied at right angles to the longitudinal axis of the structure and 6 feet (1.829 m) above the deck as a wind load on a moving live load. When a reinforced concrete floor slab or a steel grid deck is keyed to or attached to its supporting members, it may be assumed that the deck resists, within its plane, the shear resulting from the wind load on the moving live load.

(B) Substructure Design

Forces transmitted to the substructure by the superstructure and forces applied directly to the substructure by wind loads shall be assumed to be as follows:

(1) Forces from Superstructure

The transverse and longitudinal forces transmitted by the superstructure to the substructure for varying angles of wind direction shall be as set forth in the following table. The skew angle is measured from the perpendicular to the longitudinal axis. The assumed wind direction shall be that which produces the maximum stress in the substructure being designed. The transverse and longitudinal forces shall be applied simultaneously at the elevation of the center of gravity of the exposed area of the superstructure.

	Trusses		Girders	
Skew Angle of Wind	Lateral Load	Longitudinal Load	Lateral Load	Longitudinal Load
(Degrees)	PSF (Pa)	PSF (Pa)	PSF (Pa)	PSF (Pa)
0	75(3591)	0(0)	50(2394)	0(0)
15	70(3352)	12(575)	44(2107)	6(287)
30	65(3112)	28(1341)	41(1963)	12(575)
45	47(2250)	41(1963)	33(1580)	16(766)
60	24(1197)	50(2394)	17(814)	19(910)

The loads listed above shall be used in Group II loading as given in Article 1.2.22.

For Group III loading, these loads may be reduced 70 percent and there shall be added thereto, as a wind load on a moving live load, a load per linear foot (m) as given in the following table:

Skew Angle of Wind (Degrees)	Lateral Load lb/ft (N/M)	Longitudinal Load lb/ft (N/M)
0	100(1460)	0
15	88(1285)	12(175)
30	82(1197)	24(350)
45	66(964)	32(467)
60	34(496)	38(555)

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(3) Fatigue

Hybrid girders shall be designed for the allowable fatigue stress range given in Table 1.7.2A1 of Article 1.7.2.

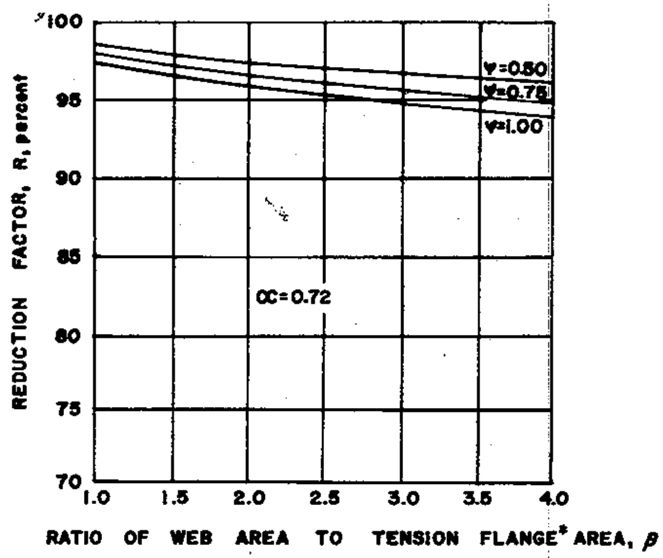


FIGURE 1.7.50B1a

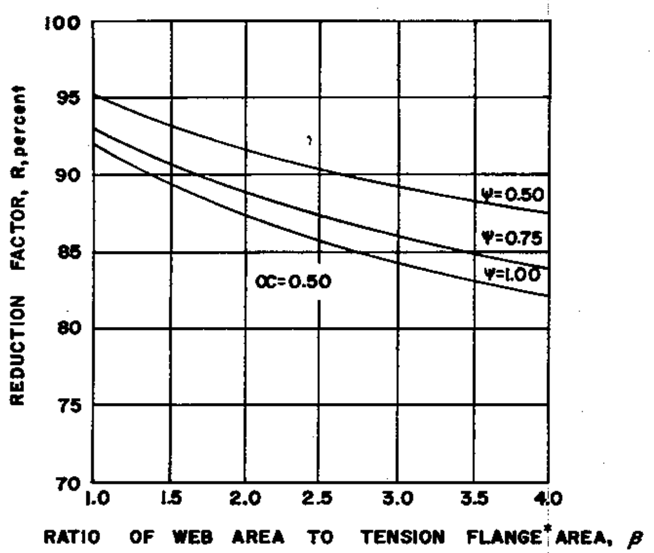


FIGURE 1.7.50B1b

*Bottom flange of orthotropic deck bridges.

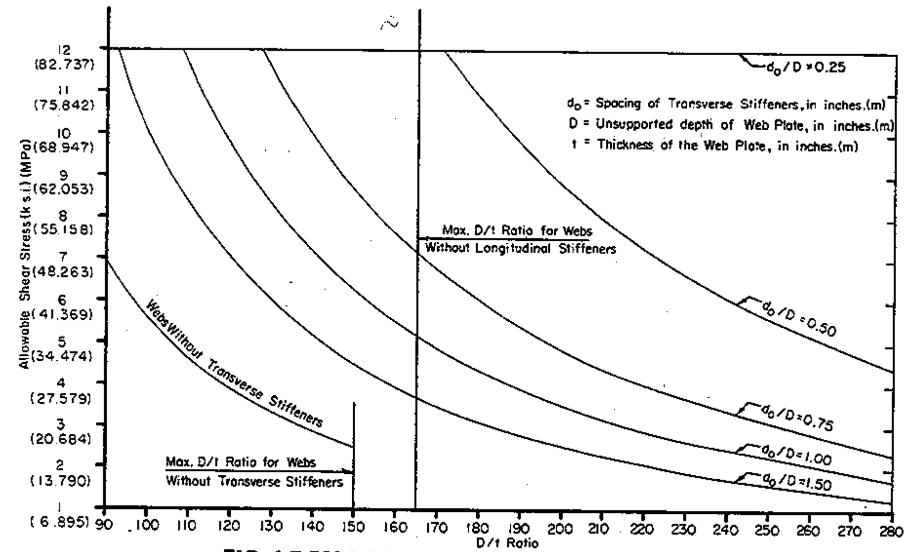


FIG. 1.7.50B2 Allowable Shear Stress VS. D/t Ratio
(M183 Steel, Fy = 36 K.S.I.)(248.211 MPa)

(C) Plate Thickness Requirements

In calculating the maximum width-to-thickness ratio of the flange plate according to Article 1.7.43(B) and the minimum thickness of the web plate according to Article 1.7.43(C), f_b shall be taken as the calculated bending stress in the compression flange divided by the reduction factor, R.

(D) Bearing Stiffener Requirements

In designing bearing stiffeners at interior supports of continuous hybrid girders for which α is less than 0.7, no part of the web shall be assumed to act in bearing.

1.7.51—ORTHOTROPIC-DECK SUPERSTRUCTURES

(A) General

This section pertains to the design of steel bridges that utilize a stiffened steel plate as a deck. Usually the deck plate is stiffened by longitudinal ribs and transverse beams; effective widths of deck plate act as the top flanges of these ribs and beams. Usually the deck, including longitudinal ribs, acts as the top flange of the main box or plate girders. As used in Articles 1.7.51(A) through 1.7.51(J), the terms, rib and beam, refer to sections that include an effective width of deck plate.

The provisions of Division I, Design, shall govern where applicable, except as specifically modified by Articles 1.7.51(A) through 1.7.51(J).

An appropriate method of elastic analysis, such as the equivalent-orthotropic-slab method or the equivalent-grid method, shall be used in designing the deck. The equivalent stiffness properties shall be selected to correctly simulate the actual deck. An appropriate method of elastic analysis, such as the thin-walled-beam method, that accounts for the effects of torsional distortions of the cross-sectional shape shall be used in designing the girders of orthotropic-deck box-girder bridges. The box-girder design shall be checked for lane or truck loading arrangements that produce maximum distortional (torsional) effects.

For an alternate design method (Load Factor Design) see Article 1.7.75.

(B) Wheel Load Contact Area

The wheel loads specified in Article 1.2.5 shall be uniformly distributed to the deck plate over the rectangular area defined below:

Wheel Load, kip (kN)	Width Perpendicular to Traffic, inch (m)	Length in Direction of Traffic, inch (m)
8 (36)	20+2t (.508+2t)	8+2t(.203+2t)
12 (54)	20+2t (.508+2t)	8+2t(.203+2t)
16 (72)	24+2t (.610+2t)	8+2t(.203+2t)

In the above table, t is the thickness of the wearing surface in inches (m).

(C) Effective Width of Deck Plate

(1) Ribs and Beams

The effective width of deck plate acting as the top flange of a longitudinal rib or a transverse beam may be calculated by accepted approximate methods.*

(2) Girders

The full width of deck plate may be considered effective in acting as the top flange of the girders if the effective span of the girders is not less than: (1) 5 times the maximum distance between girder webs and (2) 10 times the maximum distance from edge of the deck to the nearest girder web. The effective span shall be taken as the actual span for simple spans and the distance between points of contraflexure for continuous spans. Alternatively, the effective width may be determined by accepted analytical methods.

The effective width of the bottom flange of a box girder shall be determined according to the provisions of Article 1.7.49(D)(1).

*Design Manual for "Orthotropic Steel Plate Deck Bridges," AISC, 1963 or "Orthotropic Bridges, Theory and Design," by M.S. Troitsky, Lincoln Arc Welding Foundation, 1967.

(D) Allowable Stresses

(1) Local Bending Stresses in Deck Plate

The term local bending stresses refers to the stresses caused in the deck plate as it carries a wheel load to the ribs and beams. The local transverse bending stresses caused in the deck plate by the specified wheel load plus 30 percent impact shall not exceed 30,000 psi (206.842 MPa) unless a higher allowable stress is justified by a detailed fatigue analysis or by applicable fatigue-test results. For deck configurations in which the spacing of transverse beams is at least 3 times the spacing of longitudinal-rib webs, the local longitudinal and transverse bending stresses in the deck plate need not be combined with the other bending stresses covered in paragraphs (2) and (3) below.

(2) Bending Stresses in Longitudinal Ribs

The total bending stresses in longitudinal ribs due to a combination of (1) bending of the rib and, (2) bending of the girders may exceed the allowable bending stresses in Articles 1.7.1 and 1.7.2 by 25 percent. The bending stress due to each of the two individual modes shall not exceed the allowable bending stresses in Articles 1.7.1 and 1.7.2.

(3) Bending Stresses in Transverse Beams

The bending stresses in transverse beams shall not exceed the allowable bending stresses in Articles 1.7.1 and 1.7.2.

(4) Intersections of Ribs, Beams, and Girders

Connections between ribs and the webs of beams, holes in the webs of beams to permit passage of ribs, connections of beams to the webs of girders, and rib splices may affect the fatigue life of the bridge when they occur in regions of tensile stress. Where applicable, the number of cycles of maximum stress and the allowable fatigue stresses given in Article 1.7.2 shall be applied in designing these details; elsewhere, a rational fatigue analysis shall be made in designing the details. Connections between webs of longitudinal ribs and the deck plate shall be designed to sustain the transverse bending fatigue stresses caused in the webs by wheel loads.

(E) Thickness of Plate Elements

(1) Longitudinal Ribs and Deck Plate

Plate elements comprising longitudinal ribs, and deck-plate elements between webs of these ribs, shall meet the minimum thickness requirements of Article 1.7.44(N). f_a may be taken as 75 percent of the sum of the compressive stresses due to (1) bending of the rib and, (2) bending of the girder, but not less than the compressive stress due to either of these two individual bending modes.

(2) Girders and Transverse Beams

Plate elements of box girders, plate girders, and transverse beams shall meet the requirements of Articles 1.7.43(B) through 1.7.43(F) and 1.7.49(D).

(F) Maximum Slenderness of Longitudinal Ribs

The slenderness, L/r, of a longitudinal rib shall not exceed the value given by the following formula unless it can be shown by a detailed analysis that overall buckling of the deck will not occur as a result of compressive stress induced by bending of the girders:

$$\left(\frac{L}{r}\right)_{max} = 1000\sqrt{\frac{1500}{F_y} - \frac{2700F}{F_y^2}} \quad \text{or} \quad \left(1000\sqrt{\frac{10.34}{F_y} - \frac{18.62F}{F_y^2}}\right)$$

where

- L = distance between transverse beams
- r = radius of gyration about the horizontal centroidal axis of the rib including an effective width of deck plate
- F = maximum compressive stress in psi (MPa) in the deck plate as a result of the deck acting as the top flange of the girders; this stress shall be taken as positive
- F_y = yield strength of rib material in psi (MPa).

(G) Diaphragms

Diaphragms, cross frames, or other means shall be provided at each support to transmit lateral forces to the bearings and to resist transverse rotation, displacement, and distortion. Intermediate diaphragms or cross frames shall be provided at locations consistent with the analysis of the girders. The stiffness and strength of the intermediate and support diaphragms or cross frames shall be consistent with the analysis of the girders.

(H) Stiffness Requirements

(1) Deflections

The deflections of ribs, beams, and girders due to live load plus impact may exceed the limitations in Article 1.7.6, but preferably shall not exceed 1/500 of their span. The calculation of the deflections shall be consistent with the analysis used to calculate the stresses.

To prevent excessive deterioration of the wearing surface, the deflection of the deck plate due to the specified wheel load plus 30 percent impact preferably shall be less than 1/300 of the distance between webs of ribs. The stiffening effect of the wearing surface shall not be included in calculating the deflection of the deck plate.

2.1.19.4.3
0117

(2) Vibrations

The vibrational characteristics of the bridge shall be considered in arriving at a proper design.

(I) Wearing Surface

A suitable wearing surface shall be adequately bonded to the top of the deck plate to provide a smooth, nonskid riding surface and to protect the top of the plate against corrosion and abrasion. The wearing surface material shall provide (1) sufficient ductility to accommodate, without cracking or debonding, expansion and contraction imposed by the deck plate, (2) sufficient fatigue strength to withstand flexural cracking due to deck-plate deflections, (3) sufficient durability to resist rutting, shoving, and wearing, (4) imperviousness to water and motor-vehicle fuels and oils, and (5) resistance to deterioration from deicing salts, oils, gasolines, diesel fuels, and kerosenes.

(J) Closed Ribs

Closed ribs without access holes for inspection, cleaning, and painting are permitted. Such ribs shall be sealed against the entrance of moisture by continuously welding (1) the rib webs to the deck plate, (2) splices in the ribs, and (3) diaphragms, or transverse beam webs, to the ends of the ribs.

STRENGTH DESIGN METHOD
LOAD FACTOR DESIGN

1.7.52—SCOPE

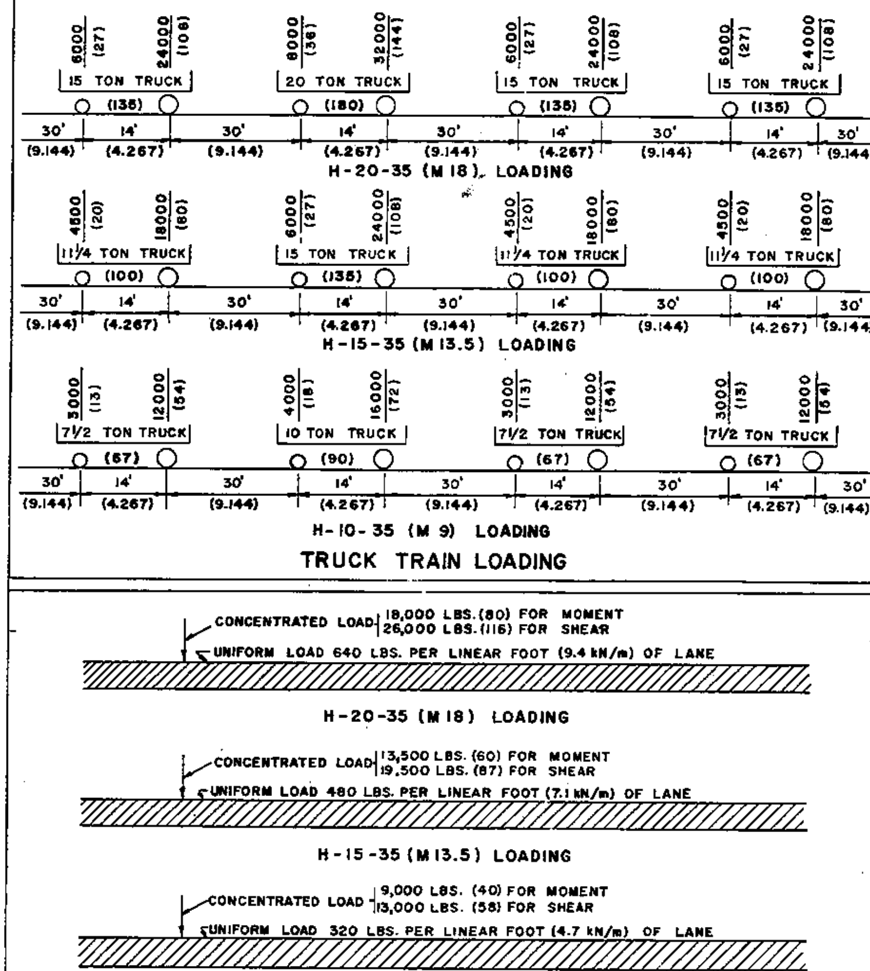
Load Factor design is an alternate method for design of simple and continuous beam and girder structures of moderate length. It is a method of proportioning structural members for multiples of the design loads. To insure serviceability and durability, consideration is given to the control of permanent deformations under overloads, to the fatigue characteristics under service loadings and to the control of live load deflections under service loadings.

1.7.53—NOTATIONS

- A = area of cross section (in.^{2})(m^2)
- A_f = area of one flange of beam or girder (in.^{2})(m^2)
- A_s = total area of steel section including cover plates (in.^{2})(m^2)
- A_s = gross effective area of column cross section (in.^{2})(m^2)
- A_w = area of web of beam (in.^{2})(m^2)
- b′ = width of projecting flange element (in.)(m)
- b′ = width of outstanding stiffener element (in.)(m)
- D = dead load
- D = distance center to center of box girder flange plates (in.)(m)
- d = depth of member (in.)(m)
- d_b = depth of beam
- d_c = depth of column

APPENDIX B

TRUCK TRAIN AND EQUIVALENT LOADINGS-1935 SPECIFICATIONS
AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS



EQUIVALENT LOADING
LANE WIDTH 10 FEET (3.048)
METRIC EQUIVALENT: LOADS IN kN, DISTANCES IN m

Appendix C:

Excerpts from AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007, Section 2 & 3.

- Computed girder rotations at bearings should be accumulated over the Engineer's assumed construction sequence. Computed rotations at bearings shall not exceed the specified rotational capacity of the bearings for the accumulated factored loads corresponding to the stage investigated.

- Camber diagrams shall satisfy the provisions of Article 6.7.2 and may reflect the computed accumulated deflections due to the Engineer's assumed construction sequence.

2.5.2.6.2 Criteria for Deflection

The criteria in this section shall be considered optional, except for the following:

- The provisions for orthotropic decks shall be considered mandatory.

- The provisions in Article 12.14.5.9 for precast reinforced concrete three-sided structures shall be considered mandatory.

- Metal grid decks and other lightweight metal and concrete bridge decks shall be subject to the serviceability provisions of Article 9.5.2.

In applying these criteria, the vehicular load shall include the dynamic load allowance.

If an Owner chooses to invoke deflection control, the following principles may be applied:

- When investigating the maximum absolute deflection for straight girder systems, all design lanes should be loaded, and all supporting components should be assumed to deflect equally;

- For curved steel box and I-girder systems, the deflection of each girder should be determined individually based on its response as part of a system;

- For composite design, the stiffness of the design cross-section used for the determination of deflection should include the entire width of the roadway and the structurally continuous portions of the railings, sidewalks, and median barriers;

C2.5.2.6.2

These provisions permit, but do not encourage, the use of past practice for deflection control. Designers were permitted to exceed these limits at their discretion in the past. Calculated deflections of structures have often been found to be difficult to verify in the field due to numerous sources of stiffness not accounted for in calculations. Despite this, many owners and designers have found comfort in the past requirements to limit the overall stiffness of bridges. The desire for continued availability of some guidance in this area, often stated during the development of these Specifications, has resulted in the retention of optional criteria, except for orthotropic decks, for which the criteria are required. Deflection criteria are also mandatory for lightweight decks comprised of metal and concrete, such as filled and partially filled grid decks, and unfilled grid decks composite with reinforced concrete slabs, as provided in Article 9.5.2.

Additional guidance regarding deflection of steel bridges can be found in Wright and Walker (*1971*).

Additional considerations and recommendations for deflection in timber bridge components are discussed in more detail in Chapters 7, 8, and 9 in Ritter (*1990*).

For a straight multibeam bridge, this is equivalent to saying that the distribution factor for deflection is equal to the number of lanes divided by the number of beams.

For curved steel girder systems, the deflection limit is applied to each individual girder because the curvature causes each girder to deflect differently than the adjacent girder so that an average deflection has little meaning. For curved steel girder systems, the span used to compute the deflection limit should be taken as the arc girder length between bearings.

- For straight girder systems, the composite bending stiffness of an individual girder may be taken as the stiffness determined as specified above, divided by the number of girders;

- When investigating maximum relative displacements, the number and position of loaded lanes should be selected to provide the worst differential effect;

- The live load portion of Load Combination Service I of Table 3.4.1-1 should be used, including the dynamic load allowance, IM;

- The live load shall be taken from Article 3.6.1.3.2;

- The provisions of Article 3.6.1.1.2 should apply; and

- For skewed bridges, a right cross-section may be used, and for curved and curved skewed bridges, a radial cross-section may be used.

In the absence of other criteria, the following deflection limits may be considered for steel, aluminum, and/or concrete construction:

- Vehicular load, general Span/800,

- Vehicular and/or pedestrian loads Span/1000,

- Vehicular load on cantilever arms...................... .... Span/300, and

- Vehicular and/or pedestrian loads on cantilever arms.. Span/375.

For steel I-shaped beams and girders, and for steel box and tub girders, the provisions of Articles 6.10.4.2 and 6.11.4, respectively, regarding the control of permanent deflections through flange stress controls, shall apply.

In the absence of other criteria, the following deflection limits may be considered for wood construction:

- Vehicular and pedestrian loads ... Span/425, and

- Vehicular load on wood planks and panels (extreme relative deflection between adjacent edges) .... 0.10 in.

The following provisions shall apply to orthotropic plate decks:

- Vehicular load on deck plate.............. Span/300,

- Vehicular load on ribs of orthotropic metal decks .... Span/1000, and

From a structural viewpoint, large deflections in wood components cause fasteners to loosen and brittle materials, such as asphalt pavement, to crack and break. In addition, members that sag below a level plane present a poor appearance and can give the public a perception of structural inadequacy. Deflections from moving vehicle loads also produce vertical movement and vibrations that annoy motorists and alarm pedestrians (*Ritter 1990*).

Excessive deformation can cause premature deterioration of the wearing surface and affect the performance of fasteners, but limits on the latter have not yet been established.

The intent of the relative deflection criterion is to protect the wearing surface from debonding and fracturing due to excessive flexing of the deck.

- Vehicular load on ribs of orthotropic metal decks (extreme relative deflection between adjacent ribs) .... 0.10 in.

The 0.10 in. relative deflection limitation is tentative.

2.5.2.6.3 Optional Criteria for Span-to-Depth Ratios

C2.5.2.6.3

Unless otherwise specified herein, if an Owner chooses to invoke controls on span-to-depth ratios, the limits in Table 1, in which *S* is the slab span length and *L* is the span length, both in ft., may be considered in the absence of other criteria. Where used, the limits in Table 1 shall be taken to apply to overall depth unless noted.

For curved steel girder systems, the span-to-depth ratio, L_{as}/D, of each steel girder should not exceed 25 when the specified minimum yield strength of the girder in regions of positive flexure is 50.0 ksi or less, and:

- When the specified minimum yield strength of the girder is 70.0 ksi or less in regions of negative flexure, or

- When hybrid sections satisfying the provisions of Article 6.10.1.3 are used in regions of negative flexure.

For all other curved steel girder systems, L_{as}/D of each steel girder should not exceed the following:

$$\frac{L_{as}}{D} \leq 25\sqrt{\frac{50}{F_{yc}}} \qquad (2.5.2.6.3\text{-}1)$$

where:

F_{yc} = specified minimum yield strength of the compression flange (ksi)

D = depth of steel girder (ft.)

L_{as} = an arc girder length defined as follows (ft.):

- arc span for simple spans;

- 0.9 times the arc span for continuous end-spans;

- 0.8 times the arc span for continuous interior spans.

Traditional minimum depths for constant depth superstructures, contained in previous editions of the AASHTO *Standard Specifications for Highway Bridges*, are given in Table 1 with some modifications.

A larger preferred minimum girder depth is specified for curved steel girders to reflect the fact that the outermost curved girder receives a disproportionate share of the load and needs to be stiffer. In curved skewed bridges, cross-frame forces are directly related to the relative girder deflections. Increasing the depth and stiffness of all the girders in a curved skewed bridge leads to smaller relative differences in the deflections and smaller cross-frame forces. Deeper girders also result in reduced out-of-plane rotations, which may make the bridge easier to erect.

An increase in the preferred minimum girder depth for curved steel girders not satisfying the conditions specified herein is recommended according to Eq. 1. In such cases, the girders will tend to be significantly more flexible and less steel causes increased deflections without an increase in the girder depth.

A shallower curved girder might be used if the Engineer evaluates effects such as cross-frame forces and bridge deformations, including girder rotations, and finds the bridge forces and geometric changes within acceptable ranges. For curved composite girders, the recommended ratios apply to the steel girder portion of the composite section.

3.6.1.1.2 Multiple Presence of Live Load

The provisions of this Article shall not be applied to the fatigue limit state for which one design truck is used, regardless of the number of design lanes. Where the single-lane approximate distribution factors in Articles 4.6.2.2 and 4.6.2.3 are used, other than the lever rule and statical method, the force effects shall be divided by 1.20.

Unless specified otherwise herein, the extreme live load force effect shall be determined by considering each possible combination of number of loaded lanes multiplied by a corresponding multiple presence factor to account for the probability of simultaneous lane occupation by the full HL93 design live load. In lieu of site specific data, the values in Table 1:

- Shall be used when investigating the effect of one lane loaded,
- May be used when investigating the effect of three or more lanes loaded.

For the purpose of determining the number of lanes when the loading condition includes the pedestrian loads specified in Article 3.6.1.6 combined with one or more lanes of the vehicular live load, the pedestrian loads may be taken to be one loaded lane.

The factors specified in Table 1 shall not be applied in conjunction with approximate load distribution factors specified in Articles 4.6.2.2 and 4.6.2.3, except where the lever rule is used or where special requirements for exterior beams in beam-slab bridges, specified in Article 4.6.2.2.2d, are used.

Table 3.6.1.1.2-1 Multiple Presence Factors *m*.

Number of Loaded Lanes	Multiple Presence Factors *m*
1	1.20
2	1.00
3	0.85
>3	0.65

C3.6.1.1.2

The multiple presence factors have been included in the approximate equations for distribution factors in Articles 4.6.2.2 and 4.6.2.3, both for single and multiple lanes loaded. The equations are based on evaluation of several combinations of loaded lanes with their appropriate multiple presence factors and are intended to account for the worst case scenario. Where use of the lever rule is specified in Article 4.6.2.2 and 4.6.2.3, the Engineer must determine the number and location of vehicles and lanes, and, therefore, must include the multiple presence. Stated another way, if a sketch is required to determine load distribution, the Engineer is responsible for including multiple presence factors and selecting the worst design case. The factor 1.20 from Table 1 has already been included in the approximate equations and should be removed for the purpose of fatigue investigations.

The entry greater than 1.0 in Table 1 results from statistical calibration of these Specifications on the basis of pairs of vehicles instead of a single vehicle. Therefore, when a single vehicle is on the bridge, it can be heavier than each one of a pair of vehicles and still have the same probability of occurrence.

The consideration of pedestrian loads counting as a "loaded lane" for the purpose of determining a multiple presence factor (m) is based on the assumption that simultaneous occupancy by a dense loading of people combined with a 75-year design live load is remote. For the purpose of this provision, it has been assumed that if a bridge is used as a viewing stand for eight hours each year for a total time of about one month, the appropriate live load to combine with it would have a one-month recurrence interval. This is reasonably approximated by use of the multiple presence factors, even though they are originally developed for vehicular live load.

Thus, if a component supported a sidewalk and one lane, it would be investigated for the vehicular live load alone with $m = 1.20$, and for the pedestrian loads combined with the vehicular live load with $m = 1.0$. If a component supported a sidewalk and two lanes of vehicular live load, it would be investigated for:

- One lane of vehicular live load, $m = 1.20$;
- The greater of the more significant lanes of vehicular live load and the pedestrian loads or two lanes of vehicular live load, $m = 1.0$, applied to the governing case; and
- Two lanes of vehicular live load and the pedestrian loads, $m = 0.85$.

The multiple presence factor of 1.20 for a single lane does not apply to the pedestrian loads. Therefore, the case of the pedestrian loads without the vehicular live load is a subset of the second bulleted item.

The multiple presence factors in Table 1 were developed on the basis of an ADTT of 5,000 trucks in one direction. The force effect resulting from the appropriate number of lanes may be reduced for sites with lower ADTT as follows:

- If $100 \leq ADTT \leq 1{,}000$, 95 percent of the specified force effect may be used; and
- If $ADTT < 100$, 90 percent of the specified force effect may be used.

This adjustment is based on the reduced probability of attaining the design event during a 75-year design life with reduced truck volume.

3.6.1.2 Design Vehicular Live Load

3.6.1.2.1 General

Vehicular live loading on the roadways of bridges or incidental structures, designated HL-93, shall consist of a combination of the:

- Design truck or design tandem, and
- Design lane load.

C3.6.1.2.1

Consideration should be given to site-specific modifications to the design truck, design tandem, and/or the design lane load under the following conditions:

- The legal load of a given jurisdiction is significantly greater than typical;
- The roadway is expected to carry unusually high percentages of truck traffic;
- Flow control, such as a stop sign, traffic signal, or toll booth, causes trucks to collect on certain areas of a bridge or to not be interrupted by light traffic; or
- Special industrial loads are common due to the location of the bridge.

See also discussion in Article C3.6.1.3.1.

The live load model, consisting of either a truck or tandem coincident with a uniformly distributed load, was developed as a notional representation of shear and moment produced by a group of vehicles routinely permitted on highways of various states under "grandfather" exclusions to weight laws. The vehicles considered to be representative of these exclusions were based on a study conducted by the Transportation Research Board (*Cohen 1990*). The load model is called "notional" because it is not intended to represent any particular truck.

Except as modified in Article 3.6.1.3.1, each design lane under consideration shall be occupied by either the design truck or tandem, coincident with the lane load, where applicable. The loads shall be assumed to occupy 10.0 ft. transversely within a design lane.

In the initial development of the notional live load model, no attempt was made to relate to escorted permit loads, illegal overloads, or short duration special permits. The moment and shear effects were subsequently compared to the results of truck weight studies (*Csagoly and Knobel 1981; Nowak 1992*), selected WIM data, and the 1991 OHBDC live load model. These subsequent comparisons showed that the notional load could be scaled by appropriate load factors to be representative of these other load spectra.

The following nomenclature applies to Figures C1 through C6, which show results of live load studies involving two equal continuous spans or simple spans:

M POS 0.4L	=	positive moment at 4/10 point in either span
M NEG 0.4L	=	negative moment at 4/10 point in either span
M SUPPORT	=	moment at interior support
Vab	=	shear adjacent to either exterior support
Vba	=	shear adjacent to interior support
Mss	=	midspan moment in a simply supported span

The "span" is the length of the simple-span or of one of each of the two continuous spans. The comparison is in the form of ratios of the load effects produced in either simple-span or two-span continuous girders. A ratio greater than 1.0 indicates that one or more of the exclusion vehicles produces a larger load effect than the HS20 loading. The figures indicate the degree by which the exclusion loads deviate from the HS loading of designation, e.g., HS25.

Figures C1 and C2 show moment and shear comparisons between the envelope of effects caused by 22 truck configurations chosen to be representative of the exclusion vehicles and the HS20 loading, either the HS20 truck or the lane load, or the interstate load consisting of two 24.0-kip axles 4.0 ft. apart, as used in previous editions of the AASHTO Standard Specifications. The largest and smallest of the 22 configurations can be found in Kulicki and Mertz (*1991*). In the case of negative moment at an interior support, the results presented are based on two identical exclusion vehicles in tandem and separated by at least 50.0 ft.

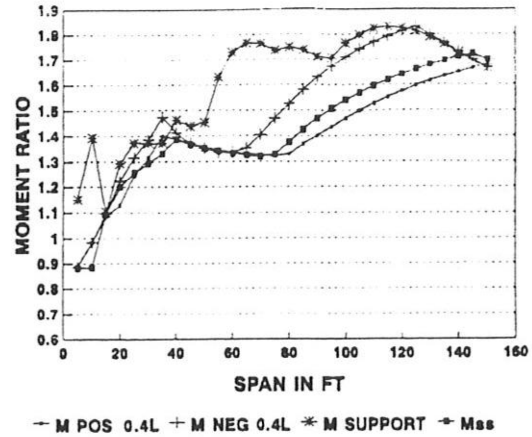


Figure C3.6.1.2.1-1 Moment Ratios: Exclusion Vehicles to HS20 (truck or lane) or Two 24.0-kip Axles at 4.0 ft.

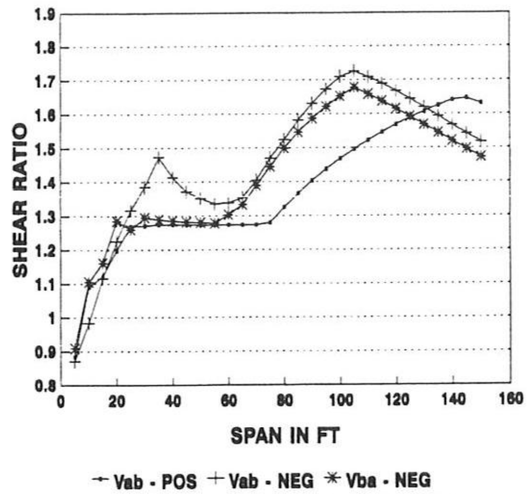


Figure C3.6.1.2.1-2 Shear Ratios: Exclusion Vehicles to HS20 (truck or lane) or Two 24.0-kip Axles at 4.0 ft.

Figures C3 and C4 show comparisons between the force effects produced by a single exclusion truck per lane and the notional load model, except for negative moment, where the tandem exclusion vehicles were used. In the case of negative moment at a support, the provisions of Article 3.6.1.3.1 requiring investigation of 90 percent of the effect of two design trucks, plus 90 percent of the design lane load, has been included in Figures C3 and C5. Compared with Figures C1 and C2, the range of ratios can be seen as more closely grouped:

- Over the span range,
- Both for shear and moment, and
- Both for simple-span and continuous spans.

The implication of close grouping is that the notional load model with a single-load factor has general applicability.

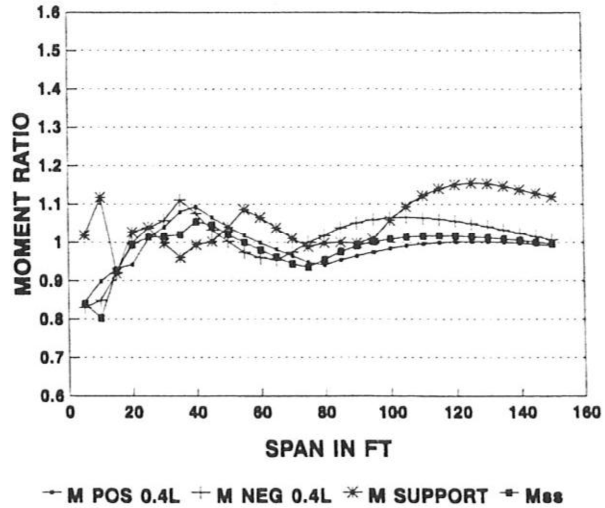


Figure C3.6.1.2.1-3 Moment Ratios: Exclusion Vehicles to Notional Model.

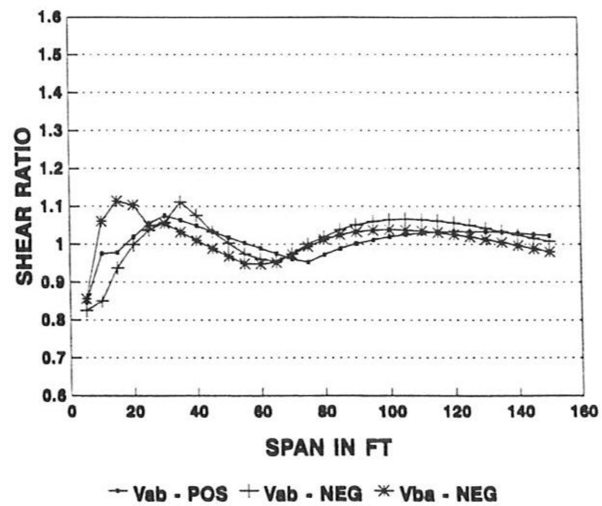


Figure C3.6.1.2.1-4 Shear Ratios: Exclusion Vehicles to Notional Model.

Figures C5 and C6 show the ratios of force effects produced by the notional load model and the greatest of the HS20 truck or lane loading, or Alternate Military Loading.

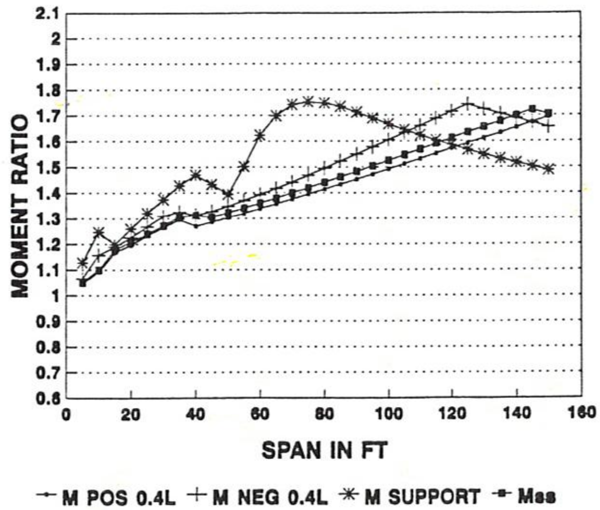


Figure C3.6.1.2.1-5 Moment Ratios: Notional Model to HS20 (truck or lane) or Two 24.0-kip Axles at 4.0 ft.

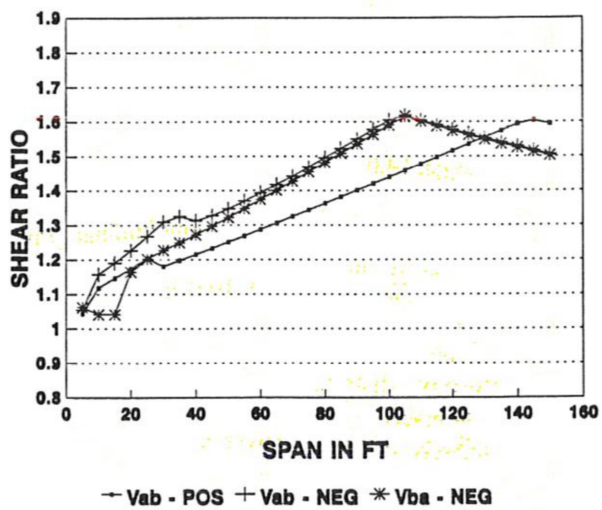


Figure C3.6.1.2.1-6 Shear Ratios: Notional Model to HS20 (truck and lane) or Two 24.0-kip Axles at 4.0 ft.

In reviewing Figures C5 and C6, it should be noted that the total design force effect is also a function of load factor, load modifier, load distribution, and dynamic load allowance.

3.6.1.2.2 Design Truck

The weights and spacings of axles and wheels for the design truck shall be as specified in Figure 1. A dynamic load allowance shall be considered as specified in Article 3.6.2.

Except as specified in Articles 3.6.1.3.1 and 3.6.1.4.1, the spacing between the two 32.0-kip axles shall be varied between 14.0 ft. and 30.0 ft. to produce extreme force effects.

3.6.1.2.6 Distribution of Wheel Loads Through Earth Fills

Revise as follows:

Where the depth of fill is less than 2.0 ft., live loads shall be distributed to the top slabs of culverts as specified in Article 4.6.2.10.

In lieu of a more precise analysis, or the use of other acceptable approximate methods of load distribution permitted in Section 12, where the depth of fill is 2.0 ft. or greater, wheel loads may be considered to be uniformly distributed over a rectangular area with sides equal to the dimension of the tire contact area, as specified in Article 3.6.1.2.5, and increased by ~~either 1.15 times the depth of the fill in select granular backfill, or~~ the depth of the fill in all other cases. The provisions of Articles ~~3.6.1.1.2 and~~ 3.6.1.3 shall apply.

C3.6.1.2.6

Add a new 3rd and 4th paragraphs as follows:

Select granular backfill is not used for embankments in California.

The multiple presence factor should not be applied when designing the top slab of culverts.

3.6.1.3 Application of Design Vehicular Live Loads

3.6.1.3.1 General

Add a new 4th bullet as follows:

- For both negative moment between points of contraflexure under a uniform load on all spans, and reaction at interior piers only, 100 percent of the effect of two design tandems spaced anywhere from 26.0 ft. to 40 ft. from the lead axle of one tandem to the rear axle of the other, combined with the design lane load specified in Article 3.6.1.2.4.

C3.6.1.3.1

Revise the 3rd paragraph as follows:

The notional design loads were based on the information described in Article C3.6.1.2.1, which contained data on "low boy" type vehicles weighing up to about 110 kip. ~~Where multiple lanes of heavier versions of this type of vehicle are considered probable, consideration should be given to investigating negative moment and reactions at interior supports for pairs of the design tandem spaced from 26.0 ft. to 40.0 ft. apart, combined with the design lane load specified in Article 3.6.1.2.4. One hundred percent of the combined effect of the design tandems and the design lane load should be used.~~ In California, side-by-side occurrences of the "low boy" truck configuration are routinely found. This amendment is consistent with Article 3.6.1.2.1, will control negative bending serviceability in two-span continuous structures with 20-ft to 60-ft span lengths, and should not be considered a replacement for the Strength II Load Combination.

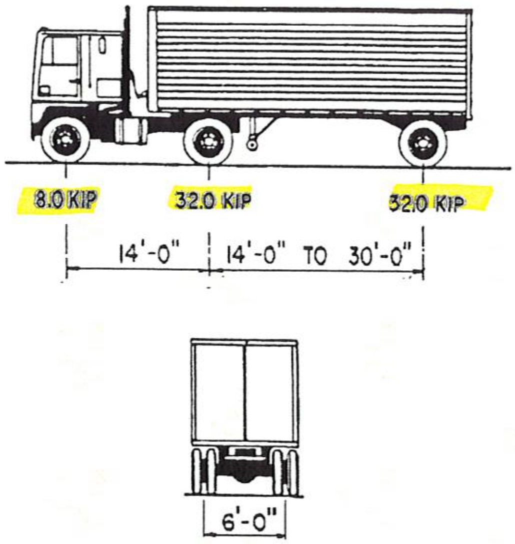


Figure 3.6.1.2.2-1 Characteristics of the Design Truck.

3.6.1.2.3 Design Tandem

The design tandem shall consist of a pair of 25.0-kip axles spaced 4.0 ft. apart. The transverse spacing of wheels shall be taken as 6.0 ft. A dynamic load allowance shall be considered as specified in Article 3.6.2.

3.6.1.2.4 Design Lane Load

The design lane load shall consist of a load of 0.64 klf uniformly distributed in the longitudinal direction. Transversely, the design lane load shall be assumed to be uniformly distributed over a 10.0-ft. width. The force effects from the design lane load shall not be subject to a dynamic load allowance.

3.6.1.2.5 Tire Contact Area

The tire contact area of a wheel consisting of one or two tires shall be assumed to be a single rectangle, whose width is 20.0 in. and whose length is 10.0 in.

The tire pressure shall be assumed to be uniformly distributed over the contact area. The tire pressure shall be assumed to be distributed as follows:

- On continuous surfaces, uniformly over the specified contact area, and
- On interrupted surfaces, uniformly over the actual contact area within the footprint with the pressure increased in the ratio of the specified to actual contact areas.

C3.6.1.2.5

The area load applies only to the design truck and tandem. For other design vehicles, the tire contact area should be determined by the engineer.

As a guideline for other truck loads, the tire area in in.^{2} may be calculated from the following dimensions:

Tire width = $P/0.8$

Tire length = $6.4\gamma(1 + IM/100)$

where:

γ = load factor

IM = dynamic load allowance percent

P = design wheel load (kip)

3.6.1.2.6 Distribution of Wheel Loads Through Earth Fills

Where the depth of fill is less than 2.0 ft., live loads shall be distributed to the top slabs of culverts as specified in Article 4.6.2.10.

In lieu of a more precise analysis, or the use of other acceptable approximate methods of load distribution permitted in Section 12, where the depth of fill is 2.0 ft. or greater, wheel loads may be considered to be uniformly distributed over a rectangular area with sides equal to the dimension of the tire contact area, as specified in Article 3.6.1.2.5, and increased by either 1.15 times the depth of the fill in select granular backfill, or the depth of the fill in all other cases. The provisions of Articles 3.6.1.1.2 and 3.6.1.3 shall apply.

Where such areas from several wheels overlap, the total load shall be uniformly distributed over the area.

For single-span culverts, the effects of live load may be neglected where the depth of fill is more than 8.0 ft. and exceeds the span length; for multiple span culverts, the effects may be neglected where the depth of fill exceeds the distance between faces of end walls.

Where the live load and impact moment in concrete slabs, based on the distribution of the wheel load through earth fills, exceeds the live load and impact moment calculated according to Article 4.6.2.10, the latter moment shall be used.

C3.6.1.2.6

Elastic solutions for pressures produced within an infinite half-space by loads on the ground surface can be found in Poulos and Davis (*1974*), NAVFAC DM-7.1 (*1982*), and soil mechanics textbooks.

This approximation is similar to the 60° rule found in many texts on soil mechanics. The dimensions of the tire contact area are determined at the surface based on the dynamic load allowance of 33 percent at depth = 0. They are projected through the soil as specified. The pressure intensity on the surface is based on the wheel load without dynamic load allowance. A dynamic load allowance is added to the pressure on the projected area. The dynamic load allowance also varies with depth as specified in Article 3.6.2.2. The design lane load is applied where appropriate and multiple presence factors apply.

This provision applies to relieving slabs below grade and to top slabs of box culverts.

Traditionally, the effect of fills less than 2.0 ft. deep on live load has been ignored. Research (*McGrath, et al. 2004*) has shown that in design of box sections allowing distribution of live load through fill in the direction parallel to the span provides a more accurate design model to predict moment, thrust, and shear forces. Provisions in Article 4.6.2.10 provide a means to address the effect of shallow fills.

3.6.1.3 Application of Design Vehicular Live Loads

3.6.1.3.1 General

Unless otherwise specified, the extreme force effect shall be taken as the larger of the following:

- The effect of the design tandem combined with the effect of the design lane load, or
- The effect of one design truck with the variable axle spacing specified in Article 3.6.1.2.2, combined with the effect of the design lane load, and

C3.6.1.3.1

The effects of an axle sequence and the lane load are superposed in order to obtain extreme values. This is a deviation from the traditional AASHTO approach, in which either the truck or the lane load, with an additional concentrated load, provided for extreme effects.

The lane load is not interrupted to provide space for the axle sequences of the design tandem or the design truck; interruption is needed only for patch loading patterns to produce extreme force effects.

- For negative moment between points of contraflexure under a uniform load on all spans, and reaction at interior piers only, 90 percent of the effect of two design trucks spaced a minimum of 50.0 ft. between the lead axle of one truck and the rear axle of the other truck, combined with 90 percent of the effect of the design lane load. The distance between the 32.0-kip axles of each truck shall be taken as 14.0 ft. The two design trucks shall be placed in adjacent spans to produce maximum force effects.

Axles that do not contribute to the extreme force effect under consideration shall be neglected.

Both the design lanes and the 10.0-ft. loaded width in each lane shall be positioned to produce extreme force effects. The design truck or tandem shall be positioned transversely such that the center of any wheel load is not closer than:

- For the design of the deck overhang—1.0 ft. from the face of the curb or railing, and
- For the design of all other components—2.0 ft. from the edge of the design lane.

Unless otherwise specified, the lengths of design lanes, or parts thereof, that contribute to the extreme force effect under consideration, shall be loaded with the design lane load.

The notional design loads were based on the information described in Article C3.6.1.2.1, which contained data on "low boy" type vehicles weighing up to about 110 kip. Where multiple lanes of heavier versions of this type of vehicle are considered probable, consideration should be given to investigating negative moment and reactions at interior supports for pairs of the design tandem spaced from 26.0 ft. to 40.0 ft. apart, combined with the design lane load specified in Article 3.6.1.2.4. The design tandems should be placed in adjacent spans to produce maximum force effect. One hundred percent of the combined effect of the design tandems and the design lane load should be used. This is consistent with Article 3.6.1.2.1 and should not be considered a replacement for the Strength II Load Combination.

Only those areas or parts of areas that contribute to the same extreme being sought should be loaded. The loaded length should be determined by the points where the influence surface meets the centerline of the design lane.

Where a sidewalk is not separated from the roadway by a crashworthy traffic barrier, consideration should be given to the possibility that vehicles can mount the sidewalk.

3.6.1.3.2 Loading for Optional Live Load Deflection Evaluation

If the Owner invokes the optional live load deflection criteria specified in Article 2.5.2.6.2, the deflection should be taken as the larger of:

- That resulting from the design truck alone, or
- That resulting from 25 percent of the design truck taken together with the design lane load.

C3.6.1.3.2

As indicated in C2.5.2.6.1, live load deflection is a service issue, not a strength issue. Experience with bridges designed under previous editions of the AASHTO Standard Specifications indicated no adverse effects of live load deflection per se. Therefore, there appears to be little reason to require that the past criteria be compared to a deflection based upon the heavier live load required by these Specifications.

The provisions of this Article are intended to produce apparent live load deflections similar to those used in the past. The current design truck is identical to the HS20 truck of past Standard Specifications. For the span lengths where the design lane load controls, the design lane load together with 25 percent of the design truck, i.e., three concentrated loads totaling 18.0 kip, is similar to the past lane load with its single concentrated load of 18.0 kip.

3.6.1.3.3 Design Loads for Decks, Deck Systems, and the Top Slabs of Box Culverts

The provisions of this Article shall not apply to decks designed under the provisions of Article 9.7.2, "Empirical Design."

Where the approximate strip method is used to analyze decks and top slabs of culverts, force effects shall be determined on the following basis:

- Where the slab spans primarily in the transverse direction, only the axles of the design truck of Article 3.6.1.2.2 or design tandem of Article 3.6.1.2.3 shall be applied to the deck slab or the top slab of box culverts.
- Where the slab spans primarily in the longitudinal direction:
    - For top slabs of box culverts of all spans and for all other cases, including slab-type bridges where the span does not exceed 15.0 ft., only the axle loads of the design truck or design tandem of Articles 3.6.1.2.2 and 3.6.1.2.3, respectively, shall be applied.
    - For all other cases, including slab-type bridges (excluding top slabs of box culverts) where the span exceeds 15.0 ft., all of the load specified in Article 3.6.1.2 shall be applied.

Where the refined methods are used to analyze decks, force effects shall be determined on the following basis:

- Where the slab spans primarily in the transverse direction, only the axles of the design truck of Article 3.6.1.2.2 or design tandem of Article 3.6.1.2.3 shall be applied to the deck slab.
- Where the slab spans primarily in the longitudinal direction (including slab-type bridges), all of the loads specified in Article 3.6.1.2 shall be applied.

Wheel loads shall be assumed to be equal within an axle unit, and amplification of the wheel loads due to centrifugal and braking forces need not be considered for the design of decks.

C3.6.1.3.3

This Article clarifies the selection of wheel loads to be used in the design of bridge decks, slab bridges, and top slabs of box culverts.

The design load is always an axle load; single wheel loads should not be considered.

The design truck and tandem without the lane load and with a multiple presence factor of 1.2 results in factored force effects that are similar to the factored force effects using earlier specifications for typical span ranges of box culverts.

Individual owners may choose to develop other axle weights and configurations to capture the load effects of the actual loads in their jurisdiction based upon local legal-load and permitting policies. Triple axle configurations of single unit vehicles have been observed to have load effects in excess of the HL-93 tandem axle load.

It is theoretically possible that an extreme force effect could result from a 32.0-kip axle in one lane and a 50.0-kip tandem in a second lane, but such sophistication is not warranted in practical design.

3.6.2 Dynamic Load Allowance: *IM*

3.6.2.1 General

Revise the 1st paragraph as follows:

Unless otherwise permitted in Articles 3.6.2.2 and 3.6.2.3, the static effects of the design truck, ~~or~~ design tandem, or permit vehicle other than centrifugal and braking forces....

Revise Table 3.6.2.1-1 as follows:

Component	*IM*
Deck Joints—All Limit States	75%
All Other Components	
• Fatigue and Fracture Limit State	15%
• Strength II Limit State	25%
• All Other Limit States	33%

C3.6.2.1

Revise the 4th and 5th paragraphs as follows:

Field tests indicate that in the majority of highway bridges, the dynamic component of the response does not exceed 25 percent of the static response to vehicles. This is the basis for dynamic load allowance with the exception of deck joints. However, the specified live load combination of the design truck and lane load, represents a group of exclusion vehicles that are at least 4/3 of those caused by the design truck alone on short- and medium-span bridges. The specified value of 33 percent in Table 1 is the product of 4/3 and the basic 25 percent. California removed the 4/3 factor for Strength II because a lane load isn't a part of the design permit vehicle used. Furthermore, force effects due to shorter permit vehicles approach those due to the HL93. The HL93 tandem*1.33 + lane generally has a greater force effect than that due to the P15 on short-span bridges.

Generally speaking, the dynamic amplification of trucks follows the following general trends:

- As the weight of the vehicle goes up, the apparent amplification goes down.
- Multiple vehicles produce a lower dynamic amplification than a single vehicle.
- More axles result in a lower dynamic amplification.

~~For heavy permit vehicles which have many axles compared to the design truck, a reduction in the dynamic load allowance may be warranted.~~ A study of dynamic effects presented in a report by the Calibration Task Group (*Nowak 1992*) contains details regarding the relationship between dynamic load allowance and vehicle configuration.

Appendix D:
Excerpts from Caltrans Bridge Design Specifications 2004, Section 10.41.4.8.1

2.1.19.4.3
0117

10.41.4.2 Bending Stresses in Longitudinal Ribs

The total bending stresses in longitudinal ribs due to a combination of (1) bending of the rib and (2) bending of the girders may exceed the allowable bending stresses in Article 10.32 by 25 percent. The bending stress due to each of the two individual modes shall not exceed the allowable bending stresses in Article 10.32.

10.41.4.3 Bending Stresses in Transverse Beams

The bending stresses in transverse beams shall not exceed the allowable bending stresses in Article 10.32.

10.41.4.4 Intersections of Ribs, Beams, and Girders

Connections between ribs and the webs of beams, holes in the webs of beams to permit passage of ribs, connections of beams to the webs of girders, and rib splices may affect the fatigue life of the bridge when they occur in regions of tensile stress. Where applicable, the number of cycles of maximum stress and the allowable fatigue stresses given in Article 10.3 shall be applied in designing these details; elsewhere, a rational fatigue analysis shall be made in designing the details. Connections between webs of longitudinal ribs and the deck plate shall be designed to sustain the transverse bending fatigue stresses caused in the webs by wheel loads.

10.41.4.5 Thickness of Plate Elements

10.41.4.5.1 Longitudinal Ribs and Deck Plate

Plate elements comprising longitudinal ribs, and deck-plate elements between webs of these ribs, shall meet the minimum thickness requirements of Article 10.35.2. The quantity f_a may be taken as 75 percent of the sum of the compressive stresses due to (1) bending of the rib and (2) bending of the girder, but not less than the compressive stress due to either of these two individual bending modes.

10.41.4.5.2 Girders and Transverse Beams

Plate elements of box girders, plate girders, and transverse beams shall meet the requirements of Articles 10.34.2 to 10.34.6 and 10.39.4.

10.41.4.6 Maximum Slenderness of Longitudinal Ribs

The slenderness, L/r, of a longitudinal rib shall not exceed the value given by the following formula unless it can be shown by a detailed analysis that overall buckling of the deck will not occur as a result of compressive stress induced by bending of the girders:

$$\left(\frac{L}{r}\right)_{max} = 1{,}000\sqrt{\frac{1{,}500}{F_y} - \frac{2{,}700f}{F_y}} \qquad (10\text{-}91)$$ +

where:

L = distance between transverse beams (in.) +

r = radius of gyration about the horizontal centroidal axis of the rib including an effective width of deck plate (in.) +

f = maximum compressive stress in the deck plate as a result of the deck acting as the top flange of the girders; this stress shall be taken as positive (psi) +

F_y = specified minimum yield strength of rib material (psi) +

10.41.4.7 Diaphragms

Diaphragms, cross frames, or other means shall be provided at each support to transmit lateral forces to the bearings and to resist transverse rotation, displacement, and distortion. Intermediate diaphragms or cross frames shall be provided at locations consistent with the analysis of the girders. The stiffness and strength of the intermediate and support diaphragms or cross frames shall be consistent with the analysis of the girders.

10.41.4.8 Stiffness Requirements

10.41.4.8.1 Deflections

The deflections of ribs, beams, and girders due to live load plus impact may exceed the limitations in Article 10.6 but preferably shall not exceed $^1/_{500}$ of their span. The calculation of the deflections shall be consistent with the analysis used to calculate the stresses.

To prevent excessive deterioration of the wearing surface, the deflection of the deck plate due to the specified wheel load plus 30-percent impact preferably 0.K shall be less than $^1/_{300}$ of the distance between webs of ribs. The stiffening effect of the wearing surface shall not be included in calculating the deflection of the deck plate.

10.41.4.8.2 Vibrations

The vibrational characteristics of the bridge shall be considered in arriving at a proper design.

10.41.4.9 Wearing Surface

A suitable wearing surface shall be adequately bonded to the top of the deck plate to provide a smooth, nonskid riding surface and to protect the top of the plate against corrosion and abrasion. The wearing surface material shall provide (1) sufficient ductility to accommodate, without cracking or debonding, expansion and contraction imposed by the deck plate, (2) sufficient fatigue strength to withstand flexural cracking due to deck-plate deflections, (3) sufficient durability to resist rutting, shoving, and wearing, (4) imperviousness to water and motor-vehicle fuels and oils, and (5) resistance to deterioration from deicing salts, oils, gasolines, diesel fuels, and kerosenes.

10.41.4.10 Closed Ribs

Closed ribs without access holes for inspection, cleaning, and painting are permitted. Such ribs shall be sealed against the entrance of moisture by continuously welding (1) the rib webs to the deck plate, (2) splices in the ribs, and (3) diaphragms, or transverse beam webs, to the ends of the ribs.

Appendix E:
Technical Brief 980204 Rev. 4, Steel Reactive Tension System Quickchange Moveable Barrier (SRTS-12" QMB), Barrier Systems Inc.

BARRIER SYSTEMS INC

TECHNICAL
BRIEF

180 River Road • Rio Vista, CA 94571 • Tel 707-374-6800 • Fax 707-374-6801
Email: info@barriersystemsinc.com • Website: barriersystemsinc.com

Steel Reactive Tension System Quickchange Moveable Barrier (SRTS-12"QMB)

Specification

The Steel Reactive Tension System Quickchange Moveable Barrier (SRTS-12"QMB) is designed to meet the rigid requirements of deployment in moveable barrier applications where positive separation technology is required and where lane widths and lateral space are limited.

Description

The barrier elements shall be 32" (810 mm) high, 39" (1000 mm) in length, and 12" (305 mm) wide. The profile is 12" (305 mm) wide except for a heavy steel base, which shall be 24" (610 mm) wide. The barrier shall allow for the incorporation of a lane line at the intersection of the base and the vertical section. The small protrusion of the base shall provide an audible signal upon contact by an errant vehicle's tire (Attachment Figure 1, B980144). The individual elements shall weigh approximately 1500 pounds (680 kg) and shall rest on four rubber feet to increase the coefficient of friction between the barrier element and the road surface.

The barrier elements are connected in an end-to-end fashion with tensioning hinge mechanisms and steel pins that are at least 1.3 " (33mm) in diameter. The minimum length of SRTS-QMB to create a longitudinal barrier is 100 feet (30 meters). Each end of the SRTS-QMB must be anchored to the roadway with an anchorage that is capable of reacting a 100,000-pound (450,000 Newton) tensile load in the barrier in order to perform with the minimum deflection characteristics. If the ends of the SRTS-QMB are not anchored to the roadway, a minimum length of 80 sections must be deployed upstream of the point where minimum deflection is required. Minimum deflection characteristics for the SRTS-QMB system are shown in the attached RTS-QMB Deflection Curve.

Materials

The primary elements of the SRTS-QMB shall be constructed of ASTM, A-36 steel and high strength concrete. All external steel shall be hot dipped galvanized in accordance with ASTM, A 123. All structural welds shall be continuous. The shell shall be constructed of 0.25" (6.4 mm) A-36 steel.

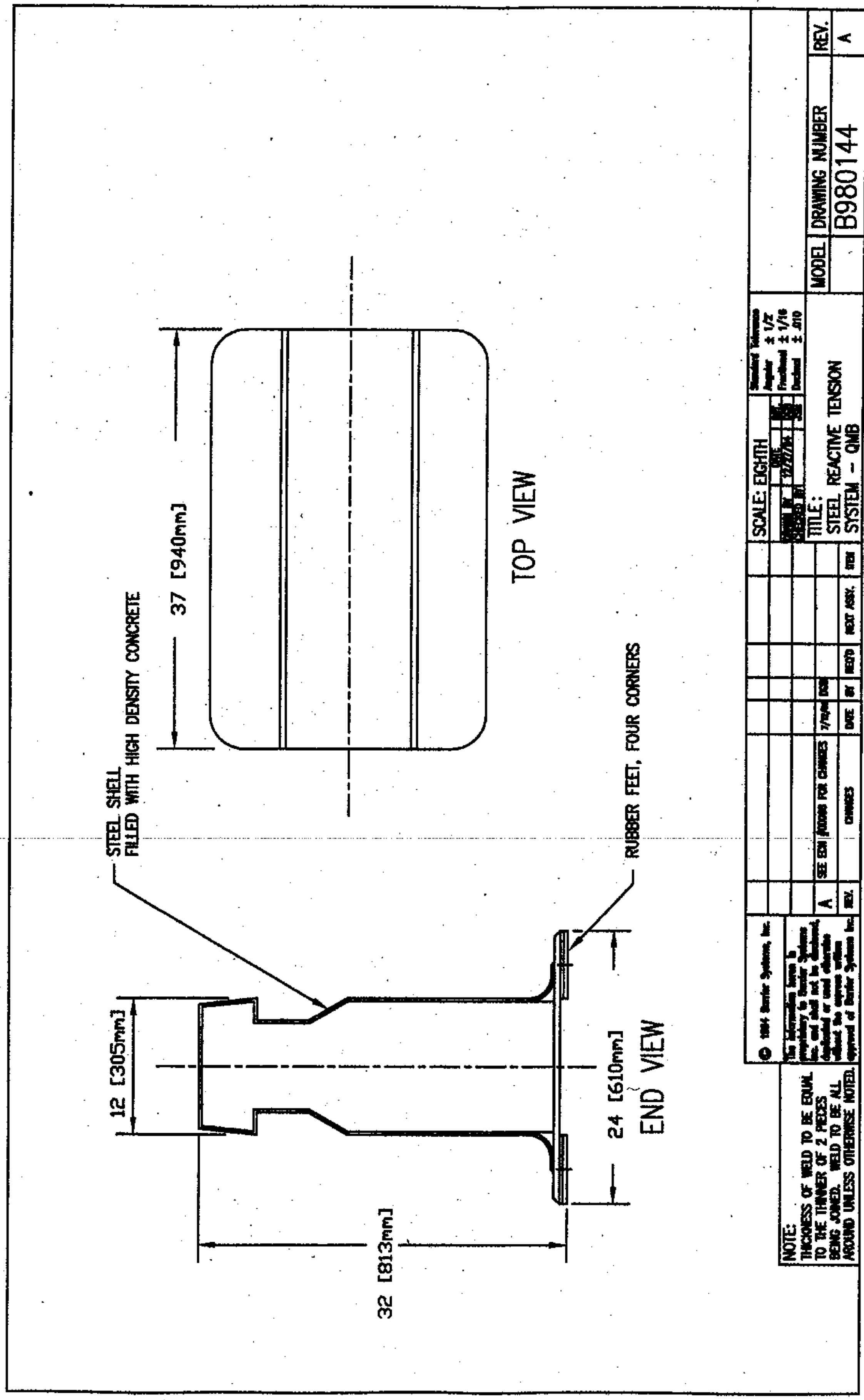
System Requirements

The SRTS-QMB system, when installed in accordance with the manufacturers instructions, shall function as a longitudinal barrier and be able to resist the impact of vehicles in accordance with the National Cooperative Highway Research Program Report 350 (NCHRP 350) Test Level 3.

The system shall minimize lateral displacement upon impact. The system shall minimize clearance between barrier hinges, resulting in a nominal metal to metal connection. During impact by an errant vehicle, the tension in the barrier system resists the penetration of the vehicle and limits the lateral displacement of the barrier.

Reactive Tension System Variable Length Barriers (RTS-VLBs) shall be added to the length of the SRTS-QMB installation in order to allow a smooth lateral transfer through the Barrier Transfer Machine. The number and location of RTS-VLB units that shall be required will vary depending on specifics of the application, number and degree of curves, changes in elevation, etc.

2.1.19.4.3
0117



TB 980204 Rev. 4
Page 3 of 4

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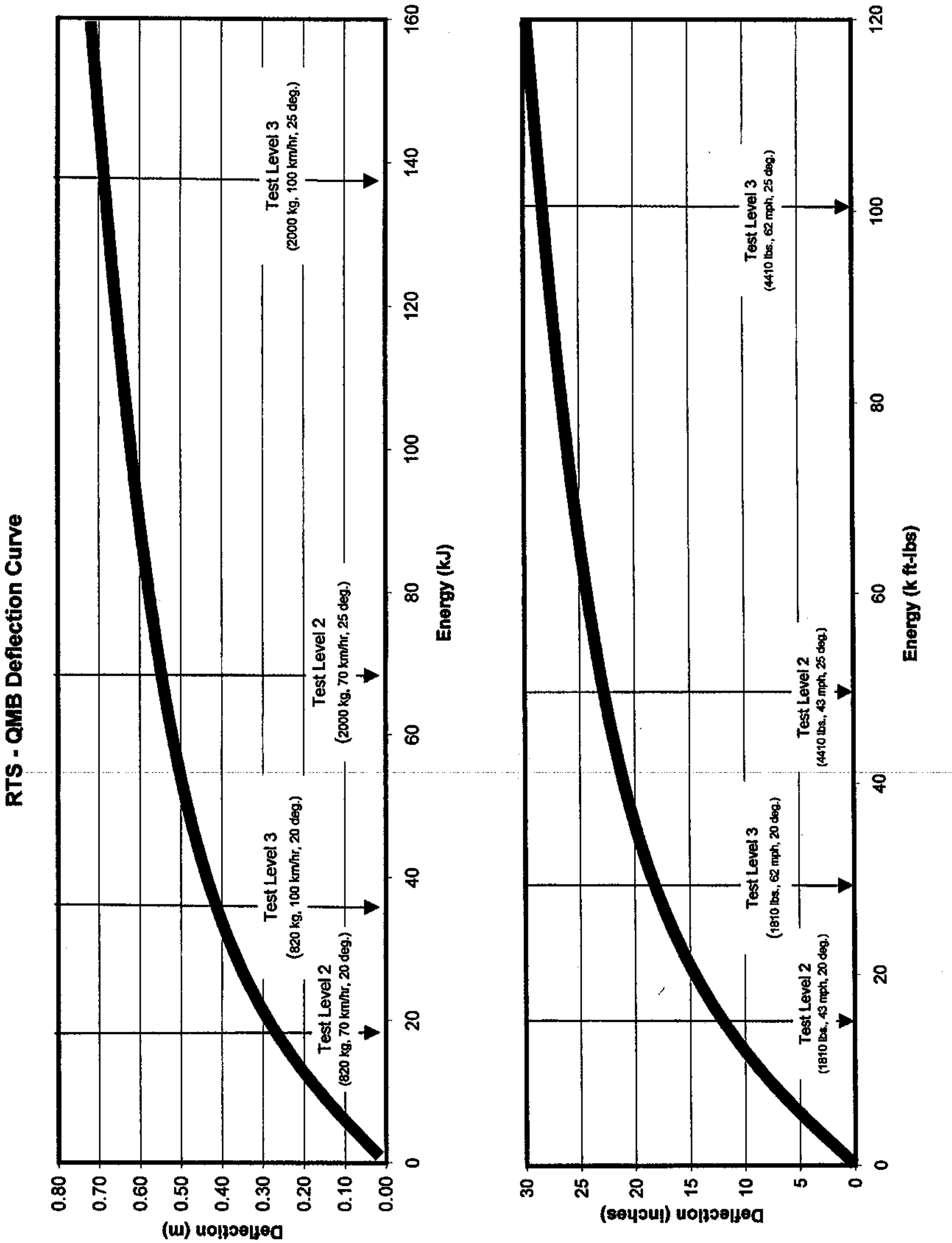

Based on NCHRP Report 350 testing for Reactive Tension QMB system with foundation or mass anchorage

Appendix F:
MS 060126 Rev.1, "Specifications for Products", Barrier Transfer Machine Specifications (Mid Sized Permanent Machines), Barrier Systems Inc.

SPECIFICATIONS FOR "PRODUCTS"

BARRIER TRANSFER MACHINE SPECIFICATIONS (Mid Sized Permanent Machines)

PART I

1.0 This specification describes a Barrier Transfer Machine (BTM) used to reposition continuous segments of Quickchange® Movable Barrier (QMB) or Reactive Tension Barrier (RTS) elements including Variable Length Barriers (VLB's), from a set position, to a lateral and approximate parallel position, a distance, that may be varied during the transfer operation or fixed, between 8'6" and 14', by means of a roller transfer system. The machine shall safely operate in all circumstances of weather, except untreated ice; and on variable surfaces such as concrete and asphalt. The machine shall not encroach upon adjacent travel lanes while making a lateral transfer of the QMB or RTS of more than 10', and must be able to reposition QMB or RTS when a deflection occurs due to normal impacts. The machine shall be equipped with two (2) fully enclosed operator's stations (cabs), one (1) located at each end of the machine. The machine shall be equipped with any special provisions required to actuate tether points or gates. The minimum service life of the machine shall be not less than 10 years. The machine must operate in both forward and reverse direction in order to reposition the QMB or RTS without turning the machine around. The machine shall be able to position the QMB or RTS to within 3 inches of existing permanent barriers. The machine must accomplish a continuous lateral transfer of the QMB or RTS at a working speed of at least 5 –10 mph depending on factors such as grade, road surface conditions weather and other factors. Top travel speed of the machine when unloaded shall not be less than 20 mph. Machine(s) furnished to these specifications must meet or exceed all requirements herein.

2.0 LIQUID-COOLED DIESEL ENGINE: The machine shall be equipped with a liquid cooled, turbo-charged, diesel-powered engine meeting, but not limited to, the following:

2.1 Minimum 300 SAE HP Cummins engine at governed RPM.

2.2 Minimum six (6) cylinders.

2.3 Twelve (12) volt electrical system consisting of a minimum 160-ampere alternator, electric starter, total minimum battery rating of at least 1,825 cold cranking amperes (CCA) at 0 degrees Fahrenheit (0°F) with a reserve capacity of 425 minutes at 80 degrees Fahrenheit (80°F).

2.4 Ignition switch at engine shall be keyed with a safety device to prevent engine starting when transmission is in gear.

2.5 Diesel engine fuel filtration system to include a screen at the fuel tank or transfer pump and at least two (2) stages of filtration. Filter stages may consist of a primary and secondary filter, or a two (2) stage filter in a common housing. At least one (1) drain shall be provided in the system to prevent water damage to the injection system. All items to be factory approved and factory installed.

2.6 Full-flow oil filtration system with replaceable filter and provision for bypassing oil to the engine as the filter becomes clogged.

2.7 Dry type air cleaner, including a primary element, a safety element, an internal or external precleaner and a restriction (service) indicator.

2.7.1 Air cleaner and connections shall be waterproof and dustproof and mounted in such a manner so as to withstand abrasions, wear and vibrations.

2.7.2 The air cleaner assembly shall not extend above the roof of the machine.

2.8 The radiator shall be suitable for ambient temperatures up to 110° Fahrenheit.

2.9 Cooling system with anti-freeze protection to at least minus 30 degrees Fahrenheit (-30°F).

2.9.1 Screw-on type water (coolant) filter.

2.10 All cooling system hoses including heater hoses shall be silicone type, if available.

2.11 Hour meter.

2.12 Positive crankcase ventilation

2.13 Drip pan mounted underneath engine with drain provision.

2.14 Engine shall be completely housed with louvered access panels for servicing and ventilation.

2.15 Manufacturer's standard fuel tank, minimum 100 gallon capacity.

2.16 Exhaust muffler, and vertical exhaust pipe with rain cap to exhaust fumes through roof and above machine.

3.0 MACHINE FRAME, DIMENSIONS AND CAPACITY: Machine shall meet the following requirements:

3.1 MACHINE FRAME: The machine frame shall be constructed of steel and have front, rear, and side structural members for impact protection.

3.2 MACHINE DIMENSIONS: The overall machine dimensions shall not exceed the following:

3.2.1 Width –120 INCHES

3.2.2 Length –624 INCHES

3.2.3 Height –148 INCHES without stack, 156 INCHES with stack,

3.3 MACHINE CAPACITY: The maximum unloaded weight of the machine shall not exceed 60,000 pounds. The loaded weight (when moving barrier segments) shall not exceed 86,000 pounds. The maximum wheel load shall be 17,000 pounds when unloaded and 25,000 pounds when fully loaded. The capacity rating shall be 60,000 pounds at the machine's top travel speed of 20 mph and 86,000 pounds at the working speed of 5 –10 mph depending on factors such as grade, road surface conditions weather and other factors.

3.4 TURNING RADIUS: The outside turning radius of the machine shall not exceed 75 feet.

4.0 CABS: The machine shall be equipped with a fully enclosed, all-metal machine cab at each operator station. The cabs shall be equipped with, but not limited to, the following:

4.1 Cab grab handles.

4.2 Access steps or ladder.

4.3 Lockable access door.

4.4 Roll-down or sliding window.

4.5 Windshield and side windows, shall be tinted shatterproof automotive type glass or Lexan.

4.6 Air ventilation, heater and windshield defroster.

4.7 Dome light.

4.8 Windshield washers, and two (2) speed or variable-speed electric windshield wiper.

4.9 Air conditioner will be engine driven

4.10 Headliner, cab insulation and rubber floor mats.

4.11 Operator's seat, cushioned, adjustable type, with padded backrest and armrests, and seat belt.

4.12 All machine controls shall be mounted in the cab and conveniently located at the operator's station.

4.13 Operator's station shall be located next to roller transfer system inlet.

4.14 Electric horn.

4.15 Access door to midsection in each cab.

5.0 INSTRUMENTATION: Each machine cab shall be equipped with, but not limited to, the following gauges or indicators that must be easily visible to the operator and shall have non-glare light(s) for night time visibility:

5.1 Speedometer.

5.2 Odometer

5.3 Tachometer.

5.4 Engine coolant temperature gauge.

5.5 Engine oil pressure gauge.

5.6 Hydraulic oil temperature gauge.

5.7 Hydraulic oil pressure gauges.

5.8 Voltmeter.

5.9 Fuel quantity gauge.

5.10 A warning light for the engine shutdown conditions.

5.10.1 High temperature.

5.10.2 Low oil pressure.

6.0 STEERING AND GUIDANCE SYSTEM

6.1 STEERING

6.1.1 The machine shall be equipped with a hydraulic, power-assist primary steering system on each axle.

6.1.2 Backup gasoline-powered, hydraulic pump system shall be provided to supply power for steering in the event the primary power system fails.

6.1.3 Control of steering for each axle shall be in the corresponding operator station performing the barrier transfer.

6.2 GUIDANCE SYSTEM

6.2.1 The machine shall be equipped with a guidance system to automatically guide the machine for pick-up and placement of the movable concrete barrier to within plus or minus 2 inches.

6.2.2 When the machine is being operated in autoguidance mode an indicator shall be provided in each cab to indicate if the machine is traveling on or to the right or left of the nominal position.

6.2.3 The system shall have an override feature to allow the machine to be steered manually.

6.2.4 The guidance or wire in or on the road surface and any electrical wiring, power systems, cabinetry and non-machine mounted peripheral items are the responsibility of others. The supplier shall be responsible for all guidance equipment on the Barrier Transfer Machine.

7.0 TRANSMISSION

7.1 The machine shall be equipped with a continuously variable, reversible hydrostatic drive system.

7.2 Drive system to provide forward and reverse working speed of at least 10mph for transfer of barrier and a top travel speed of 20 mph, in either direction.

8.0 WHEELS, TIRES AND AXLES

8.1 WHEELS

8.1.1 The machine shall be equipped with a minimum of four (4) steel disc wheels suitable for use with pneumatic, foam filled or solid rubber tires.

8.1.2 All wheels of the same size shall have the same bolt-hole pattern.

8.2 TIRES

8.2.1 The machine shall be equipped with a minimum of four urethane filled tires approved for the application by their manufacturers.

8.2.2 The total combined load rating of the tires and wheels shall exceed the gross machine weight rating (GVWR) of the machine. Load ratings shall be determined by the tire and wheel manufacturers.

8.3 AXLES

8.3.1 The machine shall be equipped with running gear of sufficient capacity to support the fully loaded weight of the machine.

8.3.2 Tractive power shall be provided to at least 2 wheels.

8.3.3 Vertical height adjustments shall be made with shims at the axles or by hydraulic cylinders, or other means.

9.0 BRAKES

9.1 In addition to the dynamic braking of the hydrostatic drive system, the machine shall be equipped with a spring applied fail safe emergency/parking brake system.

9.2 Brakes must be operable from both operator cabs.

9.3 Brakes shall be capable of holding the empty or loaded machine on 15 percent grade.

9.4 Emergency/parking brake system must be capable of being released for emergency towing purposes.

10.0 HYDRAULIC SYSTEM: System(s) as normally provided by the manufacturer shall be of a size, type and capacity to perform all required operations.

10.1 System shall be sealed against all contaminants and any necessary air vents shall be filtered.

10.2 A means shall be provided to maintain hydraulic oil at a satisfactory operating temperatures with ambient temperatures of up to 120°F.

10.3 Adjustable-pressure relief valve(s) shall be installed and preset as recommended by the manufacturer to provide overload protection to the hydraulic system.

10.4 Hydraulic reservoir shall have a minimum capacity of 60-gallon and be properly baffled.

10.5 A ten (10) micron filter (or finer) will be provided in the return line circuit with a by-pass valve to prevent restricted flow and be of a canister type design for easy servicing and replacement.

10.6 A valve shall be provided in the inlet and/or outlet lines to the filter housing in order to minimize hydraulic fluid loss when filter is being changed.

10.7 Hydraulic reservoir shall be equipped with a sight level gauge.

11.0 TRANSFER AND CAPSTAN SYSTEM

11.1 Transfer System

11.1.1 The machine shall be equipped with a roller transfer system which is capable of laterally transferring the QMB, a fixed or variable distance between 8 feet and 14 feet.

11.1.2 Roller transfer system shall lift the QMB approximately 4 inches off the road surface, pass the barrier diagonally through the intermediate roller section of the machine to the opposite side, and lower the barrier to the road surface a predetermined lateral distance.

11.1.3 No power, other than that provided by the machine, shall be required to transfer the movable barrier.

11.2 Capstan System

11.2.1 The machine shall be equipped with a multi-wheeled capstan system capable of maintaining the longitudinal position of the QMB wall within 12 to 18 inches of the specified deployment location.

11.2.2 The capstan system shall be hydraulically powered and shall be automatically or manually adjustable. If automatic controls are used there shall be manual override capability.

11.2.3 The capstan system shall maintain full capability at all allowed transfer speeds and shall be capable of imparting a longitudinal push or pull in either direction of machine travel.

12.0 MIDSECTION

12.1 The midsection shall be enclosed on the top and sides.

12.2 The engine shall be housed, skid mounted and approximately centered in the midsection of the machine with a walkway around both sides of the engine.

12.3 The floor shall be skid resistant grating.

12.4 Vertical distance between floor and roof shall be nominally 6 feet.

12.5 A top access panel shall be provided for vertical removal of engine.

12.6 Top access panel shall have four (4) lifting eyes, one (1) at each corner.

13.0 LIGHTING AND ACCESSORIES

13.1 LIGHTING: Each cab to have separate lighting controls for headlights, turn signals, rear brake and tail lamps. Both ends of the machine shall be equipped as follows:

13.1.1 Red tail lamp, red stop lamp, turn indicator and red reflector on each side. The lamps and/or reflectors may be combined, and shall be screw or bolt mounted at the same level and as widely spaced laterally as practicable. The lamps shall be located at a height of not less than 15 inches or more than 72 inches above the ground.

13.1.2 Two (2) white, halogen sealed beam headlights, located at a height of not more than 54 inches or less than 24 inches above the road surface.

13.1.3 Switchable amber and red clearance lamps shall be provided to indicate the extreme length and width of the machine.

13.1.4 Turn signal controls in each cab shall be 4-way flasher type for off, flash left, flash right and flash both lights.

13.1.5 Clearance and side marker lamps may be mounted in combination.

13.1.7 All electrical wiring shall be insulated and enclosed to maximum extent practical for protection from external damage and short circuits. Wiring shall be securely fastened at sufficient intervals to prevent sagging and insure clearance of mechanical parts. Routing of the wiring through the sub-frame, deck or the like shall be in such a manner so as not to interfere with normal operation and use or present a safety hazard. A sealed, splice-free modular wiring harness is acceptable. Rubber grommets shall be used wherever wires or harness pass through metal.

13.1.8 Where ever practical all electrical components will be suitable for operating in a marine environment to reduce the potential of corrosion.

13.2 ACCESSORY LIGHTING: The machine shall be equipped with the following:

13.2.1 Flashing amber warning lights shall be mounted on top of the machine on each corner to be clearly visible from all directions.

13.2.2 The machine shall be equipped with at least one work light at each end of the machine. On/off switch shall be convenient to the operator.

14.0 <u>SAFETY AND SPECIAL EQUIPMENT</u>

14.1 SAFETY: The machine shall be equipped with the following safety items:

14.1.1 A backup alarm system located at each end of the machine, distinguishable from the surrounding noise levels. Backup alarm system shall be activated when the machine is put in reverse from either operator station.

14.1.2 All rotating or reciprocating parts and all parts subject to high temperature, that are of such nature or so located as to become a hazard to operating personnel, shall be insulated, fully enclosed or properly guarded.

14.2 SPECIAL EQUIPMENT: The machine shall be equipped or provided with the following special equipment:

14.2.1 Tow hooks, one (1) set for each end.

14.2.2 Communication system to allow operators to communicate internally between both cabs.

14.2.3 Spare drive wheel and tire.

15.0 INSTRUCTION ON SAFETY, OPERATION AND PREVENTIVE MAINTENANCE: The successful bidder shall provide the Department a minimum 15 continuous days of instruction on safety, operation and preventive maintenance of the machine by factory-trained personnel after the machine has been delivered and is ready for operation. This training shall be concurrent with other equipment training to the maximum extent possible.

16.0 SAFETY PLAQUES OR DECALS: Safety plaques or decals shall be furnished and affixed at the operator's station and at any hazardous area. Plaques or decals shall include necessary warnings and precautions.

17.0 SERVICE POINT ACCESSIBILITY: All lubrication and frequent service items must be accessible to the operator/technician.

18.0 PAINTING: The exterior of the machine shall be painted a manufacturers approved standard color, except for glass, rubber and those metallic accessories or fixtures constructed of rust-resistant or plated material not normally painted. Lead-free paint, in accordance with Federal Standards, with be accepted. All interior compartment surfaces shall be painted a light green or light grey non-glare color.

19.0 MANUALS: Manual(s) containing illustrated parts list(s) and operating and service instructions for the machine and engine(s) shall be delivered with each machine.

19.1 Manual(s) shall be as detailed as possible outlining all necessary service and operating instructions for each machine delivered. Parts list(s) shall cover all components of the machine. Each part shall be identified by part number, description and component location. Full schematics of the electrical and hydraulic system will be supplied with each manual. Necessary warnings and safety precautions shall be included.

19.2 The following additional information shall be provided by the vendor at time of delivery if it is not included in the manuals required above.

19.2.1 Manufacturer's recommended service/preventive maintenance intervals.

19.2.2 Recommended fluids, lubricants and their SAE/API equivalents.

20.0 MANUFACTURER'S STATEMENT OF ORIGIN (MSO):
Vendor shall furnish an MSO with each machine at time of delivery.

Appendix G:
Email dated June 28, 2010 regarding BSI BTM Specifications, specifically BTM wheel load.

Gau, Michelle

From: Jack Mazer [jack.mazer@barriersystemsinc.com]
Sent: Monday, June 28, 2010 10:48 AM
To: Gau, Michelle; Harry Villegas
Cc: Tang, Xiuwei; Rubendall, Geoffrey; Gary Kaplan; Byron West; FILE; Stephen Dickie
Subject: RE: BSI BTM Spec

Michelle,

The BTM weight of 60,000 lbs is accurate. Occasionally we get it under that by a small amount but it depends on the components available during construction.

The other 26,000 lbs is the weight of the barrier in the machine during transfer, basically (1600# x 16 units) in the air at any time.

There is no impact load involved - or very little – since the barrier are picked up on a rubber inclined ramp and are lowered the same way.

What we can do is address the max tire load... it should not really be 25,000 per tire. That can be controlled to be 86,000/4 or 21,500 +/- perhaps 500 #'s. This can be done by proper shimming of the machine axles and hydraulic controls.

As for footprint, we do not have the length and width, we do have the areas. Our standard area is 149 sq inches per tire, with our oversized tire it goes up to 202 sq. inches, but those tires add 2400 to 2800 pounds to the total weight of the machine, and are useful when the issue is contact pressure not total load.

This machine typically does not meet truck standards, but then again it makes a limited number of trips over the bridge at a max loaded speed of 10 MPH.

Let us know if there is anything else we can help with.

Jack Mazer
QMB Applications Manager
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A LINDSAY TRANSPORTATION SOLUTIONS COMPANY
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01-707-374-6800 Office
01-209-610-8713 Cell

From: Gau, Michelle [mailto:Michelle.Gau@aecom.com]
Sent: Monday, June 28, 2010 9:23 AM
To: Harry Villegas; Jack Mazer
Cc: Tang, Xiuwei; Rubendall, Geoffrey
Subject: BSI BTM Spec

Harry/ Jack,

I'm working on the Golden Gate Bridge Moveable Median Barrier project with Bill Burton of AECOM. Specifically, I'm working on calculating the additional load due to the Steel Reactive Tension Quickchange Moveable Barrier (SRTS-12"

QMB) System, and have a question regarding an empty BTM versus a loaded BTM in order to properly calculate the wheel load.

In the specs for the BTM (MS 060126, Rev 1), it states an unloaded BTM weighs 60,000 lbs with a maximum wheel load of 17,000, and a loaded BTM weighs 86,000 lbs with a maximum wheel load of 25,000 lbs. This is a great increase, especially considering the HS20-44 truck wheel load of 16,000 lbs the bridge designed to carry. Most of our resources state 60,000 as the BTM weight, so this is what has been used thus far. If in fact we are to use the weight due to a loaded BTM, it has a great impact on our load calculation.

This being said, we are looking to get the **wheel tire contact area dimension**, both the width and length, of the BTM we are looking to use. Also, we'd like an explanation of where the 86,000 lbs is coming from (i.e. what is this including exactly?, any impact or safety factors?, etc.). Xiuwei and I put a call into Harry early Friday afternoon and left both of our names and phone numbers, but have not heard back yet.

Please feel free to contact Xiuwei Tang (copied on this email) or myself. Thank you in advance for your help.

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Bridge Engineer
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GOLDEN GATE BRIDGE
ENGINEERING DEPARTMENT
21.19.4.3

c: DJ Mulligan
EZ Bauer
JR Eberle
Mike Connerari
JYF Lee

DATE: 11/15/10
JOB NO.: 60097852
PROJECT: Golden Gate Bridge Moveable Median Barrier
SUBJECT: *Draft* Structural Analysis Report

TO:
Mr. Jeffrey Y. Lee, PE. Project Manager
Golden Gate Bridge, Hwy, Transportation District
Toll Plaza Administration Building/Receiving
Engineering Department
San Francisco, CA 94129-0601
T. 415.923.2031

FROM: Bill Burton/Geoffrey Rubendall

We are forwarding:

- ☐ Enclosed
- ☐ Separate Cover
- ☐ By Pickup
- ☐ Hand Delivered
- ■ By Messenger
- ☐ Other by UPS

ORIGINAL	UNIT	DESCRIPTION
10	1	*Draft* Structural Analysis Report November 2010
1	1	CD containing the above report

COMMENTS:

Signature: [signature]
Bill Burton
