


To:		New York State Department of Transportation ENGINEERING BULLETIN	EB 03-040
<i>Expires one year after issue unless replaced</i>			
Title: NYSDOT STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES - 2003 UPDATE			
Distribution: <input type="checkbox"/> Manufacturers (18) <input type="checkbox"/> Surveyors (33) <input checked="" type="checkbox"/> Main Office (30) <input checked="" type="checkbox"/> Consultants (34) <input checked="" type="checkbox"/> Local Govt. (31) <input type="checkbox"/> Contractors (39) <input checked="" type="checkbox"/> Regions/Agencies (32) <input type="checkbox"/> _____ ()		Approved: <u>/s/ G.A. Christian</u> <u>07/24/03</u> George A. Christian Date Acting Deputy Chief Engineer (Structures)	

ADMINISTRATIVE INFORMATION:

- This Engineering Bulletin (EB) shall be considered in effect for all structural design projects in New York State.
- This EB does not supersede any current issuances.

PURPOSE:

This EB issues two new "Blue Pages" to add to the set of "Blue Pages" dated July 2002. This revised set of "Blue Pages" constitute the New York State modifications to the AASHTO Standard Specifications for Highway Bridges. The "Blue Pages", as revised, combined with the AASHTO Standard Specifications For Highway Bridges, 17th Edition (or the 16th Edition with all of the Interim Specifications 1997 through 2003) form the New York State Department of Transportation Standard Specifications (2003).

TRANSMITTED MATERIALS:

The two attached "Blue Pages" are to be added to the existing "Blue Pages" dated July 2002.

CONTACT:

Direct questions regarding this EB to Peter C. McCowan of the Main Office Structures Design and Construction Division at (518) 485-2770 or by e-mail to pmccowan@dot.state.ny.us.

10.39**COMPOSITE BOX GIRDERS**

Add the following as Article 10.39.9

10.39.9**Top Lateral Bracing**

Open top box girders (also referred to as tub girders or trough girders), where the composite concrete deck completes the box girder enclosure, shall include a full length lateral bracing system in the form of a horizontal truss between the top flanges of the box girder. The lateral bracing in the horizontal truss shall be proportioned to resist torsional shear and lateral deformations caused by all anticipated loads on the non-composite open box section including construction loads and load eccentricities encountered during placement of the concrete deck.

An equivalent top lateral bracing system or a partial length top lateral bracing system may be considered adequate if it is demonstrated by a full analysis to be effective, submitted to and approved by the Deputy Chief Engineer (Structures).

10.51

COMPOSITE BOX GIRDERS


Add the following as Article 10.51.8

10.51.8

Top Lateral Bracing

Open top box girders (also referred to as tub girders or trough girders), where the composite concrete deck completes the box girder enclosure, shall include a full length lateral bracing system in the form of a horizontal truss between the top flanges of the box girder. The lateral bracing in the horizontal truss shall be proportioned to resist torsional shear and lateral deformations caused by all anticipated loads on the non-composite open box section including construction loads and load eccentricities encountered during placement of the concrete deck.

An equivalent top lateral bracing system or a partial length top lateral bracing system may be considered adequate if it is demonstrated by a full analysis to be effective, submitted to and approved by the Deputy Chief Engineer (Structures).

To:		New York State Department of Transportation ENGINEERING BULLETIN	EB 03-016
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ADMINISTRATIVE INFORMATION:

- **Effective Date:** This Engineering Bulletin (EB) is effective immediately.
- **Superseded Issuances:** This EB does not supersede any current issuances.
- **Disposition of Transmitted Materials:** The materials transmitted by this EB will be incorporated into the next edition of the "Blue Pages".

PURPOSE:

This EB issues three new "Blue Pages" to add to the set of "Blue Pages" dated July 2002.

TECHNICAL INFORMATION:

This revised set of "Blue Pages" constitute the New York State modifications to the AASHTO Standard Specifications for Highway Bridges. The "Blue Pages", as revised, combined with the AASHTO Standard Specifications For Highway Bridges, 17th Edition (or the 16th Edition with all of the Interim Specifications 1997 through 2002) form the New York State Department of Transportation Standard Specifications (2002).

TRANSMITTED MATERIALS:

The three attached "Blue Pages" are to be added to the existing "Blue Pages" dated July 2002.

CONTACT:

Direct questions regarding this EB to Harry L. White of the Main Office Structures Design and Construction Division at (518) 485-1148 or by e-mail to hwhite@dot.state.ny.us.

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2.7 RAILINGS

Delete the entire text of Article 2.7 and replace with the following:

“For the New York State requirements for railings and NYS standard railing and barrier types, refer to Section 6, *Bridge Railing*, of the NYSDOT *Bridge Manual*.

Any other type of railing or barrier system proposed for use on a bridge in New York State must meet the requirements established in NCHRP Report 350, the design criteria of Section 13 of the AASHTO *LRFD Bridge Design Specifications* and be approved by the Deputy Chief Engineer Structures.”

3.24.5.2 Railing Loads

Delete the text of Article 3.24.5.2 and replace with the following:


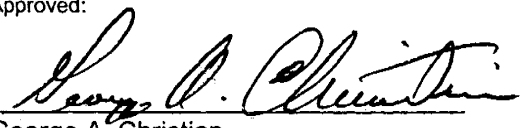
“The railing loads on the cantilever slab shall be applied in accordance with Section A13.4 of AASHTO *LRFD Bridge Design Specifications*.”

5.8.12.2 Traffic Loads and Barriers

Delete the term “of 45 kN (10 kips)” in Article 5.8.12.2 (two places).

Add the following to the end of the first paragraph of Article 5.8.12.2:

“The horizontal loads from a traffic barrier shall be calculated using the criteria of Section 13 of the AASHTO *LRFD Bridge Design Specifications*.”

To:		New York State Department of Transportation ENGINEERING BULLETIN	EB 02-038
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ADMINISTRATIVE INFORMATION:

- This Engineering Bulletin (EB) shall be considered in effect for all structural design projects in New York State unless implementation will result in undue delay to projects currently under design.
- This EB does not supersede any current issuances.

PURPOSE:

The purpose of this EB is to announce the availability of "Blue Pages" dated July 2002 which constitute New York State modifications to the AASHTO Standard Specifications for Highway Bridges, 16th Edition dated 1996 and all Interim Specifications (1997-2002). The 17th Edition of the Standard Specifications is expected to be printed soon and will be the final release. It will consist of the 16th Edition dated 1996 and all Interim Specifications (1997-2002). These "Blue Pages" have been written such that they may be inserted into the 16th or the 17th editions.

The "Blue Pages" dated July 2002 replace all existing "Blue Pages" dated 6/99 and are to be combined with the AASHTO Standard Specifications for Highway Bridges, 17th Edition dated 2002, or the 16th Edition dated 1996 and all Interim Specifications (1997-2002). The resulting specification shall be considered as the New York State Department of Transportation Standard Specifications for Highway Bridges, dated July 2002.

It should be noted that the NYSDOT - *Bridge Manual* has been referenced frequently in order to avoid redundant information.

TRANSMITTED MATERIALS:

The Department will provide only the New York State modifications in the form of "Blue Pages" as inserts for distribution to consulting engineers and others thru the Department's Plan Sales Section.

Consultants and others may purchase the "Blue Pages" for \$15 by contacting Plan Sales at the following address:

Plan Sales Section
 New York State Department of Transportation
 Building 5, Room 109A
 1220 Washington Avenue
 Albany, New York 12232-0204
 Phone: (518) 457-2124

CONTACT:

Direct questions regarding this EB to John F. Sadowski Jr. of the Main Office Structures Design and Construction Division at (518) 457-6827 or by e-mail to jsadowski@gw.dot.state.ny.us.

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APPENDIX E Page 652 Article 3.24.3.2

1.3

WATERWAYS

Add the following paragraph to Article 1.3:

The designer shall refer to the Structures Division's Bridge Safety Assurance (BSA) Hydraulic Vulnerability Manual for determining the relative risk of hydraulic induced failure at a bridge site.

1.3.2 Hydraulic Studies

Articles 1.3.2.1, 1.3.2.2 and 1.3.2.3:

These articles have been revised and the changes are incorporated in the AASHTO Model Drainage Manual - 1999 (Metric Edition).

1.5 ROADWAY DRAINAGE

Add the following to Article 1.5:

For additional information see sub-section "Deck Drainage", Section 5 of the NYSDOT - *Bridge Manual*.

1.7

SUPERELEVATION

Delete the text of Article 1.7 and replace with the following:

The superelevation treatment for a bridge deck shall follow the criteria outlined in sub-section "Superelevation", Section 2 of the NYSDOT - *Bridge Manual*.

2.1 GENERAL

2.1.2 Width of Roadway and Sidewalk

2.2 STANDARD HIGHWAY CLEARANCES - GENERAL

2.2.1 Navigational

2.2.2 Roadway Width

2.2.3 Vertical Clearance

Delete the texts of Articles 2.1.2, 2.2.1, 2.2.2 and 2.2.3 and replace with the following note:

The geometric design standards provided in the appropriate sub-sections of Section 2 of the NYSDOT - *Bridge Manual* shall be applicable, unless otherwise approved as a non-standard feature.

2.2.5 Curbs and Sidewalks

Delete the first three paragraphs of Article 2.2.5 and replace with the following:

Concrete barrier is the preferred vehicle retention system for all types of bridges. The use of this type of system may introduce a curbed section on the bridge when the highway section is not curbed. In some cases a curb/rail system may be proposed. Whatever the option, curbs shall always be used under the following conditions:

- On all structures crossing a highway or railroad.
- Where it is necessary to control bridge deck drainage (ie. Viaducts, reservoirs, etc.)

The curb height on the bridge shall match the curb height on the roadway approach. Where a curb is being introduced on the bridge, the height of the bridge curb above the roadway shall not be less than 6 inches (150 mm).

The face of the curb is defined as the vertical or sloping surface on the roadway side of the curb. Horizontal measurements of roadway and curb width are given from the bottom of the face, or, in the case of stepped back curbs, from the bottom of the lower face for roadway width.

For additional guidelines see sub-section "Miscellaneous Bridge Width Considerations", Section 2 of the NYSDOT - *Bridge Manual*.

2.3 HIGHWAY CLEARANCES FOR BRIDGES

Delete the text of Article 2.3 and replace with the following:

For required clearances, refer to Section 2 of the NYSDOT - *Bridge Manual*.

2.4 HIGHWAY CLEARANCES FOR UNDERPASSES

Delete the text of Article 2.4 and replace with the following:

For required clearances, refer to Section 2 of the NYSDOT - *Bridge Manual*.

2.6 HIGHWAY CLEARANCES FOR DEPRESSED ROADWAYS

Delete the text of Article 2.6 and replace with the following:

For required clearances, refer to Section 2 of the NYSDOT - *Bridge Manual*.

3.3 DEAD LOAD

Delete Article 3.3.3, 3.3.4, and 3.3.5 and replace with the following:

- 3.3.3** Where traffic is to bear directly on the concrete slab, an additional 1-1/2 inch (38mm) cover of the top of the reinforcement shall be added.
- 3.3.4** All structures having a monolithic wearing surface shall be designed for a possible additional wearing surface weighing 20 pounds per square foot (960N/m²).
- 3.3.5** For additional bridge deck loads see sub-section "Forming", Section 5, and sub-section "Railing/Parapet Design Dead Loads", Section 6 of the NYSDOT - *Bridge Manual*.

3.7 HIGHWAY LOADS

3.7.2 Classes of Loading

Add the following to Article 3.7.2:

For design of all new highway structures HS25 (MS23) loading (125% of the standard HS20 [MS18] loading) shall be used.

However, for fatigue analysis of all new highway structures the standard HS20 (MS18) loading shall be used.

3.7.3 Designation of Loadings

Add the following as last line to the second paragraph.

H25-S20 loading, 1992 New York Edition shall be designated -- HS25.

3.7.4 Minimum Loading

Delete "HS20-44" and replace with "HS25" (MS23).

3.8 IMPACT

3.8.1.1 Group A - Impact shall be included

Add the following to Article 3.8.1.1:

(4) All bearings except elastomeric bearings.

3.8.2 Impact Formula

3.8.2.3 Delete the text of Article 3.8.2.3 and replace with the following:

For box culverts with fill heights less than 3 feet (900 mm), the Impact Fraction, I, shall be equal to:

$$50 / (S + 125) \leq 0.3 \quad (15 / (S + 38) \leq 0.3)$$

where S is the perpendicular distance, in feet (m), between wall centerlines.

For three sided culverts and other buried structures with fill heights:

0'-0" to 1'-0" inc.	(0 mm to 300 mm)	I = 0.3
1'-1" to 2'-0" inc.	(301 mm to 600 mm)	I = 0.2
2'-1" to 3'-0"	(601 mm to 899 mm)	I = 0.1

Impact factor is equal to one plus the Impact Fraction (1.0 + I)

3.9 LONGITUDINAL FORCES

Add the following paragraph to Article 3.9

For low steel sliding type bearings, this longitudinal force due to friction may be taken as 15% of the dead load. For rocker type bearings, this force shall be based on a 15% friction coefficient on the pin, and shall be reduced in proportion to the radii of the pin and rocker.

For elastomeric bearings, the longitudinal force due to shear deformation for design purposes may be taken as defined by Equation (14.5.3.1-2), under Article 14.5.3.1.

For multi-rotational bearings, (pot & disc type) the longitudinal force due to friction may be taken, for design purposes, as 5% of the dead load.

Add the following to "Figure 3.7.6B Lane Loading:"

For HS25 (MS23) loading the concentrated load for moment shall be 22,500 lbs.(100kN) and for shear 32,500 lbs(145kN). The uniform load for HS25 (MS23) shall be 800 lbs. per linear foot (12kN/m) of lane load.

FIGURE 3.7.7A STANDARD HS TRUCKS

Add the following to "Figure 3.7.7A Standard HS Trucks":

For HS25 (MS23) loading, the front axle shall be 10,000 lbs. (44.5 kN) and the rear two axles shall be 40,000 lbs. (178.0 kN) each.

3.15**WIND LOADS**

Delete the first paragraph of Article 3.15 and replace with the following:

The following wind load forces per square foot of exposed area shall be applied to all structures (see Article 3.22 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 (161 km/hour).

The wind loads may be reduced in intensity as indicated below. The reduction is based on a Map of Wind Speeds Throughout the United States, developed by the U.S. Weather Bureau.

In Regions 10 and 11, the area of Long Island south of a line from Rockaway Inlet through Port Jefferson, the wind loads may be reduced 10% up to 30 feet (9.15 m) in height and no reduction over 30 feet (9.15 m).

In Regions 3, 4, 5, 6, the area of Region 7 within 20 miles (32.2 km) of Lake Ontario or the St. Lawrence River and Regions 10 and 11, except the area indicated above, the reduction may be 30% up to 30 feet (9.15 m); 24% over 30 feet (9.15 m) to 50 feet (15.25 m); 12% over 50 feet (15.25 m) to 100 feet (30.50 m) and no reduction over 100 feet (30.50 m).

In Regions 1, 2, 8, 9, and 7 (except the area indicated above) the reduction may be 50% up to 30 feet (9.15 m); 46% over 30 feet (9.15 m) to 50 feet (15.25 m); 38% over 50 feet (15.25 m) to 100 feet (30.50 m); 26% over 100 feet (30.50 m) to 300 feet (91.50 m) and 18% over 300 feet (91.50 m).

All heights are measured from the average ground or water surface to the center of the superstructure in the span having the greatest clearance above the ground or water.

3.16

THERMAL FORCES

Add the following paragraphs to Article 3.16:

Regions 10 and 11 may be considered having a moderate climate and the remainder of the State having a cold climate.

For elastomeric bearing design, refer to Article 14.6.6.2.

**3.18 FORCES FROM STREAM CURRENT AND FLOATING ICE,
AND DRIFT CONDITIONS**

3.18.1 Force of Stream Current on Piers

3.18.1.1 Stream Pressure

Add the following as the first two paragraphs:

Based on site conditions and the design high water elevation for a site, parts of both the superstructure and substructure can be subjected to stream and debris forces. Under pressure flow conditions, the lateral forces on the superstructure should be accounted for in design. For steel structures, a need for additional cross-frame bracing and bottom lateral bracing should be investigated. Buoyant forces must also be considered and accounted for.

Additional information to provide a better understanding of the design requirements, is given under AASHTO Commentary to Interim Specifications - Bridges - 1993, Division 1.

3.18.2 Force of Ice on Piers**3.18.2.2 Dynamic Ice Force**

Add the following at the end of Article 3.18.2.2.1:

The following ice thickness criteria shall be used when determining the criteria for equation (3-5). The resulting pressure may be applied to both the superstructure and substructure or to the substructure unit only.

Light Ice	0" to 6"	(0 to 150 mm)
Medium Ice	6" to 1-6"	(150 to 450 mm)
Heavy Ice	1-6" to 2-6"	(450 to 750 mm)
Extreme Ice Condition	>2-6"	(>750 mm)

When it is anticipated that the superstructure will be affected by ice and debris the resulting force shall also be applied to the superstructure.

Unless better information is available, debris force shall be considered same as the ice force.

TABLE 3.22.1A TABLE OF COEFFICIENTS γ AND β

Delete the 1.3 in the Gamma (γ) column for Group Load VII in the Load Factor portion of the table, and replace with 1.0.

Add the following footnote to Table 3.22.1A

Under Group "X", using the Load Factor Design Method, the design of box culverts with a total span length of 20 feet (6096 mm) or less shall be based on the design parameters given in the current New York State Highway Design Manual (Chapter 19).

**3.23 DISTRIBUTION OF LOADS TO STRINGERS, LONGITUDINAL BEAMS,
AND FLOOR BEAMS**

3.23.2 Bending Moment in Stringers and Longitudinal Beams

Add the following sentence to 3.23.2.3.1.1:

Sidewalk liveload may be distributed in the same manner as superimposed Dead Load.

Delete the text of Article 3.23.2.3.1.4 and replace with the following:

For straight stringers, in no case shall a fascia stringer have less carrying capacity than an interior stringer.

3.23.4 Precast Concrete Beams used in Multi-Beam Decks

Add the following at the end of Article 3.23.4.3:

When the structure has a reinforced concrete slab the wheel load fraction as computed above shall not be greater than

- a. 0.67 for 4 foot (1220 mm) wide beams.
- b. 0.50 for 3 foot (915 mm) wide beams.

3.24 DISTRIBUTION OF LOADS AND DESIGN OF CONCRETE SLABS***3.24.3 Bending Moment****3.24.3.1 Case A - Main Reinforcement Perpendicular to Traffic (Spans 2 Feet (610 mm) to 24 Feet (7320 mm) Inclusive)**

Add the following as equation (3-16A):

For HS25 (MS23) loading:

Use 1.25 times the values obtained by the formulae for HS20 (MS18) loading.

3.24.3.2 Case B - Main Reinforcement Parallel to Traffic

Add the following note:

For HS25 (MS23) loading:

Use 1.25 times the values obtained by the formulae for HS20 (MS18) loading.

Add the following to Article 3.24.3 as Article 3.24.3.3:

3.24.3.3 Reinforced Concrete Box Culverts

Reinforced concrete box culverts shall be designed for HS25 (MS23) vehicle live load.

When the minimum depth of fill is less than 2 feet (600 mm), wheel loads shall be distributed in accordance with AASHTO Article 3.24.3.2 Case B and modified as follows:

Wheel loads shall be distributed over a distribution slab width (E), measured in feet (m), equal to $4 + 0.06S$ ($1.22 + 0.06S$), where "S" is the perpendicular distance in feet (m) between wall centerlines.

When the culvert is skewed relative to the over roadway, the distribution width (E), shall be reduced by multiplying "E" by the cosine of the skew angle. In no instance shall the distribution width exceed 7 feet (2.13 m) nor the section length of the precast units.

3.24.3.3 Reinforced Concrete Box Culverts (continued)

For fill heights less than 2 feet (600 mm), the lateral earth pressure shall have added to it a live load surcharge pressure equivalent to 2 feet (600 mm) of earth fill.

When the depth of fill is 2 feet (600mm) or more, wheel loads shall be distributed in accordance with AASHTO Articles 6.4.1 and 6.4.2.

3.24.8 Longitudinal Edge Beams

In Article 3.24.8.2 - Replace "P" with the following:

P = Load in pounds on one rear wheel of the corresponding load truck.

3.24 DISTRIBUTION OF LOADS AND DESIGN OF CONCRETE SLABS

Add the following to Article 3.24 as Article 3.24.11:

3.24.11 Transverse Bridge Deck Slab Reinforcement

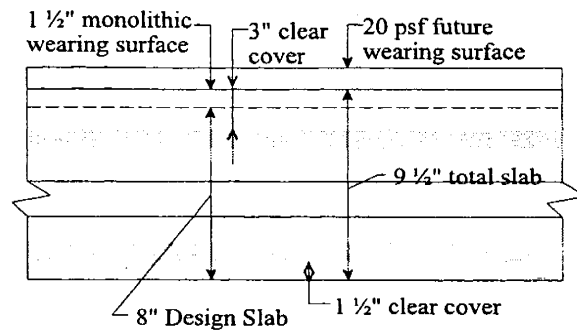
Isotropic reinforcement of bridge decks may be as outlined in the NYSDOT - *Bridge Manual*. Where isotropic reinforcement cannot be used, the transverse reinforcement for traditionally reinforced bridge decks may be as given in Tables 3.24.11A and B. In either case, the overhang reinforcement shall be as follows:

For overhangs 4'-0" (1.22 m) or less, as measured from the centerline of fascia beam web to the deck fascia, the minimum required reinforcement shall be 0.96 in²/ft (2,032 mm²/m).

For overhangs greater than 4'-0" (1.22 m), as measured from the centerline of fascia beam web to the deck fascia, the required reinforcement shall be designed in accordance with the latest version of the AASHTO LRFD Bridge Design Specification.

TABLE 3.24.11 A - SLAB DESIGN

Unit	(ENGLISH)	
Conc weight	150	pcf
Slab thickness(t)	9.5	in
Design depth	8	in
f'_c	3000	psi
f_y	60000	psi
FWS	20	psf
SIP form	4	psf

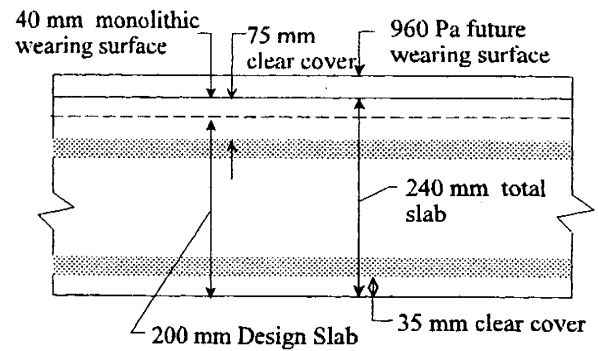


Design Reinforcement perpendicular to traffic			Design Reinforcement parallel to traffic		
Span ft	Bar Size	Bar spacing (in)	Span ft	Bar Size	Bar spacing (in)
6.00	5	9.75	6.00	5	7.00
6.25	5	9.50	6.25	5	6.75
6.50	5	9.25	6.50	5	6.50
6.75	5	8.75	6.75	5	6.25
7.00	5	8.50	7.00	5	6.00
7.25	5	8.25	7.25	5	5.75
7.50	5	8.00	7.50	5	5.50
7.75	5	7.75	7.75	5	5.25
8.00	5	7.50	8.00	5	5.25
8.25	5	7.25	8.25	6	7.25
8.50	5	7.00	8.50	6	7.00
8.75	5	6.75	8.75	6	6.75
9.00	5	6.50	9.00	6	6.50
9.25	5	6.50	9.25	6	6.25
9.50	5	6.25	9.50	6	6.25
10.00	5	6.00	10.00	6	5.75
10.50	6	8.00	10.50	6	5.50
11.00	6	7.75	11.00	6	5.25
11.50	6	7.25	11.50	6	4.75
12.00	6	7.00	12.00	6	4.50
12.50	6	6.50	12.50	6	4.50
13.00	6	6.25	13.00	6	4.25

This reinforcement table was developed using LFD with an AASHTO HS20 Live load. Based upon the conservative assumptions used in traditional slab design criteria, and an acceptable performance history, this table is suitable for use with bridges that are designed for HS25 loading using the LFD or WSD methods, also for bridges designed with the LRFD loading.

TABLE 3.24.11 B - SLAB DESIGN

Unit	(SI)	
Conc weight	25000	kN/m ³
Slab thickness(t)	240.0	mm
Design depth	200.0	mm
f _c	21	MPa
f _y	400	MPa
FWS	960	Pa
SIP form	190	Pa



Design Reinforcement perpendicular to traffic			Design Reinforcement parallel to traffic		
Span m	Bar Size	Bar spacing (mm)	Span m	Bar Size	Bar spacing (mm)
1.80	16	250.00	1.80	16	180.00
1.90	16	240.00	1.90	16	170.00
2.00	16	220.00	2.00	16	160.00
2.10	16	210.00	2.10	16	150.00
2.20	16	200.00	2.20	16	140.00
2.30	16	200.00	2.30	16	140.00
2.40	16	190.00	2.40	16	130.00
2.50	16	180.00	2.50	16	120.00
2.60	16	170.00	2.60	16	120.00
2.70	16	170.00	2.70	19	160.00
2.80	16	160.00	2.80	19	150.00
2.90	16	150.00	2.90	19	150.00
3.00	16	150.00	3.00	19	140.00
3.10	16	140.00	3.10	19	140.00
3.20	16	140.00	3.20	19	130.00
3.30	16	130.00	3.30	19	130.00
3.40	19	180.00	3.40	19	120.00
3.50	19	180.00	3.50	19	120.00
3.60	19	170.00	3.60	19	110.00
3.70	19	160.00	3.70	19	110.00
3.80	19	160.00	3.80	19	110.00
3.90	19	150.00	3.90	19	100.00

4.2 FOUNDATION TYPE AND CAPACITY

4.2.2.1 Bearing Capacity

Add the following paragraph to Article 4.2.2.1:

When considering the Seismic loads (Group VII loading) the maximum bearing capacity of the foundation soil may be taken as twice the allowable bearing capacity.

4.4 SPREAD FOOTINGS

4.4.5 Depth

4.4.5.1 Minimum Embedment and Bench Width

Delete Article 4.4.5.1 and refer to sub-section "Footing Depth", Section 11 of the NYSDOT - *Bridge Manual*.

4.4.5.2 Scour Protection

Add the following paragraph to Article 4.4.5.2:

Consideration shall be given to the possible scour of the footings, for substructures that can be underwater during a large flood (Q100). Anticipated scour depths shall be computed using the prevalent scour formulae with reasonable modifications based on the probability of any adverse effects due to debris or ice.

4.5 DRIVEN PILES

4.5.2.4 Batter Piles

Add the following paragraph to 4.5.2.4 and refer to sub-section "Pile Spacing and Placement Details", Section 11 of the NYSDOT - *Bridge Manual*.

When the boring data indicates that it takes 6 or more blows per foot of a 300 pound (1335N) hammer falling 18 inches or its equivalent energy to drive the casing, the amount of lateral resistance allowed per pile shall be 12,000 pounds (53,400N) for wooden piles, 15,000 pounds (66,750N) for cast-in-place piles and 20,000 pounds (89,000N) for steel bearing piles. When the casing is driven with fewer blows, smaller values shall be used. Sufficient batter piles shall be used so that when the horizontal components of the batter piles are added to the lateral resistance of the piles as given above, the resultant shall not be less than the total lateral or horizontal force acting at the bottom of the footing.

4.5.6.6 Uplift Loads on Piles

Delete the text of 4.5.6.6.1 and replace with the following:

- 4.5.6.6.1** Individual piles may be considered to resist an intermittent but not sustained uplift. Obtain the value of uplift resistance from the DCES.

Delete the text of 4.5.6.6.2 and replace with the following:

- 4.5.6.6.2** Group uplift capacity may be taken as the sum of the uplift resistance of individual piles.

4.5.7 Structural Capacity of Pile Section

4.5.7.1 Load Capacity Requirements

Add to Article 4.5.7.1:

The maximum design load for piles shall be as shown on the Foundation Design Report. However, when considering the seismic loads (Group VII loading), the maximum design load for piles may be taken as twice the allowable pile load unless liquefaction is a design consideration.

4.6 DRILLED SHAFTS

4.6.7.2 Load Testing Procedures

Delete the second sentence in the first paragraph and replace with the following:

Standard (approved) pile load testing procedures shall include:

- FHWA-SA-91-042 Static Testing of Deep Foundations.
- New York State Department of Transportation's current Geotechnical Control Procedure for Static Testing of Deep Foundations.

**5.5 RIGID GRAVITY AND SEMI-GRAVITY
WALL DESIGN**

5.5.2 Earth Pressure and Surcharge Loadings

Add the following to the end of the first paragraph of Article 5.5.2:

Retaining walls can be considered to yield sufficiently to mobilize an active earth stress condition. Therefore, they should be designed for active earth pressures as a method of determining lateral earth forces. The required unit weights and friction angles will be supplied in the Foundation Design Report.

5.5.6 Structure Design**5.5.6.4 Reinforcement**

Delete the first paragraph of Article 5.5.6.4 and refer to sub-section "Temperature and Shrinkage Reinforcement", Section 15 of the NYSDOT - *Bridge Manual*.

7.4 TUBULAR PIERS**7.4.2 Configuration**

Add the following to Article 7.4.2:

The minimum thickness of the metal in the shells of tubular steel piers shall be 5/16 inch (8mm). This thickness shall be increased where necessary to secure strength and rigidity for placing the shell. In all cases, the pier shall be designed for safe pile or soil bearing values as specified herein, but when the diameter required by these values is greater than that required for the superstructure bearing, the diameter may be reduced at any splice point. The minimum diameter of steel cylinders used for piers shall be 30 inches (750mm).

7.5 ABUTMENTS

7.5.2 Loading

Add the following paragraph to 7.5.2

High abutments shall also be investigated for a construction condition where the superstructure is erected before the fill is placed in back of abutment and the construction condition where fill is placed in back of abutment before the superstructure is erected. Since these are temporary stresses, use 150% of the allowable stresses. Passive earth pressure in front of walls shall not be used to counteract overturning or sliding. Passive earth pressure below the footing may only be used on a key below the footing to counteract sliding.

7.5.2.1 Stability

Delete the first bullet item of 7.5.2.1 and replace with the following:

- The Factor of Safety against overturning about the toe of footing shall be 1.5 for footing on rock and 2.0 for footing on soil for permanent loading conditions (1.25 for temporary loading conditions). The Factor of Safety against sliding shall be 1.5 or greater for permanent loading conditions (1.25 for temporary loading conditions).

7.5.2.2 Reinforcement for Temperature

Delete the text of 7.5.2.2 and replace with the following:

In the case of a continuous deck slab sliding over the bridge seat and/or backwall vertical reinforcement at the front face of the abutment stem and/or backwall shall be investigated, but in no case shall the vertical reinforcement be less than No. 5 (#16) bars at 1'-0" (300 mm).

For additional reinforcement requirements refer to sub-section "Temperature and Shrinkage Reinforcement", Section 15 of the NYSDOT *Bridge Manual*.

7.5.6 Wingwalls

Add Article 7.5.6.3 Approach Slabs

7.5.6.3 Approach Slabs

When asphalt concrete approach pavement abuts the abutment backwall and wingwalls, the effect of live load surcharge on the approach fill shall be considered in the design of both the abutment and wingwalls.

However, with reinforced concrete approach slabs resting on the abutment backwall or wingwalls, the effect of live load surcharge on the approach fill need not be considered for the substructure element which supports the approach slab.

8.20 SHRINKAGE AND TEMPERATURE REINFORCEMENT

Add the following sentence at the end of the paragraph to Article 8.20.1:

Refer to sub-section "Temperature and Shrinkage Reinforcement", Section 15 of the NYSDOT - *Bridge Manual*.

8.22 PROTECTION AGAINST CORROSION

Delete the text of Article 8.22.1 and refer to sub-section "Cover", Section 15 of the NYSDOT - *Bridge Manual*.

8.29 DEVELOPMENT OF STANDARD HOOKS IN TENSION

Add the following Article:

8.29.6 Further extension of the straight portion at the free end of a standard hook shall not be considered as additional embedment.

"Standard hooks" are defined in Article 8.23.

9.7.2 Bridges Composed of Simple Span Precast Prestressed Girders Made Continuous

Add the following as second paragraph to Article 9.7.2.1:

9.7.2.1 General

Refer to sub-section "Continuous Design", Section 9 of the NYSDOT - *Bridge Manual*.

9.10.3 Box Girders

9.10.3.2 Delete the text of Article 9.10.3.2 and refer to sub-section “Diaphragms and Transverse Tendons”, Section 9 of the NYSDOT - *Bridge Manual*.

9.13.1 Design Theory and General Considerations

Add Article 9.13.1.5

9.13.1.5 Refer to sub-section "Camber", Section 9 of the NYSDOT - *Bridge Manual*.

9.15 ALLOWABLE STRESSES

- 9.15** Delete the first paragraph of Article 9.15 and refer to sub-section "Concrete Strength", Section 9 of the NYSDOT - *Bridge Manual*.

9.28 EMBEDMENT OF PRESTRESSED STRAND

Add the following as Article 9.28.4:

- 9.28.4** Refer to sub-section "Debonding of Prestressing Strands", Section 9 of the
NYSDOT - *Bridge Manual*.

10.3.1 Allowable Fatigue Stress

Add the following Special Note at the end of Article 10.3.1.

SPECIAL NOTE - LATERAL CONNECTIONS

High strength bolts shall be used for lateral connections. Tension members and tension flanges of flexural members to which lateral gusset plates are attached using high strength bolted connections shall be considered in stress Category "B" for fatigue. Stress shall be computed as outlined in Article 10.18 and 10.19.

FIGURE 10.3.1C ILLUSTRATIVE EXAMPLES

Add the following note to "Figure 10.3.1C Illustrative Examples"

NOTE:

Connection plates or gusset plates attached to tension flanges by groove welds shall always be provided with a transition radius (minimum transition radius = $3/4$ " (19 mm)) and weld ends ground smooth. Therefore, for this type of connection Illustrative Example 16 shall be used. The square corner plates shown in Illustrative Example 15 will not be acceptable for groove weld tension connections.

10.8 MINIMUM THICKNESS OF METAL

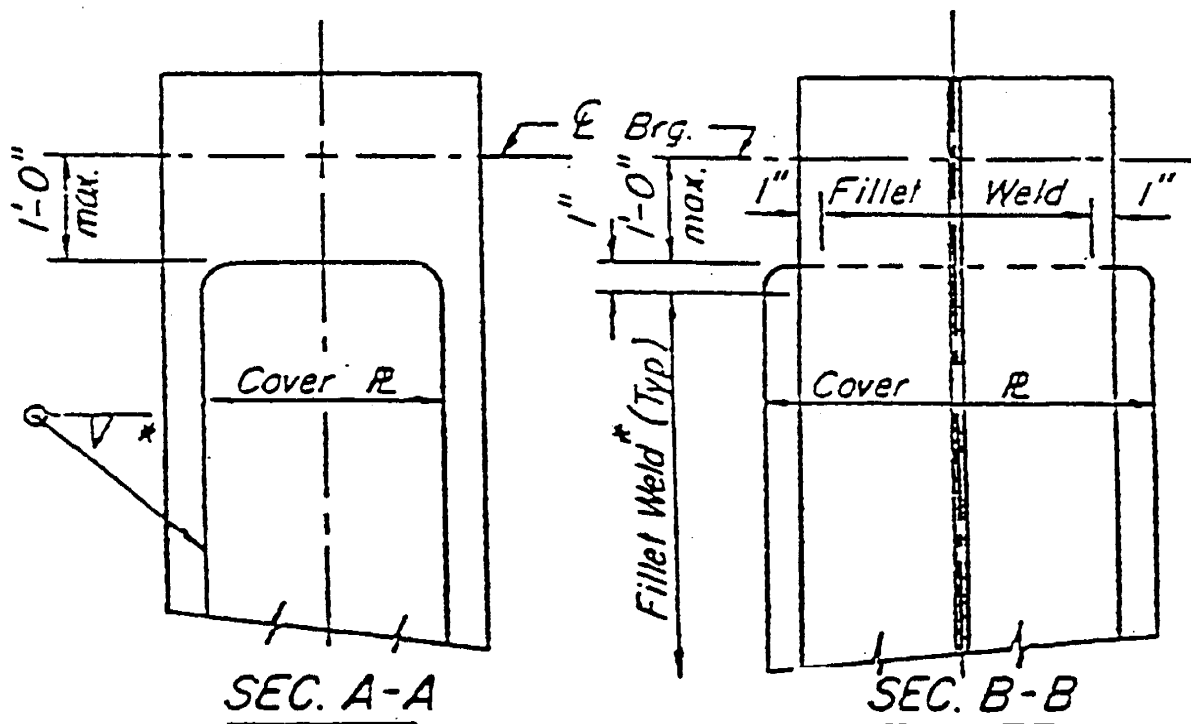
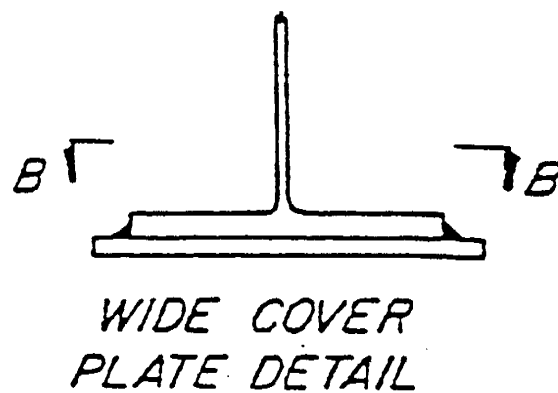
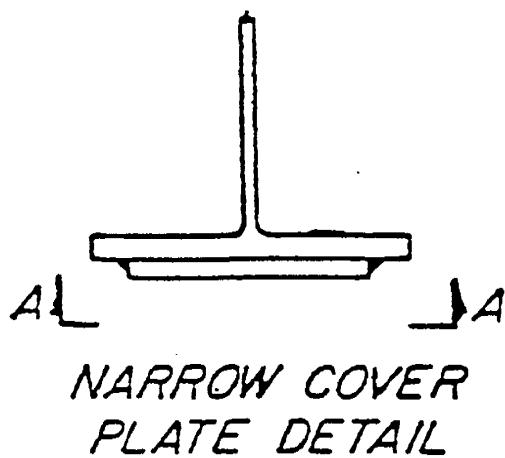
Delete the text of Article 10.8.1 and refer to sub-section "Metal Thicknesses",
Section 8 of the NYSDOT - *Bridge Manual*.

10.13 COVER PLATES

Add the following paragraph as the first paragraph under Article 10.13:

Generally cover plates should be used only on simple span structures. Cover plates welded to rolled beams shall be full length or a partial length using the end bolted detail (as shown in Fig. 10.3.1C, illustration number 22). Welded cover plates shall be limited to one, on any flange. The thickness of the cover plates may be varied by butt welding parts of different thickness with the transitions conforming to the requirements of Article 10.18.5. The plates shall be assembled, welded and radiographed before attachment to the flange. The full length cover plates shall be welded to the flange in accordance with Figure 10.13.

10.13.4 Delete the last three sentences of Article 10.13.4.



* Weld size to be determined by designer

COVER PLATE CONNECTIONS

FIG. 10.13

10.14 CAMBER

Delete the text of 10.14 and replace with the following:

Girders should be cambered to compensate for dead load deflections as specified in Article 10.6 and in addition thereto the camber should be increased and/or decreased for the flanges to parallel the profile grade line when it is on a vertical curve. Generally, a sagging appearance of the lower flange of the girder should be avoided. Generally, camber for fascia girders shall be not less than that provided for interior girders.

The effects of stage construction and constructions loads (e.g. temporary barriers) should be considered in preparing the camber table.

For additional camber requirements, refer to sub-section "Camber", Section 8 of the NYSDOT - *Bridge Manual*.

10.15.3 Camber

Delete the text of Article 10.15.3 and replace with the following:

Compensation for loss of camber pertaining to heat-curved Rolled Beams and Welded Plate girders shall be in accordance with the latest edition of the New York State *Steel Construction Manual*.

10.16 TRUSSES

10.16.4 Diaphragms

Add the following to Article 10.16.4.1.

Diaphragms preferably shall extend the full depth of the chord.

10.16.5 Camber

Add the following to Article 10.16.5

The camber shall also be increased or decreased as necessary when the profile grade is on a vertical curve.

10.18.5 Welding

Delete the text of Article 10.18.5.3 and replace with the following:

Materials of different widths spliced by butt welds shall have transitions conforming to Figure NY 10.18 5A(b). At butt weld splices joining material of different thicknesses there shall be a uniform slope between the offset surfaces of not more than 1 in 2 1/2 with respect to the surface of either part.

In Figure 10 . 18 . 5A Splice details, delete detail (b) , Straight Tapered Transition, and replace with the following Figure NY 10 . 18 . 5A (b).

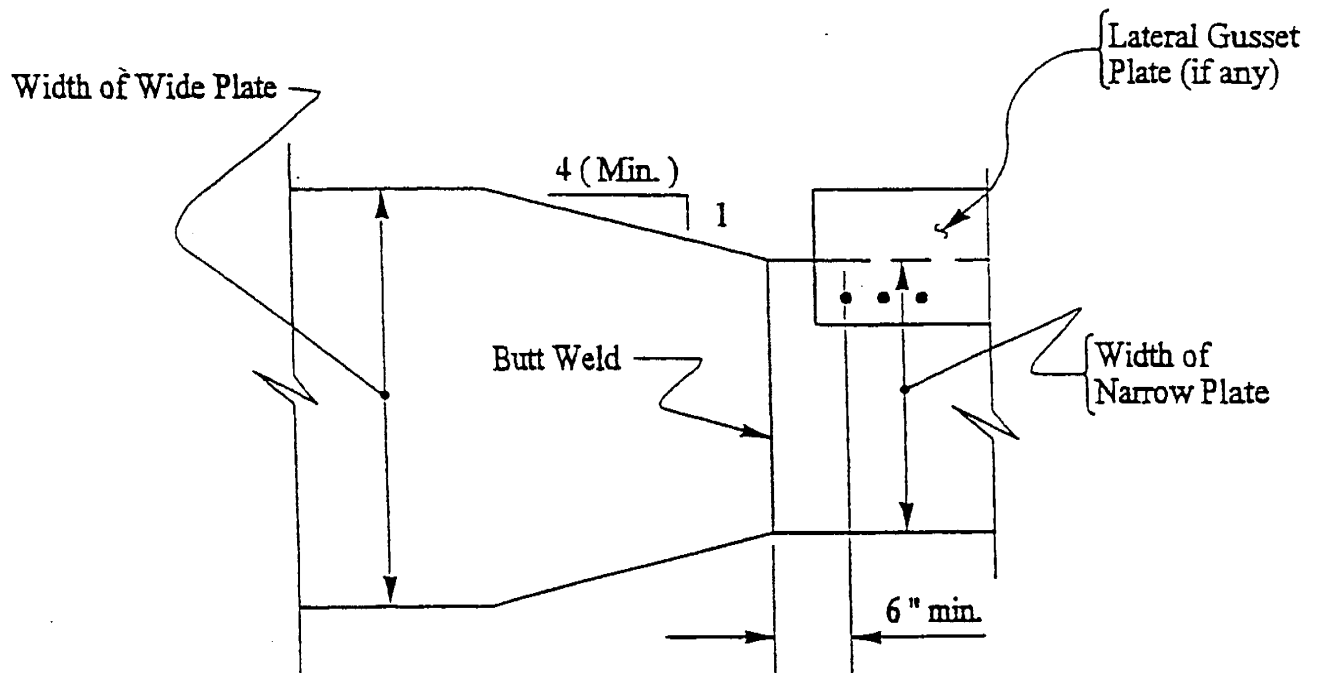
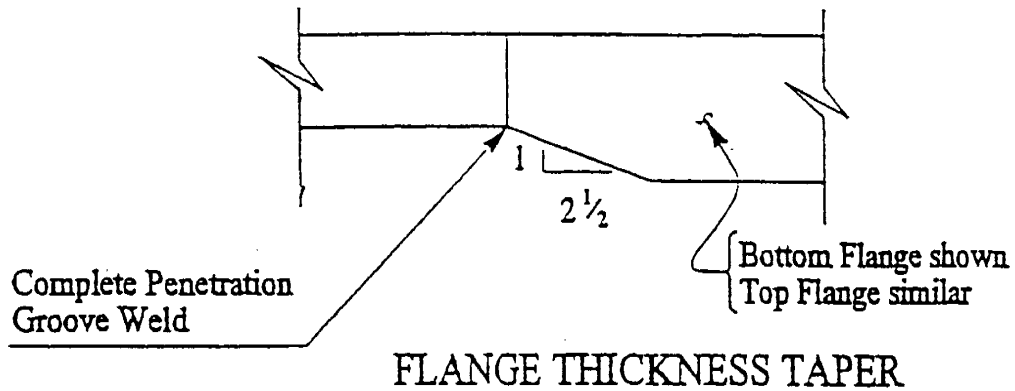


FIGURE NY 10 . 18 . 5A (b)

10.19.2 End Connections of Floorbeams and Stringers.

07/02

Add Article 10.19.2.5

10.19.2.5 Where floorbeams are connected directly to stiffeners, knee braces or connection plates, the floorbeams shall not be coped. The flanges shall be cut and chipped to provide a smooth faying surface as shown in Figure NY 10.19.2.

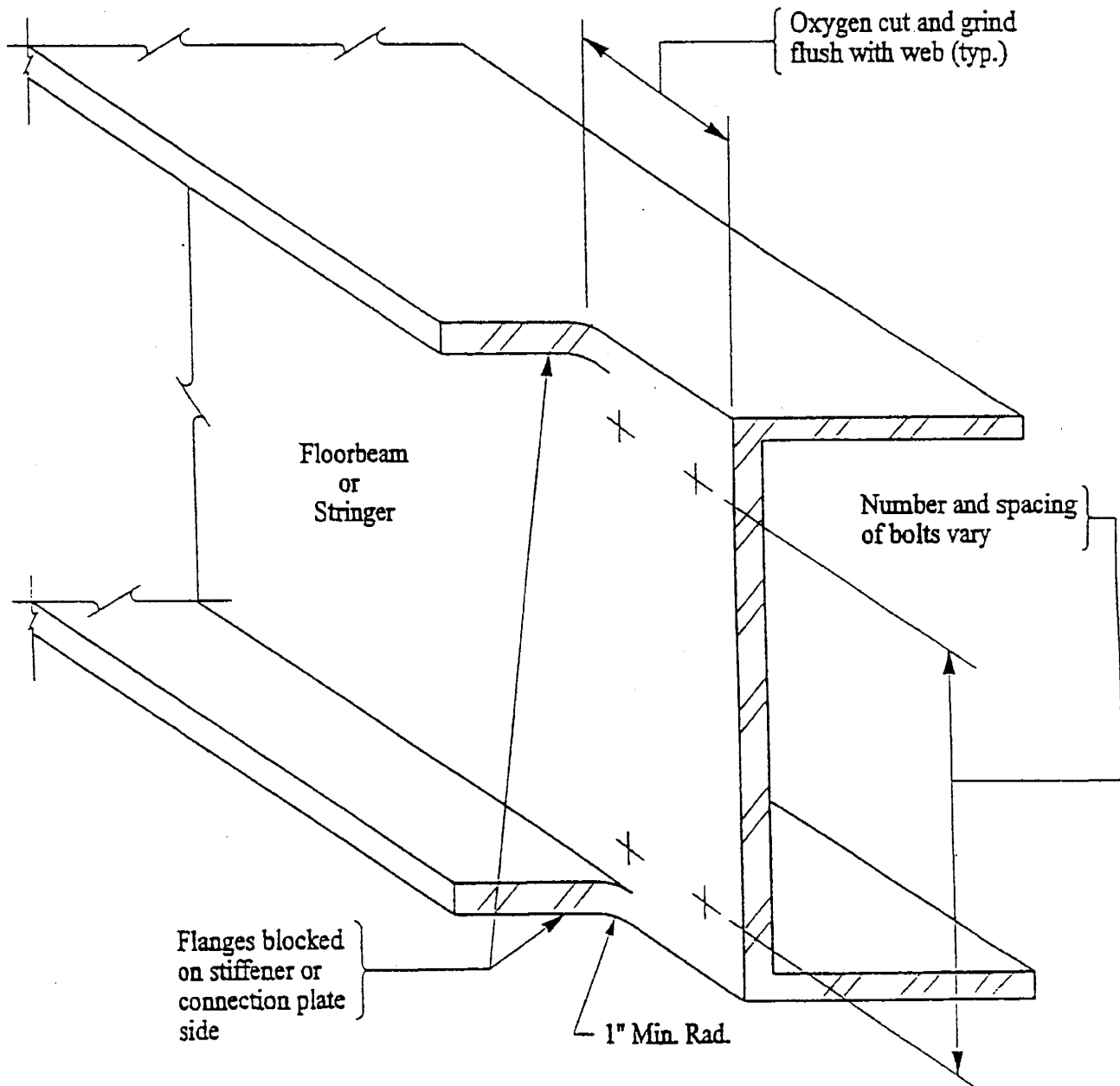


FIG. NY 10.19.2

10.20 DIAPHRAGMS AND CROSS FRAMES**10.20.1 General**

Add the following at the end of Article 10.20.1:

All connection plates for straight girders shall be a minimum thickness of 7/16 inch (11 mm). All connection plates for curved girders shall be a minimum thickness of 9/16 inch (14 mm). Minimum thickness for diaphragms, cross frames and bracing shall conform to Article 10.8. Greater thickness shall be used when required by design.

All outside bay diaphragms for bridges shall be provided with a top strut. This strut is required to prevent torsional displacement of the fascia stringer during concrete deck placement operations*. It therefore must be in place prior to the start of the deck concreting operation.

On existing structures, if the existing diaphragm does not include a top strut, consideration must be given to include one whenever the deck is to be replaced.

*During stage construction, a top strut is required in the bay adjacent to the screed rail support (temporary fascia bay) to prevent top flange displacement.

10.20 DIAPHRAGMS AND CROSS FRAMES (cont'd.)

Delete the text of Article 10.20.3 and replace with the following:

10.20.3 Stresses Due to Wind Loading When Top Flanges are not Continuously Supported.

A horizontal wind force of 50 pounds per square foot (2394N/sq.m) shall be applied to the area of the superstructure exposed in elevation. Half of this force shall be applied in the plane of each flange. The stress induced shall be computed using the summation of transverse bending resistances of all flanges in the loaded plane.

Lateral bracing will be required if the resulting lateral stress, when combined with the stress due to all dead loads and the lateral stress due to curvature, if any, exceeds 125 percent of the allowable stress.

If the lateral bracing is required, it shall be placed in the outside (fascia) bays between diaphragms or crossframes. A sufficient length of flange shall be braced so that the stress computation when repeated for the unbraced length shall result in a flange stress less than 125 percent of the allowable stress. It may be desirable to provide lateral bracing in adjacent interior bays to limit the size of the bracing members. All required lateral bracing shall be placed in or near the plane of the flange being braced. Cross frames or diaphragms shall be placed in all bays.

10.21 LATERAL BRACING

Delete the last two sentences of Article 10.21.2 and add the following paragraphs:

Bottom lateral wind bracing may or may not be provided at the discretion of the Design Engineer providing the stresses in the bottom flanges due to wind loading are accounted for as specified in Articles 10.20.2 or 10.20.3.

Article 10.20.2 shall be used to investigate the need for lateral bracing if the top flanges are continuously supported and the diaphragms are not placed on a skew exceeding 20 degrees. If either of these conditions are not met, Article 10.20.3 shall be used.

When applying Article 10.20.2, horizontal wind forces in accordance with Article 3.15 shall be applied to the area of the superstructure exposed in elevation. Half of the force shall be applied in the plane of each flange. The allowable stress shall be factored in accordance with Article 3.22.

Add the following to the end of Article 10.21.4.

For secondary members the edge of the gusset or connection plate shall be stiffened if the outstanding width of that portion of the plate outside the main member is equal to or greater than the following number of times its thickness:

58 for steel with 36,000 psi Y.P. min.

49 for steel with 50,000 psi Y.P. min.

10.23 WELDING

10.23.1 General

Delete Article 10.23.1.1 and replace with the following:

- 10.23.1.1** Steel base metal to be welded, weld metal, and welding design details shall conform to the provisions of New York State *Steel Construction Manual*.

Delete Article 10.23.1.2 and 10.23.1.3 and replace with the following:

- 10.23.1.2** Fabrication shall conform to the provisions of the New York State *Steel Construction Manual*.

10.23.2 Effective Size of Fillet Welds

Delete Article 10.23.2.2 and replace with the following:

- 10.23.2.2** The minimum size of fillet weld shall be as shown in Table 703.1 of the "New York State *Steel Construction Manual*".

10.24 FASTENERS (RIVETS AND BOLTS)**10.24.1 General**

Delete the text of Article 10.24.1.6 and replace with the following:

- 10.24.1.6** Bolted bearing type connections using high strength bolts shall be limited to members in compression and secondary members, excluding diaphragms and cross-frames.

10.24 FASTENERS (RIVETS AND BOLTS)**10.24.2 Hole Types**

Delete the text of 10.24.2.2 in it's entirety and replace with the following:

- 10.24.2.2** Oversize holes shall be used on secondary members only in accordance with the current New York State *Steel Construction Manual*. Oversize holes will be allowed in cross frames and lateral bracing for curved girders. Slotted holes in cross frames and lateral bracing for straight and curved girders are not allowed unless approved by Deputy Chief Engineer (Structures).

10.24.7 Edge Distance of Fasteners

10.24.7.1 General

Delete the text of Article 10.24.7.1 and refer to the NYSDOT *Steel Construction Manual*.

10.29 FIXED AND EXPANSION BEARINGS

10.29.1 General

Delete the text of Articles 10.29.1.2, 10.29.1.3, 10.29.2 and 10.29.3 and replace with the following:

See Section 14 for appropriate bridge bearings.

TABLE 10.32.1A

The use of High Yield Strength Quenched and Tempered Alloy Steels, ASTM Designations A514, A517 and A852 shall not be permitted. See footnote "c" in TABLE 10.32.1A.

Delete the second line in the TABLE 10.32.1A (cont'd) and replace with the following:

$$F_a = \frac{\pi^2 E}{F.S. (KL/r)^2} = \frac{135,008,740}{(KL/r)^2}$$

TABLE 10.32.3B

Add the following notes to TABLE 10.32.3B:

- NOTE 1: Use ASTM A325 (A325 M) bolts only. Use ASTM A490 (A490 M) bolts only when approved by Deputy Chief Engineer (Structures).
- NOTE 2: Use of oversize and slotted holes shall be in accordance with Article 10.24.2.2.
- NOTE 3: Allowable shear stress for the design of anchor bolts ASTM A449 (A449 M), for masonry plates placed on concrete surfaces shall be 19.0 ksi (131.0 MPa).

TABLE 10.32.3C

Add the following footnote:

All slip-critical connections shall be designed for Class A surface conditions unless otherwise approved by Deputy Chief Engineer (Structures).

10.34 PLATE GIRDERS**10.34.2 Flanges****10.34.2.1 Welded Girders**

Delete the first 2 sentences in paragraph 10.34.2.1.1 and replace with the following:

- 10.34.2.1.1** Each flange may comprise a series of plates joined end to end by complete penetration butt welds. Changes in flange areas may be accomplished by varying the thickness and/or width of adjacent flange plates. The thickness ratio of flange plates at a joint shall be 2 to 1 maximum. The minimum flange thickness shall be 3/4 inch (20 mm). Flange plates may be reduced in width at a butt weld or tapered between butt welds. The minimum flange width shall be 12 inches (300 mm).

10.34.3 Thickness of Web Plates

10.34.3.1 Girders not Stiffened Longitudinally

Change the last line of Article 10.34.3.1.1 to read:

“...but in no case shall the thickness be less than $D/170$ or $3/8$ inch (10mm).

10.34.3 Thickness of Web Plates

10.34.3.2 Girders Stiffened Longitudinally

Change the first sentence of Article 10.34.3.2.1. to read:

"...but in no case shall the thickness be less than $D/340$ or 1/2 inch (12 mm),..."

10.34.4 Transverse Intermediate Stiffeners

Add the following at the end of Article 10.34.4.2:

If transverse stiffeners are required, the maximum spacing shall be one half of the diaphragm spacing.

10.34.5 Longitudinal Stiffeners

Delete the text of Article 10.34.5.4 and refer to sub-section “Longitudinal Stiffeners”, Section 8 of the NYSDOT - *Bridge Manual*.

10.34.6 Bearing Stiffeners

10.34.6.1 Welded Girders

Add the following at the end of 10.34.6.1:

For additional information refer to sub-section "Bearing Stiffeners", Section 8 of the NYSDOT - *Bridge Manual*.

10.34 PLATE GIRDERS

Add the following at the end as Article 10.34.7

10.34.7 Stability During Erection

Girders shall be checked for stability during erection by analyzing the buckling stress in the compression flange.

The equation for buckling stress is:

$$f_{cr} = \frac{E}{L} \left[\left[\frac{.65A_f}{D} \right]^2 + \left[\frac{5W_t^2}{\left[\frac{A_w}{A_f} + 6 \right] L} \right]^2 \right]^{1/2}$$

where

- f_{cr} = Critical buckling stress (kips/sq in.) (MPa)
- E = Modulus of Elasticity of steel (kips/sq. in.) (MPa)
- W_t = Width of compression flange (inches) (mm)
- D = Total depth of girder (inches) (mm)
- A_w = Area of web (sq. in.) (mm²)
- A_f = Area of compression flange (sq. in.) (mm²)
- L = Unsupported length of compression flange (inches) (mm)

The critical section shall be taken at the point of maximum moment due to steel weight and span length.

The Factor of Safety provided against buckling shall be 1.1 minimum.

$$F.S. = (f_{cr}/fb) \geq 1.1$$

where fb = Compression stress due to bending (ksi) [Mpa]

For more information on performing the stability check refer to sub-section "Stability During Erection", Section 8 of the NYSDOT - *Bridge Manual*.

10.38 COMPOSITE GIRDERS**10.38.2 Shear Connectors**

Delete paragraph 10.38.2.1 and replace with the following:

The mechanical means which are used at the junction of the girder and slab for the purpose of developing the shear resistance necessary to produce composite action shall conform to the current requirements for stud shear connectors of the New York State Department of Transportation Standard Specifications. The shear connectors shall be 3/4 inch (20 mm) diameter stud shear connectors having a minimum length of 4 inch (100 mm). Channel shear connectors shall not be used unless approved by the Deputy Chief Engineer (Structures).

10.38.4 Stresses

Delete the text of Article 10.38.4.3 and replace with the following:

In the negative moment regions of continuous spans, the minimum longitudinal reinforcement including the longitudinal distribution reinforcement must equal or exceed 1.0% of the cross sectional area of the concrete slab. The area of the concrete slab shall be taken as equal to the design slab thickness times the entire width of the bridge deck.

The required additional longitudinal reinforcement shall be No. 19 bars or smaller spaced at 300 mm or less. Two-thirds of this required additional reinforcement shall be placed in the top mat of reinforcement. One-third of the required additional shall be placed in the bottom mat of reinforcement. The required additional longitudinal reinforcement shall, at a minimum, extend beyond the extreme limit of tension stress in the top flange a distance equal to the development length of the bar.

10.46 DESIGN STRESS FOR STRUCTURAL STEEL

Add the following paragraphs at the end of Article 10.46:

The unsupported compression flange of straight girders shall be checked for lateral buckling during erection in accordance with the procedure outlined in Article 10.34.7.

For additional information refer to sub-section "Flanges", Section 8 of the NYSDOT - *Bridge Manual*.

10.48.8 Shear

Delete the first sentence of the first paragraph of Article 10.48.8.3 and replace with the following:

Where transverse intermediate stiffeners are required, transverse stiffeners shall be spaced at a distance, d_o , according to shear capacity as specified in Article 10.48.8.1, but not more than $3D$ or half the diaphragm space, whichever controls.

10.56 SPLICES, CONNECTIONS AND DETAILS**10.56.1 Connectors****10.56.1.1 General**

Delete the text of 10.56.1.1 and replace with the following:

Connectors shall be proportioned so that their maximum strength multiplied by the reduction factor, ϕ shall be least equal to the effects of design loads multiplied by their respective load factors specified in Article 10.47. The maximum strengths multiplied by the reduction factors are listed in Table 10.56A.

Add the following as Article 10.62:

10.62

LOAD RATING

Inventory rating and Operating rating of bridge structures shall be computed using the provisions of the AASHTO Manual for Condition Evaluation of Bridges, 1994 with current revisions.

**WORKING STRESS DESIGN CRITERIA FOR
CURVED STEEL I-GIRDER BRIDGES**

10.100 GENERAL

The specifications of this section pertain to bridge superstructures composed of steel members which are curved in plan. It is applicable to simple and continuous span, composite or noncomposite bridges of moderate length, employing either rolled or fabricated I-shaped steel sections. The design of these members shall be based on unit working stresses. The provisions of Division I, Design shall govern where applicable, except as specifically modified by the following Articles 10.101 through 10.121.

10.101 NOTATION

- A_f = area of one flange of beam or girder (in.²)
- A_s = total area of steel section, including cover plates (in.²)
- A_s^r = total of longitudinal reinforcing steel at the interior support within the effective flange width (in.²)
- A_t = area of the tension flange (in.²)
- A_w = area of web of beam (in.²)
- b = compression flange width
- b = effective flange width (Comp. Girder) given in Article 10.38.3(in.)
- b' = width of outstanding stiffener element (in.)
- c = thickness of concrete slab (in.)
- D = the unsupported distance between flange components (in.)
- d = required distance between transverse stiffeners (in.)
- d_o = actual distance between transverse stiffeners (in.)
- E = modulus of elasticity (psi)
- F = maximum slab force acting on the shear connectors due to curvature (lb.)
- F_b = maximum allowable compressive stress in extreme fibers of rolled shapes, girders, and built-up sections subject to bending (psi)

- F_v = maximum allowable shear stress (psi)
 F_y = specified minimum yield point or yield strength for the type of steel being used (psi)
 F_y^r = specified minimum yield point of the reinforcing steel (psi)
 F_{yf} = specified minimum yield strength of flange (psi)
 F_{yw} = specified minimum yield strength of web (psi)
 f_b = calculated bending stress (psi)
 f_c' = compressive strength of concrete at age of 28 days (psi)
 f_v = the average calculated unit shearing stress in the gross section of the web plate at the point considered (psi)
 f_w = calculated normal stress at the edge of the flange due to nonuniform torsion (lateral flange bending) for plate girders (psi)
 I = impact factor
 I = moment of inertia (in.⁴)
 I_w = warping constant (in.⁶)
 J = the required ratio of rigidity of one transverse stiffener to that of the web plate
 K = modification factor
 K_T = torsional constant (in.⁴)
 L = span length (ft.)
 l = length of the unsupported compression flange between cross frames or diaphragms (in.)
 N = number of shear connectors between points of maximum positive moment and adjacent end supports or dead load points of contraflexure, or between points of maximum negative moment and adjacent dead load points of contraflexure
 N_s = number of shear connectors at a section
 n = modular ratio of elasticity of steel to that of concrete

- P = maximum tangential slab force acting on the shear connectors (lb.)
- P_c = maximum resultant slab force acting on the shear connectors (lb.) as defined in Article 10.38.
- R = radius of curvature of the girder web (in.)
- R = reduction factor
- R_c = radius of centerline of bridge (ft.)
- r = radius of gyration of stiffener (in.)
- r' = radius of gyration of the compression flange about axis in the plane of the web (in.)
- S_u = ultimate strength of a shear connector (lb.) as given in Article 10.38.5.1.2
- t = thickness (in.)
- X = curvature correction factor for transverse stiffener requirements
- Y_o = distance from the neutral axis to the extreme outer fiber (maximum distance for nonsymmetrical sections) (in.)
- Z = curvature parameter
- α = the minimum specified yield strength of the web divided by the minimum specified yield strength of the tension flange
- α' = α times a correction factor based on the ratio of lateral flange bending to bending stress
- β = area of web divided by area of tension flange
- Δ = amount of camber (in.)
- Δ_{DL} = camber at any point along the span calculated by usual procedures to compensate for deflection due to dead loads or any other specified loads (in.)
- Δ_m = maximum value of Δ_{DL} within the span calculated by usual procedures to compensate for deflection due to dead loads or any other specified loads (in.)
- θ = angle subtended between the points of maximum moment (positive or negative) and adjacent point of contraflexure or support (radians)

- $\rho_B \rho_W$ = curvature correction factors for allowable compressive bending stress
- ϕ = reduction factor=0.85
- Ψ = ratio of total cross-sectional area to the cross-sectional area of both flanges
- ψ = distance from outer edge of tension flange to the neutral axis (of the transformed section for composite girders) divided by the depth of the steel section of Hybrid Beam.

10.102 LOADS

The loads shall be in accordance with Section 3 except as modified in Article 10.103.

A. Uplift

In addition to analyzing curved structures for moments and shears, a condition of live loading shall be investigated that would produce a minimum reaction. If the load that produces minimum reaction is one where more than two lanes of live loads are on the bridge, the reaction should be computed without the reduction for the extra lanes as specified in Article 3.12. The provisions of Article 3.17 and 10.29.6 shall apply.

Investigation shall also be made to insure that uplift movement will not occur due to the sequence of placing the concrete deck.

B Impact

The impact loads shall be in accordance with Article 3.8.2

C Superelevation and Centrifugal Forces

The effect of superelevation and centrifugal forces shall be taken into account when determining the distribution of an axle load to the individual wheels. Values of the centrifugal forces and their application shall be in accordance with Article 3.10

D Thermal Forces

When temperature movements (as specified in Articles 10.105 and 10.106) are allowed to occur, no allowances need to be made in superstructure design for thermal forces.

10.103 DESIGN THEORY

A. General

The moments, shears, and other forces required to proportion the individual members shall be based on an analysis of the entire structure which takes into account the complete distribution of loads to the various members.

The influence of torsion and the stresses resulting therefrom must be included in the design. If the rational analysis considers the system as a plane grid and not a space frame and bottom lateral bracing is specified, the resulting maximum live load stresses can be modified in accordance with the provisions stated in Section E. When the radius is such that the central angle subtended by each span is less than the values given Table 10.103A the effects of the curvature may be neglected in determining the primary bending moments in the longitudinal members.

TABLE 10.103A-Limiting Central Angle for Neglecting Curvature in Determining Primary Bending Moments

<u>Number of Girders</u>	<u>Angle for 1 Span</u>	<u>Angle/Span for 2 or more spans continuous</u>
2	2°	3°
3 or 4	3°	4°
5 or more	4°	5°

B Torsion

Intermediate transverse diaphragms or cross frames must be provided between the longitudinal members for the purpose of distributing the internal torsion at any cross section to the individual members.

C Nonuniform Torsion

Analysis shall be based on any rational method which takes into account the normal stresses developed in the curved longitudinal members due to nonuniform torsion (also known as lateral flange bending). This effect shall be included in the design of all curved bridges, even those with central angles less than the limits given in Article 10.103(A).

D Composite Design

Curved bridges may be designed with the concrete deck acting compositely with the steel members provided the shear connectors are designed in accordance with Article 10.118.

E Load Distribution

If a single or continuous span I girder bridge system is composite and has bottom lateral wind bracing in at least two bays, the resulting maximum live load (normal bending + warping) bottom flange stress in the outside girder (f_{be}), computed using a plane grid analysis can be computed from the following:

$$f_{be} = f_{ube} \times (DF)_b$$

where: $(DF)_b$ = Distribution Factor bottom flange

f_{ube} = Maximum live load flange stress in the outside exterior girder based on grid analysis.

When all bays have bottom lateral bracing,

$$(DF)_b = \frac{3.0 - 0.06(L)}{(SG)^{3/2}} (L/R) + 0.9$$

When bottom lateral is in every other bay,

$$(DF)_b = \frac{3.0 - 0.06(L)}{(SG)^{3/2}} (L/R) + 0.95$$

The maximum stress in the inside exterior girder (f_{bi}) can be computed from:

$$f_{bi} = f_{ube} \times (DF)_b \times M$$

where:

$M = -0.366(L/R) + 0.944$ - when all bays are braced

$M = -0.473(L/R) + 0.934$ - for bracing in every other bay

SG = Girder Spacing (ft); R = Radius of Curvature of outside Exterior Girder (ft); L = Outside Exterior Girder Span Length (ft)

The distribution of stresses in the interior girders can be assumed proportional between the f_{be} and f_{bi} values.

The above expressions are valid for radial or skewed supported bridges, span lengths from 80' to 300', girder spacing from 8' to 14', a minimum radius of 300', central angle less than 57° and a maximum of six girders.

10.104 FATIGUE

The requirements of Article 10.3 concerning fatigue stresses shall apply to curved steel bridges. Due consideration should be given to the evaluation of stresses in curved members to insure that accurate stress magnitudes are obtained at connection details. The calculated stress shall include the contribution from torsion.

10.105 EXPANSION AND CONTRACTION

In applying the provisions of Article 10.11 thermal movements must be allowed in directions radiating from the fixed supports. In general, these directions will not be tangential to the center lines of girders at supports and joints.

10.106 BEARINGS

Bearings shall be designed as a system to resist forces resulting from horizontal loads as well as vertical loads and simultaneously to permit horizontal movements within the superstructure resulting from temperature changes. Each bearing must not prevent angular rotation in a tangential vertical plane.

If uplift occurs under the loading conditions described in Article 10.102(A), suitable hold downs shall be provided to overcome this action.

All sliding type expansion bearings shall be provided with guiding system unless otherwise approved by DCEs. Provisions of Articles 10.11 and 10.29.6 and Section 14 shall also apply.

10.107 DIAPHRAGMS, CROSS FRAMES, AND LATERAL BRACING**(A) Diaphragms and Cross Frames**

In addition to the applicable provisions of Article 10.20 and 10.21 the provisions of this Article shall apply.

Cross frames or diaphragms shall be provided at each support and at intermediate intervals between supports with spacings as determined by design considerations.

Each line of diaphragms or cross frames shall extend in a single plane across the width of the bridge with diaphragms or cross frames included between all longitudinal girders. Diaphragms or cross frames at interior skew supports need not be located along the line of support.

The diaphragms or cross frames shall be full depth members designed as main structural elements to distribute torsional forces to the longitudinal girders. The cross frames and the flanges of diaphragms as well as lateral bracing members shall be framed in

such a way to transfer the horizontal and vertical forces to the flanges and web as necessary.

The diaphragm or cross frame plates attached to the girder web shall be connected to flange(s) as well in a manner that will prevent distortion of the web at each end of the connection plate. All connection plates shall be coped for a length of 4 to 6 times the web thickness from the near edge of the longitudinal weld at points of intersection with longitudinal weldments.

(B) Lateral Bracing

When bottom lateral wind bracing is specified, for curved composite girders the maximum stress in the bottom laterals can be computed from the following:

$$f_b = f_d \chi (DF)_{bl}$$

where:

f_d = Maximum stress in cross frames as determined from the grid solution.

$(DF)_{bl}$ = Distribution Factor for bottom laterals.

$$(DF)_{bl} = 2.5 \left[\frac{-SD + (0.16L-10)}{(SG)^{4/3}} \right] (L/R) + ((L/160)+0.652)$$

SD = The distance between bracing at the outside exterior girder and L, SG, R are as defined previously.

When steel girders are fabricated curved, although designed as straight, the attachment details for the lateral bracing gusset plate to the girder flange shall be the standard details for curved girders. When steel girders are fabricated straight, although the roadway alignment is curved, the attachment details for the lateral bracing gusset plate to the girder flange may be the standard details for straight girders.

CURVED STEEL I - GIRDERS

10.108 - GENERAL

The provisions in Articles 10.109 to 10.113 are not applicable to riveted or bolted girders. Curved beams and girders shall be proportioned to resist normal stress due to bending and nonuniform torsion (also known as lateral flange bending). The normal stress due to bending shall be determined using the moment of inertia method. The normal stress due to nonuniform torsion shall be determined by any rational method of analysis. For design against fatigue, a stress analysis shall be performed using a rational method of analysis capable of accurately determining stresses under combined bending and nonuniform torsion.

Therefore, consideration of the lateral shears in estimating the full load capacity of the member is not required. The effect of the eccentricity of the load on the bending moment capacity need only be considered as previously given.

10.109 - ALLOWABLE FLANGE NORMAL STRESS

(A) Compression

The ratio of compression flange width to thickness shall not exceed the value determined by the formula:

$$\frac{b}{t} = \frac{4400}{(F_y)^{1/2}}$$

The flexural stress (f_b) in the compression flange shall not exceed the allowable compressive stress, F_b , as determined by:

$$F_b = 0.55 F_y \left[1 - \frac{(\ell/r')^2 F_y}{4\pi^2 E} \right] \rho_B \rho_w \quad (a)$$

Where:

$$\rho_B = \frac{1}{1 + (\ell/R)(\ell/b)} \quad (b)$$

$$\rho_w = \frac{1}{1 - (f_w/f_b) \left[1 - \frac{(\ell/b)}{75} \right]} \quad (c)$$

$$\text{or } \rho_w = \frac{0.95 + \frac{\ell/b}{30 + 8000(0.1 - \ell/R)^2}}{1 + 0.6(f_w/f_b)} \quad (d)$$

and ℓ = length of the unsupported compression flange between cross frames or diaphragms (in.)

R = radius of curvature of the girder web (in.)

$r' = \left[\frac{b^2}{12} \right]^{1/2}$ = radius of gyration of the compression flange about axis in plane of web (in.)

b = Width of compression flange (in.)

$\frac{f_w}{f_b}$ = ratio at the diaphragm location of the flange tip stress caused by nonuniform torsion (lateral flange bending) to the calculated bending stress in the flange: the ratio shall be taken as positive if f_w is compression on the flange tip further away from the center of curvature and negative if f_w is compression on the flange tip closer to the center of curvature.

If f_w/f_b is positive, the smaller value for ρ_w given by Eqs. (c) and (d) shall be used in Eq. (a).

If f_w/f_b is negative, Eq. (c) shall be used for ρ_w in Eq. (a). In no case shall the maximum total stress ($f_w + f_b$) at the compression flange tip exceed $0.55F_y$.

The allowable stress in Eq. (a) shall be computed using the larger of the two bending moments at either end of the braced flange and the corresponding value of f_w at that location.

The following additional controls shall apply:

The absolute value of f_w/f_b shall be less than or equal to 0.5 at the point of maximum bending stress;

l/b shall be less than or equal to 25; and
 l/R shall be less than or equal to 0.1

Values of the product $\rho_b \rho_w$ determined from Eqs. (b), (c) and (d) are given in Table 10.109.4.B. Values of the Allowable Compression flange stresses are given in TABLE 10.109.A. When the value of l/R lies between the two values listed in the tables, it is recommended to use the equations (a), (b), (c) and (d) to actually compute the allowable compressive stress (F_b). Since interpolation will give incorrect results.

B. Tension

The total stress ($f_w + f_b$) at the tension flange tip shall not exceed $0.55 F_y$.

TABLE 10.109A

ALLOWABLE COMPRESSION FLANGE STRESS - (50,000 PSI YIELD STRESS)

l/R	f_w/f_b	l/B = (LENGTH BETWEEN DIAPH./FLANGE WIDTH)						
		6	8	10	12	14	16	18
0.008	0.000	25.75	24.98	24.13	23.20	22.19	21.11	19.96
0.008	0.050	25.28	25.02	24.65	24.16	23.13	21.97	20.75
0.008	0.100	24.57	24.32	23.95	23.48	22.89	22.18	21.35
0.008	0.150	23.89	23.65	23.29	22.83	22.26	21.57	20.77
0.008	0.200	23.25	23.01	22.67	22.22	21.66	20.99	20.21
0.008	0.250	22.64	22.41	22.08	21.64	21.10	20.44	19.68
0.008	0.300	22.07	21.84	21.52	21.09	20.56	19.92	19.18
0.008	0.350	21.52	21.30	20.98	20.57	20.05	19.43	18.71
0.008	0.400	21.00	20.79	20.48	20.07	19.56	18.96	18.25
0.008	0.450	20.50	20.30	19.99	19.60	19.10	18.51	17.82
0.008	0.500	20.03	19.83	19.53	19.14	18.66	18.08	17.41
0.010	0.000	25.45	24.61	23.69	22.70	21.64	20.53	19.35
0.010	0.050	25.04	24.71	24.28	23.70	22.56	21.37	20.11
0.010	0.100	24.33	24.01	23.59	23.06	22.41	21.66	20.81
0.010	0.150	23.66	23.35	22.94	22.42	21.80	21.07	20.23
0.010	0.200	23.03	22.73	22.33	21.82	21.21	20.50	19.69
0.010	0.250	22.43	22.13	21.74	21.25	20.66	19.97	19.18
0.010	0.300	21.86	21.57	21.19	20.71	20.13	19.46	18.69
0.010	0.350	21.32	21.04	20.66	20.20	19.64	18.98	18.23
0.010	0.400	20.80	20.53	20.16	19.71	19.16	18.52	17.79
0.010	0.450	20.31	20.04	19.69	19.24	18.71	18.08	17.37
0.010	0.500	19.84	19.58	19.23	18.80	18.28	17.66	16.96
0.012	0.000	25.17	24.25	23.27	22.22	21.13	19.98	18.78
0.012	0.050	24.81	24.41	23.92	23.20	22.02	20.79	19.52
0.012	0.100	24.11	23.72	23.24	22.65	21.97	21.18	20.29
0.012	0.150	23.44	23.07	22.60	22.03	21.36	20.60	19.74
0.012	0.200	22.82	22.45	21.99	21.44	20.79	20.05	19.21
0.012	0.250	22.22	21.87	21.42	20.88	20.25	19.52	18.71
0.012	0.300	21.65	21.31	20.88	20.35	19.73	19.03	18.23
0.012	0.350	21.12	20.78	20.36	19.85	19.24	18.56	17.78
0.012	0.400	20.61	20.28	19.87	19.37	18.78	18.11	17.35
0.012	0.450	20.12	19.80	19.40	18.91	18.34	17.68	16.94
0.012	0.500	19.66	19.34	18.95	18.47	17.91	17.27	16.55
0.014	0.000	24.89	23.90	22.86	21.77	20.63	19.45	18.24
0.014	0.050	24.58	24.13	23.57	22.72	21.51	20.25	18.96
0.014	0.100	23.89	23.44	22.90	22.27	21.55	20.73	19.74
0.014	0.150	23.23	22.80	22.27	21.66	20.95	20.16	19.27
0.014	0.200	22.61	22.19	21.68	21.08	20.39	19.62	18.75
0.014	0.250	22.02	21.61	21.11	20.53	19.86	19.11	18.26
0.014	0.300	21.46	21.06	20.58	20.01	19.36	18.62	17.80
0.014	0.350	20.93	20.54	20.07	19.51	18.88	18.16	17.36
0.014	0.400	20.42	20.04	19.58	19.04	18.42	17.72	16.94
0.014	0.450	19.94	19.57	19.12	18.59	17.98	17.30	16.54
0.014	0.500	19.48	19.12	18.68	18.16	17.57	16.90	16.16

TABLE 10.109A
ALLOWABLE COMPRESSION FLANGE STRESS - (50,000 PSI YIELD STRESS)

t/R	f_w/f_b	$l/B = (\text{LENGTH BETWEEN DIAPH.}/\text{FLANGE WIDTH})$						
		6	8	10	12	14	16	18
0.016	0.000	24.62	23.56	22.46	21.33	20.16	18.96	17.73
0.016	0.050	24.36	23.85	23.24	22.26	21.01	19.73	18.43
0.016	0.100	23.68	23.17	22.58	21.91	21.15	20.30	19.18
0.016	0.150	23.02	22.54	21.96	21.31	20.57	19.74	18.83
0.016	0.200	22.41	21.93	21.37	20.74	20.01	19.21	18.33
0.016	0.250	21.82	21.36	20.82	20.19	19.49	18.71	17.85
0.016	0.300	21.27	20.82	20.29	19.68	19.00	18.24	17.40
0.016	0.350	20.74	20.30	19.79	19.19	18.53	17.78	16.97
0.016	0.400	20.24	19.81	19.31	18.73	18.08	17.35	16.56
0.016	0.450	19.76	19.34	18.85	18.29	17.65	16.94	16.17
0.016	0.500	19.30	18.90	18.42	17.86	17.24	16.55	15.79
0.018	0.000	24.35	23.23	22.08	20.91	19.71	18.49	17.24
0.018	0.050	24.15	23.58	22.93	21.83	20.54	19.24	17.92
0.018	0.100	23.47	22.91	22.28	21.56	20.77	19.90	18.66
0.018	0.150	22.82	22.28	21.67	20.97	20.20	19.35	18.43
0.018	0.200	22.21	21.69	21.08	20.41	19.66	18.83	17.93
0.018	0.250	21.63	21.12	20.53	19.88	19.14	18.34	17.47
0.018	0.300	21.08	20.58	20.01	19.37	18.66	17.87	17.02
0.018	0.350	20.56	20.07	19.52	18.89	18.20	17.43	16.60
0.018	0.400	20.06	19.59	19.04	18.43	17.75	17.01	16.20
0.018	0.450	19.59	19.12	18.59	18.00	17.34	16.61	15.82
0.018	0.500	19.14	18.68	18.17	17.58	16.94	16.22	15.45
0.020	0.000	24.09	22.91	21.72	20.50	19.28	18.04	16.79
0.020	0.050	23.95	23.32	22.63	21.40	20.09	18.78	17.45
0.020	0.100	23.27	22.66	21.99	21.23	20.41	19.52	18.17
0.020	0.150	22.63	22.04	21.38	20.65	19.85	18.98	18.04
0.020	0.200	22.02	21.45	20.81	20.10	19.32	18.47	17.56
0.020	0.250	21.45	20.89	20.26	19.57	18.82	17.99	17.10
0.020	0.300	20.90	20.36	19.75	19.08	18.34	17.53	16.67
0.020	0.350	20.39	19.85	19.26	18.60	17.88	17.10	16.26
0.020	0.400	19.89	19.37	18.79	18.15	17.45	16.69	15.86
0.020	0.450	19.42	18.92	18.35	17.72	17.04	16.29	15.49
0.020	0.500	18.97	18.48	17.93	17.31	16.64	15.92	15.13
0.022	0.000	23.83	22.60	21.36	20.11	18.86	17.61	16.35
0.022	0.050	23.75	23.08	22.33	21.00	19.66	18.33	17.00
0.022	0.100	23.08	22.42	21.70	20.92	20.07	19.12	17.70
0.022	0.150	22.44	21.81	21.11	20.35	19.52	18.64	17.69
0.022	0.200	21.84	21.22	20.54	19.80	19.00	18.14	17.21
0.022	0.250	21.27	20.67	20.01	19.28	18.50	17.66	16.76
0.022	0.300	20.73	20.14	19.50	18.79	18.03	17.21	16.34
0.022	0.350	20.22	19.64	19.01	18.33	17.59	16.79	15.93
0.022	0.400	19.73	19.17	18.55	17.88	17.16	16.38	15.55
0.022	0.450	19.26	18.72	18.12	17.46	16.75	15.99	15.18
0.022	0.500	18.82	18.28	17.70	17.06	16.37	15.62	14.83

TABLE 10.109A

ALLOWABLE COMPRESSION FLANGE STRESS - (50,000 PSI YIELD STRESS)

l/R	f_w/f_b	l/B = (LENGTH BETWEEN DIAPH./FLANGE WIDTH)						
		6	8	10	12	14	16	18
0.024	0.000	23.58	22.30	21.02	19.74	18.47	17.20	15.94
0.024	0.050	23.56	22.84	21.97	20.61	19.25	17.91	16.57
0.024	0.100	22.89	22.19	21.44	20.62	19.75	18.67	17.25
0.024	0.150	22.26	21.58	20.85	20.06	19.21	18.31	17.35
0.024	0.200	21.66	21.00	20.29	19.52	18.70	17.82	16.89
0.024	0.250	21.10	20.45	19.76	19.01	18.21	17.35	16.44
0.024	0.300	20.56	19.93	19.26	18.53	17.74	16.91	16.03
0.024	0.350	20.05	19.44	18.78	18.07	17.30	16.49	15.63
0.024	0.400	19.57	18.97	18.32	17.63	16.89	16.09	15.25
0.024	0.450	19.10	18.52	17.89	17.21	16.49	15.71	14.89
0.024	0.500	18.66	18.09	17.48	16.82	16.11	15.35	14.55
0.026	0.000	23.34	22.00	20.68	19.38	18.09	16.82	15.55
0.026	0.050	23.37	22.61	21.62	20.23	18.86	17.50	16.17
0.026	0.100	22.71	21.97	21.18	20.34	19.45	18.25	16.83
0.026	0.150	22.08	21.36	20.60	19.78	18.91	18.00	17.03
0.026	0.200	21.49	20.79	20.04	19.25	18.41	17.52	16.58
0.026	0.250	20.93	20.25	19.52	18.75	17.93	17.06	16.15
0.026	0.300	20.40	19.73	19.03	18.27	17.47	16.63	15.73
0.026	0.350	19.89	19.24	18.55	17.82	17.04	16.21	15.34
0.026	0.400	19.41	18.78	18.10	17.39	16.63	15.82	14.97
0.026	0.450	18.95	18.34	17.68	16.98	16.23	15.45	14.62
0.026	0.500	18.52	17.91	17.27	16.58	15.86	15.09	14.28
0.028	0.000	23.10	21.71	20.36	19.03	17.73	16.44	15.18
0.028	0.050	23.19	22.39	21.28	19.86	18.48	17.12	15.78
0.028	0.100	22.53	21.75	20.93	20.07	19.16	17.85	16.43
0.028	0.150	21.91	21.15	20.36	19.52	18.64	17.71	16.74
0.028	0.200	21.33	20.59	19.81	18.99	18.14	17.23	16.29
0.028	0.250	20.77	20.05	19.29	18.50	17.66	16.78	15.86
0.028	0.300	20.24	19.54	18.80	18.03	17.21	16.36	15.46
0.028	0.350	19.74	19.06	18.34	17.58	16.79	15.95	15.08
0.028	0.400	19.26	18.60	17.89	17.16	16.38	15.57	14.71
0.028	0.450	18.81	18.16	17.47	16.75	15.99	15.20	14.37
0.028	0.500	18.37	17.74	17.07	16.36	15.62	14.85	14.03
0.030	0.000	22.87	21.43	20.05	18.69	17.38	16.09	14.82
0.030	0.050	23.01	22.17	20.95	19.51	18.11	16.75	15.41
0.030	0.100	22.36	21.55	20.70	19.81	18.89	17.46	16.04
0.030	0.150	21.75	20.95	20.13	19.27	18.37	17.43	16.46
0.030	0.200	21.16	20.39	19.59	18.75	17.88	16.97	16.02
0.030	0.250	20.16	19.86	19.08	18.26	17.41	16.52	15.60
0.030	0.300	20.09	19.36	18.59	17.80	16.97	16.10	15.20
0.030	0.350	19.59	18.88	18.13	17.36	16.55	15.71	14.83
0.030	0.400	19.12	18.42	17.69	16.94	16.15	15.33	14.47
0.030	0.450	18.67	17.98	17.28	16.54	15.77	14.96	14.13
0.030	0.500	18.23	17.57	16.88	16.16	15.40	14.62	13.80

TABLE 10.109A

ALLOWABLE COMPRESSION FLANGE STRESS - (50,000 PSI YIELD STRESS)

l/R	f_w/f_b	l/B = (LENGTH BETWEEN DIAPH./FLANGE WIDTH)						
		6	8	10	12	14	16	18
0.032	0.000	22.64	21.16	19.74	18.37	17.04	15.75	14.49
0.032	0.050	22.85	21.97	20.64	19.18	17.76	16.39	15.06
0.032	0.100	22.20	21.35	20.47	19.57	18.55	17.09	15.68
0.032	0.150	21.59	20.76	19.91	19.03	18.12	17.18	16.20
0.032	0.200	21.01	20.20	19.38	18.52	17.63	16.72	15.76
0.032	0.250	20.46	19.68	18.87	18.04	17.17	16.28	15.35
0.032	0.300	19.94	19.18	18.39	17.58	16.74	15.87	14.96
0.032	0.350	19.45	18.70	17.93	17.14	16.32	15.47	14.59
0.032	0.400	18.98	18.25	17.50	16.73	15.93	15.10	14.24
0.032	0.450	18.53	17.82	17.09	16.33	15.55	14.74	13.90
0.032	0.500	18.10	17.41	16.69	15.96	15.19	14.40	13.58
0.034	0.000	22.41	20.89	19.45	18.06	16.72	15.42	14.16
0.034	0.050	22.68	21.77	20.33	18.85	17.43	16.05	14.72
0.034	0.100	22.04	21.16	20.26	19.34	18.20	16.74	15.33
0.034	0.150	21.34	20.58	19.70	18.80	17.88	16.93	15.95
0.034	0.200	20.86	20.02	19.17	18.30	17.40	16.48	15.52
0.034	0.250	20.32	19.50	18.67	17.82	16.95	16.05	15.12
0.034	0.300	19.80	19.01	18.20	17.37	16.52	15.64	14.73
0.034	0.350	19.31	18.53	17.75	16.94	16.11	15.25	14.37
0.034	0.400	18.84	18.09	17.32	16.53	15.72	14.88	14.02
0.034	0.450	18.40	17.66	16.91	16.14	15.35	14.53	13.69
0.034	0.500	17.97	17.25	16.52	15.77	14.99	14.20	13.37
0.036	0.000	22.19	20.63	19.16	17.75	16.41	15.11	13.85
0.036	0.050	22.52	21.59	20.03	18.53	17.10	15.73	14.40
0.036	0.100	21.89	20.97	20.05	19.11	17.86	16.40	14.99
0.036	0.150	21.28	20.40	19.50	18.59	17.66	16.70	15.64
0.036	0.200	20.71	19.85	18.98	18.09	17.18	16.25	15.30
0.036	0.250	20.17	19.33	18.48	17.62	16.74	15.83	14.90
0.036	0.300	19.66	18.84	18.01	17.17	16.31	15.43	14.52
0.036	0.350	19.17	18.37	17.57	16.74	15.91	15.04	14.16
0.036	0.400	18.71	17.93	17.14	16.34	15.52	14.68	13.82
0.036	0.450	18.27	17.51	16.74	15.95	15.15	14.33	13.49
0.036	0.500	17.85	17.10	16.35	15.59	14.80	14.00	13.18
0.038	0.000	21.97	20.38	18.88	17.46	16.11	14.81	13.56
0.038	0.050	22.37	21.33	19.74	18.23	16.79	15.41	14.09
0.038	0.100	21.74	20.80	19.86	18.90	17.53	16.07	14.67
0.038	0.150	21.14	20.23	19.31	18.38	17.44	16.48	15.30
0.038	0.200	20.57	19.68	18.79	17.89	16.98	16.04	15.09
0.038	0.250	20.04	19.17	18.30	17.42	16.53	15.62	14.69
0.038	0.300	19.53	18.68	17.84	16.98	16.11	15.23	14.32
0.038	0.350	19.04	18.22	17.39	16.56	15.71	14.85	13.96
0.038	0.400	18.58	17.78	16.97	16.16	15.33	14.49	13.63
0.038	0.450	18.14	17.36	16.57	15.78	14.97	14.15	13.30
0.038	0.500	17.73	16.96	16.19	15.41	14.63	13.82	13.00

TABLE 10.109A

ALLOWABLE COMPRESSION FLANGE STRESS - (50,000 PSI YIELD STRESS)

l/R	f_w/f_b	l/B = (LENGTH BETWEEN DIAPH./FLANGE WIDTH)						
		6	8	10	12	14	16	18
0.040	0.000	21.76	20.13	18.61	17.18	15.82	14.52	13.27
0.040	0.050	22.22	21.08	19.46	17.93	16.49	15.11	13.80
0.040	0.100	21.60	20.63	19.67	18.70	17.22	15.76	14.37
0.040	0.150	21.00	20.06	19.13	18.19	17.24	16.28	14.98
0.040	0.200	20.44	19.52	18.61	17.70	16.78	15.84	14.89
0.040	0.250	19.91	19.01	18.13	17.24	16.34	15.43	14.50
0.040	0.300	19.40	18.53	17.67	16.80	15.93	15.04	14.13
0.040	0.350	18.92	18.07	17.23	16.38	15.53	14.66	13.78
0.040	0.400	18.46	17.63	16.81	15.99	15.16	14.31	13.45
0.040	0.450	18.02	17.22	16.42	15.61	14.80	13.97	13.13
0.040	0.500	17.61	16.82	16.04	15.25	14.46	13.65	12.83
0.042	0.000	21.55	19.89	18.35	16.90	15.54	14.24	13.00
0.042	0.050	22.08	20.82	19.18	17.65	16.20	14.82	13.51
0.042	0.100	21.46	20.47	19.49	18.45	16.91	15.46	14.07
0.042	0.150	20.87	19.90	18.95	18.00	17.05	16.08	14.67
0.042	0.200	20.31	19.37	18.44	17.52	16.59	15.65	14.70
0.042	0.250	19.78	18.87	17.96	17.06	16.16	15.25	14.32
0.042	0.300	19.28	18.39	17.51	16.63	15.75	14.86	13.95
0.042	0.350	18.80	17.93	17.07	16.22	15.36	14.49	13.61
0.042	0.400	18.34	17.50	16.66	15.83	14.99	14.14	13.28
0.042	0.450	17.91	17.08	16.27	15.45	14.63	13.80	12.96
0.042	0.500	17.50	16.69	15.89	15.10	14.30	13.49	12.66
0.044	0.000	21.35	19.66	18.10	16.64	15.27	13.97	12.74
0.044	0.050	21.95	20.58	18.92	17.37	15.92	14.55	13.24
0.044	0.100	21.23	20.31	19.32	18.17	16.62	15.17	13.79
0.044	0.150	20.74	19.75	18.79	17.83	16.87	15.84	14.38
0.044	0.200	20.18	19.22	18.28	17.35	16.42	15.48	14.52
0.044	0.250	19.66	18.72	17.81	16.90	15.99	15.07	14.14
0.044	0.300	19.16	18.25	17.35	16.47	15.58	14.69	13.78
0.044	0.350	18.68	17.79	16.92	16.06	15.20	14.32	13.44
0.044	0.400	18.23	17.36	16.51	15.67	14.83	13.98	13.12
0.044	0.450	17.80	16.95	16.12	15.30	14.48	13.65	12.81
0.044	0.500	17.39	16.56	15.75	14.95	14.14	13.33	12.51
0.046	0.000	21.15	19.43	17.85	16.38	15.01	13.72	12.49
0.046	0.050	21.81	20.34	18.66	17.10	15.65	14.28	12.98
0.046	0.100	21.20	20.16	19.15	17.88	16.34	14.89	13.52
0.046	0.150	20.61	19.61	18.63	17.66	16.70	15.55	14.10
0.046	0.200	20.06	19.08	18.13	17.19	16.25	15.31	14.36
0.046	0.250	19.54	18.58	17.65	16.74	15.83	14.91	13.98
0.046	0.300	19.04	18.11	17.21	16.31	15.42	14.53	13.63
0.046	0.350	18.57	17.66	16.78	15.91	15.04	14.17	13.29
0.046	0.400	18.12	17.23	16.37	15.52	14.68	13.83	12.97
0.046	0.450	17.69	16.83	15.99	15.16	14.33	13.50	12.66
0.046	0.500	17.28	16.44	15.62	14.81	14.00	13.19	12.37

TABLE 10.109A

ALLOWABLE COMPRESSION FLANGE STRESS - (50,000 PSI YIELD STRESS)

t/R	f_w/f_b	$l/B = (\text{LENGTH BETWEEN DIAPH.}/\text{FLANGE WIDTH})$						
		6	8	10	12	14	16	18
0.048	0.000	20.95	19.20	17.61	16.13	14.76	13.47	12.25
0.048	0.050	21.68	20.10	18.40	16.84	15.38	14.02	12.73
0.048	0.100	21.07	20.02	19.00	17.61	16.06	14.62	13.26
0.048	0.150	20.49	19.47	18.47	17.50	16.53	15.27	13.82
0.048	0.200	19.94	18.95	17.98	17.03	16.09	15.15	14.20
0.048	0.250	19.42	19.45	17.51	16.59	15.67	14.75	13.83
0.048	0.300	18.93	17.98	17.07	16.17	15.27	14.38	13.48
0.048	0.350	18.46	17.54	16.64	15.76	14.89	14.02	13.15
0.048	0.400	18.01	17.11	16.24	15.38	14.53	13.68	12.83
0.048	0.450	17.59	16.71	15.86	15.02	14.19	13.36	12.52
0.048	0.500	17.18	16.32	15.49	14.67	13.86	13.05	12.24
0.050	0.000	20.75	18.98	17.37	15.89	14.51	13.23	12.02
0.050	0.050	21.56	19.87	18.16	16.59	15.13	13.77	12.49
0.050	0.100	20.95	19.88	18.85	17.35	15.80	14.36	13.00
0.050	0.150	20.37	19.33	18.33	17.35	16.38	15.00	13.56
0.050	0.200	19.83	18.81	17.84	16.88	15.94	15.00	14.05
0.050	0.250	19.31	18.32	17.37	16.44	15.52	14.61	13.69
0.050	0.300	18.82	17.86	16.93	16.03	15.13	14.24	13.34
0.050	0.350	18.35	17.42	16.51	15.63	14.75	13.88	13.01
0.050	0.400	17.91	16.99	16.11	15.25	14.40	13.55	12.69
0.050	0.450	17.49	16.59	15.73	14.89	14.06	13.23	12.39
0.050	0.500	17.08	16.21	15.37	14.55	13.73	12.92	12.11

TABLE 10.109A

ALLOWABLE COMPRESSION FLANGE STRESS - (36,000 PSI YIELD STRESS)

t/R	f_w/f_b	$t/B = (\text{LENGTH BETWEEN DIAPH.}/\text{FLANGE WIDTH})$						
		6	8	10	12	14	16	18
0.008	0.000	18.64	18.16	17.64	17.08	16.49	15.86	15.19
0.008	0.050	18.30	18.19	18.02	17.79	17.19	16.51	15.79
0.008	0.100	17.78	17.68	17.51	17.29	17.01	16.66	16.26
0.008	0.150	17.29	17.19	17.03	16.81	16.54	16.20	15.81
0.008	0.200	16.83	16.73	16.58	16.36	16.10	15.77	15.38
0.008	0.250	16.39	16.29	16.14	15.94	15.68	15.36	14.98
0.008	0.300	15.97	15.88	15.73	15.53	15.28	14.97	14.60
0.008	0.350	15.58	15.49	15.34	15.15	14.90	14.60	14.24
0.008	0.400	15.20	15.11	14.97	14.78	14.54	14.24	13.90
0.008	0.450	14.84	14.75	14.62	14.43	14.19	13.91	13.57
0.008	0.500	14.50	14.41	14.28	14.10	13.87	13.59	13.25
0.010	0.000	18.43	17.89	17.32	16.72	16.08	15.42	14.73
0.010	0.050	18.13	17.97	17.75	17.45	16.77	16.05	15.31
0.010	0.100	17.61	17.46	17.25	16.98	16.66	16.28	15.84
0.010	0.150	17.13	16.98	16.77	16.51	16.20	15.83	15.40
0.010	0.200	16.67	16.52	16.32	16.07	15.76	15.40	14.99
0.010	0.250	16.24	16.09	15.90	15.65	15.35	15.00	14.60
0.010	0.300	15.82	15.68	15.49	15.25	14.96	14.62	14.23
0.010	0.350	15.43	15.29	15.11	14.87	14.59	14.26	13.87
0.010	0.400	15.06	14.92	14.74	14.51	14.24	13.91	13.54
0.010	0.450	14.70	14.57	14.40	14.17	13.90	13.58	13.22
0.010	0.500	14.36	14.24	14.06	13.84	13.58	13.27	12.91
0.012	0.000	18.22	17.63	17.01	16.37	15.70	15.01	14.29
0.012	0.050	17.96	17.75	17.49	17.08	16.36	15.62	14.86
0.012	0.100	17.45	17.25	16.99	16.68	16.32	15.91	15.45
0.012	0.150	16.97	16.77	16.52	16.22	15.87	15.47	15.02
0.012	0.200	16.52	16.32	16.08	15.79	15.45	15.06	14.62
0.012	0.250	16.08	15.90	15.66	15.38	15.05	14.67	14.24
0.012	0.300	15.68	15.49	15.26	14.99	14.66	14.29	13.88
0.012	0.350	15.29	15.11	14.89	14.62	14.30	13.94	13.53
0.012	0.400	14.92	14.74	14.52	14.26	13.95	13.60	13.21
0.012	0.450	14.56	14.40	14.18	13.93	13.62	13.28	12.89
0.012	0.500	14.23	14.06	13.85	13.60	13.31	12.97	12.60
0.014	0.000	18.02	17.38	16.71	16.03	15.33	14.61	13.88
0.014	0.050	17.80	17.54	17.23	16.73	15.98	15.21	14.43
0.014	0.100	17.29	17.04	16.75	16.40	16.01	15.57	15.02
0.014	0.150	16.82	16.57	16.29	15.95	15.57	15.14	14.67
0.014	0.200	16.37	16.13	15.85	15.52	15.15	14.74	14.28
0.014	0.250	15.94	15.71	15.44	15.12	14.76	14.35	13.90
0.014	0.300	15.53	15.31	15.04	14.73	14.38	13.99	13.55
0.014	0.350	15.15	14.93	14.67	14.37	14.03	13.64	13.21
0.014	0.400	14.78	14.57	14.32	14.02	13.69	13.31	12.89
0.014	0.450	14.43	14.23	13.98	13.69	13.36	13.00	12.59
0.014	0.500	14.10	13.90	13.66	13.37	13.05	12.70	12.30

TABLE 10.109A

ALLOWABLE COMPRESSION FLANGE STRESS - (36,000 PSI YIELD STRESS)

l/R	f_w/f_b	l/B = (LENGTH BETWEEN DIAPH./FLANGE WIDTH)						
		6	8	10	12	14	16	18
0.016	0.000	17.82	17.13	16.42	15.71	14.98	14.24	13.49
0.016	0.050	17.64	17.34	16.99	16.40	15.62	14.82	14.03
0.016	0.100	17.14	16.85	16.51	16.41	15.71	15.25	14.60
0.016	0.150	16.67	16.38	16.06	15.69	15.28	14.83	14.34
0.016	0.200	16.22	15.94	15.63	15.27	14.87	14.43	13.95
0.016	0.250	15.80	15.53	15.22	14.87	14.48	14.06	13.59
0.016	0.300	15.40	15.13	14.83	14.49	14.12	13.70	13.24
0.016	0.350	15.01	14.76	14.47	14.13	13.77	13.36	12.92
0.016	0.400	14.65	14.40	14.12	13.79	13.43	13.04	12.60
0.016	0.450	14.30	14.06	13.78	13.47	13.12	12.73	12.31
0.016	0.500	13.97	13.74	13.46	13.16	12.81	12.43	12.02
0.018	0.000	17.63	16.89	16.15	15.40	14.65	13.89	13.13
0.018	0.050	17.48	17.14	16.76	16.07	15.27	14.46	13.64
0.018	0.100	16.99	16.66	16.29	15.88	15.43	14.95	14.21
0.018	0.150	16.52	16.20	15.84	15.44	15.01	14.54	14.03
0.018	0.200	16.08	15.77	15.42	15.03	14.61	14.15	13.65
0.018	0.250	15.66	15.35	15.01	14.64	14.23	13.78	13.30
0.018	0.300	15.26	14.96	14.63	14.27	13.86	13.43	12.96
0.018	0.350	14.88	14.59	14.27	13.91	13.52	13.10	12.64
0.018	0.400	14.52	14.24	13.92	13.58	13.19	12.78	12.33
0.018	0.450	14.18	13.90	13.60	13.25	12.88	12.48	12.04
0.018	0.500	13.85	13.58	13.28	12.95	12.58	12.19	11.76
0.020	0.000	17.44	16.66	15.88	15.10	14.32	13.55	12.78
0.020	0.050	17.34	16.96	16.54	15.76	14.93	14.11	13.28
0.020	0.100	16.84	16.48	16.07	15.64	15.17	14.66	13.83
0.020	0.150	16.38	16.02	15.63	15.21	14.75	14.26	13.74
0.020	0.200	15.94	15.59	15.21	14.80	14.36	13.88	13.37
0.020	0.250	15.53	15.19	14.82	14.41	13.98	13.52	13.02
0.020	0.300	15.13	14.80	14.44	14.05	13.63	13.17	12.69
0.020	0.350	14.76	14.43	14.08	13.70	13.29	12.85	12.37
0.020	0.400	14.40	14.08	13.74	13.37	12.97	12.54	12.07
0.020	0.450	14.06	13.75	13.42	13.05	12.66	12.24	11.79
0.020	0.500	13.73	13.43	13.11	12.75	12.37	11.96	11.52
0.022	0.000	17.25	16.43	15.62	14.81	14.02	13.23	12.45
0.022	0.050	17.19	16.78	16.32	15.46	14.61	13.77	12.94
0.022	0.100	16.70	16.30	15.87	15.41	14.92	14.36	13.47
0.022	0.150	16.24	15.85	15.43	14.98	14.51	14.00	13.46
0.022	0.200	15.81	15.43	15.02	14.58	14.12	13.62	13.10
0.022	0.250	15.40	15.03	14.63	14.20	13.75	13.27	12.76
0.022	0.300	15.01	14.64	14.26	13.84	13.40	12.93	12.44
0.022	0.350	14.63	14.28	13.90	13.50	13.07	12.61	12.13
0.022	0.400	14.28	13.93	13.57	13.17	12.75	12.31	11.83
0.022	0.450	13.94	13.61	13.25	12.86	12.45	12.02	11.56
0.022	0.500	13.62	13.29	12.94	12.56	12.16	11.74	11.29

TABLE 10.109A

ALLOWABLE COMPRESSION FLANGE STRESS - (36,000 PSI YIELD STRESS)

l/R	f_w/f_b	$l/B = (\text{LENGTH BETWEEN DIAPH.}/\text{FLANGE WIDTH})$						
		6	8	10	12	14	16	18
0.024	0.000	17.07	16.21	15.37	14.54	13.72	12.92	12.14
0.024	0.050	17.05	16.60	16.06	15.17	14.31	13.45	12.62
0.024	0.100	16.57	16.13	15.67	15.19	14.68	14.03	13.13
0.024	0.150	16.11	15.69	15.24	14.77	14.27	13.75	13.21
0.024	0.200	15.68	15.27	14.83	14.37	13.89	13.39	12.85
0.024	0.250	15.27	14.87	14.45	14.00	13.53	13.04	12.52
0.024	0.300	14.88	14.49	14.08	13.64	13.19	12.70	12.20
0.024	0.350	14.51	14.13	13.73	13.31	12.86	12.39	11.90
0.024	0.400	14.16	13.79	13.40	12.98	12.55	12.09	11.61
0.024	0.450	13.83	13.47	13.08	12.68	12.25	11.80	11.34
0.024	0.500	13.51	13.15	12.78	12.38	11.97	11.53	11.07
0.026	0.000	16.90	15.99	15.12	14.27	13.44	12.63	11.84
0.026	0.050	16.92	16.44	15.81	14.90	14.01	13.15	12.31
0.026	0.100	16.44	15.97	15.48	14.98	14.45	13.71	12.81
0.026	0.150	15.99	15.53	15.06	14.57	14.06	13.52	12.97
0.026	0.200	15.56	15.12	14.66	14.18	13.68	13.16	12.62
0.026	0.250	15.15	14.72	14.27	13.81	13.32	12.82	12.29
0.026	0.300	14.77	14.35	13.91	13.46	12.98	12.49	11.98
0.026	0.350	14.40	13.99	13.57	13.12	12.66	12.18	11.68
0.026	0.400	14.05	13.65	13.24	12.80	12.36	11.89	11.40
0.026	0.450	13.72	13.33	12.92	12.50	12.06	11.61	11.13
0.026	0.500	13.40	13.02	12.63	12.21	11.78	11.34	10.87
0.028	0.000	16.72	15.79	14.89	14.02	13.17	12.35	11.56
0.028	0.050	16.79	16.28	15.56	14.63	13.73	12.86	12.01
0.028	0.100	16.31	15.81	15.31	14.78	14.24	13.41	12.51
0.028	0.150	15.86	15.38	14.88	14.37	13.85	13.30	12.74
0.028	0.200	15.44	14.97	14.49	13.99	13.48	12.95	12.40
0.028	0.250	15.03	14.58	14.11	13.62	13.12	12.61	12.08
0.028	0.300	14.65	14.21	13.75	13.28	12.79	12.29	11.77
0.028	0.350	14.29	13.85	13.41	12.95	12.47	11.98	11.48
0.028	0.400	13.94	13.52	13.08	12.64	12.17	11.69	11.20
0.028	0.450	13.61	13.20	12.77	12.34	11.88	11.42	10.94
0.028	0.500	13.30	12.89	12.48	12.05	11.61	11.15	10.68
0.030	0.000	16.55	15.58	14.66	13.77	12.91	12.09	11.29
0.030	0.050	16.66	16.12	15.32	14.37	13.46	12.58	11.73
0.030	0.100	16.19	15.66	15.13	14.59	14.04	13.12	12.21
0.030	0.150	15.74	15.23	14.72	14.19	13.65	13.10	12.53
0.030	0.200	15.32	14.83	14.32	13.81	13.28	12.75	12.19
0.030	0.250	14.92	14.44	13.95	13.45	12.94	12.41	11.88
0.030	0.300	14.54	14.07	13.59	13.11	12.61	12.10	11.57
0.030	0.350	14.18	13.72	13.26	12.78	12.30	11.80	11.29
0.030	0.400	13.84	13.39	12.94	12.47	12.00	11.51	11.01
0.030	0.450	13.51	13.07	12.63	12.18	11.72	11.24	10.75
0.030	0.500	13.20	12.77	12.34	11.90	11.45	10.98	10.50

TABLE 10.109A

ALLOWABLE COMPRESSION FLANGE STRESS - (36,000 PSI YIELD STRESS)

t/R	f_w/f_d	t/B = (LENGTH BETWEEN DIAPH./FLANGE WIDTH)						
		6	8	10	12	14	16	18
0.032	0.000	16.39	15.38	14.43	13.53	12.66	11.83	11.03
0.032	0.050	16.54	15.97	15.09	14.12	13.20	12.31	11.46
0.032	0.100	16.07	15.52	14.97	14.41	13.78	12.84	11.93
0.032	0.150	15.63	15.09	14.56	14.01	13.46	12.90	12.33
0.032	0.200	15.21	14.69	14.17	13.64	13.10	12.56	12.00
0.032	0.250	14.81	14.31	13.80	13.28	12.76	12.23	11.69
0.032	0.300	14.44	13.94	13.45	12.95	12.44	11.92	11.39
0.032	0.350	14.08	13.60	13.11	12.62	12.13	11.62	11.11
0.032	0.400	13.74	13.27	12.80	12.32	11.84	11.34	10.84
0.032	0.450	13.41	12.95	12.49	12.03	11.56	22.07	10.58
0.032	0.500	13.10	12.66	12.21	11.75	11.29	10.82	10.34
0.034	0.000	16.22	15.19	14.22	13.30	12.42	11.59	10.78
0.034	0.050	16.42	15.83	14.86	13.88	12.95	12.06	11.21
0.034	0.100	15.95	15.38	14.81	14.24	13.52	12.57	11.67
0.034	0.150	15.52	14.96	14.40	13.85	13.29	12.72	12.14
0.034	0.200	15.10	14.56	14.02	13.48	12.93	12.38	11.82
0.034	0.250	14.71	14.18	13.65	13.13	12.59	12.06	11.51
0.034	0.300	14.33	13.82	13.31	12.79	12.27	11.75	11.22
0.034	0.350	13.98	13.47	12.98	12.47	11.97	11.46	10.94
0.034	0.400	13.64	13.15	12.66	12.17	11.68	11.18	10.67
0.034	0.450	13.32	12.84	12.36	11.89	11.40	10.92	10.42
0.034	0.500	13.01	12.54	12.08	11.61	11.14	10.66	10.18
0.036	0.000	16.06	15.00	14.01	13.08	12.19	11.35	10.55
0.036	0.050	16.30	15.69	15.64	13.65	12.71	11.81	10.96
0.036	0.100	15.84	15.25	14.66	14.08	13.27	12.32	11.41
0.036	0.150	15.41	14.83	14.26	13.69	13.12	12.55	11.90
0.036	0.200	14.99	14.43	13.88	13.32	12.77	12.21	11.65
0.036	0.250	14.60	14.06	13.51	12.98	12.44	11.89	11.34
0.036	0.300	14.23	13.70	13.17	12.65	12.12	11.59	11.05
0.036	0.350	13.88	13.36	12.84	12.33	11.82	11.30	10.78
0.036	0.400	13.54	13.03	12.53	12.03	11.53	11.03	10.52
0.036	0.450	13.22	12.73	12.24	11.75	11.26	10.77	10.27
0.036	0.500	12.92	12.43	11.95	11.48	11.00	10.52	10.03
0.038	0.000	15.90	14.82	13.81	12.86	11.97	11.12	10.32
0.038	0.050	16.19	15.51	14.43	13.42	12.48	11.58	10.73
0.038	0.100	15.74	15.12	14.52	13.92	13.03	12.07	11.17
0.038	0.150	15.30	14.70	14.12	13.54	12.96	12.38	11.65
0.038	0.200	14.89	14.31	13.74	13.18	12.61	12.05	11.48
0.038	0.250	14.50	13.94	13.38	12.83	12.29	11.74	11.18
0.038	0.300	14.14	13.58	13.04	12.51	11.97	11.44	10.90
0.038	0.350	13.79	13.25	12.72	12.20	11.68	11.15	10.63
0.038	0.400	13.45	12.93	12.41	11.90	11.39	10.89	10.37
0.038	0.450	13.13	12.62	12.12	11.62	11.12	10.63	10.13
0.038	0.500	12.83	12.33	11.84	11.35	10.87	10.38	9.89

TABLE 10.109A

ALLOWABLE COMPRESSION FLANGE STRESS - (36,000 PSI YIELD STRESS)

l/R	f_w/f_b	l/B = (LENGTH BETWEEN DIAPH./FLANGE WIDTH)						
		6	8	10	12	14	16	18
0.040	0.000	15.75	14.64	13.61	12.65	11.75	10.91	10.10
0.040	0.050	16.09	15.32	14.23	13.21	12.25	11.35	10.50
0.040	0.100	15.63	15.00	14.38	13.77	12.79	11.84	10.94
0.040	0.150	15.20	14.58	13.98	13.40	12.81	12.23	11.40
0.040	0.200	14.80	14.19	13.61	13.04	12.47	11.90	11.33
0.040	0.250	14.41	13.82	13.25	12.70	12.14	11.59	11.04
0.040	0.300	14.04	13.47	12.92	12.37	11.83	11.30	10.76
0.040	0.350	13.69	13.14	12.60	12.07	11.54	11.02	10.49
0.040	0.400	13.36	12.82	12.29	11.77	11.26	10.75	10.24
0.040	0.450	13.05	12.52	12.00	11.50	11.00	10.50	9.99
0.040	0.500	12.75	12.23	11.73	11.23	10.74	10.25	9.76
0.042	0.000	15.60	14.46	13.42	12.45	11.55	10.70	9.90
0.042	0.050	15.98	15.14	14.03	13.00	12.04	11.14	10.29
0.042	0.100	15.53	14.88	14.25	13.59	12.57	11.61	10.71
0.042	0.150	15.11	14.47	13.86	13.26	12.67	12.08	11.17
0.042	0.200	14.70	14.08	13.49	12.90	12.33	11.76	11.19
0.042	0.250	14.32	13.72	13.13	12.57	12.01	11.45	10.90
0.042	0.300	13.95	13.37	12.80	12.25	11.70	11.16	10.62
0.042	0.350	13.61	13.03	12.48	11.94	11.41	10.89	10.36
0.042	0.400	13.28	12.72	12.18	11.65	11.14	10.62	10.11
0.042	0.450	12.96	12.42	11.89	11.38	10.87	10.37	9.87
0.042	0.500	12.67	12.13	11.62	11.12	10.62	10.13	9.64
0.044	0.000	15.45	14.29	13.23	12.25	11.35	10.50	9.70
0.044	0.050	15.89	14.96	13.83	12.79	11.83	10.93	10.08
0.044	0.100	15.44	14.77	14.12	13.38	12.35	11.39	10.50
0.044	0.150	15.01	14.36	13.74	13.13	12.53	11.90	10.95
0.044	0.200	14.61	13.98	13.37	12.78	12.20	11.63	11.06
0.044	0.250	14.23	13.61	13.02	12.44	11.88	11.32	10.77
0.044	0.300	13.87	13.26	12.69	12.13	11.58	11.03	10.49
0.044	0.350	13.52	12.94	12.37	11.83	11.29	10.76	10.23
0.044	0.400	13.20	12.62	12.07	11.54	11.02	10.50	9.99
0.044	0.450	12.88	12.32	11.79	11.27	10.76	10.25	9.75
0.044	0.500	12.59	12.04	11.52	11.01	10.51	10.02	9.52
0.046	0.000	15.31	14.12	13.05	12.06	11.15	10.30	9.51
0.046	0.050	15.79	14.78	13.64	12.59	11.63	10.73	9.88
0.046	0.100	15.34	14.66	14.00	13.17	12.14	11.18	10.29
0.046	0.150	14.92	14.25	13.62	13.01	12.41	11.68	10.73
0.046	0.200	14.52	13.87	13.25	12.66	12.07	11.50	10.93
0.046	0.250	14.14	13.51	12.91	12.33	11.76	11.20	10.64
0.046	0.300	13.78	13.17	12.58	12.01	11.46	10.92	10.37
0.046	0.350	13.44	12.84	12.27	11.72	11.18	10.64	10.12
0.046	0.400	13.12	12.53	11.97	11.43	10.91	10.39	9.87
0.046	0.450	12.81	12.23	11.69	11.16	10.65	10.14	9.64
0.046	0.500	12.51	11.95	11.42	10.90	10.40	9.91	9.942

TABLE 10.109A

ALLOWABLE COMPRESSION FLANGE STRESS - (36,000 PSI YIELD STRESS)

l/R	f_w/f_b	l/B = (LENGTH BETWEEN DIAPH./FLANGE WIDTH)						
		6	8	10	12	14	16	18
0.048	0.000	15.16	13.96	12.87	11.88	10.97	10.12	9.32
0.048	0.050	15.70	14.61	13.46	12.40	11.43	10.53	9.69
0.048	0.100	15.25	14.55	13.89	12.97	11.94	10.98	10.09
0.048	0.150	14.83	14.15	13.51	12.89	12.29	11.47	10.52
0.048	0.200	14.44	13.77	13.15	12.54	11.96	11.38	10.81
0.048	0.250	14.06	13.41	12.80	12.22	11.64	11.08	10.53
0.048	0.300	13.70	13.07	12.48	11.91	11.35	10.80	10.26
0.048	0.350	13.36	12.75	12.17	11.61	11.07	10.53	10.01
0.048	0.400	13.04	12.44	11.87	11.33	10.80	10.28	9.76
0.048	0.450	12.73	12.15	11.59	11.06	10.54	10.04	9.53
0.048	0.500	12.44	11.87	11.33	10.81	10.30	9.81	9.31
0.050	0.000	15.02	13.80	12.70	11.70	10.79	9.94	9.15
0.050	0.050	15.61	14.45	13.28	12.22	11.24	10.34	9.51
0.050	0.100	15.17	14.45	13.78	12.78	11.74	10.79	9.90
0.050	0.150	14.75	14.05	13.40	12.78	12.17	11.27	10.32
0.050	0.200	14.35	13.68	13.04	12.43	11.84	11.27	10.70
0.050	0.250	13.98	13.32	12.70	12.11	11.54	10.97	10.42
0.050	0.300	13.62	12.98	12.38	11.80	11.24	10.70	10.15
0.050	0.350	13.29	12.66	12.07	11.51	10.96	10.43	9.90
0.050	0.400	12.96	12.35	11.78	11.23	10.70	10.18	9.66
0.050	0.450	12.66	12.06	11.50	10.97	10.45	9.94	9.44
0.050	0.500	12.37	11.78	11.24	10.71	10.20	9.71	9.22

TABLE 10.109B

Curvature Reduction Factor ρ_{red} for Allowable Stress

$\frac{e}{R}$	$\frac{e}{L}$	$\frac{e}{b}$										
		7	8	9	10	12	14	16	18	20	22	24
0.008	0.50	0.74	0.75	0.75	0.75	0.75	0.76	0.76	0.76	0.77	0.77	0.77
	0.25	0.84	0.84	0.85	0.85	0.85	0.85	0.86	0.86	0.87	0.87	0.87
	0.00	0.95	0.94	0.93	0.93	0.91	0.90	0.89	0.87	0.86	0.85	0.84
	-0.25	0.77	0.77	0.76	0.76	0.75	0.75	0.74	0.73	0.73	0.72	0.72
	-0.50	0.65	0.65	0.65	0.65	0.64	0.64	0.64	0.63	0.63	0.63	0.63
0.010	0.50	0.74	0.74	0.74	0.74	0.74	0.74	0.74	0.74	0.74	0.75	0.75
	0.25	0.83	0.83	0.83	0.83	0.84	0.84	0.84	0.84	0.84	0.84	0.84
	0.00	0.93	0.93	0.92	0.91	0.89	0.88	0.86	0.85	0.83	0.82	0.81
	-0.25	0.76	0.76	0.75	0.75	0.74	0.73	0.72	0.71	0.70	0.70	0.69
	-0.50	0.64	0.64	0.64	0.63	0.63	0.62	0.62	0.61	0.61	0.61	0.60
0.014	0.50	0.72	0.72	0.72	0.72	0.71	0.71	0.71	0.71	0.71	0.70	0.70
	0.25	0.81	0.81	0.81	0.81	0.81	0.80	0.80	0.80	0.80	0.80	0.79
	0.00	0.91	0.90	0.89	0.88	0.86	0.84	0.82	0.80	0.78	0.76	0.75
	-0.25	0.74	0.74	0.73	0.72	0.71	0.69	0.68	0.67	0.66	0.65	0.64
	-0.50	0.63	0.62	0.62	0.61	0.60	0.59	0.59	0.58	0.57	0.56	0.56
0.018	0.50	0.71	0.70	0.70	0.70	0.69	0.69	0.68	0.68	0.67	0.67	0.66
	0.25	0.80	0.79	0.79	0.79	0.78	0.78	0.77	0.77	0.76	0.76	0.75
	0.00	0.89	0.87	0.86	0.85	0.82	0.80	0.78	0.76	0.74	0.72	0.70
	-0.25	0.72	0.71	0.71	0.70	0.68	0.66	0.65	0.63	0.62	0.61	0.60
	-0.50	0.61	0.60	0.60	0.59	0.58	0.57	0.56	0.55	0.54	0.53	0.52
0.022	0.50	0.69	0.69	0.68	0.68	0.67	0.66	0.66	0.65	0.64	0.64	0.63
	0.25	0.78	0.78	0.77	0.77	0.76	0.75	0.74	0.73	0.73	0.72	0.71
	0.00	0.87	0.85	0.83	0.82	0.79	0.76	0.74	0.72	0.69	0.67	0.65
	-0.25	0.71	0.70	0.68	0.67	0.65	0.64	0.62	0.60	0.59	0.57	0.56
	-0.50	0.60	0.59	0.58	0.57	0.56	0.54	0.53	0.52	0.51	0.50	0.49
0.026	0.50	0.68	0.67	0.67	0.66	0.65	0.64	0.63	0.63	0.62	0.61	0.60
	0.25	0.77	0.76	0.76	0.75	0.74	0.73	0.72	0.71	0.70	0.69	0.68
	0.00	0.85	0.83	0.81	0.79	0.76	0.73	0.71	0.68	0.66	0.64	0.62
	-0.25	0.69	0.68	0.66	0.65	0.63	0.61	0.59	0.57	0.56	0.54	0.53
	-0.50	0.58	0.57	0.56	0.55	0.54	0.52	0.51	0.49	0.48	0.47	0.46
0.030	0.50	0.67	0.66	0.65	0.65	0.64	0.62	0.61	0.60	0.60	0.59	0.58
	0.25	0.76	0.75	0.74	0.73	0.72	0.71	0.69	0.68	0.67	0.66	0.66
	0.00	0.83	0.81	0.79	0.77	0.74	0.70	0.68	0.65	0.63	0.60	0.58
	-0.25	0.67	0.66	0.65	0.63	0.61	0.59	0.56	0.55	0.53	0.51	0.50
	-0.50	0.57	0.56	0.55	0.54	0.52	0.50	0.48	0.47	0.46	0.45	0.43
0.040	0.50	0.64	0.63	0.62	0.62	0.60	0.59	0.57	0.56	0.55	0.54	0.53
	0.25	0.73	0.72	0.71	0.70	0.68	0.66	0.65	0.64	0.62	0.61	0.60
	0.00	0.78	0.76	0.74	0.71	0.68	0.64	0.61	0.58	0.56	0.53	0.51
	-0.25	0.64	0.62	0.60	0.59	0.56	0.53	0.51	0.49	0.47	0.45	0.44
	-0.50	0.54	0.52	0.51	0.50	0.48	0.46	0.44	0.42	0.41	0.39	0.38
0.050	0.50	0.62	0.61	0.60	0.59	0.57	0.56	0.54	0.53	0.52	0.51	0.50
	0.25	0.70	0.69	0.68	0.67	0.65	0.63	0.61	0.60	0.59	0.58	0.55
	0.00	0.74	0.71	0.69	0.67	0.65	0.59	0.56	0.53	0.50	0.48	0.45
	-0.25	0.60	0.58	0.57	0.55	0.52	0.49	0.46	0.44	0.42	0.40	0.39
	-0.50	0.51	0.49	0.48	0.47	0.44	0.42	0.40	0.38	0.37	0.35	0.34

TABLE 10.109B (Continued)

Curvature Reduction Factor μ_{pc} for Allowable Stress

$\frac{e}{R}$	$\frac{L}{L_c}$	$\frac{e}{b}$										
		7	8	9	10	12	14	16	18	20	22	24
0.060	0.50	0.60	0.59	0.58	0.57	0.55	0.53	0.52	0.51	0.50	0.49	0.48
	0.25	0.58	0.67	0.66	0.64	0.62	0.60	0.59	0.57	0.56	0.52	0.49
	0.00	0.70	0.58	0.55	0.53	0.58	0.54	0.51	0.48	0.45	0.43	0.41
	-0.25	0.57	0.55	0.53	0.51	0.48	0.45	0.43	0.40	0.38	0.37	0.35
	-0.50	0.48	0.47	0.45	0.44	0.41	0.39	0.37	0.35	0.33	0.32	0.31
0.070	0.50	0.59	0.57	0.56	0.55	0.53	0.52	0.50	0.49	0.48	0.47	0.46
	0.25	0.56	0.65	0.64	0.62	0.60	0.58	0.57	0.55	0.51	0.48	0.45
	0.00	0.67	0.64	0.61	0.59	0.54	0.51	0.47	0.44	0.42	0.39	0.37
	-0.25	0.55	0.52	0.50	0.48	0.45	0.42	0.39	0.37	0.35	0.33	0.32
	-0.50	0.46	0.44	0.43	0.41	0.38	0.36	0.34	0.32	0.30	0.29	0.28
0.080	0.50	0.57	0.56	0.55	0.53	0.51	0.50	0.48	0.47	0.46	0.45	0.44
	0.25	0.55	0.63	0.62	0.60	0.58	0.56	0.55	0.51	0.47	0.44	0.41
	0.00	0.64	0.61	0.58	0.56	0.51	0.47	0.44	0.41	0.38	0.36	0.34
	-0.25	0.52	0.50	0.48	0.46	0.42	0.39	0.37	0.34	0.33	0.31	0.29
	-0.50	0.44	0.42	0.40	0.39	0.36	0.34	0.31	0.30	0.28	0.27	0.26
0.090	0.50	0.56	0.54	0.53	0.52	0.50	0.48	0.46	0.45	0.44	0.43	0.42
	0.25	0.63	0.61	0.60	0.58	0.56	0.54	0.51	0.47	0.44	0.41	0.38
	0.00	0.61	0.58	0.55	0.53	0.48	0.44	0.41	0.38	0.36	0.34	0.32
	-0.25	0.50	0.48	0.45	0.43	0.40	0.37	0.34	0.32	0.30	0.29	0.27
	-0.50	0.42	0.40	0.38	0.37	0.34	0.31	0.29	0.28	0.26	0.25	0.24
0.100	0.50	0.54	0.52	0.51	0.49	0.47	0.45	0.44	0.43	0.41	0.40	0.40
	0.25	0.61	0.59	0.57	0.56	0.53	0.51	0.48	0.44	0.41	0.38	0.35
	0.00	0.59	0.56	0.53	0.50	0.45	0.42	0.38	0.36	0.33	0.31	0.29
	-0.25	0.48	0.45	0.43	0.41	0.38	0.35	0.32	0.30	0.28	0.27	0.25
	-0.50	0.40	0.38	0.37	0.35	0.32	0.30	0.28	0.26	0.24	0.23	0.22

10.110 ALLOWABLE WEB SHEAR STRESS

The average calculated shear stress in the web of a curved girder shall not exceed:

$$F_v = 0.33F_y$$

10.111 THICKNESS OF WEB PLATES**A Girders Not Stiffened Longitudinally**

The web plate thickness of curved plate shall not be less than that determined by the formula:

$$t = \frac{D(f_b)^{1/2}}{23,000} \left[\frac{1}{1 - 4(d_o/R)} \right]$$

Where f_b is the calculated flexural compressive bending stress in the flange.

But in no case shall the thickness be less than $D/170$ or $3/8"$, whichever controls.

Where the calculated average normal stress in the compression flange equals the allowable normal flange stress, the thickness of the web plate in Eq. (a) shall not be less than the limiting values in Article 10.34.3.1.2.

B Girders Stiffened Longitudinally

The web plate thickness of curved plate girders equipped with a single longitudinal stiffener, located in the compressive stress region a distance $D/5$ from the compression flange, shall not be less than that determined by the formula:

$$t = \frac{D(f_b)^{1/2}}{46,000} \left[\frac{1}{1 - 2.9(d_o/R)^{1/2} + 2.2(d_o/R)} \right]$$

Where f_b is the calculated compressive bending stress in the flange.

But in no case shall the thickness be less than $D/340$ or $1/2"$, whichever controls.

Where the calculated flexural compressive stress in the flange equals the allowable flexural compressive stress, and the girder is equipped with a single longitudinal stiffener located in the compressive stress region as well as a single longitudinal stiffener in the tension stress region the thickness of the web plate shall not be less than the limiting values in Article 10.34.3.2.2.

10.112

TRANSVERSE INTERMEDIATE STIFFENERS

Transverse intermediate stiffeners may be omitted if the average calculated shear stress in the gross section of the web plate at the point considered meets the requirements of Article 10.34.4.1.

Where transverse stiffeners are required, the spacing of the transverse stiffener shall be such that the actual shear stress shall not exceed the value given by the following equation:

$$F_v = CF_y/3 \leq F_y/3$$

The value of C is determined from the provisions of Article 10.34.4.2.

For that portion of the span where stiffeners are required, the maximum intermediate transverse stiffener spacing is limited to the web depth, D or half the diaphragm spacing, whichever controls. The spacing of the first transverse stiffener at a simply supported end is limited to 0.5d.

For curved girders, the moment of inertia with reference to the mid-plane of the web of any intermediate transverse stiffener shall not be less than:

$$I = \frac{d_o t^3 J}{10.92}$$

$$\text{Where } J = \left[25 \left[\frac{D^2}{d^2} \right] - 20 \right] (X), \text{ but not less than } 5.0.$$

In this expression,

I = the minimum permissible moment of inertia of any type of transverse intermediate stiffener (in⁴)

J = the required ratio of rigidity of one transverse stiffener to that of the web plate

d = the required distance between transverse stiffeners (in.)

d_o = the actual distance between transverse stiffeners (in.)

D = The unsupported distance between flange components (in.)

t = the thickness of the web plate (in.)

$$X = 1.0; \text{ when } \frac{d}{D} \leq 0.78$$

$$X = 1.0 + \frac{[(d/D) - 0.78]Z^4}{1775} : \text{when } 0.78 \leq d/D \leq 1.0 \text{ and } 0 \leq Z \leq 10$$

$$Z = 0.95[d^2/(Rt)]$$

R = radius of curvature of girder web (in.)

The thickness of a transverse stiffener shall not be less than:

$$\frac{b'(F_y)^{1/2}}{2600}$$

Where b' is the width of the outstanding stiffener element (in) and F_y is the yield strength of the transverse stiffener (PSI).

When stiffeners are in pairs, the moment of inertia shall be taken about the centerline of the web plate. When single stiffeners are used, the moment of inertia shall be taken about the face in contact with the web plate.

Where transverse intermediate stiffeners are required, the spacing of the transverse intermediate stiffener shall not exceed:

- a) D where D is Depth of Web.
- b) 0.5 diaph. spacing

10.113

LONGITUDINAL STIFFENERS

The longitudinal stiffeners on curved web panels shall satisfy the requirements of Article 10.34.5. In addition, the radius of gyration of the stiffener shall not be less than

$$r = \frac{d_o(F_y)^{1/2}}{23,000}$$

In computing r , a centrally located web strip not more than 18t in width shall be considered as part of the longitudinal stiffener.

For plate girders equipped with a single longitudinal stiffener in the compressive stress region as well as a single longitudinal stiffener in the tension stress region, both stiffeners shall have the same size. The stiffener in the tension region shall be placed at a distance $D/5$ from the inner surface of the tension flange.

CURVED COMPOSITE I-GIRDERS

10.114 GENERAL

This section pertains to structures composed of steel girders which are curved in plan with concrete slabs connected by shear connectors in the positive dead load moment area. The general requirements of Article 10.38.1 shall also apply to such structures.

10.115 EFFECTIVE FLANGE WIDTH

For purposes of computing the properties of a composite section to be used in analysis for bending moments, twisting moments, and deformations, Article 10.38.3 shall be applied.

10.116 NONCOMPOSITE DEAD LOAD STRESSES

The normal noncomposite dead-load stresses in the steel section alone due to flexure, f_b , and restrained warping, f_w , shall be determined by elastic theory. The noncomposite dead-load flexural stresses, f_b , and tip stresses, $f_b + f_w$, in the steel beam alone shall not exceed the values given in Article 10.109.

10.117 COMPOSITE SECTION STRESSES

The normal stresses in the composite section due to flexure, f_b , and restrained warping, f_w , for the composite dead load plus live load after the concrete deck has hardened shall be determined by elastic theory. After adding in the non-composite dead load stresses, the total flexural stresses, f_b , and tip stresses, $f_b + f_w$, in the composite section shall not exceed the values given in Article 10.109. Design for shear shall be according to the requirements specified in Articles 10.110 and 10.112.

Top compression flanges in composite sections shall be considered braced against lateral distortions by the hardened concrete deck when computing the allowable normal flange stresses in the composite section for the total dead plus live load. The allowable total normal flange stress for the top compression flange in composite sections after the concrete deck has hardened shall therefore be limited to $0.55F_y$. The added warping stresses, f_w , in the top flange due to the composite dead load and live load may be neglected for composite sections after the concrete deck has hardened.

The provisions of Article 10.3 shall also apply to curved composite girders.

10.118

SHEAR CONNECTORS

The provisions of Article 10.38.2 shall also apply to the design of shear connectors for curved composite girders except that the requirements of Article 10.118 shall be used in place of those in Article 10.38.5.1.2 to determine the capacity of the shear connectors.

(A) Fatigue

The provisions of Article 10.38.5.1.1 shall govern for fatigue of shear connectors in curved composite girders.

(B) Ultimate Strength

The number of connectors so provided for fatigue shall be checked to ensure that adequate connectors are provided for ultimate strength. The number of shear connectors required between points of maximum positive moment and the end supports or dead load points of contraflexure shall be such that:

$$P_c \leq \phi S_u$$

where:

ϕ = a reduction factor = 0.85

S_u = the ultimate strength of the shear connectors as given in Art. 10.38.5.1.2.

P_c = force on the connector defined by

$$P_c = \left[\bar{P}^2 + F^2 + 2\bar{P}F(\sin(\theta/2)) \right]^{1/2}$$

$\frac{P}{N}$ = $\frac{P}{N}$

P = $0.85f'_c b c$ or $P = A_s F_y$, whichever is smaller; at points of maximum positive moment; at points of maximum negative moment, $P = A_s^T F_y^T$ as defined in Article 10.38.5.1.2.

N = number of connectors between points of maximum positive moment and adjacent end supports or dead load points of contraflexure, or between points of maximum negative moment and adjacent dead load points of contraflexure

$$F = \frac{P(1 - \cos \theta)}{4K(N_s) \sin(\theta/2)}$$

θ = angle subtended between the point of maximum moment (positive or negative) and adjacent point of contraflexure or support

b = effective flange width (in.) as given in Article 10.38.3

c = thickness of concrete slab (in.)

A_s = total area of steel section (in.²) including cover plates

$$K = 0.166 \left[\frac{N}{N_s} - 1 \right] + 0.375$$

N_s = number of connectors at a section

Additional connectors to develop slab stresses, Same as Art.10.38.5.1.3.

CURVED HYBRID GIRDERS

10.119 GENERAL

This section pertains to the design of hybrid I-girders which have a vertical axis of symmetry through the middle-plane of the web plate and which meet the general requirements of Article 10.40.1. The following additional requirements apply for curved hybrid girders.

10.120 ALLOWABLE STRESSES

A. Bending - Noncomposite Girders

For noncomposite girders the bending stress in the web may exceed the allowable stress for web steel provided that the stress in each flange does not exceed the allowable stress from Article 10.109 for the steel in that flange multiplied by the reduction factor.

$$R = 1 - \left[\frac{\beta\psi(1 - \alpha')^2(3 - \psi + \psi\alpha')}{6 + \beta\psi(3 - \psi)} \right] \quad (a)$$

where:

F_y = minimum yield strength of the compression flange (psi)

β = the area of the web divided by the area of the tension flange

ψ = the distance from the outer edge of the tension flange to the neutral axis (of the transformed section for composite girders) divided by the depth of the steel section

α' = $\alpha(1 + |f_w/f_b|_t)$ (b)

α = the minimum specified yield strength of the web divided by the minimum specified yield strength of the tension flange

$|f_w/f_b|_t$ = the absolute value of the tension flange tip stress due to lateral flange bending divided by the bending stress in the tension flange

If $|f_w/f_b|_t \geq \frac{1 - \alpha}{\alpha}$, then $R = 1$. (c)

B. Bending - Composite Girders

In the positive moment region of composite girders the bending stress in the web may exceed the allowable stress for the web steel provided that the stress in the tension flange does not exceed the allowable stress from Article 10.109 for the steel in that flange multiplied by the reduction factor determined from Eq. (a) in Article 10.120A where α' is given by Eq. (b).

In the negative moment region of continuous span composite girders in which the tension flange is connected to the concrete slab by shear connectors, the reduction factor shall be calculated using Eq. (a) in Article 10.120A where α' shall be the appropriate value given below:

$$\text{when } |f_w/f_b|_c \leq \frac{2\psi - 1}{1 - \psi} ; \quad \alpha' = \alpha \quad (d)$$

$$\text{when } \frac{2\psi - 1}{1 - \psi} < |f_w/f_b|_c < \frac{\psi}{\alpha(1 - \psi)} - 1 ; \quad \alpha' = \alpha \left[1 + |f_w/f_b|_c \right] \frac{1 - \psi}{\psi} \quad (e)$$

where α and ψ are defined in Article 10.120 (A) and

$|f_w/f_b|_c$ = the absolute value of the compression flange tip stress due to lateral flange bending divided by the bending stress in the compression flange

$$\text{If } |f_w/f_b|_c \geq \frac{\psi}{\alpha(1 - \psi)} - 1 , \quad \text{then } R = 1 \quad (f)$$

C. Shear

Design for shear shall be according to the requirements of Article 10.112.

D. Fatigue

Design for fatigue shall be according to the requirements of Article 10.40.2.3.

10.121**PLATE THICKNESS REQUIREMENTS**

In calculating the minimum thickness of the web plate according to Article 10.111, f_b shall be taken as the calculated bending stress in the compression flange divided by the reduction factor, R , determined from the appropriate formula in Article 10.120. The limiting width to thickness ratio of the compression flange plate for hybrid girders shall be determined according to Article 10.109 using the yield stress of the compression flange material and need not be reduced due to the lower yield strength of the web material.

**PREFACE TO "WORKING STRESS DESIGN CRITERIA FOR
CURVED STEEL, COMPOSITE AND HYBRID I-GIRDER BRIDGES: COMMENTARY"**

PREFACE

The analytical and experimental studies which form the basis for the proposed design recommendations have been documented in the various CURT project reports and elsewhere. The purpose of this commentary is to briefly summarize these results for each specific recommendation. Appropriate references are made to the detailed project reports for those interested in a comprehensive study of the original of these recommendations.

In order to assist the designer in understanding the various recommendations and clarify questions regarding their application, some of the formulas have been presented in tabular and/or graphical form. Other forms of design aids can also be prepared depending upon the preference of the individual designer. In some cases, it may be advantageous to program these equations on a digital computer to assist in proportioning the members in curved bridges to meet these recommendations.

Since it is impossible to cover every conceivable design situation in these recommendations, it is suggested that the designer refer to the various references cited in this commentary for such unusual cases or when the design problem falls outside the scope of the various recommendations.

**WORKING STRESS DESIGN CRITERIA FOR
CURVED STEEL I-GIRDER BRIDGES: COMMENTARY**

10.100 GENERAL

The design recommendations presented in the first part of this specification are based on analytical and experimental studies of curved I-shaped members. An attempt was made in developing these recommendations to include not only results obtained from the CURT project but to recognize all the existing applicable work on curved bridges. These recommendations, therefore, should reflect the state of knowledge in this area as of mid-1974.

The provisions are presented in the form of requirements applicable to an "allowable stress" type design procedure. As such the requirements parallel those in the current edition of The Standard Specifications for Highway Bridges adopted by AASHTO. It should be noted, however, that some information on ultimate strength behavior of curved members was obtained in the CURT studies. This information may be useful at a later date should it be decided to extend these design provisions to a load factor type approach. In most cases, as the radius of the bridge becomes large or approaches infinity, i.e., the curved bridge becomes straight, the recommendations herein reduce to the AASHTO requirements for straight bridges.

The recommendations presented obviously do not cover every aspect of the design and construction of curved highway bridges. Only those areas for which specific information was available were included. For problems not specifically covered herein, the designer should resort to solutions based on sound engineering judgement until additional information becomes available.

10.102 LOADS

(A) Uplift

Because of the overall behavior of curved girder structures, there is a possibility that uplift will occur at the supports. This uplift condition is not as readily apparent in curved girder unit systems as in other structural systems, and for this reason, uplift is specified in this section. Besides checking the system for uplift due to dead load and the live load, an investigation should be made to insure that uplift does not occur because of the method of placing the deck concrete. Since uplift is a condition of stability rather than stress, it is recommended that if it is possible to have more than two lanes of load on the bridge and produce uplift, a reduction of the loads should not be made.

(B) Impact

Numerous studies have been made on impact. These studies indicate a need to study straight girders as well as a need to study a greater range of parameters for curved girders. Until this information is available it is recommended that the existing equation for straight girders be used for curved girders.

The following is a proposed specification that should be reviewed along with further investigations.

Proposed Specification

The dynamic effects, including centrifugal forces, produced by HS loadings on the bridge shall be accounted for by using the impact factors given in Table 10.102B. Thus, the value of a live load quantity (e.g., reaction), considering the dynamic effects, is the product of the value of that live load quantity calculated by a static analysis and $(1+I)$, or

$$\text{Dynamic live load value} = (1+I) \times (\text{Static live load value})$$

where I is the impact factor, the impact factors are valid within the following parameter values:

$$50 \text{ ft.} \leq L \leq 200 \text{ ft.}$$

$$200 \text{ ft.} \leq R_c \leq 1000 \text{ ft.}$$

$$v \leq 70 \text{ mph}$$

$$\text{Number of I-girders} \leq 6$$

$$\text{Number of continuous spans} \leq 2$$

$$\frac{\text{Weight of Vehicle}}{\text{Weight of Bridge}} \leq 0.6$$

If the above ranges are exceeded, then a dynamic analysis should be made.

The impact factors are:

TABLE 10.102B

<u>Quantity</u>	<u>Impact Factor, I</u>
Reactions and shear forces	0.30
Moments in longitudinal girders	0.25
Torsional moments in longitudinal girders	0.40
Moments in slab	0.20
Bimoments in longitudinal girders	0.25
Forces and moments in diaphragms	0.25
Deflections	0.25

(C) Superelevation and Centrifugal Forces

The horizontal radial force resulting from live load on a horizontally curved bridge is dependent upon the velocity of the vehicle and the radius of curvature of the structure. This force should be included in the analysis of the bridge. The vertical component of live load produces essentially the same moments, twisting moments and deflections for any angle of superelevation between 0 and 10 percent. This has been shown through application of the Syracuse University Three Dimensional Analysis program (19).

(D) Thermal Forces

A comprehensive analytical study (11) has been made, in which temperature effects on curved bridges have been compared with those on straight bridges. No significant differences were noted unless the mean horizontal radius was less than 100' and the subtended angle was greater than 45°. Consequently, no allowances need to be made for thermal forces in superstructure design, for curved bridges of practical proportions.

10.103 DESIGN THEORY**(A) General**

Simplifications are made in structural analysis to facilitate design. In the case of a bridge with straight girders, for example, torsion in the girders is ignored, since it is a secondary effect and stability can be achieved by considering bending as the primary structural action. For a horizontally curved beam carrying vertical loads, however, the torsion which is developed is necessary for stability, and must be considered in its design.

Since the twisting of the curved girders is resisted by interaction with adjacent girders, it is strongly recommended that the entire structure be analyzed as a system.

Methods of Analysis

As of this date, the following methods of analysis of I-section curved girder highway bridges provide some or all of the information needed to proportion the individual members:

- * USSteel V-Load Method (12)
- * STRESS and STRUDL
- * Berkeley's CURVBRG program (13)
- * University of Maryland's COBRA programs (14)

- * University of Rhode Island's CUGAR program
(15,16, 17, 18)
- * Syracuse University's Three-Dimensional Analysis
(19)
- * University of Pennsylvania's STACRB program (20)

For most of the above, loads are applied directly to the structure, and the method of analysis "distributes" the loads to the various girders. For a grid analysis, this distribution is accomplished by the diaphragms, whether the bridge is curved or straight. For a straight or nearly straight bridge, use of a grid method produces a structure of lower total capacity than for a straight bridge of the same span designed using the distribution factors of Article 3.23; the difference has been found to be as much as 20%.

Limiting Central Angle

An extensive analytical study (21) was made to determine the effects of curvature on the primary bending moment. One-, two-, and three-span structures were analyzed having from 2 to 10 girders spaced 8', 12' and 16' on centers, for curvatures varying from straight to 10° subtended angle per span. A wide variety of distributed and concentrated loads were used to obtain extreme load conditions. Moments, obtained at midspans of each girder, were adjusted to obtain values for the same lengths as the corresponding straight bridge. The differences between corresponding moments in curved and straight beam with similar loading.

Table 10.103A was prepared assuming that an increase of moment in the order of 10% would be a reasonable upper limit on use of the distribution factors given in Article 3.23.

Preliminary Method For Dead Load Analysis

A computer program for preliminary design of single-span horizontally curved bridges has been developed by the New York State Department of Transportation (22).

Preliminary Method For Live Load Analysis

1. Live Load Distribution - Composite Girders

In order to determine a preliminary size of a curved girder, it is desirable to have probable induced maximum girder forces. Such forces can be determined by the following equations, which were obtained by analyzing many curved girder systems (23). These equations are analogous to the well known and presently used distribution factors

for straight girders. It should be emphasized that the procedure that was used in development of these curved girder factors follows the same technique employed in determining the straight girder equations.

The concept of live load distribution in a bridge system requires a thorough investigation of the interaction of all bridge elements. In developing the presently used straight girder distribution factors, a thorough analysis of many bridge systems was required. In order to develop similar factors for curved bridge system and its solution, a mathematical model of a curved bridge system and its solution is required.

The development of such a model and its solution (24) has resulted in a tool for use in developing distribution factors. The data obtained from the computer-oriented solution has also been correlated with data obtained from the testing of curved bridge models (25), giving good results.

As will be recalled, the concept of the distribution factor requires a relation between the response of the forces in a system to those forces developed in a single isolated girder subjected to a set of wheel loads; i.e.

$$D.F. = \frac{\text{Curved System Function}}{\text{Curved Single Girder Function}}$$

The resulting live load distribution factors relative to induced bending moments and bimoments* for each girder are:

$$\begin{aligned} \text{Bending Moments: } DF_B &= \frac{S}{5.5} \frac{[(N+3) L + 0.7]}{4R} \\ \text{Bimoment: } DF_{Bi} &= \frac{S}{5.5} \left[[0.0008L + 0.13] \right. \\ &\quad \left. + [0.0022L^2 - 0.59L + 40] R \times 10^{-4} \right] \end{aligned}$$

where: S = Girder Spacing, ft. ($7' \leq S \leq 12'$)

L = Span Length, ft.

R = Radius, ft. ($R > 100'$)

*As used in the Specifications, the term "lateral flange bending" is analogous to "bimoment."

$$N = \frac{R}{100} \quad (R > 100')$$

These distribution factors are multiplied by the wheel load (front and rear), which is then applied to the individual girder.

2. Interior and Exterior Composite Beam Forces

The above distribution factors would only be applicable if an analysis was now performed on a single curved girder subjected to a line of wheel loads. The resulting forces obtained from this analysis, when multiplied by the distribution factors, represent those forces in a system. However, the analysis of a single curved girder is complex (26). In order to eliminate this complexity the response of a single curved girder to that of an equivalent straight girder, has been studied (23). These results give a modification factor, which then accounts for curvature, and is of the form:

$$M.F. = \frac{\text{Curved Single Girder Function}}{\text{Single Straight Girder Maximum Bending Moment}}$$

in which the curved single girder function is bending moment or bimoment. The modification factors are:

$$\text{Bending Moment: } MF_B = \left[\frac{0.15}{N} (L/R) + 1.0 \right]$$

$$\text{Bimoment: } MF_{Bi} = \frac{35N(L/R) - 15.0}{0.108L - 1.68} (L/R)$$

These modification factors are then multiplied by the equivalent straight girder bending moment, which is then multiplied by the D.F. to obtain the resulting maximum system girder forces.

The following general equation would then be used:

$$\text{Curved Girder Bending Moment: } M = (MF_B) \times M_s$$

where: M_s = equivalent straight girder moment due to a set of wheel loads (front and rear) when multiplied by the distribution factor (DF_B) as previously given. The straight girder will have a length equal to the arc length of the curved girder.

$$\text{Curved Girder Bimoment: } B_i = (MF_{Bi}) \times M_s$$

Where M_s is as defined previously, the above equations are applicable to systems of 4 to 8 curved girders.

Applications of these equations relative to the analytical model studies from the computer (24) are shown in Figures 4 and 5 for bending moments and bimoments. The results indicate that the equations are quite satisfactory for those bridges which have severe curvature and thus high bimoments. The deviation between the computer data and the equations for the bimoments is large when $R > 600$ ft. However, the bimoments are of the order of $10 (K\text{-ft.}^2)$, which is not significant. The bending moment data at the large radii show reasonable results. This data, however, would be entirely conservative if the conventional distribution factor were to be used.

(B) Torsion

Since I-girders are relatively flexible in torsion, it is necessary to provide torsional restraints along the girders. This can be done using diaphragms or cross-frames with sufficient bending capacity to resist the generated torques.

(C) Nonuniform Torsion

If a non-circular member is twisted, planar cross-sections warp. Normal stresses will be developed in addition to torsional shearing stresses, if this warping is restrained. This phenomenon is known as nonuniform torsion or as lateral flange bending (12, 16, 57).

Even if the curvature is within the limits given in Article 10.103A (so that curvature is neglected in determining primary bending moments), the effect of nonuniform torsion (12) on normal stresses should be considered.

(D) Composite Design

Properties

The bending properties of composite sections are readily determined by applying conventional strength of material techniques. These techniques can also be adapted (27) in evaluating the torsional properties of composite sections. However, the application of these methods is tedious and difficult. Therefore, the following approximate equations may be used for design application (28). The reference material property for these equations is steel.

$$\text{Shear Center: } \alpha = \frac{d_3^3 t_3}{d_1^3 t_1 + d_3^3 t_3} (d_2)$$

$$\text{Warping Functions: } W_{nc} = \frac{\alpha d_1}{2} ; W_{ns} = \frac{(d_2 - \alpha)}{2} (d_3)$$

$$\text{Warping Statical Moment: } S_{wc} = \frac{\alpha d_1^2 t_1}{8} ; S_{ws} = \frac{(d_2 - \alpha)}{2} (d_3^2 t_3)$$

$$\text{Warping Constant: } I_w = \frac{\alpha^2}{12} t_1 d_1^3 + (d_2 - \alpha)^2 \left[\frac{t_3 d_3^3}{12} \right]$$

$$\text{Torsional Constant: } K_T = \frac{1}{3} \left[d_3 t_3^3 + d_2 t_2^3 + \frac{d_1 t_s^3}{m} \right]$$

Figure 6 shows the actual composite beam and Figure 7 represents the idealized section.

The parameters associated with these equations are:

n = ratio of modulus of elasticity of steel to that of concrete

m = ratio of shear modulus of steel to that of concrete

$t_1 = t_s / n$

$t_2 = w$

$d_2 = d_g + t_s / 2 - t_f / 2$

$d_3 = b_f$

$t_3 = t_f$

In terms of parameters shown on Figure 6.

The determination of composite girder torsional properties, using these approximate equations and the more exact equations, indicate good correlation (28). Test results of composite beams subjected to torsional loadings (29) also indicate the validity of the general theories and approximations.

Stresses:

The bending stresses induced on a composite section are determined from the conventional equations:

$$f_b = Mc/I$$

In a similar manner, the normal stresses induced by warping may be computed from an analogous equation of the form:

$$f_w = B_i W_n / I_w$$

in which the parameters W_n and I_w are torsional cross-sectional properties, as defined previously and elsewhere (30). The parameter B_i is called a Bimoment, which is force due to nonuniform torsion of the section.

10.104 FATIGUE

The primary difference between the stress distribution in horizontally curved versus straight girders is the contribution of torsional stresses. The resulting complex stress distribution, particularly in curved plate girders, requires accurate stress analyses under the appropriate loading conditions when designing against fatigue.

10.105 EXPANSION AND CONTRACTION

If the superstructure were completely unrestrained, the effect of a long time temperature change would be to increase (or decrease) all dimensions uniformly; i.e., the deformed structure would be geometrically similar to its original shape (11). To minimize thermal forces in the superstructure and in the substructure, movements must be allowed in directions radiating from the fixed supports.

When designing expansion and contraction joints, it should be remembered that the primary direction of movement will not generally be tangential to the girder webs. If the curvature is sharp, a significant component of displacement can occur in the direction perpendicular to the web.

10.106 BEARINGS

The system of bearings must provide horizontal stability for the superstructure and still permit thermal movements (31). Movement at each expansion bearing should be permitted in a direction approximately along a line drawn from the fixed bearing to the expansion bearing, and at least one bearing should be restrained in a direction perpendicular to this line.

At a support only the usual bending rotation needs to be provided for, since the diaphragm or cross frame at the support will prevent most of the twisting from occurring. Thus, regardless of the direction of displacement allowed at a support, if rotation is permitted about only one axis, that axis should be perpendicular to the centerline of the web at the bearing.

10.107 DIAPHRAGMS, CROSS FRAMES AND LATERAL BRACING

Out-of-plane web distortion causing severe displacement-induced transverse bending stresses in the web can result in very low fatigue strength unless the connection plates are also attached to the adjacent flange(s) of the plate girder. Coping is required to avoid intersecting welds.

In order to keep flange normal stresses caused by nonuniform torsion to reasonable values, diaphragms and cross frames should be more closely spaced as curvature increases.

The following spacing limits are suggested:

<u>Centerline Radius Of Bridge (feet)</u>	<u>Suggested Maximum Spacing (feet) for Outermost Girder</u>
Below 200	15
200 to 500	17
500 to 1000	20
over 1000	25

It should also be noted that the requirement of $l/R \leq 0.1$, given in Article 10.109A, also serves to limit diaphragms and cross frame spacing.

When using a grid method of analysis, the diaphragms or cross frames become primary load carrying members. Since their function is to provide lateral support for the flanges, they must be attached close to the flanges.

CURVED STEEL I - GIRDERS**10.108****GENERAL**

Since no information is available on the behavior of riveted or bolted curved girders, the design recommendations were limited to rolled and welded plate girders. This limitation is not that severe since the majority of curved plate girder bridges are of this type.

Field tests (32) and laboratory tests (33, 34, 35) indicated the importance of normal stresses due to nonuniform torsion or lateral flange bending for curved plate girders. The design recommendations require, therefore, that these normal stresses, in addition to those due to bending, be taken into account in designing curved plate girder bridges.

Fatigue strengths can be seriously under or overestimated, especially at intersections of diaphragms with curved plate girders, unless an accurate stress analysis of the three dimensional structure is performed.

10.109**ALLOWABLE NORMAL FLANGE STRESS****(A) Compression**

In order to prevent local buckling of the outstanding elements of the compression flange prior to reaching a given stress level, limiting values of the width-thickness ratio of the elements are specified (36, 37). These existing straight girder requirements are based on solutions for plate elements subjected to a uniform compressive stress across the flange width. In a curved girder normal stresses due to torsion or lateral flange bending develop and the stress varies across the flange width. Since the stress is high only near the flange tip, it would appear that local buckling would be less critical. Some designers, in fact, have used the average stress, i.e., the stress due to primary bending in determining the flange width-thickness requirements. Studies in the first phase of the PennDot Project (38) indicated that although curvature has little effect on local buckling for practical curved girders, the stress gradient across the flange is important. Localized yielding only near the flange tip, for example, causes a sudden decrease in the buckling coefficient. Thus, if the flange width-thickness requirements presently used for straight girders designed using the working stress method (36) were adopted for curved girders, allowable design stresses would have to be based on applying a factor of safety to the stress at the flange tip (39). Test results obtained in the CURT project substantiate this finding (33). This procedure is somewhat inefficient since the major portion of the

flange is less highly stressed and would not be utilized effectively. Results based on inelastic buckling indicated that if the flange proportions are limited to those for straight compact sections designed on the basis of load factor design (40), the full flange strength could be utilized prior to local buckling. Test results from the CURT project also substantiate this fact (33, 34, 35). Allowable stresses based on applying a factor of safety to the full plastic moment under combined bending and torsion could then be used.

Allowable compression flange stresses for curved girders, dependent upon the flange proportions, b/t , were developed by Culver (41). Two cases were considered. In the first, applicable to noncompact flanges $[3200/(F_y)^{1/2}] < b/t \leq [4400/(F_y)^{1/2}]$, the allowable stress was based on providing a factor of safety against initial yield at the flange tip. The limiting value of $b/t = 4400/(F_y)^{1/2}$ corresponds to the requirement for noncompact sections in the load factor specification (40) and is the same as the AASHTO value (36) obtained using $f_b = 0.55 F_y$. The lower limit of $3200/(F_y)^{1/2}$ corresponds to the load factor value for compact sections. Based on analytical studies, the use of a transition formula which permits an increase in allowable stress as b/t decreases from $4400/(F_y)^{1/2}$ to $3200/(F_y)^{1/2}$, (37) was not advisable for curved beams. In the second case, $b/t \leq 3200/(F_y)^{1/2}$, these stresses were based on the ultimate strength of the girder. In establishing these flange proportions in both cases, the effects of residual stresses due to fabrication were taken into account and the rotational restraint provided by the web in resisting flange buckling was minimized. The magnitude of the allowable flange stress was based on lateral buckling considerations. A comprehensive analytical study (42) of the behavior of curved plate girders subjected to bending and torsion was conducted to establish these allowable stresses. Second order effects as well as effects due to deformation of the cross section were included in this study.

In this analysis, the lateral buckling strength of a curved girder segment between points of lateral bracing was found to be dependent on the direction and magnitude of the warping bimoment (lateral flange bending moment) at the bracing points. If the bimoment tends to bend the compression flange toward the center of curvature, thereby restraining the outward displacement of the compression flange, the lateral buckling strength of the girder will be enhanced. This type of bimoment produces a positive warping to bending stress ratio, f_w/f_b , at the bracing points based on the sign convention given in Article 10.109.

On the other hand, a bi-moment that produces a negative value for f_w/f_b at the braces will reduce the lateral buckling strength of the girder. This behavior is reflected in the allowable stress formulae in Article 10.109. In most cases the values of f_w/f_b at the lateral braces will be positive.

As noted above, two sets of formulae for F_b were developed by Culver (41). The first set for noncompact sections was based on providing a factor of safety against initial yield. The second set for compact girders provides the same factor of safety against full plastification under combined bending and nonuniform torsion. The design formulae were presented in the same form as those for straight girders, using a factor $\rho_B \rho_w$ to represent the reduction in allowable stress due to curvature effects. The factor of safety used to develop these equations was the same as that used for straight girders (F.S. = 1.82). As the curvature decreased and the girder became straight, the allowable stresses for both cases reduced to those presently used in working stress design. Since the formulas for the first case were based on initial yield, the total stress at the flange tip (warping plus bending, $f_w + f_b$) was less than or equal to the allowable stress for a straight girder with the same l/b . In the second case, this total flange tip stress exceeded the straight girder value in some instances. It is important to realize, however, that the resulting factor of safety against failure was still 1.82. Permitting this flange tip stress to exceed the present AASHTO values for straight girders was also justified in view of the conservatism used in the analytical study (42) and also since any redistribution effects or postbuckling strength was neglected. The allowable stress formula in Article 10.109 is based on Culver's results for noncompact sections. The formula for compact sections was not included because the total tip stress was limited to $0.55F_y$. Future additions to the Specifications should consider removing this limitation and permitting higher allowable tip stresses for compact flanges.

The limit of $l/b \leq 25$ in the Specifications was established by Culver and McManus (42). For higher values of l/b the allowable bending stresses are extremely small and it would be uneconomical to consider such cases. The curvature reduction factors, ρ_B and ρ_w , given Eqs. (b), (c), and (d) in Article 10.109 become inaccurate for values of l/R exceeding 0.1 and for absolute values of f_w/f_b exceeding 0.5. Calculated values for these parameters in most practical curved bridge systems will be within these limits of applicability. The curvature correction factor, ρ_B , is plotted as a function of curvature, l/R , in Figure 8 for selected values of l/b .

(B) Tension

To be consistent with the criterion adopted for the compression flange allowable stress, Article 10.109, the provision in this Article is based on providing a factor of safety of 1.82 against initial yield at the flange tip. If future specifications incorporate increased allowable compressive stresses for compact girders, then consideration should be given to larger allowable tension stress based on plastic analysis.

10.110 - ALLOWABLE WEB SHEAR STRESS

The allowable shear stress specified in this article for curved girders is the same as the existing allowable shear stress in Article 10.2 and 10.32. Tests (43) on curved box girders have shown that the fully plastic shear strength of curved girder webs can be developed if the web plate is adequately proportioned or stiffened. Therefore, it is recommended that the existing maximum allowable shear stress be adopted for curved girders.

10.111 THICKNESS OF WEB PLATES**A. Girders not Stiffened Longitudinally**

Existing web depth-thickness limitations for straight girders (Ref. 44, Ch 8) are based on buckling considerations. Recent studies carried out at Lehigh University indicated that the upper limit for transversely stiffened girders could be increased to $D/t = 36,500/(F_y)^{1/2}$. This increase was based on treating the web panel as fixed along the flange web boundary rather than simply supported. Fatigue tests (45) also indicated that the web bending stresses due to initial imperfections in the web panel did not cause fatigue problems up to this limit. The increase in web slenderness ratio permitted in the load factor portion of the AASHTO specifications over the present working stress values reflects this recent work.

The limiting web slenderness ratios for curved girders are based on an analytical study (46) since fatigue test data similar to that for straight girders is presently not available. Due to curvature effects, the web of a curved girder subjected to bending tends to deform outward in the compression region and in the tension region it tends to flatten out. These deformations produce web bending stresses which vary through the thickness of the web plate. Using a simple mathematical model to account for the shell action of a curved panel, the following expression for the web slenderness ratio required to limit these web bending

stresses was developed.

$$D/t = \frac{36,500}{(F_y)^{1/2}} \left[1 - 8.6(d_o/R) + 34(d_o/R)^2 \right]$$

For a flat panel, $(d_o/R)=0$, this reduces to the limiting value given in the load factor specification (40); since this limit exceeds the value in the working stress portion of the AASHTO Specification, it was revised to be in accord with the AASHTO working stress limit. Solving for the value of d_o/R corresponding to a value of $D/t = 23,000/(f_b)^{1/2}$ with $f_b = 0.55F_y$ gives $d_o/R = 0.017$. Rounding off this value to $d_o/R = 0.02$ does not significantly affect this value of D/t and is justified in view of the approximation and conservatism of the mathematical model. Thus, for curvatures less than or equal to this value, no reduction in web slenderness ratio is required based on web bending considerations. Replacing d_o/R by $(d_o/R - 0.02)$, replacing the yield stress by the bending stress f_b and simplifying the results above gives the following expression for the limiting web slenderness ratios proposed for curved girders for $d_o/R > 0.02$.

$$D/t = \frac{23,000}{(f_b)^{1/2}} \left[1.19 - 10(d_o/R) + 34(d_o/R)^2 \right]$$

Limiting the web depth-thickness ratios in accordance with this result should insure that the fatigue effects due to web bending in a curved panel will not be any more severe than in straight girders. Until fatigue test data becomes available, therefore, these limits appear reasonable.

The test results on curved plate and box girder webs of Lehigh University have indicated that the above equations from the CURT research are too severe. Accordingly, the equation now incorporates a reduction factor which is expressed as a linear function of transverse stiffener spacing instead of the quadratic form from CURT. The same mathematical mode is used, however a smaller initial out-of-straightness has been assumed. This smaller initial out-of-straightness accounts for the effects of deflections of the curved web boundary.

B. Girders Stiffened Longitudinally

Web slenderness limitations for longitudinally stiffened straight girders are also based on buckling considerations. The position and stiffness of the longitudinal stiffener are

based on forcing a node in the buckled configuration produced by the linearly varying stress gradient through the depth of the girder due to the internal bending moment. Tests indicated that these stiffeners also limit the web bending deformations in a straight girder thus insuring that the web effectively contributes to the moment carrying capacity of the cross section (47).

The simple model mentioned above in connection with transversely stiffened webs was also used to determine the limiting value of D/t for longitudinally stiffened curved panels. Since the model treated the tension and compression region did not affect web bending in the tension area. In order to limit this web bending in the tension area to that which occurs in straight panels, therefore, a reduction in D/t with increasing panel curvature was necessary for singly stiffened curved plate girders. Since the addition of a longitudinal stiffener significantly reduces this web bending, no reduction in D/t was required for the case when longitudinal stiffeners are provided in both the tension and compression regions. The equation $t = [D(F_y)^{1/2}] / 62,000$ yields the same D/t ratios as shown in Art. 10.34.3.2.

A summary of the web plate thickness requirements for stiffened web panels is indicated in Figure 9 for an A36 girder. When transverse stiffeners only are provided, no reduction in D/t below the straight girder value is required up to a panel curvature of $d_o/R = 0.02$. Above this curvature D/t must be reduced. When a single longitudinal stiffener is provided in the compression area, D/t must be reduced from the straight girder value of 330 as the panel curvature d_o/R increases. If adequate longitudinal stiffeners are provided in both the tension and compression regions, no reduction in D/t below the straight girder value is required.

10.112

TRANSVERSE INTERMEDIATE STIFFENERS

The formula in Article 10.34.4.1 is the allowable shear stress for a straight unstiffened girder. It is based on the elastic shear buckling equation for a flat web panel with simply supported boundary conditions using a buckling coefficient $k = 5$. It provides a factor of safety of 1.8 against elastic shear buckling, which is consistent with other factors of safety inherent in the specification. It has been shown (48) that the elastic buckling strength of a curved web panel is greater than that of a straight panel with the same aspect ratio, slenderness ratio and boundary conditions. However, for practical curvature the buckling strength will not be significant in most cases. The formula in Article 10.112 to determine the transverse stiffener spacing is equivalent to the shear buckling strength of a

flat straight-girder web panel simply supported at the flange and stiffener boundaries. The nondimensional parameter C , given by the provisions of Article 10.34.4.2, is the ratio of the elastic shear buckling stress to the shear yield stress. The upper limit of $D/t_w (7500(k/F_y)^{1/2})$ for elastic shear buckling was determined by setting $C = 0.8$. The lower limit of $D/t_w (6000(k/F_y)^{1/2})$, which defines the limit between inelastic shear buckling and shear yielding, was determined by setting $C = 1.0$.

Furthermore, it has been noted in a report by Ilyasevitch and Klujev (49) that with increasing curvature in girder webs, there is an accompanying decrease in the post buckling reserve strength. A similar effect was observed in tests on curved plate girders (34, 35). Results from these tests are compared in Figure 10 to calculated results based on Basler's (50) postbuckling strength theory for straight girders. Note that while most of the curved girder specimens exhibited shear strengths V_u which exceeded the straight girder shear buckling strength V_{cr} none were able to develop the calculated straight girder ultimate strength.

In light of the above observations, it is recommended that for curved girders, the spacing of intermediate transverse stiffeners be determined from the AASHTO formula for the shear buckling capacity. Although recommendation is conservative in that the AASHTO formula does not directly include postbuckling reserve strength, it is felt that more information is needed before a postbuckling theory can be incorporated into the design of curved girder webs.

It is further recommended that the maximum intermediate transverse stiffener not exceed the depth of the web plate. At end panels, the spacing of the first transverse stiffener at the simple support end of a girder is further limited to $0.5D$. Future study may lead to an eventual relaxation of these requirements to increase the maximum intermediate transverse stiffener spacing at end panels for curved girders to the maximum values presently allowed for straight girders.

The required moment of inertia for the transverse stiffeners on curved web panels was determined for a stability analysis (48). Considering a multiple stiffened curved web subjected to pure shear, the relationship between the buckling coefficient and the stiffener rigidity was determined. The optimum value of the stiffener rigidity required to force a nodal line in the buckled panel at the stiffener locations was then evaluated. Relating this optimum value of I as a function of the panel curvature to the value for a flat web panel, the final expression for J in terms of the flat panel value modified by a curvature correction factor X was obtained. This factor is compared to theoretical results in Figure 11. Note that for the lower aspect

ratios ($d/D = 0.5, 0.75$), X is less than one. It is recommended however, that the curved girder stiffener requirements should not be less than existing requirements for straight girders. Thus, for aspect ratios $d/D \leq 0.78$, the correction factor is one. For aspect ratios in the range $0.78 < d/D \leq 1.0$, the formula in Article 10.112 for the correction factor approximates the theory to within 6 percent (48).

In order to prevent buckling of transverse stiffener, the requirement on minimum stiffener thickness used in load factor design (40) was adopted in this Article.

Test results obtained in the CURT program (35) have indicated that transverse stiffeners may be omitted in a curved girder if the web thickness complies to the limits given in Article 10.34.4. In this case measured plate bending stresses in the web due to cross sectional deformation were no more severe than in the case of a slender curved web with transverse stiffeners. Furthermore, experimental (35) and analytical investigations (51) have shown that transverse stiffeners of practical sizes do not significantly reduce lateral flange distortions in a curved girder.

10.113 LONGITUDINAL STIFFENERS

Analytical studies (46) indicated that the present AASHTO requirement for the moment of inertia of the longitudinal stiffeners is also adequate for curved web panels. In order to prevent buckling of the stiffener due to the stress associated with the internal moment in the girder, the requirement regarding the minimum radius of gyration of the stiffener used in load factor design (40) was adopted for curved girders.

CURVED COMPOSITE I - GIRDERS

10.115 EFFECTIVE FLANGE WIDTH

Because of torsional and curvature effects, strain distributions across the slab in a curved composite bridge are not uniform. However, calculated results for displacements and member forces from the Syracuse University Three Dimensional Computer Analysis (10) using member section properties based on the effective width criteria in Article 10.38.3 showed good agreement with measurements from small scale curved bridge tests (52, 53). Research is currently in progress on this criteria.

10.116 NONCOMPOSITE DEADLOAD STRESSES

Flange flexural and tip (flexural plus warping) stresses in the steel section alone in both the top and bottom flanges should first be computed under noncomposite dead loads only and checked using the provisions of Article 10.109. The deck-casting sequence should be considered in computing the noncomposite dead-load stresses.

10.117 COMPOSITE SECTION STRESSES

When computing the allowable flexural stresses in the top flange of composite sections due to the total dead plus live load after the concrete deck has hardened, it can be assumed that the hardened composite concrete deck is sufficient to prevent local and lateral distortions of the top compression flange. Therefore, the reduction factors ρ_b , ρ_w , and the factor computed from the ratio of (I/r') for the compression flange between cross frames are not necessary when computing the allowable normal flange stress, F_b , for the total dead plus live load in top compression flanges of composite sections after the concrete deck has hardened.

Also, warping stresses in the top flange due to the total dead plus live load may be neglected for composite sections after the concrete deck has hardened. In composite girders, the top flange and concrete deck act together to resist the warping stresses in the top flange. The combined section modulus of the top flange and concrete deck to resist lateral bending is so large that top-flange warping stresses in composite sections are negligible.

10.118

SHEAR CONNECTORS

A curved girder is subjected to shear forces induced by bending and torsional forces. In a composite section these forces will act at the interface of the bottom of the slab and top flange of the girder. In order to maintain equilibrium of the system and insure that these forces can develop, shear connectors must be provided. The number of shear connectors at each section must be such to resist the torsional and bending shears.

According to present design concepts, both elastic and ultimate shear forces need to be evaluated. The following will illustrate the required elastic torsional forces that are developed (54). The results will also indicate that these forces are negligible, thus the straight girder equation based on fatigue may be used.

(A) Fatigue

Pure Torsional Shear

As observed during the testing of composite beams (28), the torsional rigidity of a composite section is equal to the sum of the torsional rigidities of the plate element parts. However, this summation assumes that the shearing stresses in each element are zero at the mid-depth and maximum at the faces, as shown in Fig. 12. If this condition is assumed at the interface between the slab and girder, there would be a discontinuity in shear stress. In order to create continuity at this junction and thus evaluate the shearing stress, an idealization of the section will be made.

The original composite girder is as shown in Fig. 13 and the idealized girder is as given in Fig. 14. The idealization assumes that the concrete slab and top flange plate are now considered to

be one plate of length b_f and $t_e = t_f + t_s$. It will be assumed that this plate has an equivalent shear modulus G_e .

Assuming full composite action, i.e., no slip, the rotation of the two plates must be equal to each other and the equivalent plate, shown in Figure 15. Therefore,

$$\frac{T_{ST}}{G_e K'_e} = \frac{T_c}{G_c K_c} = \frac{T_s}{G_s K_s} \quad (1)$$

where:

T_{ST} = torque in slab and flange plate

T_c = portion of torque in concrete slab

T_s = portion of torque in tip flange plate

$G_e K'_e$ = torsional rigidity of equivalent plate

$G_c K_c$ = torsional rigidity of concrete slab

$G_s K_s$ = torsional rigidity of top flange plate

also

$$T_{ST} = T_c + T_s \quad (2)$$

Therefore,

$$T_c = \frac{T_{ST} G_c K_c}{G_e K'_e} \quad (3)$$

$$T_s = \frac{T_{ST} G_s K_s}{G_e K'_e} \quad (4)$$

Substituting Eqs. 3 and 4 into Eq. 2 gives:

$$G_e K'_e = G_c K_c + G_s K_s \quad (5)$$

Eq. 5 states that the torsional rigidity of the equivalent plate equals the sum of the rigidities of the parts. Using

$K = (1/3)(b)(t)^3$ for plate elements, Eq. 5 reduces to :

$$G_e = \frac{G_c t_s^3 + G_s t_f^3}{t_e^3} \quad (6)$$

The maximum shearing stress of this equivalent plate is computed by the equation:

$$\tau = \frac{T_{ST} t}{K_T}$$

or

$$\tau_{\max} = T_{ST} (t_e/2) (G_e/G_s) (1/K_T) \quad (8)$$

where T_{ST} is the total pure torsional moment at a given section and K_T is the total torsional stiffness of the entire idealized section, shown in Figure 14.

The total force across the steel flange, assuming a constant stress block rather than the linear condition as given in Figure 15, is equal to:

$$F_{ST} = \tau b_f t_f$$

$$F_{ST} = T_{ST} (t_e/2) (G_e/G_s) (1/K_T) (b_f t_f)$$

Substitution of Eq. 6 into the above gives:

$$F_{ST} = T_{ST} (t_e/2) (b_f t_f / K_T) \left[\frac{(m t_s^3 + t_f^3)}{t_e^3} \right] \quad (9)$$

where:

$$m = \frac{G_c}{G_s}$$

Assuming $K_T = (m/3) b_s t_s^3$, $t_e = t_s$, $t_f \ll t_s$,

Eq. (9) reduces to:

$$F_{ST} = T_{ST} \left[\frac{3 b_f t_f}{2 t_s^2 b_s} \right] \quad (10)$$

As in the case of induced shear due to changing moment, the change in force F_{ST} from section to section is the required lateral force the shear connector must take. Therefore, the change in force over a unit distance is represented by:

$$\Delta F_{ST} = [T_{ST(i)} - T_{ST(i+1)}] \left[\frac{3 b_f t_f}{2 t_s^2 b_s} \right] \quad \text{or} \quad \Delta F_{ST} = \Delta T_{ST} \left[\frac{3 b_f t_f}{2 t_s^2 b_s} \right] \quad (11)$$

where (i) and (i+1) are adjacent sections. This change with respect to X gives:

$$\frac{\Delta F_{ST}}{\Delta X} = q_{ST} = \frac{\Delta T_{ST}}{\Delta X} \left[\frac{3 b_f t_f}{2 t_s^2 b_s} \right]$$

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where q_{ST} is the shear flow in K/in.

The torque $T_{ST} = G_s K_T [\phi' + (\eta'/R)]$;

therefore, $\frac{dT_{ST}}{dX} = G_s K_T [\phi'' + (\eta''/R)]$;

however, $Bi = E_s I_w [\phi'' + (\eta''/R)]$;

$$\text{therefore, } q_{ST} = \frac{dT_{ST}}{dX} = Bi \left[\frac{G_s K_T}{E_s I_w} \right] \left[\frac{3b_f t_f}{2t_s^2 b_s} \right] \quad (12)$$

$$\text{Defining } A = \frac{G_s K_T}{E_s I_w} ; \quad q_{ST} = BiA \left[\frac{3b_f t_f}{2t_s^2 b_s} \right] \quad (13)$$

Warping Shear

In addition to pure torsional stresses, the composite girder will also be subjected to warping shear. This shear can be related to the warping torsional moment by the following simplified equation:

$$V_L d_2 = T_w \quad (14)$$

where $d_2 = d_g + (t_s/2) - (t_f/2)$ as illustrated in Figure 7. The

resulting shear flow is,

$$\text{therefore, } q_w = (dT_w/dX)(1/d_2) \quad (15)$$

Total Horizontal Shear

Summarizing vectorally the shears due to torsion and bending gives:

$$S_r = \left[(VQ/I)^2 + \left[BiA \left[\frac{3b_f t_f}{2t_s^2 b_s} \right] + (dT_w/dX)(1/d_2) \right]^2 \right]^{1/2} \quad (16)$$

This equation can be reduced by examining the magnitude of typical forces induced in curved bridges and typical properties.

Example :

As an example, consider a bridge girder W36 X 280 with 14.5 x 1-3/4 cover plate and slab 8.5" x 84", spanning 100'. The properties of this girder are:

$$A = (1/(2220)) \text{ in}^2 ; b_f = 16.6 \text{ in.} ; t_f = 1.6 \text{ in.} ; d_2 = 40.8 \text{ in.}$$

An examination of the maximum induced live load forces in a curved bridge (24) with $R_{\min.} = 100'$, $L_{\min.} = 100'$ gives

$$B_i(\max) = 10,000 \text{ K-in.}^2$$

$$\left[\Delta T_w / \Delta X \right] (\max) = 2 \text{ K-in./in.}$$

$$V (\max) = 46^K$$

Pure Torsion Effect:

$$q_{ST} = B_i A \left[\frac{3b_f t_f}{2t_s^2 b_s} \right] = (10,000/2,220) \left[\frac{3 \times 16.6 \times 1.6}{2 \times 72 \times 84} \right]$$

$$q_{ST} = 79,500 / (26.6 \times 10^5)$$

$$q_{ST} = .0298 \text{ k/in} \quad \text{which is negligible.}$$

Warping Effect:

In order to evaluate dT_w/dX , an examination of the T_w diagram is required. An examination of such typical diagrams (25) indicates that the maximum change in T_w per unit of length is generally much less than 2K-in./in. Therefore, assuming this maximum value of 2K-in./in., the shear flow is then equal to:

$$q_w = (\Delta T_w / \Delta X) (1/d_2) = 2/40.8 = .049 \text{ K/in}$$

Bending Effects:

$$q = (VQ/I)$$

$$q = 46 \times 1160/57,664 = .93^K/\text{in.}$$

The resulting shears or stress range per Eq. 15 are, therefore,

$$S_r = \left[(.93)^2 + (.030 + .049)^2 \right]^{1/2} = (.865 + .0063)^{1/2} = .935 \text{ K/in}$$

The differences when torsional effects are neglected:

$$S_r = (.935 - 0.93) / (.935) = .54 \%$$

Therefore, the warping and pure torsional shear flows may be neglected for most practical problems. Thus, the design of the shear connector for curved bridges for fatigue would be the same as for straight girders. The final equation would, therefore, be:

$$S_r = VQ/I \quad (17)$$

2. Ultimate Strength

Bending Stress

The forces on the connectors at ultimate bending load depend on the location of the neutral axis of the composite beam at failure. The force to be developed by the connectors at ultimate load when the neutral axis lies in the steel is

$$P = 0.85f_c'A_c \quad (18)$$

which is based on the usual rectangular stress block theory for ultimate strength design of reinforced concrete. When the neutral axis lies in the slab,

$$P = A_s f_y \quad (19)$$

and the entire steel section is at yield stress.

The ultimate capacity of a composite curved girder is influenced by the smaller of the two P values from Eqs. 18 and 19. For straight girders this value is used in the equation

$$P_c / (.85S_u)$$

where $P = P_c$, to determine the number of shear connectors.

Eccentricity of the Longitudinal Force Due to Bending

For a straight composite beam, the force P obtained as described above is colinear with shear connectors and according to experimental tests, the shear connectors may be spaced equally along the length of a straight beam without reducing the total load-carrying capacity of the member. This is possible since after a connector develops its maximum force capacity, any excess force to be carried will be resisted by adjacent connectors.

Figure 16 illustrates the additional complication caused by curvature of the beam axis (54). In this case, the force P developed at the section of maximum moment is not colinear with the shear connectors and, due to this eccentricity, the load on each connector must account for this torsional effect.

The effect of the eccentricity is accounted for in the following equations which are based on the following two assumptions:

1. Shear connectors are spaced uniformly along line OAB. The shear connectors are located along the curved axis AB. By assuming that they are actually located along line AOB, the resulting equations are simplified and conservative regarding the magnitude of the connector

force. Also since in most bridge structures R is large ($R > 100'$), the error involved in this assumption should not be significant.

2. The force in each connector due to the eccentricity of P with respect to the centroid of the connectors (point O in Figure 16) is proportional to the distance of the connector from point O . This assumption is conservative regarding the number of connectors required for a given force P .

Based on the above assumptions, the end force P may be repositioned at the centroid of the connectors, in addition to a torque equal to $(PR/2)(1-\cos\theta)$. The force in each extreme connector is equal to the sum of two components, namely P/N , where N is the number of connectors, and a force result in from the eccentricity of P . This latter force may be written as

$$F = \frac{PR(1-\cos\theta)}{N_s 4RK\sin(\theta/2)} = \frac{P(1-\cos\theta)}{N_s 4K\sin(\theta/2)} \quad (20)$$

$$\text{where: } K = 0.166[(N/N_s)-1] + .375 \quad \text{and} \quad (21)$$

N_s is the number of connectors placed at each section. The resultant force P_c in the extreme connector is the vectorial sum of P and F . It is assumed conservatively that the maximum value of P_c occurs at B or $(L/2)$, as shown in Figure 16, and is equal to

$$P_c = \left[[P + F\sin(\theta/2)]^2 + F^2 \cos^2(\theta/2) \right]^{1/2} \quad \text{or} \quad (22)$$

$$P_c = \left[P^2 + F^2 + 2PF\sin(\theta/2) \right]^{1/2} \quad (23)$$

$$P = P/N$$

Lateral Shear Forces

As illustrated previously, the elastic torsional shear forces are negligible. However, at ultimate loading, these forces might reduce the permissible bending moment. A study of the plastic collapse of curved composite bridges (55) indicates that if the ratio $\alpha = (T_p/M_p)$ (i.e., the full developable plastic torque to the full developable plastic moment) is greater than 0.40, the full plastic moment can then be developed with a maximum reduction of 5%. Also, with $\alpha > .40$, the collapse will be a bending mode not torsional mode. This mode of collapse can be insured with adequate lateral bracing.

Therefore consideration of lateral shears in estimating the full load capacity of the member is not required. The effect of the eccentricity of the load on the bending moment capacity need only be considered as previously given.

CURVED HYBRID GIRDERS

10.119 GENERAL

Details of the derivations for the hybrid curved girder reduction factors in Article 10.120 are given in a research report (41) to the Pennsylvania Department of Transportation. These factors were derived by means of an approximate analysis similar to that presented in the ASCE-AASHTO Subcommittee 1 Report on Hybrid Beams and Girders (56) for straight hybrid girders. Consequently, the general requirements in Article 10.40.2 will also apply to curved hybrid girders.

10.120 ALLOWABLE STRESSES

(A) Bending - Noncomposite Girders

Test Results (33, 34) reported to the U.S. Department of Transportation and 25 participating states as part of the CURT project involved bending tests on curved plate girders. Some specimens were fabricated with web plates having significantly lower yield strength ($F_{yw}/F_{yf} \sim 0.73$) than the flanges.

These test specimens behaved as homogeneous girders of the flange steel up to the load at which initial yielding in the flanges occurred. Therefore, the design of a hybrid girder can be based on initial yielding in the flanges. To account for the lower web yield strength, the allowable flange stresses given in Articles 10.109 are multiplied by a reduction factor R , Eq. (a). This factor is simply the ratio of the flange yield moment for the hybrid section to the yield moment for a homogeneous section of the flange steel. Eq. (a) is similar to the reduction factor formula in Article 10.40.3 for straight girders, except the Eq. (a) contains a parameter α' in place of the yield stress ratio α , in the straight girder formula. For noncomposite sections in which the compression flange area is equal to or greater than the tension flange area the parameter α' is given by Eq. (b). In this case, a portion of the web adjacent to the tension flange is plastified at the moment for which the total stress, bending plus warping (lateral flange bending), at the tension flange tip equals the flange yield stress. Therefore, α' becomes a function of the yield stress ratio and the warping to bending stress ratio $|f_w/f_b|_t$ of the tension flange. If the tension flange warping to bending stress ratio is equal to or greater than the limit in Eq. (c), the yielding at the flange tip will occur at a lower

moment than web yielding and a reduction factor is not required. Reduction factors based on Eq. (a) and (b) are plotted in Figure 17 as a function of the area ratio, $\beta = A_w/A_t$, and total reductions as a percentage are listed in Table 1(a). As noted above, these allowable stress reduction factors are based on initial yield criteria at the flange tips and are usually small for most cases. If future Specifications include increased allowable stresses for hybrid girders with compact compression flanges [see Commentary for Articles 10.109], then it will be necessary to include formulae for allowable stress reductions based on plastic analysis of the hybrid girder section. Factors based on such an analysis have been developed in a research report to the Pennsylvania Department of Transportation (41).

(B) Bending - Composite Girders

In the positive moment region of composite girders, the stresses in the compression flange of the steel girder are low. Therefore, the tension flange stress condition will govern the design of the steel section for bending. In this case the hybrid girder reduction factor is determined by Eq. (a) and (b) which are the same equations used for noncomposite hybrid girders.

In the negative moment region of a continuous span composite girder, the steel section will essentially behave as a noncomposite girder. Therefore, the hybrid reduction factor is given by Eq. (a) in Article 10.120.

When the tension flange is attached to the concrete slab by shear connectors, warping stresses (lateral flange bending stresses) in that flange will be small and the parameter becomes a function of the warping to bending stress ratio in the compression flange. If this ratio is less than the limit in Eq. (d) of Article 10.120, the bending stress in the tension flange will control and $\alpha' = \alpha$. For this case, Eq. (a) becomes identical to the reduction factor in Article 10.40.2. For warping to bending stress ratios in the compression flange equal to or greater than the limit in Eq. (f), the compression flange will yield at a lower moment than the web and no reduction is necessary for hybrid girder behavior. For intermediate values of the compression flange warping to bending stress ratio, the total stresses at the compression flange tip will control yield and α' is given by Eq. (e). Reduction factors based on the application of these limits are plotted in Figure 18 and listed in Table 1b as total percent reductions.

10.121

PLATE THICKNESS REQUIREMENTS

The limiting width-thickness ratio for the compression flange given in Article 10.109 is based on a flange buckling model in which the rotational restraint provided by the web to the flange at the instant of local buckling is a minimum (38). No reduction in these limiting values is required, therefore, for hybrid girders to take into account the fact that portions of the web may be yielded when flange buckling occurs.

The limiting web thickness for a hybrid curved girder can be determined from the requirements in Article 10.111 based on the yield strength and allowable stress for the web material.

HEAT-CURVED ROLLED BEAMS AND WELDED PLATE GIRDERS

(See Art. 10.15)

TABLE 1. HYBRID GIRDER REDUCTION FACTORS

(a) NON-COMPOSITE GIRDERS AND POSITIVE MOMENT REGION
OF COMPOSITE GIRDERS

Steels	Yield Point Ratio,	Neutral Axis Location, ψ	Reduction as a Percentage					
			$\beta = 1$		$\beta = 2$		$\beta = 4$	
			$f_w =$ f_b		$f_w =$ f_b		$f_w =$ f_b	
			0	0.5	0	0.5	0	0.5
A514/A36	0.36	0.50	8	4	13	7	20	11
		0.75	10	5	17	9	24	13
		1.00	12	7	19	11	28	15
A514/A441	0.50	0.50	5	1	8	2	12	3
		0.75	6	2	10	3	15	4
		1.00	8	2	12	3	18	5
A441/A36	0.72	0.50	2	0	3	0	4	0
		0.75	2	0	3	0	5	0
		1.00	3	0	4	0	6	1

(b) NEGATIVE MOMENT REGION OF COMPOSITE GIRDERS

Steels	Yield Point Ratio, α	Neutral Axis Location,	Reduction as a Percentage					
			$\beta = 1$		$\beta = 2$		$\beta = 4$	
			$f_w =$ f_b		$f_w =$ f_b		$f_w =$ f_b	
			0	0.5	0	0.5	0	0.5
A514/A36	0.36	0.50	8	4	13	7	20	11
		0.56	8	7	14	11	21	17
A514/A441	0.50	0.50	5	1	8	2	12	3
		0.56	5	4	9	6	13	9
A441/A36	0.72	0.50	2	0	3	0	4	0
		0.56	2	1	3	1	4	1

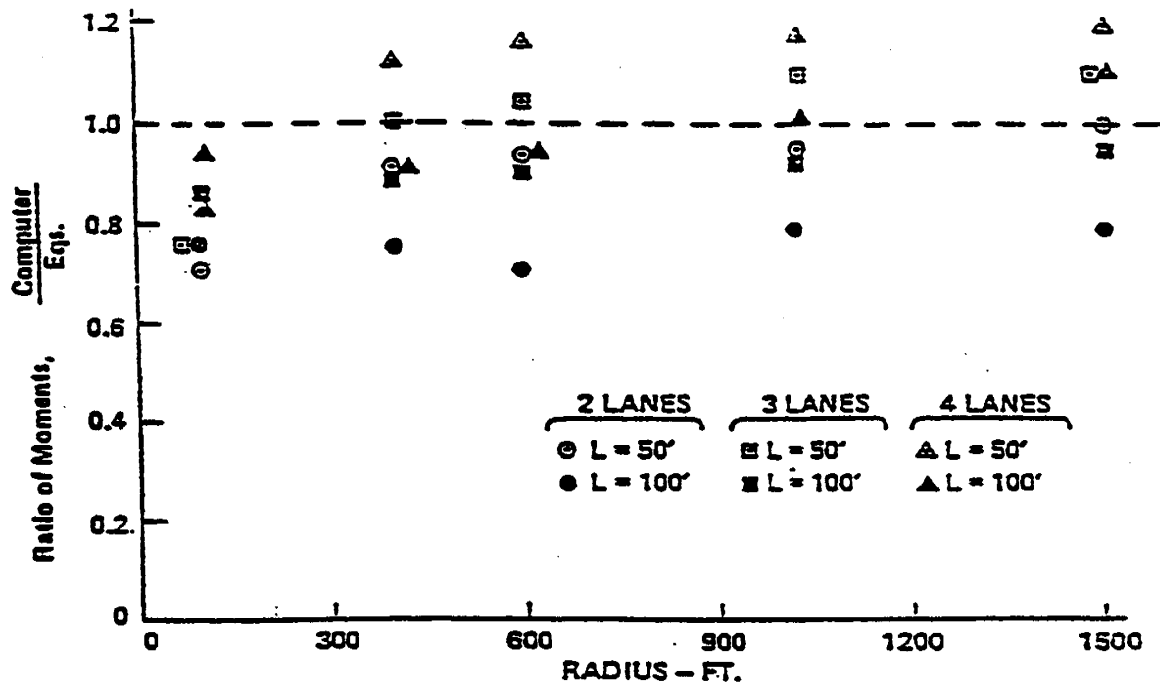


Figure 4. Computer Results vs. Equations-Moments

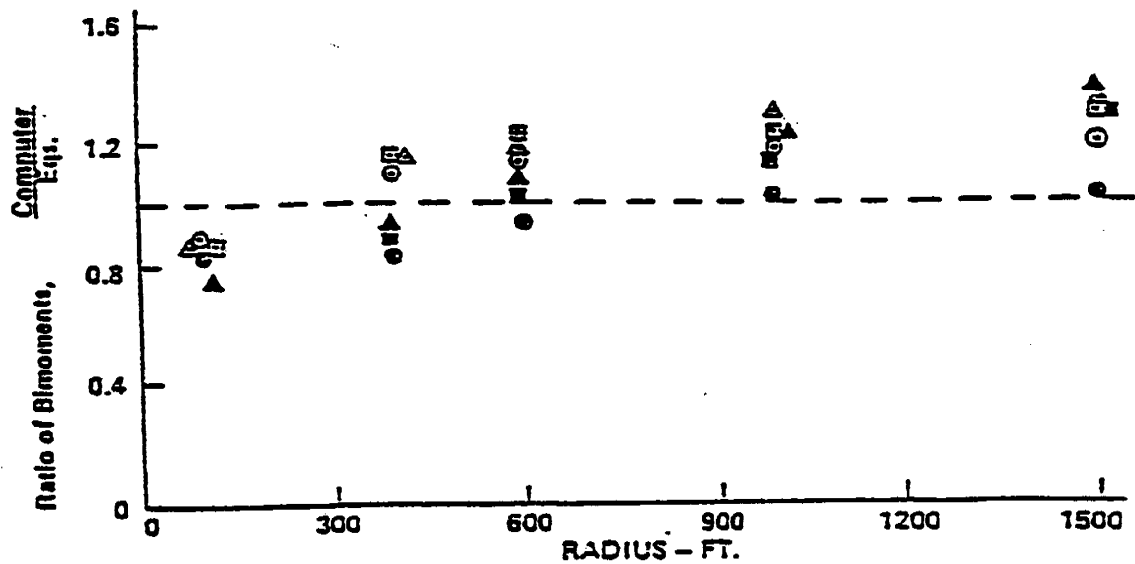


Figure 5. Computer Results vs. Equations-Bimoments

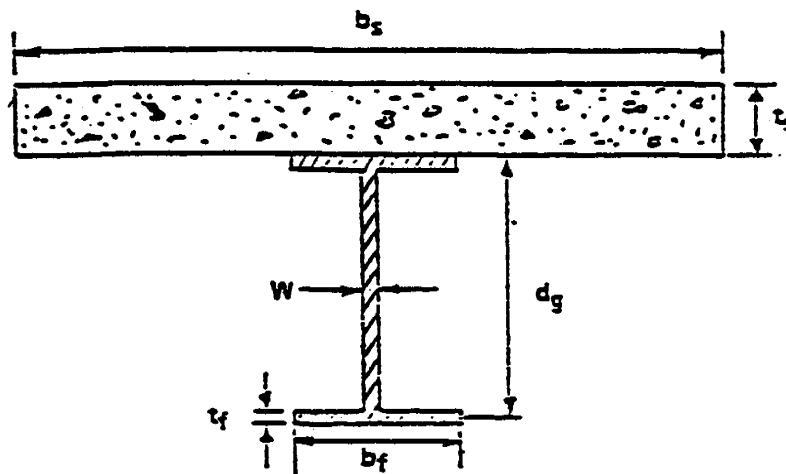


Figure 6. Typical Composite Section

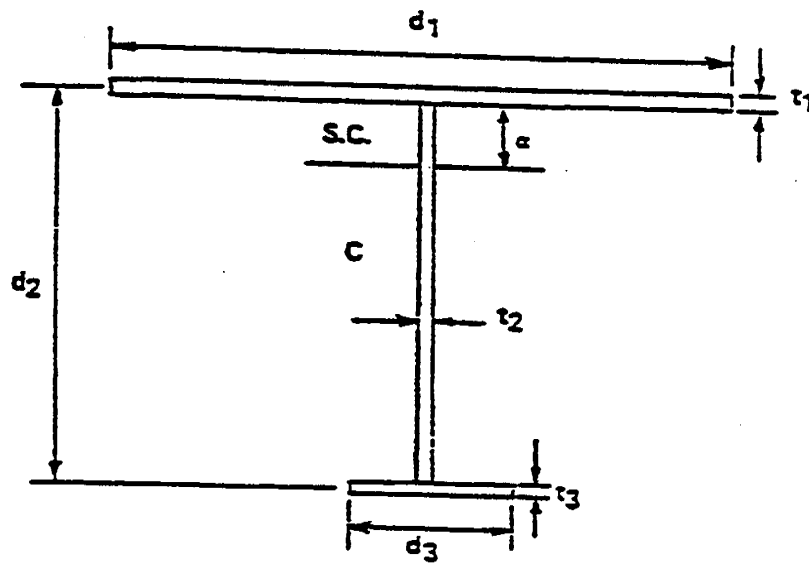


Figure 7. Idealized Composite Section

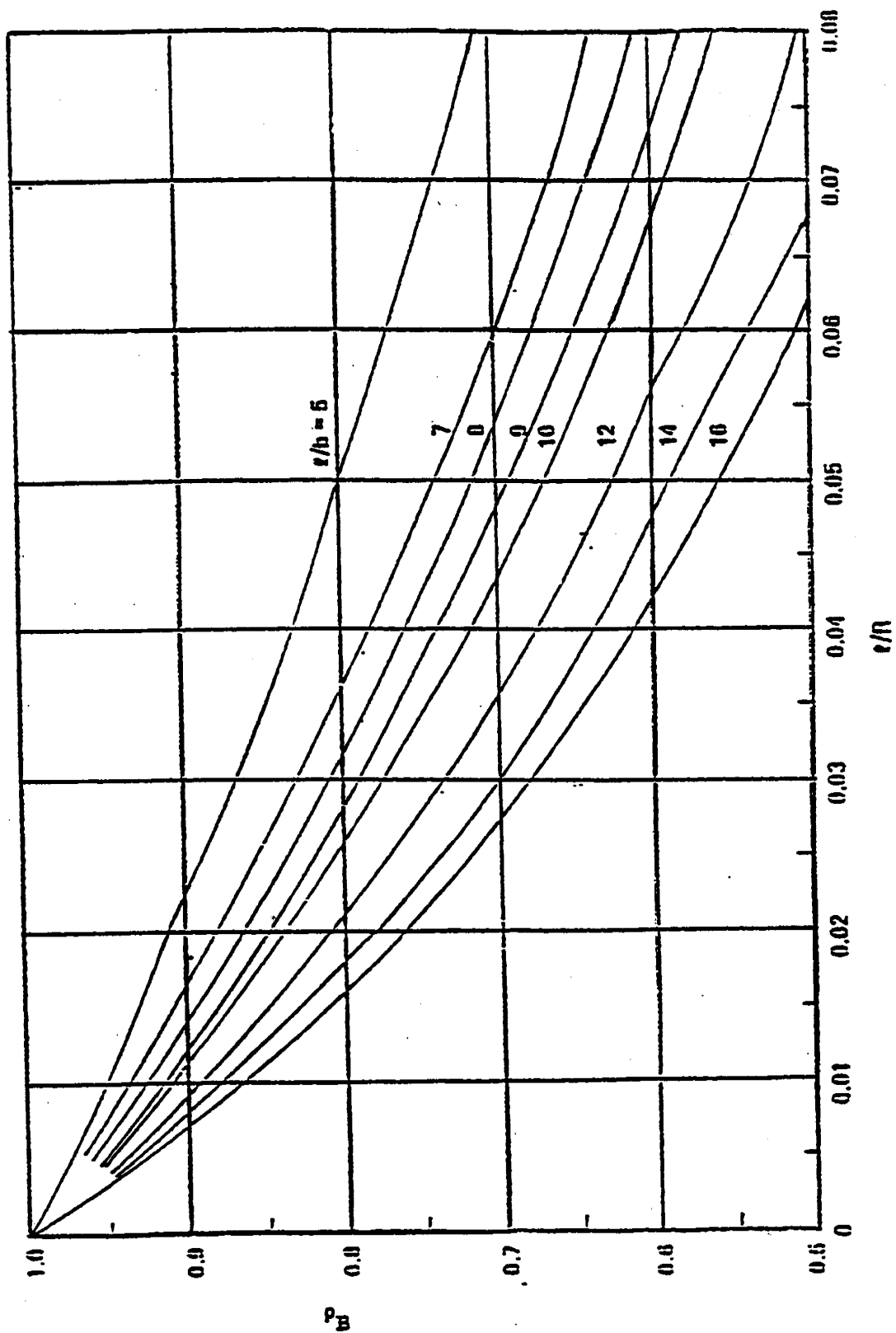


Figure 8. Curvature Correction Factor For Allowable Stress

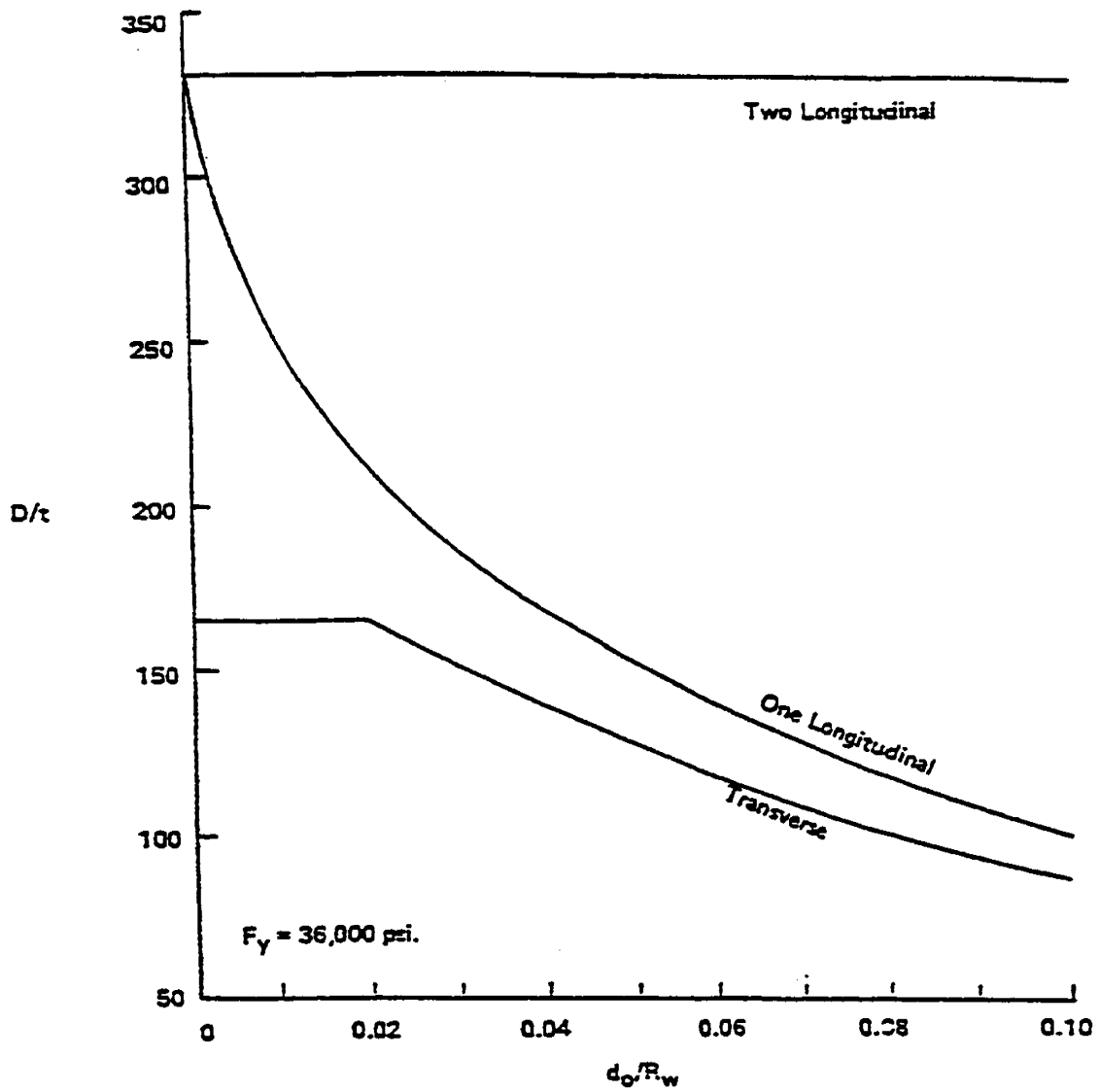


Figure 9. Limiting Web Slenderness Ratios

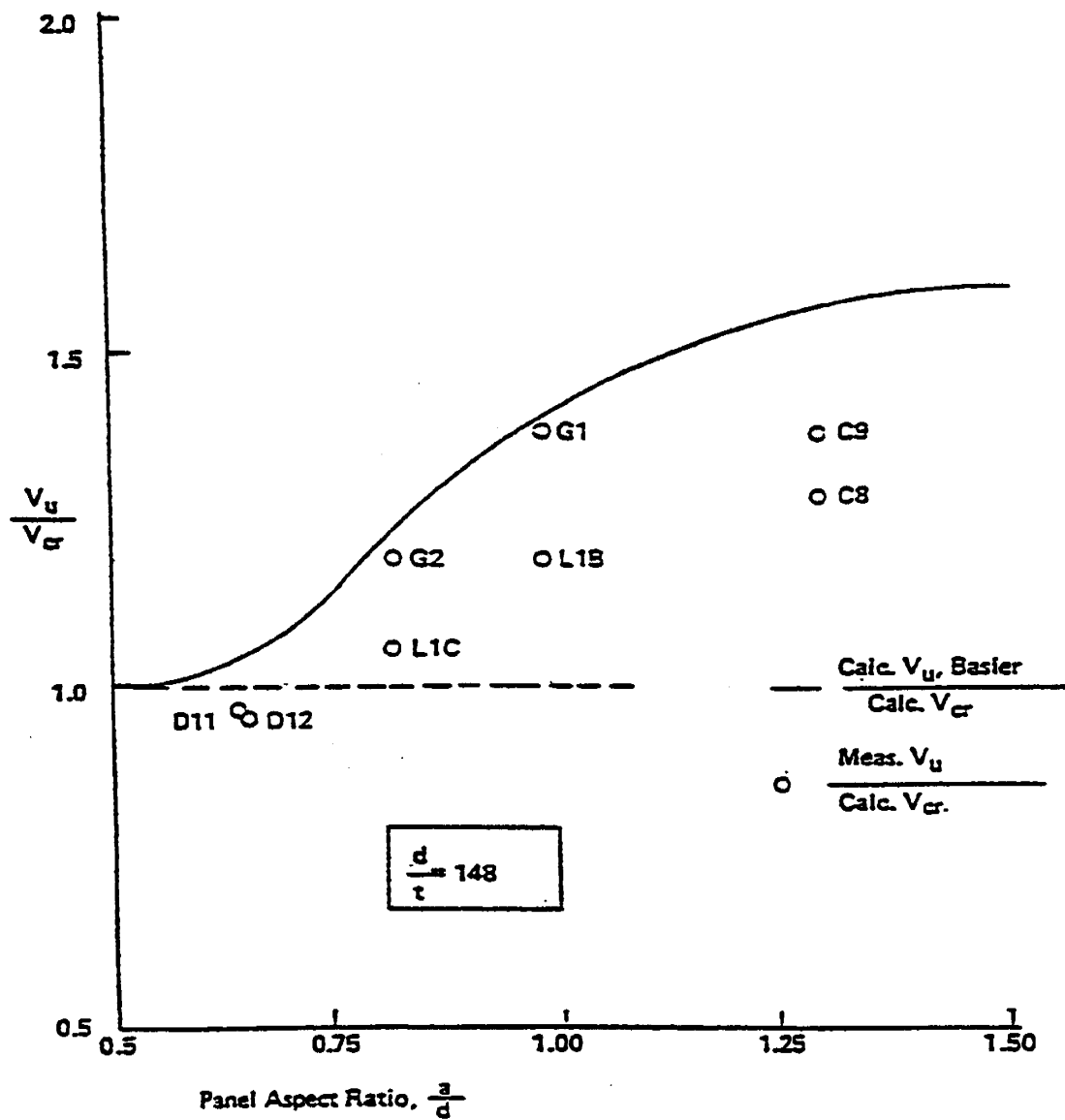


Figure 10. Comparison of Measured Shear Strength of Curved Girder Webs to Straight Girder Theory

Figure 10. Comparison of Measured Shear Strength of Curved Girder Webs to Straight Girder Theory

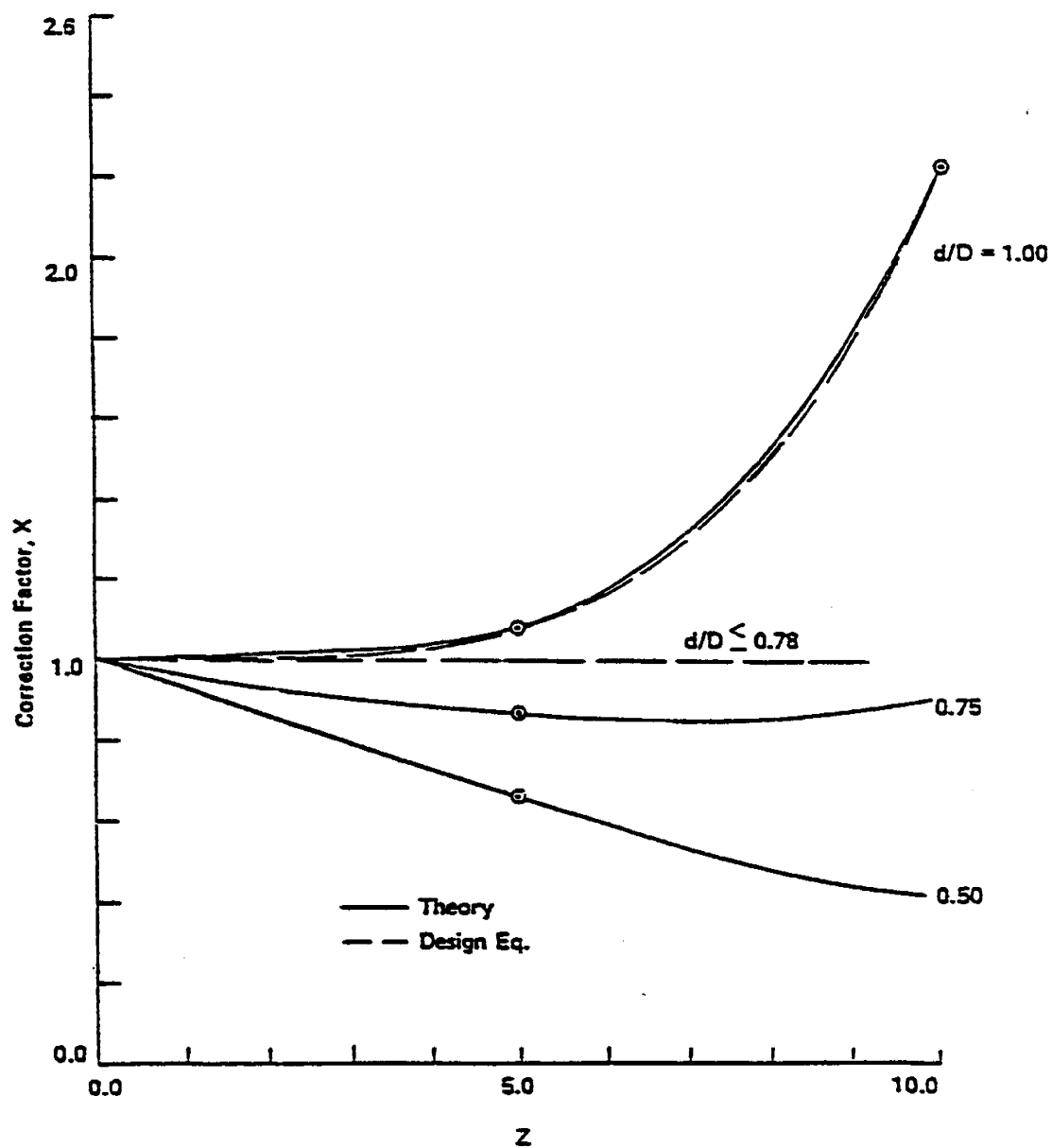
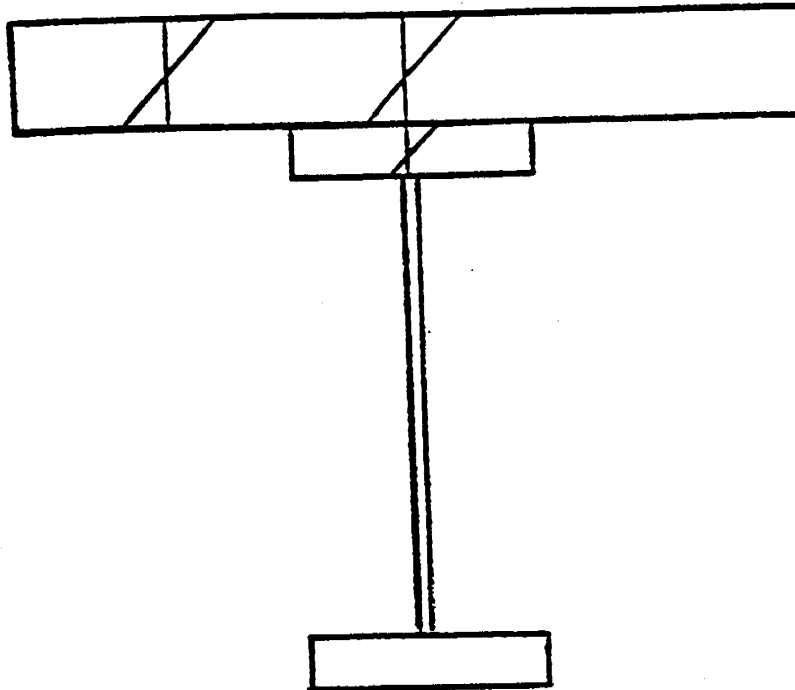
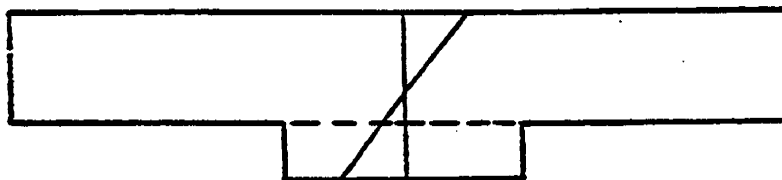


Figure 11. Curvature Correction Factor X for Transverse Stiffeners



(a) Simple Theory Stress Distribution



(b) Probable Stress Distribution

Figure 12. Composite Beam Stress Distribution

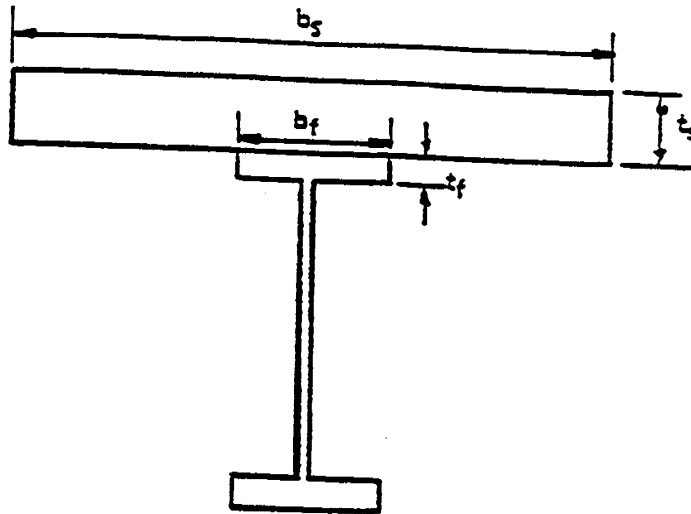


Figure 13. Original Section

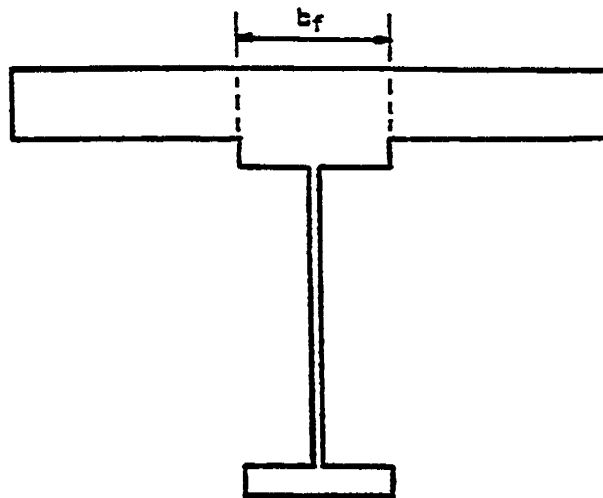


Figure 14. Idealized Section

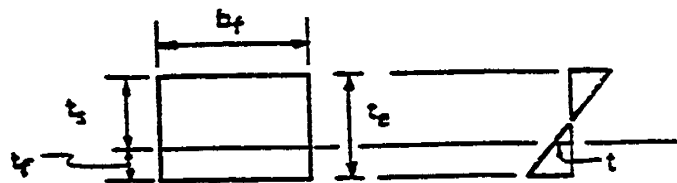


Figure 15. Equivalent Plate

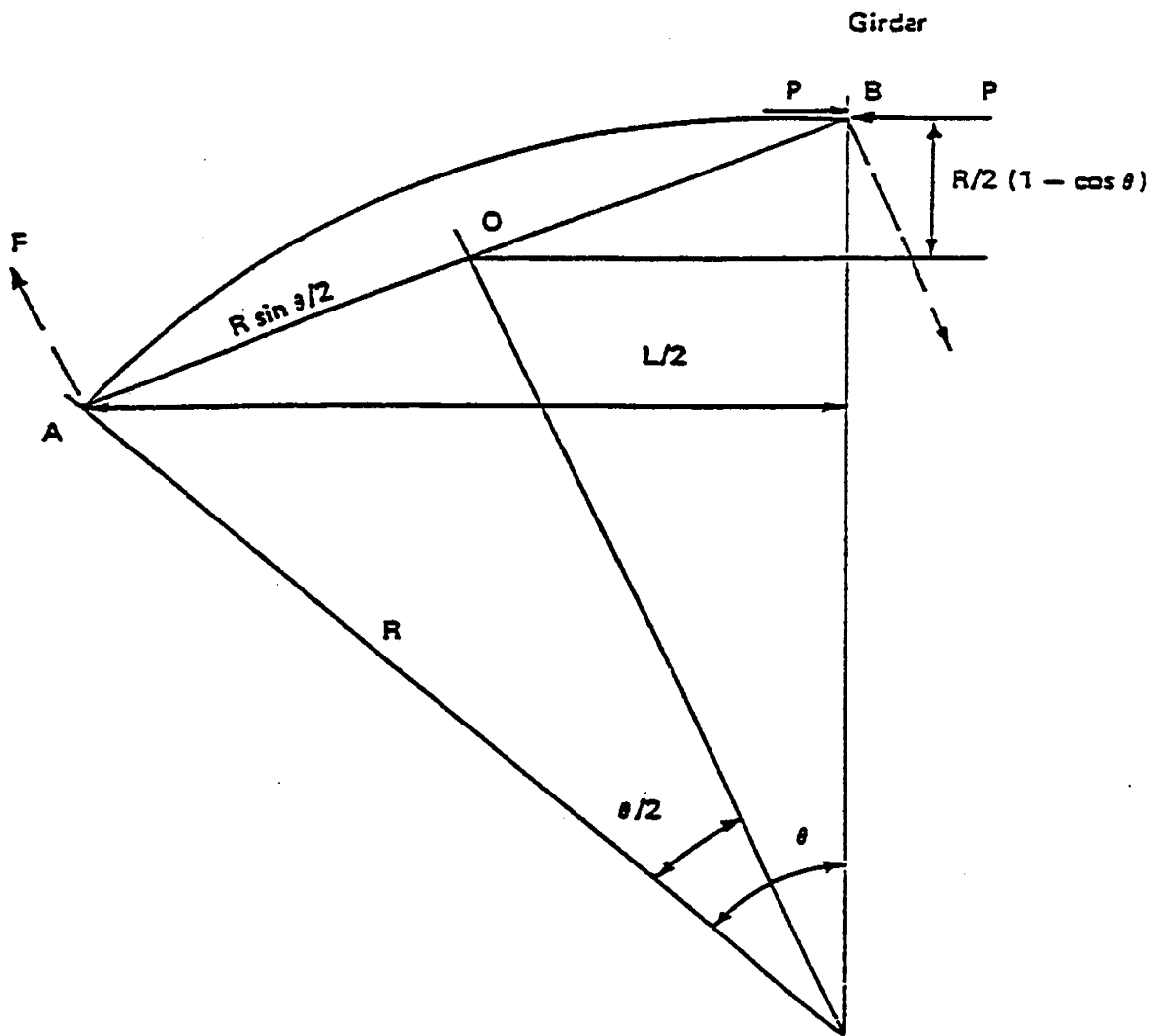


Figure 16. Eccentricity of P

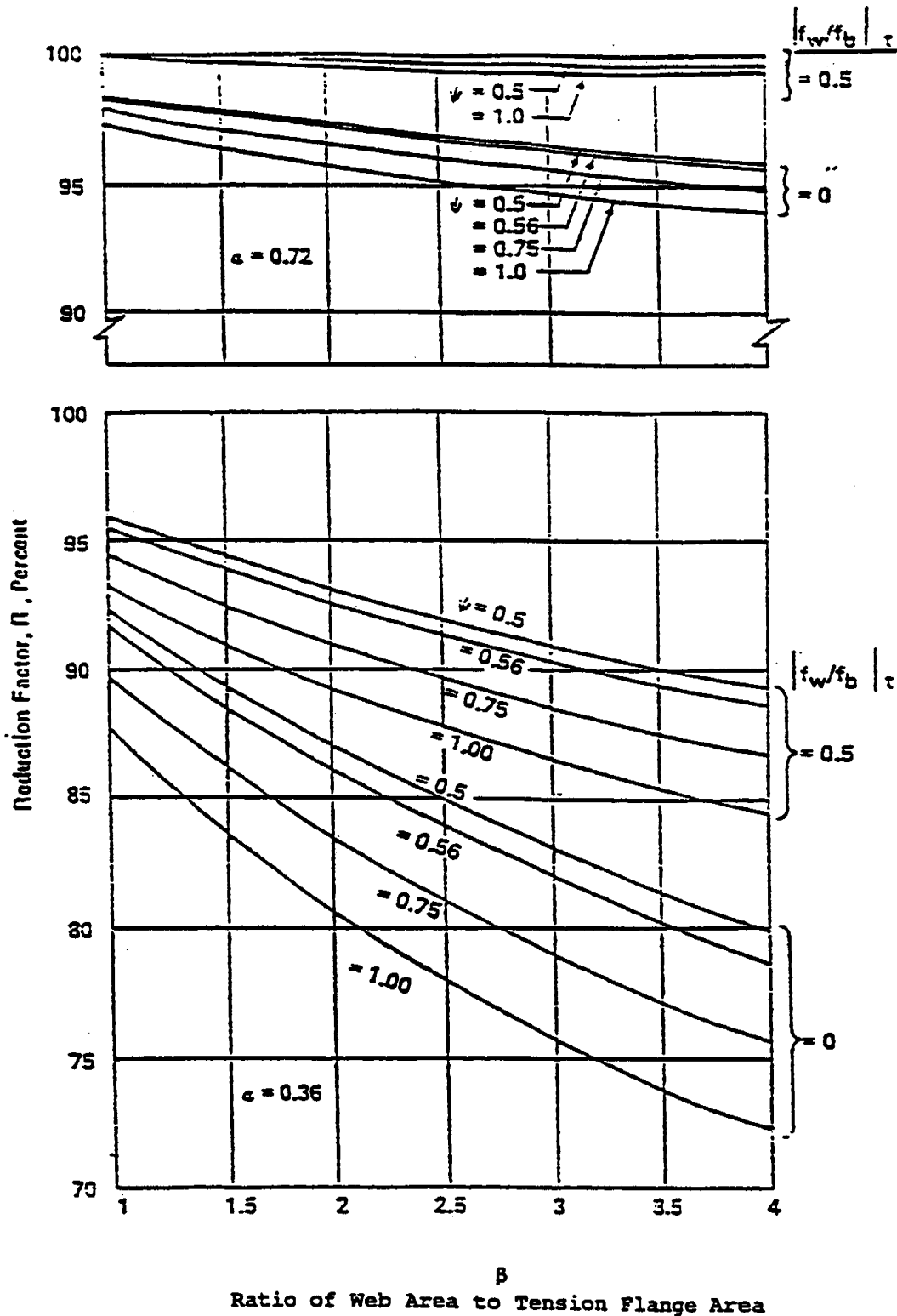
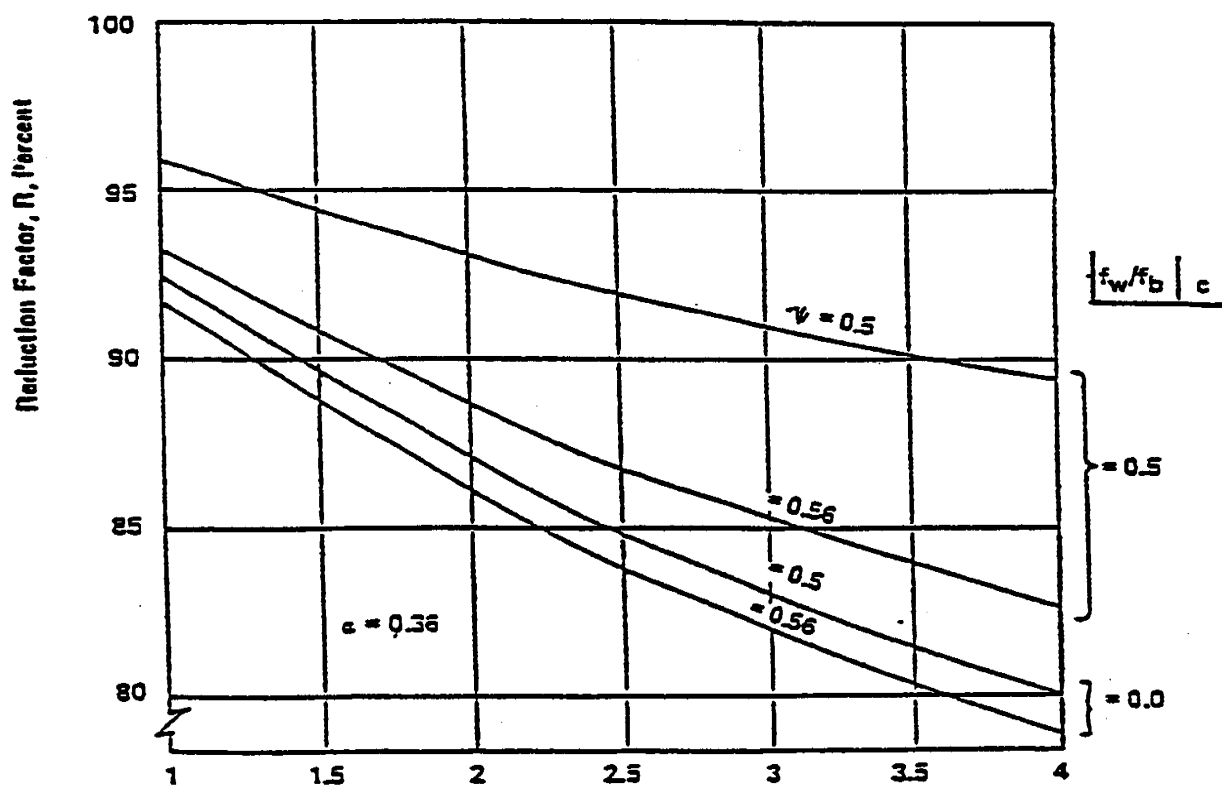
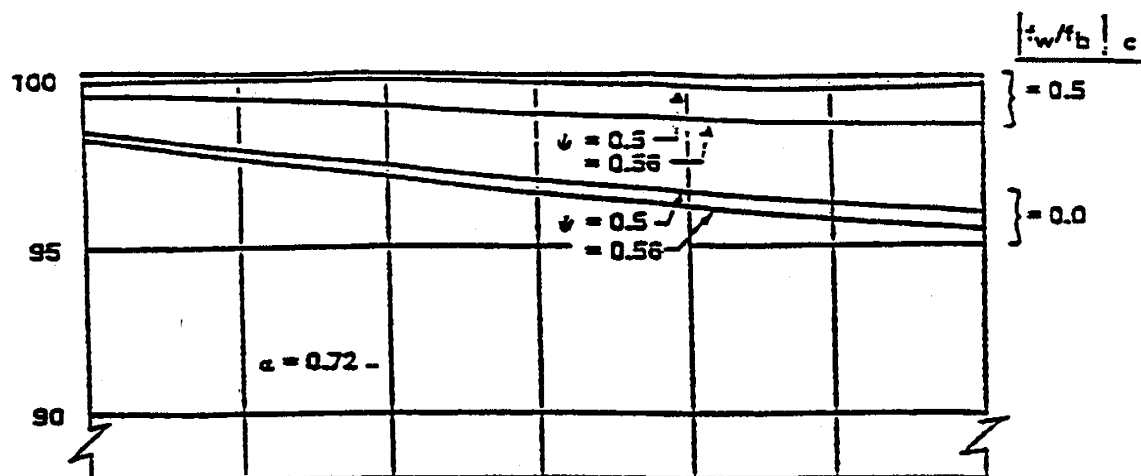


Figure 17. Hybrid Girder Reduction Factors -
Noncomposite Girders and Positive Moment Region
of Composite Girders



β
Ratio of Web Area to Tension Flange Area

Figure 18. Hybrid Girder Reduction Factors -
Negative Moment Region of Composite Girders

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12.1 GENERAL

12.1.1 Scope

Add the following paragraph to 12.1.1:

Design criteria for corrugated metal round pipe and pipe-arch culverts can be found in Chapter 8 of the NYSDOT Highway Design Manual. Buried metal conduits falling within the stated criteria shall be designed using the appropriate tables.

12.4.1.5 Minimum Cover

Change the sentence in the parentheses to read:

The minimum cover shall be measured from the bottom of pavement.

12.6.1.5 Minimum Cover

Change the sentence in the parentheses to read:

The minimum cover shall be measured from the bottom of pavement.

12.7 LONG SPAN STRUCTURAL PLATE STRUCTURES

12.7.1 General

Delete Article 12.7.1.1 and replace with the following:

- 12.7.1.1** Structural plate structures which exceed the maximum sizes are shown in Chapter 8 of the NYSDOT *Highway Design Manual*.

13.1.1 General

Add the following sentence at the end of the paragraph:

All the provisions of wood design, including the deflection criteria, under this section shall also apply to temporary detour timber structures.

14.1 SCOPE

Add the following paragraph at the end of Article 14.1:

The only types of bridge bearings allowed for installations on new and replacement bridges are Plain Elastomeric Bearings, Steel Laminated Elastomeric Bearings and Multi-Rotational Pot or Disc Type Bearings.

On rehabilitation projects, replacement bearings matching the existing bearing type found on the structure may be considered only when few of the existing bearings warrant replacement.

Any other bearing type shall not be used without prior approval by the D.C.E.S.

See the NYSDOT - *Bridge Manual* for a thorough discussion of bearing types and applications.

14.3

NOTATIONS

Add the following at the end of the definition for 'S':

When plain or laminated elastomeric bearings are used with prestressed box beams or slab units, holes are permitted for anchor rods to secure against horizontal movement. The effect of the holes must be accounted for since they reduce the loaded area and increase the area free to bulge. Suitable formulae are:

$$S = \frac{LW - \sum d_i^2 (\pi/4)}{h_{\max} (2L + 2W + \pi \sum d_i)} \text{ for rectangular bearings}$$

$$S = \frac{D^2 - \sum d_i^2}{4h_{\max} (D + \sum d_i)} \text{ for circular bearings}$$

Where:

L = Length of the rectangular bearing along the beam

W = Width of the rectangular bearing perpendicular to the beam

D = Diameter of the circular bearing

d_i = Diameter of the i^{th} hole in the bearing

h_{\max} = Thickness of thickest elastomeric layer

14.4 MOVEMENT AND LOADS

Delete the last three sentences of the second paragraph and replace with the following:

In all cases, both instantaneous and long-term effects shall be considered for all bearing designs, but the influence of impact need not be included for elastomeric bearing design.

14.4.1 Design Requirements

Add the following to the second paragraph after the statement “-the dead and live load rotations.”:

Dead load rotations may be ignored when a beveled sole plate is used to restore the bearing to a no rotation state under dead load. Otherwise, the dead load rotations must be considered.

Delete the following statement in the second paragraph:

“-an allowance for uncertainties, which is normally taken as less than 0.005 rad.”

And replace it with the following:

-an allowance for uncertainties, θ_c , equal to 0.002 rad.

Add the following to the third paragraph after the statement “-The greater of either the rotations due to all applicable factored loads or the rotation at the service limit state.”:

Dead load rotations may be ignored when a beveled sole plate is used to restore the bearing to a no rotation state under dead load. Otherwise, the dead load rotations must be considered.

Delete the following statement in the third paragraph:

“-an allowance for uncertainties, which shall be taken as 0.01 rad unless an approved quality control plan justifies a smaller value.”

And replace it with the following:

- an allowance for uncertainties, θ_c , equal to 0.005 rad.

14.5.2 Characteristics

Delete the text of Article 14.5.2 and refer to NYSDOT - *Bridge Manual* - Section 12.

14.6.5 Steel Reinforced Elastomeric Bearings-- Method "B"

Add the following notes:

Elastomeric Bearings shall be designed using Method "A", see Article 14.6.6.

14.6.6 Elastomeric Pads and Steel Reinforced Elastomeric Bearings -- Method "A"

Add the following note:

Elastomeric bearings reinforced with discrete layers of fiberglass, FGP, and pads reinforced with closely spaced layers of cotton duck, CDP, shall not be used.

14.6.6.2 Material Properties

Delete the text of Article 14.6.6.2, and replace with the following:

A nominal hardness (durometer) of 50 on the Shore "A" scale shall be used for design purposes. For calculations, the shear modulus shall be taken as the value from the range given in Table 14.6.5.2-1 that produces the most conservative bearing design.

All bearings shall be designed to the requirements of temperature Zone C as given in Table 14.6.5.2-2.

Add the following as Article 14.6.6.5:

14.6.6.5 Shear Resisting Steel Pin

The steel pin of a fixed elastomeric bearing shall be capable of resisting a horizontal force in any direction equal to the larger of the design shear force or 19% of the combined vertical dead load and superimposed dead loads.

14.6.8.5 Shear Resisting Mechanism

Delete the second sentence of the first paragraph and replace with the following:

It shall be capable of resisting a horizontal force in any direction equal to the larger of the design shear force or 19% of the combined vertical dead and superimposed dead loads.

14.6.9.2 Design Loads

Delete the second bulleted sentence and replace with the following:

- 19% of the combined vertical dead and superimposed dead loads acting on all the bearings at the bent divided by the number of guided bearings at the bent.

17.1 GENERAL

17.1.2 Notations

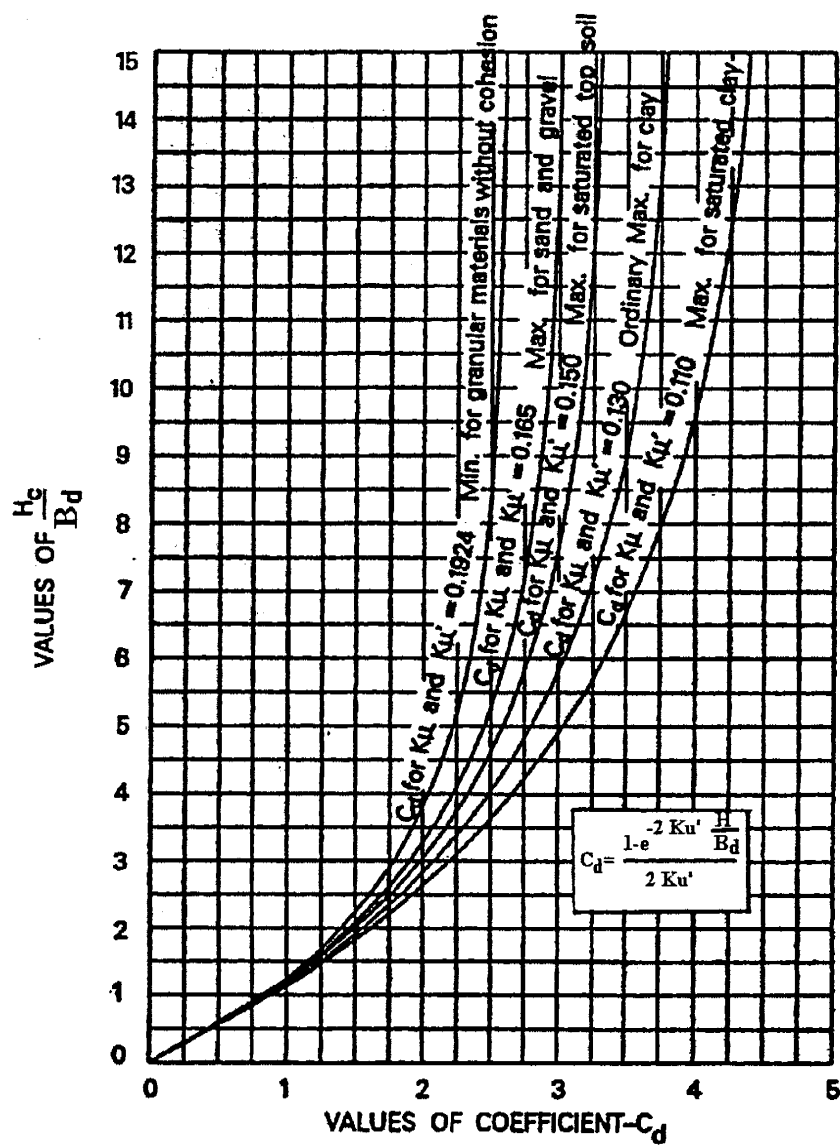
Replace the following notations:

- A_p = total active lateral pressure acting on pipe, lbs/ft
(Figure 17.4H)
- B'_c = out-to-out vertical rise of pipe, ft (Figure 17.4H)
- C_e = load coefficient for embankment installations (Figure 17.4J)
- C_d = load coefficient for trench installations (Figure 17.4.I)
- K = ratio of the active unit lateral soil pressure to unit vertical
soil pressure-Rankine's coefficient of active earth pressure
(Figures 17.4H-J)
- p = projection ratio (Article 17.4.5.2.3)
- p' = negative projection ratio (Figure 17.4G)
- μ = coefficient of internal friction of the soil (Figure 17.4I)
- μ' = coefficient of friction between backfill and trench walls
(Figure 17.4I)

17.1.4 Design

Add the following as the second paragraph:

Reinforced concrete box culverts shall be designed as rigid frames and that corner reinforcement shall be detailed to accommodate the fixed end corner moments.

Figure 17.4I Coefficient C_d

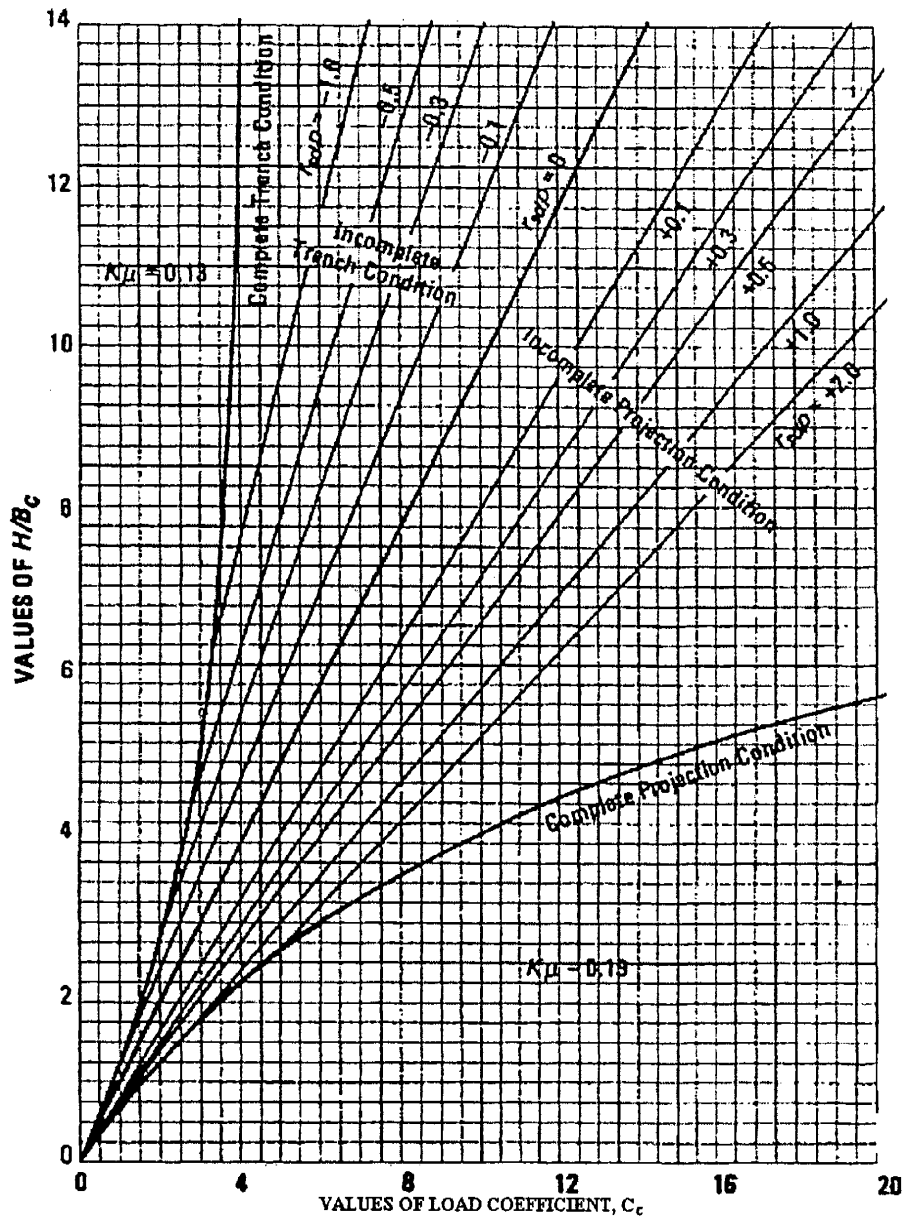


Figure 17.4J Load coefficient for positive projection embankment condition

17.6.4.1 General Requirements

Add the following to Article 17.6.4.1:

a. **Shear in Top Slabs**

The maximum shear in the top slabs of culverts under embankments shall be assumed to occur at a distance "D", out from the inside face of the wall or abutment; "D" being equal to the depth from the compression face of the slab to the centroid of the tension reinforcement.

b. **Shear in Bottom Slabs**

The shear in bottom slabs shall be computed as specified for footings.

17.6.4.2 Modification of Earth Loads for Soil Structure Interaction**17.6.4.2.2 *Trench Installations***

Change "Figure 17.4B" to "Figure 17.4I"

17.7.4.1 General Requirements

Add the following to Article 17.7.4.1:

a. **Shear in Top Slabs**

The maximum shear in the top slabs of culverts under embankments shall be assumed to occur at a distance, "D", out from the inside face of the wall or abutment; "D" being equal to the depth from the compression face of the slab to the centroid of the tension reinforcement.

b. **Shear in Bottom Slabs**

The shear in bottom slabs shall be computed as specified for footings.

17.7.4.2 Modification of Earth Loads for Soil-Structure Interaction

17.7.4.2.2 *Trench Installations*

Change “Figure 17.4B” to “Figure 17.4.I”.

17.8.5.1 General Requirements

Add the following to Article 17.8.5.1:

a. **Shear in Top Slabs**

The maximum shear in the top slabs of culverts under embankments shall be assumed to occur at a distance, "D", out from the inside face of the wall or abutment; "D" being equal to the depth from the compression face of the slab to the centroid of the tension reinforcement.

DIVISION I-A

SEISMIC DESIGN (Blue Pages)

ARTICLE NOS.

3.1
3.2
3.3
3.4
3.5
3.6
3.7
3.12
3.13

ARTICLE NOS.

4.1
6.1
6.2.3
6.4.3
Section 6A
Section 6B
Appendix to Section 6B
(6B-1, 6B-2, 6B-3, 6B-4)

3.1 APPLICATION OF SPECIFICATIONS

Add the following sentence at the end of the third sentence:

However, for all types of bridges in New York City (Downstate*), use the criteria outlined in Section 6B, "Seismic Hazard Spectra for New York City (Downstate*) Highway Structures".

3.2 ACCELERATION COEFFICIENT

Add the following as a first paragraph:

For the new design and analysis of Upstate* New York Bridges, an acceleration coefficient (A) of 0.19g, in lieu of the contour map (Figure 3.2A), shall be used.

However, vulnerability of existing structures shall be evaluated using the local contours of acceleration coefficient as shown in Figure 6A.2-1.

The new designs and retrofitting of existing structures for New York City (Downstate*) bridges shall be as outlined in Section 6B.

*See Figure 6B.1-1

3.3 IMPORTANCE CLASSIFICATION

Add the following as the first paragraph:

The new designs and retrofitting of existing structures for Upstate* New York bridges shall be as outlined in this article. The new designs and retrofitting of existing structures for New York City (Downstate*) bridges shall be as outlined in Section 6B.

3.4 Seismic Performance Categories

Delete the text of Article 3.4 and replace with the following:

The new designs for the Upstate* New York bridges shall be assigned seismic performance category (SPC) B, based on the acceleration coefficient $A=0.19g$ and the Importance Classification (IC) as shown in Table 3.4.

Retrofitting of existing bridges for the Upstate* New York bridges shall be assigned seismic performance category (SPC) A or B, as shown in Figure 6A.2-2 and the Importance Classification (IC) as shown in Table. 3.4.

The new designs and retrofitting of existing structures for the New York City (Downstate*) bridges shall be as outlined in Section 6B.

3.5 Site Effects

Add the following paragraph as the first paragraph:

The new designs and retrofitting of existing structures for Upstate* New York bridges shall be as outlined in this article. The new designs and retrofitting of existing structures for New York City (Downstate*) bridges shall be as outlined in Section 6B.

*See Figure 6B.1-1 and Figure 6A.2-2

3.6 ELASTIC SEISMIC RESPONSE COEFFICIENT

Add the following as the first two paragraphs:

The new designs and retrofitting of existing structures for New York City (Downstate*) bridges shall be as outlined in Section 6B.

The new designs and retrofitting of existing structures for Upstate* New York shall be as outlined in this article.

3.7 RESPONSE MODIFICATION FACTORS

Add the following as the first two paragraphs:

The new designs and retrofitting of existing structures for New York City (Downstate*) bridges shall be as outlined in Section 6B.

The new designs and retrofitting of existing structures for Upstate* New York bridges shall be as outlined in this article.

*See Figure 6B.1-1 and Figure 6A.2-2

3.12 REQUIREMENTS FOR TEMPORARY BRIDGES AND STAGE CONSTRUCTION

Add the following as the last paragraph to Article 3.12:

Temporary structures and stage construction structures (duration not exceeding 3 years) may be analyzed, using 50% of the Acceleration Coefficient given in Article 3.2, to compute the elastic forces and displacements.

Add the following as Article 3.13:

3.13 REQUIREMENTS FOR MOVABLE BRIDGES

Movable bridges shall be analyzed for dynamic loads, considering both the 'open' condition and 'closed' condition. However, if the bridge is going to remain 'open' for 10% or less of the time in a year, on a regular basis, the seismic loads may be reduced by 50% for analysis in the 'open' condition.

4.1 GENERAL

Add the following note to Article 4.1:

Procedure 1 and procedure2 should not be used for multi-span (continuous or multiple-simply supported) bridges, unless otherwise approved by the Deputy Chief Engineer (Structures).

Table 4.2A. Minimum Analysis Requirements:

Add the following as a footnote to Table 4.2A:

Procedure 1 and procedure 2 should not be used for multi-span (continuous or multiple-simply supported) bridges, unless otherwise approved by the Deputy Chief Engineer (Structures).

6.1**GENERAL**

Add the following as the first paragraph:

The new designs and retrofitting of existing structures for New York City (Downstate*) bridges shall be as outlined in this section. The provisions of Seismic Hazard Levels, Performance Criteria and Site-Dependent Spectra are given in Section 6B.

Add the following at the end of the last paragraph of Article 6.1:

Static effect of live load should be considered along with the dead loads in equations (6-1) and (6-2) where they are likely to be critical in the seismic analysis.

*See Figure 6B.2

6.2.3 Design Forces for Abutments and Retaining Walls

Add the following paragraph at the end of Article 6.2.3:

Typical retaining walls (retaining fill not more than 25ft. high) shall be analyzed for seismic loading when their movement is restrained by the nearby structure (e.g. not part of abutment).

6.4.3 Abutments**6.4.3 (A) *Free-Standing Abutment***

Third paragraph, add the following after the second sentence:

However, the nominal batter of 6 on 1 for the front row of piles should not be considered in the seismic analysis, since it is assumed that this batter effect does not change the seismic coefficient significantly. Hence, the seismic coefficient equal to one half the acceleration coefficient ($K_h = 0.5A$) should be used.

SECTION 6A

CRITERIA FOR SEISMIC RETROFITTING OF BRIDGES PROGRAMMED FOR REHABILITATION

6A.1 GENERAL

Existing bridges, programmed for rehabilitation shall be evaluated for seismic vulnerability. The evaluation should assess options and costs of seismic retrofit measures, necessary to eliminate or mitigate such failure vulnerability.

6A.2 CRITERIA FOR PLANNING SEISMIC RETROFITTING

The strengthening of existing bridges to the same earthquake resistance as currently required of new bridges is not always practical or cost effective. It is, therefore, the intent of these criteria to upgrade elements to be retrofitted to "new bridge" seismic criteria where feasible. However, seismic evaluations and seismic element analysis may be based on the actual rock acceleration coefficient (Figure 6A.2-1) attributable to the bridge site for SPC 'A' and SPC 'B' area (Figure 6A.2-2).

6A.3 Seismicity of Site

Seismicity is indicated by location of the project on Figure 6A.2-1 and also by the geotechnical characteristics of the particular foundation support system.

New York State is divided into three areas, based on ground motions and level of seismic hazard. The three areas are shown in Figure 6A.2-2. The New York City (Downstate*) area is covered in Section 6B of Division 1A. The rest of the state is divided in SPC 'A' and SPC 'B' areas and provisions of Articles 6A.3 and 6A.4 shall apply.

6A.4 Functional Importance

Bridges on or over highways which are critical to maintaining essential services (e.g. fire and police), regional movement of goods and services (e.g. connects commercial centers), acceptable traffic flow (for example, those serving major traffic generators), etc. should be recognized as "critical" bridges and given high seismic retrofitting priority.

As a minimum, "functionally important" bridges should include all those that carry or cross major highways, such as interstates, expressways and major arterials. Other bridges on routes that are important to regional or local well-being should also be included, at the Regional Director's discretion.

***See Figure 6B.1-1**

Following paragraphs discussed various aspects of importance classification, as given in AASHTO's Division I-A Commentary:

The determination of the Importance Classification of a bridge is necessarily subjective. Consideration should be given to the following Social/Survival and Security Defense requirements. An additional consideration would be average annual daily traffic.

The Social/Survival evaluation is largely concerned with the need for roadways during the period immediately following an earthquake. In order for civil defense, police, fire department or public health agencies to respond to a disaster situation a continuous route must be provided.

Survival and mitigation of the effects of the earthquake are of primary concern following a seismic event. Transportation routes to critical facilities such as hospitals, police and fire stations and communication centers must continue to function and bridges required for this purpose should be classified accordingly. In addition, a bridge that has the potential to impede traffic if it collapses onto an important route should also be classified equally important!

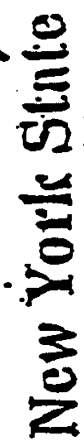
The health and well-being of the community is another major concern. Victims with critical injuries or illnesses must be treated; food, water and shelter provided and utilities restored. Routes to such facilities as schools and arenas, which could provide shelter or be converted to aid stations must suffer little or no damage. Access must be available to power installations and water treatment plants.

The importance evaluation of a bridge of Social/Survival significance in a disaster situation depends on the range of options available and the possibility of a bridge being in parallel or series with other bridges in a roadway network. Discussion may be required with highway, civil defense and police officials.

A basis for the Security Defense evaluation is the 1973 Federal-Aid Highway Act which required that a plan for defense highways be developed by each state. This plan had to include, as a minimum, the Interstate and Federal-Aid Primary routes; however, some of these routes can be deleted when such action is considered appropriate by a state. The defense highway network provides connecting routes to active military installations, industries and resources not covered by the Federal-Aid Primary routes and includes:

1. Military bases and supply depots and National Guard installations.
2. Hospitals, medical supply centers and emergency depots.
3. Major airports.
4. Defense industries and those industries that could easily or logically be converted to such.
5. Refineries, fuel storage and distribution centers.
6. Major railroad terminals, railheads, docks and truck terminals.
7. Major power plants including nuclear power facilities and hydroelectric centers at major dams.
8. Major communication centers.
9. Other facilities that the state considers important from a national defense viewpoint or during emergencies resulting from natural disasters or other unforeseen circumstances.

075




STYLING

Описание работы

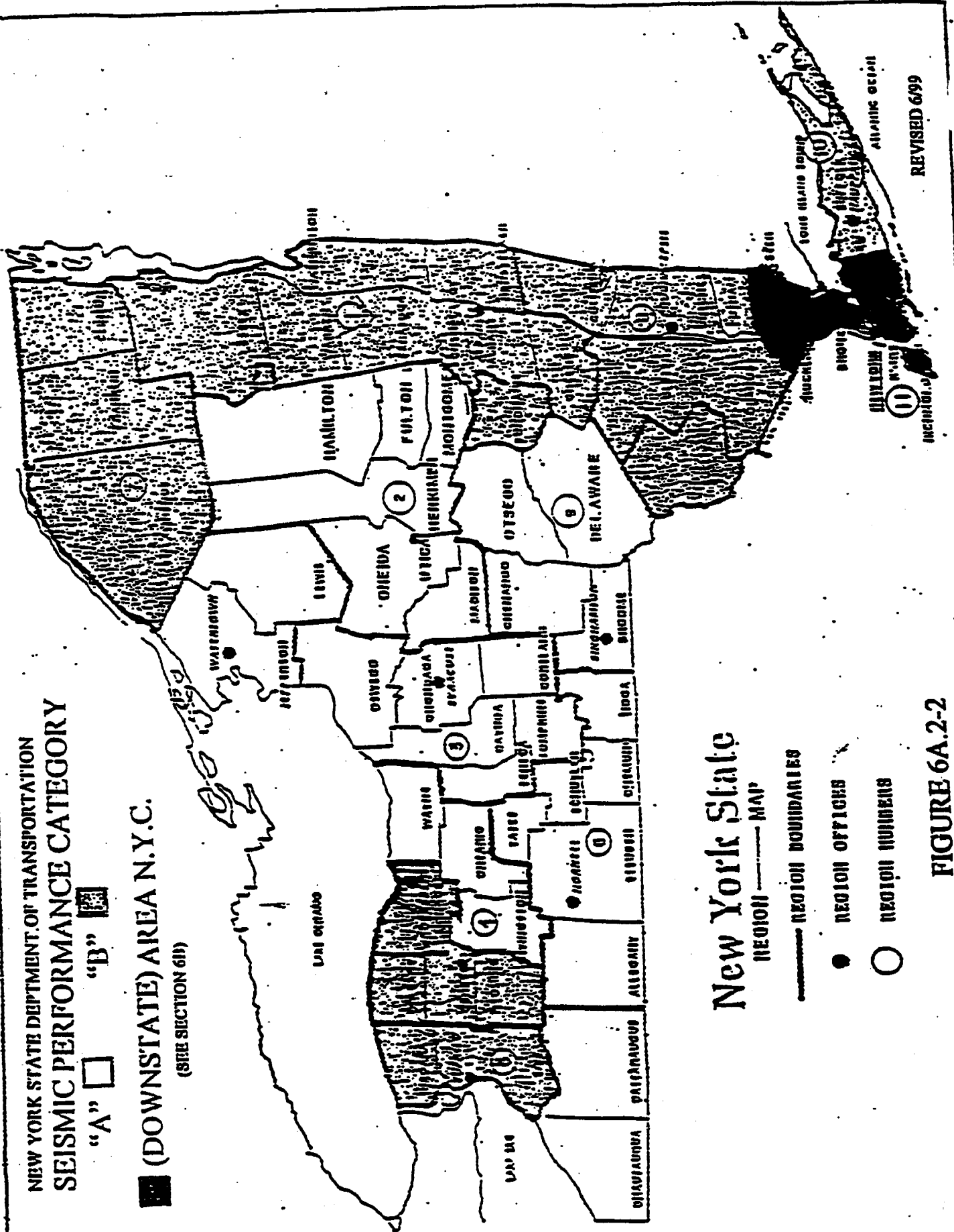
FIGURE 6A.2-1

NEW YORK STATE DEPARTMENT OF TRANSPORTATION
SEISMIC PERFORMANCE CATEGORY

"A"  "B" 

 (DOWNSTATE) AREA N.Y.C.

(SEE SECTION 611)



New York State
REGION - MAP

REGION BOUNDARIES

● REGION OFFICES

○ REGION NUMBERS

FIGURE 6A.2-2

6A.5 Rehabilitation Project Scope (SPC 'A', SPC 'B' and New York City (Downstate*) Area Bridges)

The nature and extent of scheduled rehabilitation work can influence the decision to include or defer the recommended seismic retrofit activities.

Rehabilitation work can be classified as major or minor rehabilitation.

Minor rehabilitations address non-structural repair or improvement of certain bridge elements, and include such work as concrete surface repair, deck overlays, joint and bearing restoration, secondary member steel repair and minor repair to primary steel members.

Major rehabilitations involve structural repair or replacement of primary bridge elements, and include work such as pier and pier cap beam replacement, deck replacement, superstructure replacement, bridge widenings and primary member replacement or strengthening. (Simple restoration of steel members by cover plating with H.S. bolts can usually be considered minor repair.)

6A.6 GUIDELINES FOR SEISMIC RETROFIT MEASURES

Certain bridge types (e.g. multiple simple spans), or details (e.g. high rocker bearings) which are more vulnerable to earthquake damage should be evaluated for seismic retrofit work based on the probable severity of damage and its impact on serviceability of the bridge.

Retrofitting existing bridge elements will be done in accordance with current NYSDOT seismic design policy for new bridges, whenever feasible. However, seismic evaluations and seismic element analysis may be based on the actual rock acceleration coefficient attributable to the bridge site. (See Figure 6A.2-1)

The following guidelines should be employed in determining the extent and implementation of seismic retrofit work:

6A.6.1 SPC 'A' Bridges

These bridges, in general, will not require seismic retrofit. However, for functionally important bridges programmed for major rehabilitation, it may be prudent to consider incorporating appropriate seismic resistant details if they are compatible with other work in the contract. For instance, consider replacing high steel rocker-type expansion bearings with elastomeric types, if extensive bearing and/or pedestal restoration work is already required due to deterioration.

***See Figure 6B.1-1**

Consider providing continuity at piers supporting simple spans, if major bearing/pedestal work is being done, or if deck replacement is contemplated. Cable restraints should also be considered at those piers if the available support length is inadequate and a continuity retrofit will not be done. Compatibility of the seismic retrofit work with other intended work will make the seismic actions more cost effective.

6A.6.2 New York City (Downstate*) Area Bridges and SPC 'B' Bridges

The most common approach for retrofitting a bridge is by increasing the capacity of the structure by strengthening or replacing the vulnerable element(s) to meet the demand. These conventional strengthening techniques are addressed in the current version of FHWA's "Seismic Retrofitting Manual for Highway Bridges".

6A.6.2.1 Replacement of High Steel Rocker and Low Steel Sliding Bearings

Replace high steel rocker bearings with elastomeric type bearings. Replace or retrofit companion fixed bearings. Replace low steel sliding bearings on "functionally important" structures, and on structures requiring bearing restoration work. Low steel fixed bearings which are in good condition may be kept or replaced as conditions warrant. Exceptions may be made to this general guideline when bridges are extremely wide, with many stringers in cross section, or when spans are continuous over several supports, and bearings are functioning properly and in good condition.

Fixed or expansion high steel bearings supporting non-redundant elements, either superstructure or substructure (pier caps), should always be replaced.

6A.6.2.2 Providing Continuity to Multiple Simple Spans

Provide continuity at piers for multiple simple span bridges. When conditions permit, the preferred method is to retrofit girders/beams at piers by fully splicing the girders/beams, and replacing two lines of bearings with a single line of bearings, centered over the pier. Where continuity by splicing is not feasible, cable restrainers or other connecting devices should be provided.

6A.6.2.3 Bridge Seat Extension

The bridge seat width shall satisfy the Minimum Support Length requirements given under AASHTO Division 1A, Article 6.3.1. This requirement is applicable for support lengths, considering the skew effect, parallel as well as normal to the span length. When these requirements are not met, shear blocks or restrainers as given in the current version of FHWA's "Seismic Retrofitting Manual for Highway Bridges" shall be provided to prevent the superstructure from falling off the bridge seat.

*See Figure 6B.1-1

6A.6.2.4 Retrofitting of Concrete Columns and Foundations

Most of the existing columns may not meet the shear strength and/or flexural ductility demand during a seismic event. Lack of confinement and inadequate lap splice length in the plastic hinge area seems to be the vulnerable element of the existing substructures built before 1991.

Retrofitting measures for columns and footings, outlined in the current version of FHWA's "Seismic Retrofitting Manual for Highway Bridges", should be considered.

6.A.7 RETROFITTING PRIORITY

6A.7.1 For "Functionally Important" bridges in SPC 'B' and "critical and "essential" New York City (Downstate*) area, seismic retrofit work should be included in the first scheduled general rehabilitation activity for the structure. Bridges with concrete columns should also be evaluated for column reinforcement details. Although existing column reinforcement details generally will not meet new seismic criteria, it is anticipated that only in a few instances will the deficiency be significant enough to warrant retrofit action. In those instances, retrofit procedures such as those shown in the current versions of FHWA's "Seismic Retrofitting Manual for Highway Bridges" should be followed.

6A.7.2 For Other SPC 'B' and New York City (Downstate*) bridges, the seismic retrofit work described above should be included in the next scheduled major rehabilitation work. Minor rehabilitation contracts should include as much of the seismic retrofit work as can be accommodated by project cost and compatibility of activities. At a minimum, cable restraints or continuity at piers (where necessary) and lateral restraint systems should be included. Bridges with high steel rocker bearings should be scheduled for follow-up retrofit activity, if necessary.

6A.8 BRIDGES WITH SPECIAL CONDITIONS

Certain bridge types or bridge details are particularly sensitive to seismic forces. When such conditions are identified on bridges programmed for rehabilitation, it would be prudent to consider additional retrofit measures or structure replacement. The location (SPC area) and "functional importance" of the structure should be key considerations in this decision.

The following conditions are particularly sensitive to seismic forces:

- Single or individual column pier supports.
- High, slender pier columns (when slenderness ratio exceeds 60).
- Large skews, generally in excess of 45°, with substandard support lengths.
- Severe curvature, where the subtended arc angle exceeds 75°.
- Unusual geometry causing portions of the structure to be significantly different in stiffness than the main structure, or which results in unusual support or framing details.
- Hinges or seated connections in suspended superstructures.
- Non-redundant load path superstructures.

*See Figure 6B.1-1

Bridges incorporating any of the above conditions should be evaluated for seismic vulnerability (capacity-demand ratios), as outlined in the current version of FHWA's "Seismic Retrofitting Manual for Highway Bridges".

It should be noted that only a few SPC 'A' bridges, incorporating the conditions noted above, will be candidates for extensive analysis or retrofit actions. However, the presence of these conditions should be acknowledged and considered in the project scoping phase.

6A.9 RETROFIT COSTS FOR SPC 'A' AND SPC 'B' BRIDGES

- (A) The cost of retrofitting structures will vary significantly based on the type and extent of needed work, as well as site conditions. In determining seismic retrofit actions, it may be appropriate, in some cases, to limit immediate retrofit action with a pre-determined cost ceiling, while deferring remaining less critical actions to a future project. As a guideline, a cost increase for seismic retrofit in the range of 10-15% is considered appropriate for SPC 'A' and SPC 'B' bridges with 15% being typical for projects with rehabilitation costs of less than \$2 million. Where certain bridges are judged to be highly vulnerable based on the previously noted criteria, cost increases in excess of 15% may be warranted to guarantee the structural integrity of the bridge.
- (B) In making a decision to defer work based on initial cost, the Regional Director should recognize the benefits of performing the seismic retrofit work now in conjunction with other proposed rehabilitation work. Further, in some instances, the need for major seismic retrofit may indicate that replacement rather than rehabilitation is more cost effective, and therefore the more appropriate action. A Regional Director's decision to defer seismic retrofit work for SPC 'B' and New York City (Downstate*) bridges should preferably be made with concurrence of the Deputy Chief Engineer (Structures). The basis for that decision should be documented in the project file, with copy to the DCE(S). Should the DCE(S) not concur with the Regional Director's decision, the DCE(S) reserves the right to appeal that decision, if in his/her opinion it presents a significant threat to the safety of the structure.

*See Figure 6B.1-1

SECTION 6B

SEISMIC HAZARD SPECTRA FOR NEW YORK CITY (DOWNSTATE*) HIGHWAY STRUCTURES

6B.1 Introduction

The purpose of these guidelines is to define the engineering application of the hard rock Acceleration Response Spectra developed by a panel of expert seismologists, moderated by Dr. Robin McGuire, Risk Engineering Inc., which will apply to both the design of new bridges and retrofitting of existing bridges being rehabilitated. Panel based spectra apply in the region indicated in Figure 6B.1-1.

These guidelines are intended for the New York City (Downstate*) highway bridges, unless otherwise directed by the owner of those having jurisdiction to modify or substitute provisions to suit specific circumstances.

As in the case of any rehabilitation project, judgement should be exercised in assessing options and costs of seismic retrofit measures, and to incorporate into the rehabilitation plans those retrofit measures deemed warranted to eliminate or mitigate such seismic vulnerabilities.

6B.1.1 Panel Study and Recommendations

The panel was assigned to assess and to work with the available studies, ground motion estimation models and seismic source interpretations for the region and to develop design parameters, applicable to the city area and the surrounding counties (Figure 6B.1-1). The study is very well documented by Dr. McGuire's report titled "Seismic Hazard for New York City".

Uniform hazard spectra for 5% damping developed as 84% spectra for return periods between 500 and 5000 years are shown in Figure 6B.1.1-1. Results shown are for PGA and spectral acceleration from 25 Hz to 0.5 Hz, and have been extrapolated to 0.1 Hz (10 sec.). If these long periods are critical to structural facilities in New York City, it is recommended that more quantitative calculations be developed for the longer period ground motions (>2 sec.).

For design values, it is recommended that the 84% spectra shown in Table 6B.1.1-1 (for hard rock) be used. The 84% spectra, shown in Table 6B.1.1-1, should be used for both the longitudinal and transverse direction of a structure. For vertical motions, the Report (1) recommends a method to derive vertical spectra from horizontal spectra. Appendix C to the Report (1) also outlines a recommended procedure for "Estimation of Spectra for other Dampings" and Appendix Section 6B -4 provides "Generation of Multiple-Support Artificial Ground Motions".

*See Figure 6B.1-1

(1) Dr. McGuire's Report "Seismic Hazard for New York City", January 14, 1998

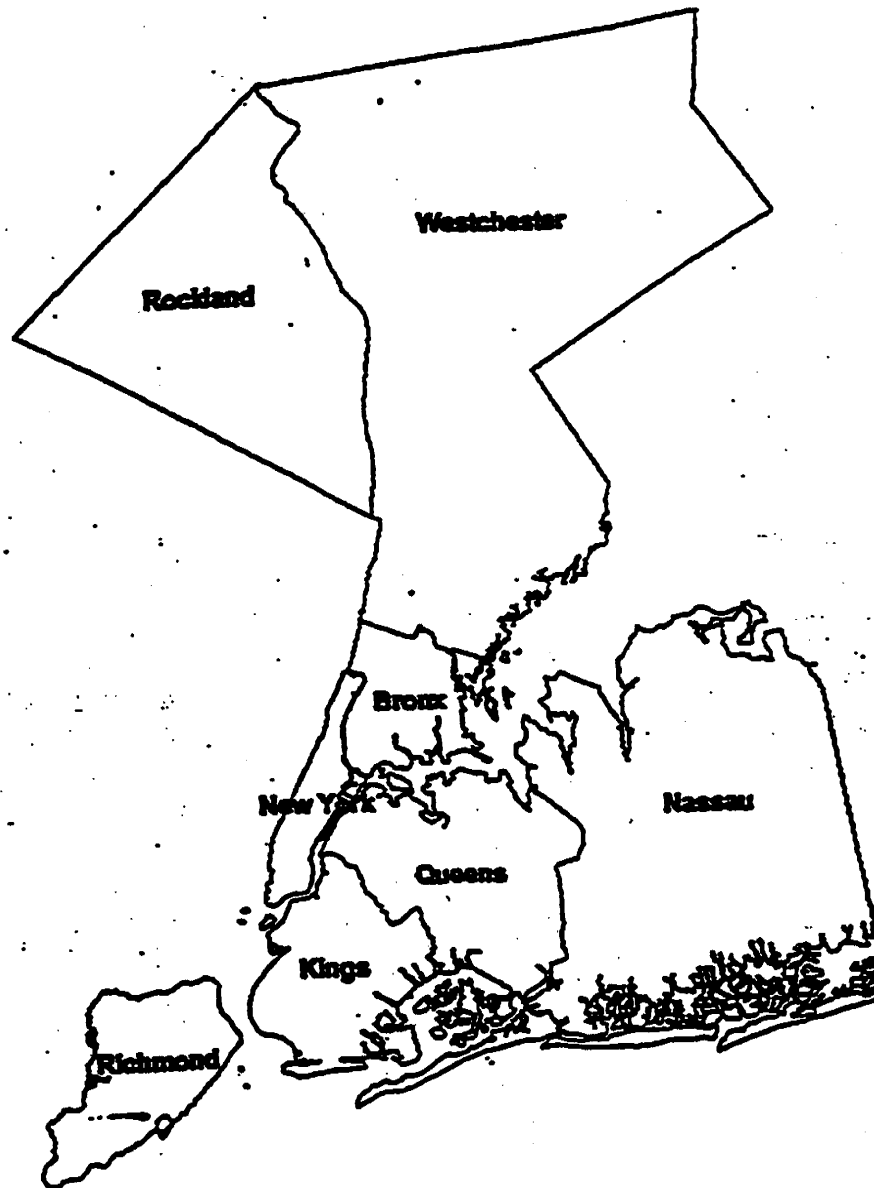


Figure 6B.1-1 New York City and Surrounding Area

TABLE 6B.1.1-1 UNIFORM HAZARD SPECTRA (84 percentile)
Spectral Acceleration in g , 5 % damping

Spectral Frequency (Period)	500 year Return period	1000 year Return period	2500 year Return period	5000 year Return period
PGA = 50 Hz (0.02 sec)	6.13 E -02	1.22 E -01	2.44 E -01	3.72 E -01
25 Hz (0.04 sec)	1.45 E - 01	2.77 E -01	5.78 E -01	8.82 E -01
10 Hz (0.1 sec)	1.44 E -01	2.54 E -01	5.08 E -01	8.13 E -01
5 Hz (0.2 sec)	1.03 E -01	2.08 E -01	3.94 E -01	5.86 E -01
2 Hz (0.5 sec)	5.14 E -02	9.46 E -02	1.95 E -01	3.11 E -01
1 Hz (1.0 sec)	3.21 E -02	5.64 E -02	1.04 E -01	1.61 E -01
0.5 Hz (2.0 sec)	1.50 E -02	2.56 E -02	4.97 E -02	7.51 E -02
0.2 Hz (5.0 sec)	5.99 E -03	1.02 E -02	1.99 E -02	3.00 E -02
0.1 Hz (10.0 sec)	1.50 E -03	2.56 E -03	4.97 E -03	7.51 E -02

6B.1.2 Site Effects

The recommended uniform hazard horizontal acceleration spectra on hard rock for return periods of 500 and 2500 years are supplemented by Dr. Dobry's work on extending the spectra to other site conditions ranging from soft rock to soft soil (2). A set of spectra (Figures 6B.1.2-1 and 6B.1.2-2), corresponding to hard rock (Site Class A) through soft soil (Site Class E), using the short and long period site amplification factors and site classification and similar to the building codes (NEHRP 1997, UBC 1997) for each of the two return periods (500 and 2500 years), is developed for the designer's use. A nine step procedure to develop similar spectra for other conditions in New York City area, if needed, and examples of application of the recommended set of spectra (Figures 6B.1.2-1 and 6B.1.2-2), are outlined in Dr. Dobry's report (2) and included in the Appendix to Section 6B. The site classes, as given in the report(2), are defined in Table 6B.1.2-1. Discussion on the site classification and eight solved examples covering different soil profiles, using shear wave velocity, the average standard penetration resistance for cohesionless soil and/or the undrained shear strength for cohesive soil, are included in the Appendix 6B-3.

TABLE - 6 B . 1 . 2 -1 SITE CLASSIFICATION
(NEHRP 1997, UBC 1997, DOBRY et al., 1997)

SOIL PROFILE TYPE	DESCRIPTION	\bar{V}_s m/sec. (ft./sec.) Top 30 meters (100 ft.)
A	Hard Rock	> 1500 (5000)
B	Rock	760-1500(2500-5000)
C	Very dense soil/soft rock	360-760 (1200-2500)
D	Stiff Soil	180-360 (600-1200)
E	Soft Soil	< 180 (600)
F	Special Soils Requiring site-specific evaluation	See Appendix to Section 6B

(2) Dr. Ricardo Dobry's Report "Earthquake Horizontal Response Spectra for Different Site Conditions in New York City",
June 19, 1998

500-Year Earthquake (84th Percentile, 5% Damping)

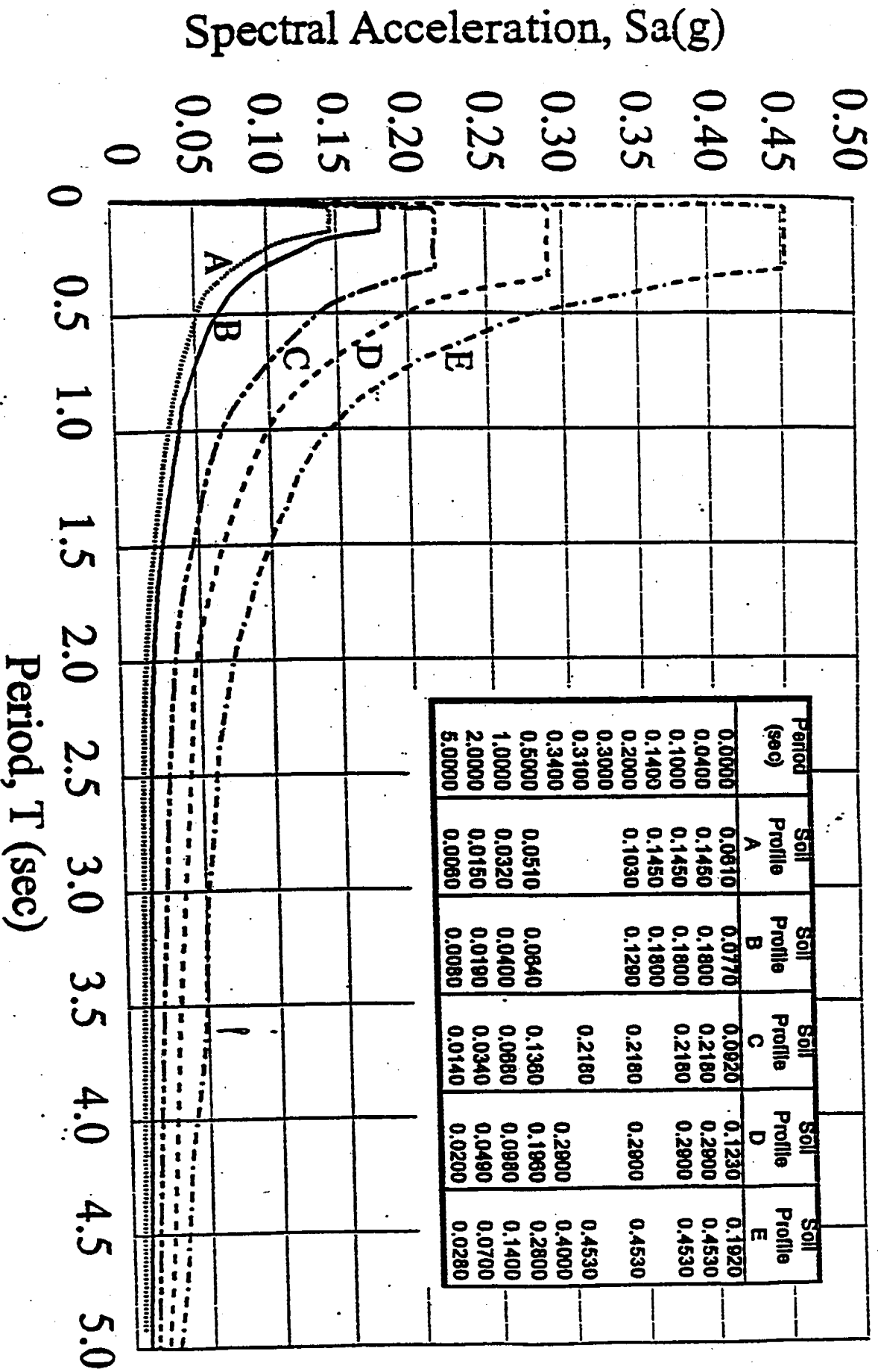


FIGURE 6B.1.2-1. Response spectra for 500-year earthquake in New City and surrounding areas.

2500-Year Earthquake (84th Percentile, 5% Damping)

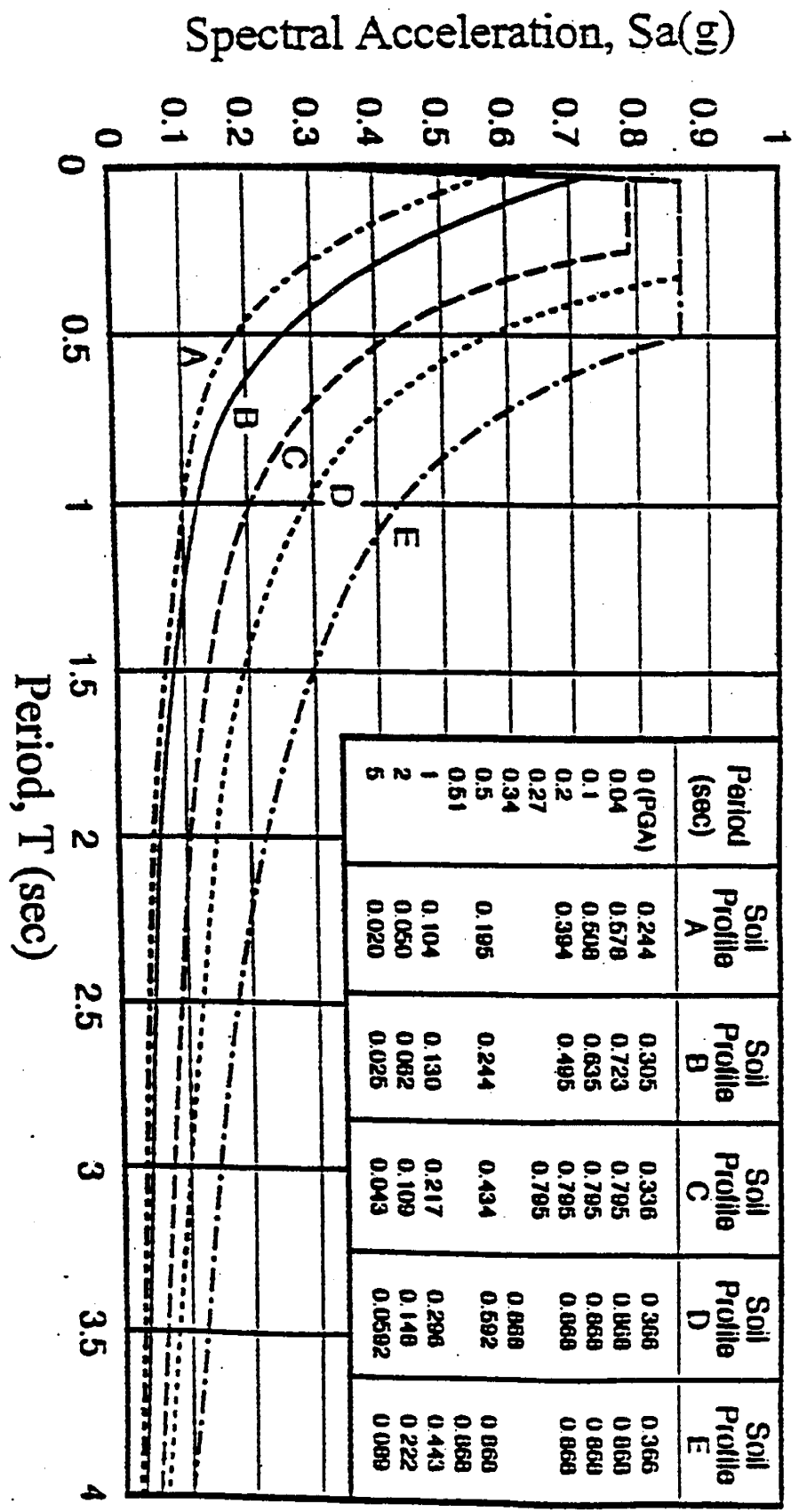


FIGURE 6B.1.2-2. Response spectra for 2500-year earthquake in New York City and surrounding areas.

6B.2 Seismic Criteria

The recommended response spectra based on the site classes (Table 6B.1.2-1), shall be used for the New York City and the surrounding area (Figure 6B.1-1) bridges. The return period(s), one or two level approach and corresponding performance criteria should be as outlined in Art. 6B.3, unless otherwise approved by the agency having the project jurisdiction.

6B.3 Performance Criteria and Seismic Hazard

Bridges in the New York City and surrounding areas should be designed to meet the performance criteria outlined in Table 6B.3-1. Bridges should be classified by the agency having jurisdiction, as "critical", "essential" or "other" meeting the following requirements:

Critical Bridge⁽¹⁾: A Critical Bridge must provide immediate access after the lower level (functional) event and limited access after the upper level (safety) event and continue to function as a part of the lifeline, social/survival network and serve as an important link for civil defense, police, fire department and/or public health agencies to respond to a disaster situation after the event, providing a continuous route. Any bridge that crosses a Critical route should be evaluated on the critical hazard levels with the performance criteria of no collapse and the bridge shall not restrict the operation of the critical highway passing below. Critical Bridges shall be analyzed for two earthquake levels: a lower level event (functional evaluation/design level) having 10% probability of being exceeded in 50 years (500 years Return Period); and an upper level event (safety evaluation/design level) having a 2% in 50 years probability of exceedance (2500 years Return Period). Site-specific soil effects and, if necessary, Soil Structure Interaction must be considered. A multimode spectral analysis must be used to establish vulnerability for either event. The seismic retrofit for the 2% in 50 years probability of exceedance event must be confirmed by either a multimodal spectral analysis augmented by non-linear static (pushover) analysis, or by nonlinear time history analysis. Critical Bridges shall survive the upper level event (2% probability of being exceeded in 50 years) with repairable damage (see definition under Damage Levels). Traffic access following this event shall be limited: within 48 hours for emergency/defense vehicles and within months for general traffic. After the lower level event (10% probability of being exceeded in 50 years), the bridge shall suffer no damage to primary structural elements and minimal damage (see definition under Damage Levels) to other components. The access after this event shall be immediate to all traffic with an allowance of a few hours for inspection.

Essential Bridge⁽¹⁾: An Essential Bridge must provide limited access after the one level evaluation/design event and serve as an important link for civil defense, police, fire department and/or public health agencies to respond to a disaster situation after the event, providing a continuous route. Any bridge that crosses an

(1) New York City Department of Transportation - Seismic Design Criteria. Guidelines prepared by Weidlinger Associates, New York City.

Essential route should be evaluated on the essential hazard level with the performance criteria of no collapse and the bridge shall not restrict the operation of the essential highway passing below. Essential bridges shall be analyzed for a one level seismic event consisting of a spectrum equal to 2/3 of the Panel's 2% probability of being exceeded in 50 years (2500 year Return Period). Essential bridges shall survive this event with repairable damage (see definition under Damage Levels). Access following the event shall be limited: one or two lanes shall be available within 3 days for emergency vehicles, full service within months.

Other Bridges:⁽¹⁾ Other bridges are those not classified as Critical or Essential. Other bridges shall be analyzed for a one level seismic event consisting of a spectrum equal to 2/3 of the panel's 2% probability of being exceeded in 50 years (2500 year return period). Other bridges may suffer significant damage (see definition under Damage Levels), although collapse shall not occur. The damage shall be in visible and pre-selected areas. Extended closures are acceptable.

6B.3.1 Damage Levels - Definitions ⁽¹⁾

Bridge component detailing or retrofit should be such that the damage caused by the earthquake would be controlled in order to allow for the desired post event service level.

Minimal Damage: The bridge should essentially behave elastically during the earthquake, although minor inelastic response could take place. Post earthquake damage should be limited to narrow flexural cracking in concrete and masonry elements. There should be no permanent deformations to structural members. Only minor damage or permanent deformations to non-structural members should take place.

Repairable Damage: The extent of damage should be limited so that the structure can be restored to its pre-earthquake condition without replacement of structural members. Inelastic response may occur resulting in: concrete cracking, minor cover spalling and reinforcement yielding; minor yielding of structural steel members; some damage to secondary members and non-structural components; some damage to masonry. Repair should not require complete closure of the bridge. Permanent offsets should be small and there should be no collapse.

Significant Damage: There is no collapse, but permanent offsets may occur. Extensive cracking, major spalling of concrete and reinforcement yielding may force closure for repair. Similar consequences could result from yielding or local buckling of steel members. There could be yielding of member connections, fracture of limited number of bolts/rivets, serious damage to secondary structural members and non-structural components, as well as to masonry. Partial or complete replacement may be required in some cases.

(1) New York City Department of Transportation - Seismic Design Criteria. Guidelines prepared by Weidlinger Associates, New York City.

6B.3.2 Two Level and One level Design Approach

Critical bridges designed for larger exposure period should have adequate ductility to survive an upper level event, avoiding collapse. The probabilistic return periods and the corresponding performance criteria for functional evaluation and safety evaluation are shown in Table 6B.3-1. In all cases, collapse shall be prevented; and repairable damage may occur for one-level design approach and lower level of the two-level approach. The extent of damage and performance criteria should be as outlined in Article 6B.3.

**TABLE 6B.3-1 PERFORMANCE CRITERIA AND SEISMIC HAZARD LEVEL
FOR DOWNSTATE BRIDGE PROJECTS**

Importance Categories	Return Period	Probability of Exceedance	Performance Criteria
Critical Bridges	2500 yrs	2% in 50 yrs	No collapse, limited access for emergency traffic in 48 hrs, full service within month(s)
	500 yrs	10% in 50 yrs	No collapse, no damage to primary structural elements, minimal repairable damage, full access to normal traffic available immediately (allow few hours for inspection)
Essential Bridges		$\frac{2}{3}$ (2% in 50 yrs)	No collapse, repairable damage, one or two lanes available within 3 days, full service within month(s)
Other Bridges		$\frac{2}{3}$ (2% in 50 yrs)	No collapse, significant but repairable damage in visible areas. Traffic interruption acceptable.

Critical Bridge: Any bridge that must continue to function as part of the lifeline, social/survival network and serve as important link for civil defense, police, fire department or/and public health agencies to respond to a disaster situation after the event, providing a continuous route. Any bridge that crosses a critical route should be evaluated on critical hazard levels with the performance criteria of no collapse and the bridge shall not restrict the operation of the critical highway passing below.

Essential Bridge: Any bridge that must provide limited access for emergency vehicles after the event and serve as important link for civil defense, police, fire department or/and public health agencies to respond to a disaster situation after the event, providing a continuous route. Any bridge that crosses an essential route should be evaluated on the essential hazard level with the performance criteria of no collapse and the bridge shall not restrict the operation of the essential highway passing below.

Other Bridges: Bridges not qualifying as critical or essential.

Downstate: Includes Bronx, Kings, Nassau, New York, Queens, Richmond, Rockland and Westchester counties.

6B.4 Design and Analysis

Multimode Response Spectrum analysis should be performed for the two-level and the one-level design approach. However, for the upper level event, a non-linear static analysis, to assess the displacement and resulting damage, shall be performed. The critical primary load carrying members shall be evaluated to prevent brittle modes of failure. Different analysis procedures for different seismic input levels in a two-level approach should be considered.

Gravity loads should include live loads where they are likely to be critical in the seismic analysis. Effect of vertical ground motions (not less than 67% of the horizontal ground motion) should be considered, especially for long span structures.

Response Modification Factors (R) shown in Table 6B.4-1 shall be used to determine the seismic design forces, by dividing the seismic forces resulting from elastic analysis, by the respective 'R' factor. However, if an inelastic time history method of analysis is used, or forces developed by the plastic hinging of column or multi-column bent, 'R' shall be taken as 1.0.

**TABLE - 6 B . 4 - 1 Response Modification Factors- Substructures
(AASHTO - LRFD 1998)**

SUBSTRUCTURES			
	CRITICAL	ESSENTIAL	OTHER
Wall-type piers	1.5	1.5	2.0
Reinforced concrete pile bents			
Vertical piles only	1.5	2.0	3.0
With batter piles	1.5	1.5	2.0
Single columns	1.5	2.0	3.0
Steel or composite steel and concrete pile bents			
Vertical pile only	1.5	3.5	5.0
With batter piles	1.5	2.0	3.0
Multiple column bents	1.5	3.5	5.0

NOTE: These Response Modification Factors (R) shall not be used if segmental hollow prestressed columns are used. Ductility of such system shall be demonstrated by conducting rigorous analysis.

Geotechnical analysis should be completed for both levels of design, rather than just the upper level. The analysis should include assessments of the potential for liquefaction, lateral spreading, soil-structure interaction and uplift. In all cases, soil behavior that degrades the structural capacity of the foundations must be prevented at the lower level event, and that movement should be less than a maximum acceptable amount during the upper level event.

6B.5 Site Specific Studies

"Critical", "Essential" and "Other" bridges with Site Class F shall require site specific studies using as input the time acceleration histories provided by Dr. McGuire (see Appendix to Section 6B) . In a similar manner, site specific studies shall be conducted for all critical bridges with Site Class E, except for deposits, that involve only cohesionless soils. Geotechnical Engineering Bureau should be contacted for further directions.

APPENDIX TO SECTION 6B

Appendix to Section 6B contains an additional information in four parts, and it is intended to provide better understanding of the provisions.

**Appendices 6B-1 and 6B-4 are taken from Dr. McGuire's Report
" SEISMIC HAZARD FOR NEW YORK CITY ".**

**Appendices 6B-2 and 6B-3 are taken from Dr. DOBRY's Report
"EARTHQUAKE HORIZONTAL RESPONSE SPECTRA FOR
DIFFERENT SITE CONDITIONS IN NEW YORK CITY"**

APPENDIX TO SECTION 6B -1

The spectra given in Table 6B.1.T-1 are for horizontal motions applied in any direction, e.g., these spectra should be used for both the longitudinal and transverse direction of a structure. For vertical motions the method recommended in Electric Power Research Institute (EPRI) (1993) can be used to derive vertical spectra from horizontal spectra. For hard rock this is summarized as follows (using the recommendation for $R = 10$ to 20 km, which includes earthquakes that dominate in this study). At 30 Hz and higher, vertical is taken as equal to horizontal. For frequencies between 20 and 30 Hz, interpolation is made, using a logarithmic frequency scale (i.e., $V = 1.517 + 0.740 \ln F$, where V is vertical/horizontal ratio and F is frequency). If vertical motions are critical to design, a more detailed site-specific investigation should be made including any local site conditions.

APPENDIX TO SECTION 6B -2

NEHRP's Soil Profile Types and Site Dependent Spectra Provisions

The small changes in nomenclature between 1994 and 1997 NEHRP are summarized in the following Table:

Parameter	Name 1994 NEHRP	Name 1997 NEHRP	Correspondence
Name site classification	Soil Profile Type	Site Class	Exact (A, B, C, D, E, and F)
Short period measure of seismic hazard on rock type B	A_s	S_s	$S_s = 2.5 A_s$
Long period measure of seismic hazard on rock type B	A_v	S_1	$S_1 = A_v$

1.4.2 SEISMIC COEFFICIENTS: The values of seismic coefficients (C_a and C_v) shall be determined from Sec. 1.4.2.3 or Tables 1.4.2.4a and 1.4.2.4b based on Soil Profile Types defined as follows:

- A Hard rock with measured shear wave velocity, $\bar{v}_s > 5,000$ ft/sec (1500 m/s)
- B Rock with $2,500$ ft/sec $< \bar{v}_s \leq 5,000$ ft/sec (760 m/s $< \bar{v}_s \leq 1500$ m/s)
- C Very dense soil and soft rock with $1,200$ ft/sec $< \bar{v}_s \leq 2,500$ ft/sec (360 m/s $< \bar{v}_s \leq 760$ m/s) or with either $N > 50$ or $\bar{s}_u \geq 2,000$ psf (100 kPa)
- D Stiff soil with 600 ft/sec $\leq \bar{v}_s \leq 1,200$ ft/sec (180 m/s $\leq \bar{v}_s \leq 360$ m/s) or with either $15 \leq N \leq 50$ or $1,000$ psf $\leq \bar{s}_u \leq 2,000$ psf (50 kPa $\leq \bar{s}_u \leq 100$ kPa)
- E A soil profile with $\bar{v}_s < 600$ ft/sec (180 m/s) or any profile with more than 10 ft (3 m) of soft clay defined as soil with $PI > 20$, $w \geq 40$ percent, and $\bar{s}_u < 500$ psf (25 kPa)
- F Soils requiring site-specific evaluations:
 - 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils.
 - 2. Peats and/or highly organic clays ($H > 10$ ft [3 m] of peat and/or highly organic clay where H = thickness of soil)
 - 3. Very high plasticity clays ($H > 25$ ft [8 m] with $PI > 75$)
 - 4. Very thick soft/medium stiff clays ($H > 120$ ft [36 m])

EXCEPTION: When the soil properties are not known in sufficient detail to determine the Soil Profile Type, Type D shall be used. Soil Profile Types E or F need not be assumed unless the regulatory agency determines that Types E or F may be present at the site or in the event that Types E or F are established by geotechnical data.

1.4.2.1 Steps for Classifying a Site (also see Table 1.4.2.1 below):

- Step 1:** Check for the four categories of Soil Profile Type F requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as Soil Profile Type F and conduct a site-specific evaluation.
- Step 2:** Check for the existence of a total thickness of soft clay > 10 ft (3 m) where a soft clay layer is defined by: $s_u < 500$ psf (25 kPa), $w \geq 40$ percent, and $PI > 20$. If these criteria are satisfied, classify the site as Soil Profile Type E.
- Step 3:** Categorize the site using one of the following three methods with \bar{v}_s , \bar{N} , and \bar{s}_u computed in all cases as specified by the definitions in Sec. 1.4.2.2:
- \bar{v}_s for the top 100 ft (30 m) (\bar{v}_s method)
 - \bar{N} for the top 100 ft (30 m) (\bar{N} method)
 - \bar{N}_{ch} for cohesionless soil layers ($PI < 20$) in the top 100 ft (30 m) and average \bar{s}_u for cohesive soil layers ($PI > 20$) in the top 100 ft (30 m) (\bar{s}_u method)

TABLE 1.4.2.1
Soil Profile Type Classification

Soil Profile Type	\bar{v}_s	\bar{N} or \bar{N}_{ch}	\bar{s}_u
E	< 600 fps (< 180 m/s)	< 15	< 1,000 psf (< 50 kPa)
D	600 to 1,200 fps (180 to 360 m/s)	15 to 50	1,000 to 2,000 psf (50 to 100 kPa)
C	1,200 to 2,500 fps (360 to 760 m/s)	> 50	> 2,000 (> 100 kPa)

NOTE: If the \bar{s}_u method is used and the \bar{N}_{ch} and \bar{s}_u criteria differ, select the category with the softer soils (for example, use Soil Profile Type E instead of D).

The shear wave velocity for rock, Soil Profile Type B, shall be either measured on site or estimated by a geotechnical engineer or engineering geologist/seismologist for competent rock with moderate fracturing and weathering. Softer and more highly fractured and weathered rock shall either be measured on site for shear wave velocity or classified as Soil Profile Type C.

The hard rock, Soil Profile Type A, category shall be supported by shear wave velocity measurements either on site or on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 100 ft (30 m), surficial shear wave velocity measurements may be extrapolated to assess \bar{v}_s .

The rock categories, Soil Profile Types A and B, shall not be used if there is more than 10 ft (3 m) of soil between the rock surface and the bottom of the spread footing or mat foundation.

1.4.2.2 Definitions: The definitions presented below apply to the upper 100 ft (30 m) of the site profile. Profiles containing distinctly different soil layers shall be subdivided into those layers designated by a number that ranges from 1 to n at the bottom where there are a total of n distinct layers in the upper 100 ft (30 m). The symbol i then refers to any one of the layers between 1 and n .

v_{si} is the shear wave velocity in ft/sec (m/s).

d_i is the thickness of any layer between 0 and 100 ft (30 m).

* \bar{v}_s is:

$$\bar{v}_s = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{v_{si}}} \quad (1.4.2-1)$$

where $\sum_{i=1}^n d_i$ is equal to 100 ft (30 m)

N_i is the Standard Penetration Resistance (ASTM D1586-84) not to exceed 100 blows/ft as directly measured in the field without corrections.

\bar{N} is:

$$\bar{N} = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{N_i}} \quad (1.4.2-2)$$

*Soil profiles involving (1) soils and rock and (2) cohesionless soils and plastic soils with $N \leq 10$,

\bar{v}_s method is preferred by converting all N and S_u data to \bar{v}_s values, using appropriate empirical equations. Geotechnical Engineering Bureau should be contacted for further directions.

\bar{N}_{ch} is:

$$\bar{N}_{ch} = \frac{d_s}{\sum_{i=1}^n \frac{d_i}{N_i}} \quad (1.4.2-3)$$

where $\sum_{i=1}^n d_i = d_s$.

(Use only d_i and N_i for cohesionless soils.)

d_s is the total thickness of cohesionless soil layers in the top 100 ft (30 m).

s_{ui} is the undrained shear strength in psf (kPa), not to exceed 5,000 psf (250 kPa), ASTM D2166-91 or D2850-87.

\bar{s}_u is

$$\bar{s}_u = \frac{d_c}{\sum_{i=1}^k \frac{d_i}{s_{ui}}} \quad (1.4.2-4)$$

where $\sum_{i=1}^k d_i = d_c$.

d_c is the total thickness (100 - d_s) of cohesive soil layers in the top 100 ft (30 m).

PI is the plasticity index, ASTM D4318-93.

w is the moisture content in percent, ASTM D2216-92.

1.4.2.3 Site Coefficients: The values for site coefficients F_a and F_v are as indicated in Tables 1.4.2.3a and 1.4.2.3b, respectively, and are used to determine seismic coefficients C_a and C_v as follows:

$$C_a = F_a A_a \quad (1.4.2.3-1)$$

and

$$C_v = F_v A_v \quad (1.4.2.3-2)$$

TABLE 1.4.2.3a
Values of F_a as a Function of Site Conditions and Shaking Intensity

Soil Profile Type	Shaking Intensity				
	$A_g \leq 0.1$	$A_g = 0.2$	$A_g = 0.3$	$A_g = 0.4$	$A_g \geq 0.5^a$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	<i>b</i>
F	<i>b</i>	<i>b</i>	<i>b</i>	<i>b</i>	<i>b</i>

NOTE: Use straight line interpolation for intermediate values of A_g .

^a Values for $A_g > 0.4$ are applicable to the provisions for seismically isolated structures in Sec. 2.6 and certain other structures (e.g., see Table 2.2.4.3).

^b Site specific geotechnical investigation and dynamic site response analyses shall be performed.

TABLE 1.4.2.3b
Values of F_v as a Function of Site Conditions and Shaking Intensity

Soil Profile Type	Shaking Intensity				
	$A_v \leq 0.1$	$A_v = 0.2$	$A_v = 0.3$	$A_v = 0.4$	$A_v \geq 0.50^a$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	<i>b</i>
F	<i>b</i>	<i>b</i>	<i>b</i>	<i>b</i>	<i>b</i>

NOTE: Use straight line interpolation for intermediate values of A_v .

^a Values for $A_v > 0.4$ are applicable to the provisions for seismically isolated structures in Sec. 2.6 and certain other structures (e.g., see Table 2.2.4.3).

^b Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

1.4.2.4 Seismic Coefficients C_a and C_v : Seismic coefficient C_a based on Soil Profile Type and A_a is determined from Table 1.4.2.4a:

TABLE 1.4.2.4a
Seismic Coefficient C_a

Soil Profile Type	$A_a < 0.05$	$A_a = 0.05$	$A_a = 0.10$	$A_a = 0.20$	$A_a = 0.30$	$A_a = 0.40$
A	A_a	0.04	0.08	0.16	0.24	0.32
B	A_a	0.05	0.10	0.20	0.30	0.40
C	A_a	0.06	0.12	0.24	0.33	0.40
D	A_a	0.08	0.16	0.28	0.36	0.44
E	A_a	0.13	0.25	0.34	0.36	0.36

NOTE: For intermediate values, the higher value or straight-line interpolation shall be used to determine the value of C_a .

Seismic coefficient C_v based on Soil Profile Type and A_v is determined from Table 1.4.2.4b:

TABLE 1.4.2.4b
Seismic Coefficient C_v

Soil Profile Type	$A_v < 0.05$	$A_v = 0.05$	$A_v = 0.10$	$A_v = 0.20$	$A_v = 0.30$	$A_v = 0.40$
A	A_v	0.04	0.08	0.16	0.24	0.32
B	A_v	0.05	0.10	0.20	0.30	0.40
C	A_v	0.09	0.17	0.32	0.45	0.56
D	A_v	0.12	0.24	0.40	0.54	0.64
E	A_v	0.18	0.35	0.64	0.84	0.96

NOTE: For intermediate values, the higher value or straight-line interpolation shall be used to determine the value of C_v .

Note that where A_a and A_v are less than 0.05, $C_a = A_a$ and $C_v = A_v$.

APPENDIX TO SECTION 6B -3

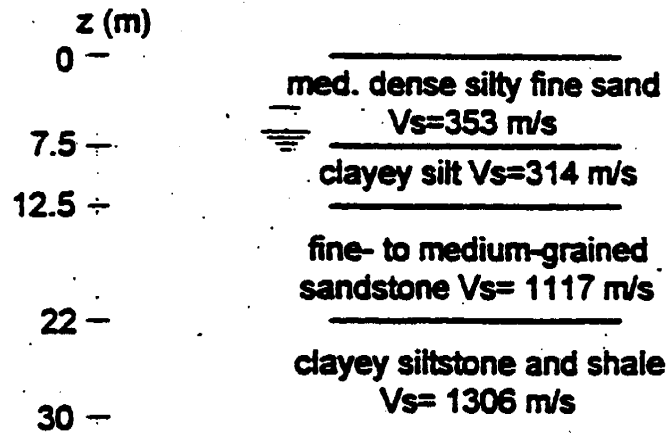
Examples of Site Classifications.

SYMBOLS OF SOIL PROPERTIES USED IN SOIL PROFILES

N (b/ft)	Standard Penetration Resistance (uncorrected)
q_u (psf)	Unconfined compressive strength of cohesive layer
PI	Plasticity Index of cohesive layer
S_u (psf) = 0.5 q_u	Undrained shear strength of cohesive layer
V_s (fps, m/s)	Shear wave velocity of soil layer
w (%)	Water content

EXAMPLE No. 1

SOIL PROFILE



SITE CLASSIFICATION

Using the \bar{v}_s method, the average the average shear wave velocity is given by (see Appendix I):

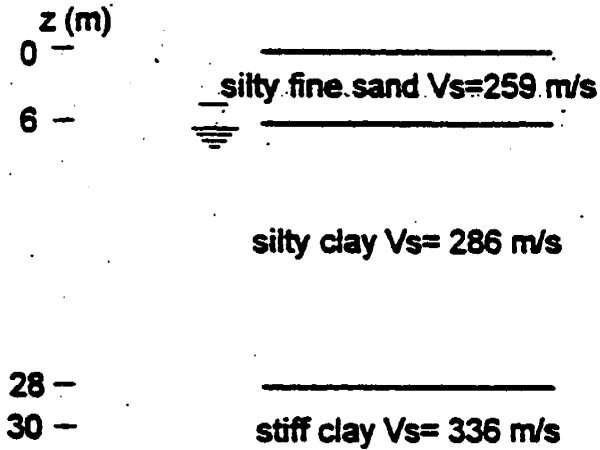
$$\bar{v}_s = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{v_{s_i}}} = \frac{30}{\frac{7.5}{353} + \frac{5}{314} + \frac{9.5}{1117} + \frac{8}{1306}} = 579 \text{ m/s}$$

$360 \text{ m/s} < \bar{v}_s \leq 760 \text{ m/s}$. Therefore, the site is classified as Soil Profile Type C.

Soil Profile Type C

EXAMPLE No. 2

SOIL PROFILE



SITE CLASSIFICATION

Using the \bar{v}_s method, the average the average shear wave velocity is given by (see Appendix I):

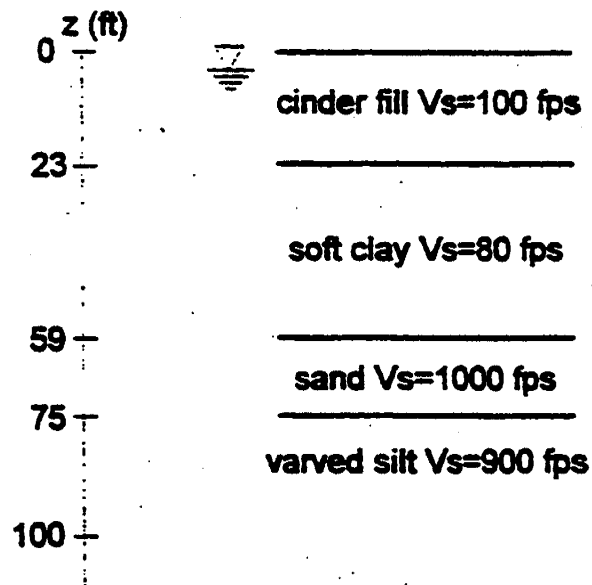
$$\bar{v}_s = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{v_{s_i}}} = \frac{30}{\frac{6}{259} + \frac{22}{286} + \frac{2}{336}} = 283 \text{ m/s}$$

$180 \text{ m/s} \leq \bar{v}_s \leq 360 \text{ m/s}$. Therefore, the site is classified as Soil Profile Type D.

Soil Profile Type D.

EXAMPLE No. 3

SOIL PROFILE



SITE CLASSIFICATION

Using the v_s method, the average the average shear wave velocity is given by (see Appendix I):

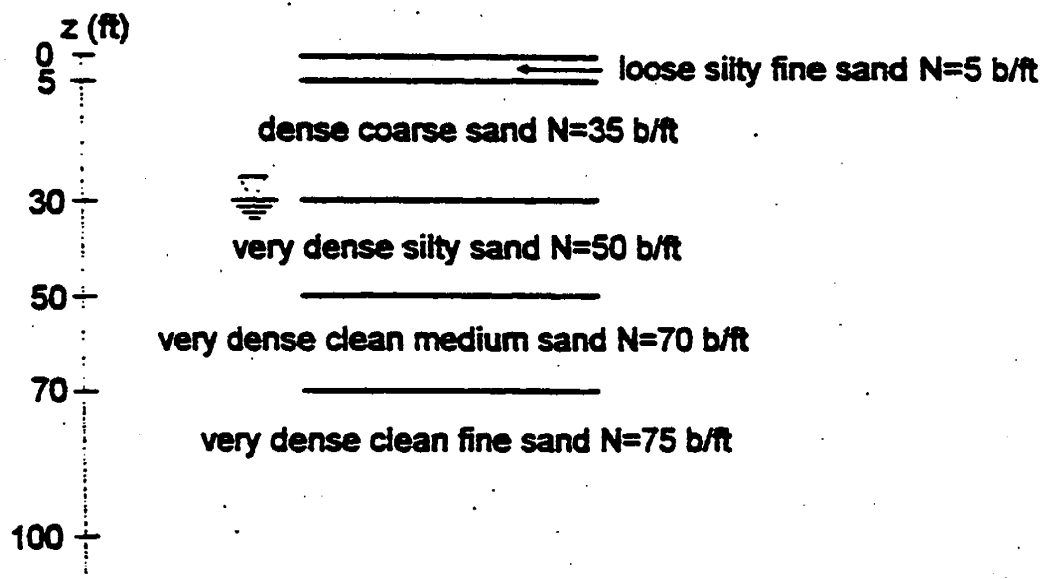
$$\bar{v}_s = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{v_{si}}} = \frac{100}{\frac{23}{100} + \frac{36}{80} + \frac{16}{1000} + \frac{25}{900}} = 138 \text{ fps}$$

$\bar{v}_s < 600 \text{ fps}$. Therefore, the site is classified as Soil Profile Type E.

Soil Profile Type E

EXAMPLE No. 4

SOIL PROFILE



SITE CLASSIFICATION

Using the \bar{N} method, the average blow count can be calculated as is given by (see Appendix I):

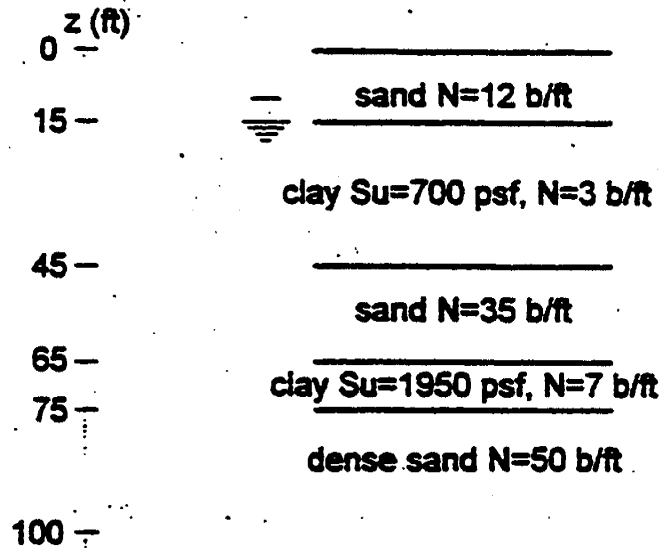
$$\bar{N} = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{N_i}} = \frac{100}{\frac{5}{5} + \frac{25}{35} + \frac{20}{50} + \frac{20}{70} + \frac{30}{75}} = 36 \text{ b/ft}$$

$\bar{N} < 50$ blows/ft. Therefore, the site is classified as Soil Profile Type D.

Soil Profile Type D

EXAMPLE No. 5

SOIL PROFILE



SITE CLASSIFICATION

N method *

If the \bar{N} method is used, the average blow count can be calculated as (see Appendix I):

$$\bar{N} = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{N_i}} = \frac{100}{\frac{15}{12} + \frac{30}{3} + \frac{20}{35} + \frac{10}{7} + \frac{25}{50}} = 7 \text{ blows/ft}$$

$\bar{N} < 15$ blows/ft. Therefore, the site is classified as Soil Profile Type E

* \bar{V}_s method is preferred by converting all \bar{N} values into \bar{V}_s values using appropriate empirical equations.

EXAMPLE No. 5 (continued)

s_u method

If the s_u method is used, two parameters are needed, the \bar{N}_{60} for the cohesionless soil layers.

and s_u for the cohesive soil layers (see Appendix I):

$$\bar{N}_{60} = \frac{d_s}{\sum_{i=1}^n \frac{d_i}{N_i}} = \frac{15 + 20 + 25}{\frac{15}{12} + \frac{20}{35} + \frac{25}{50}} = 26 \text{ blows / ft}$$

$$s_u = \frac{d_c}{\sum_{i=1}^n \frac{d_i}{s_{ui}}} = \frac{30 + 10}{\frac{30}{700} + \frac{10}{1950}} = 834 \text{ psf}$$

$15 < \bar{N}_{60} = 26 < 50$. Therefore, the site would be classified as Soil Profile Type D.

$s_u = 834 < 1000$. Therefore, the site would be classified as Soil Profile Type E.

Finally, with the s_u method, the softer classification is selected: Soil Profile Type E.

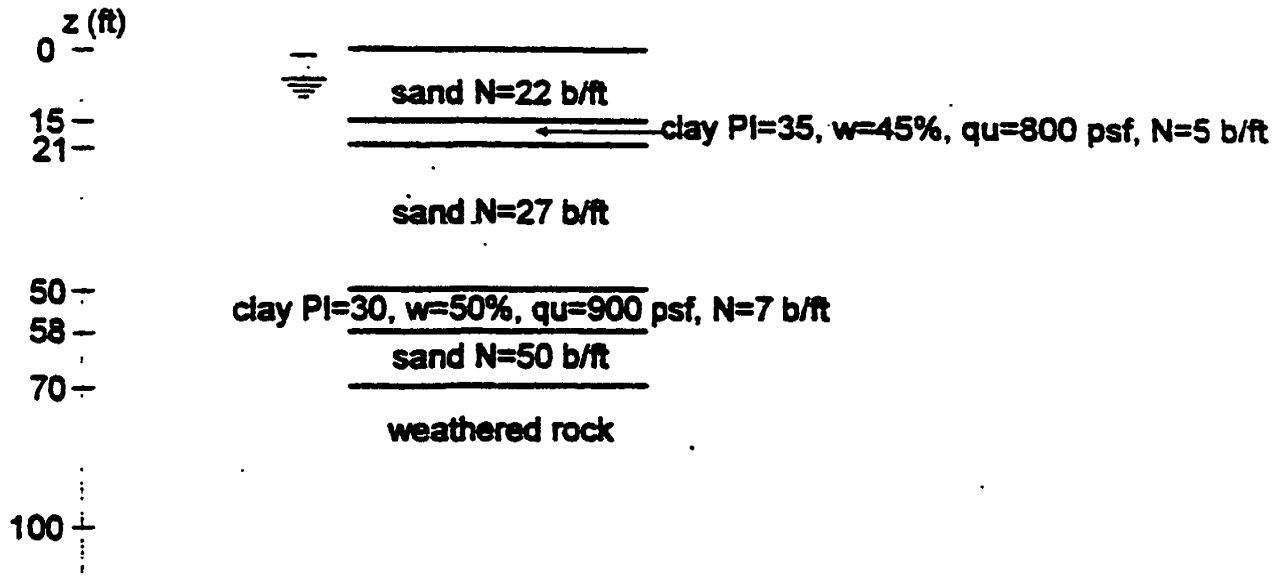
FINAL SITE CLASSIFICATION

Both the \bar{N} method and s_u method give Soil Profile Type E.

Soil Profile Type E

EXAMPLE No. 6

SOIL PROFILE



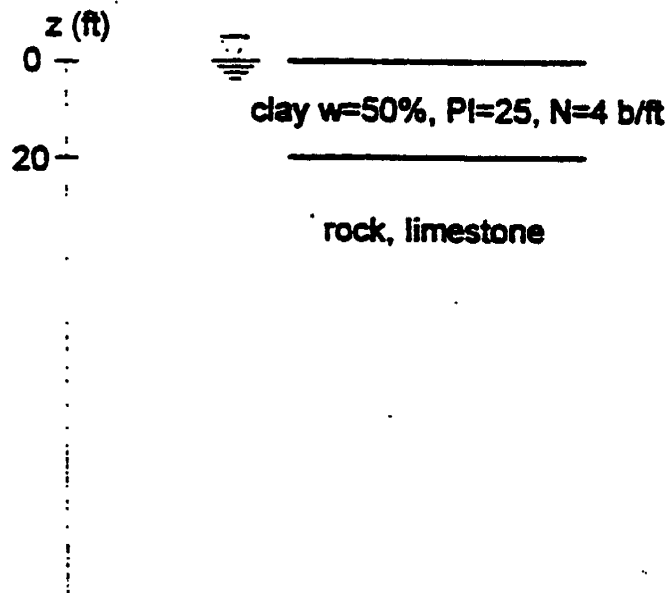
SITE CLASSIFICATION

S_u for the 6 ft layer is $0.5q_u = 400$ psf and for the 8 ft layer is $S_u = 450$ psf. For both layers the S_u is less than 500 psf. Therefore, we have a total thickness of soft clay $6+8=14$ ft > 10 ft, where soft clay is defined as $PI > 20$, $w \geq 40\%$ and $S_u < 500$ psf. Therefore, this is a Soil Profile Type E (see Appendix I).

Soil Profile Type E

EXAMPLE No. 7

SOIL PROFILE



SITE CLASSIFICATION

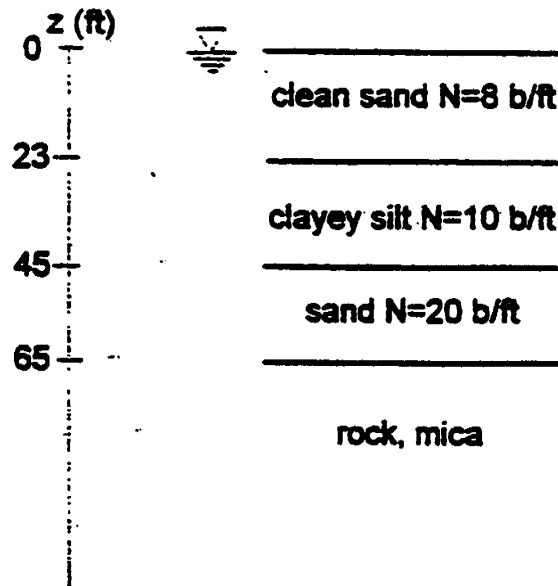
The profile contains a soft clay layer thicker than 10 ft ($N=4$ b/ft, $PI=25$, $w=50\%$).

Therefore this is a Soil Profile Type E.

Soil Profile Type E.

EXAMPLE No. 8

SOIL PROFILE



SITE CLASSIFICATION

This soil profile contains at least one saturated loose sand layer which would liquefy (Seed and Idriss, 1982). Hence the site is classified as Soil Profile Type F, and a site specific evaluation and analysis is required.

Soil Profile Type F

APPENDIX TO SECTION 6B -4

Generation of Multiple-Support Artificial Ground Motions

[Appendix D to Report (1)]

(The time histories are provided in digital form and are available on CD-ROM , for the designers, from Deputy Chief Engineer [Structures] office.)

GENERATION OF MULTIPLE-SUPPORT ARTIFICIAL GROUND MOTIONS

D.1 INTRODUCTION

To address the seismic threat to large structures such as a long bridge, it is important to consider the spatial variation of ground motions along the length of the structure. There are several factors that may induce this variation. First, unlike small structures, large structures usually have dimensions large enough such that the differences of the arrivals of seismic waves at different parts of the structure become significant in the analysis. Second, scattering and three-dimensional wave propagation effects can cause variation of ground motions along the length of the structure, even if the soil conditions are similar. The first effect, where relative ground motion along a large structure are significant due to the differences in the arrival times of seismic waves, is referred to as the wave-passage effect. The second effect, where motions along a large structure are neither identical nor independent due to spatial variations of seismic waves, is referred to as the spatial-coherency effect.

In addition to the two effects described above, there are the extended source effect (mixing of wave types and source directions) and the attenuation effect (varying distance to the fault rupture), each of which can add its own variations to ground motions. The extended source effect is usually not significant. The attenuation effect is small in this study because the bridge lengths of interest (of the order of one mile or less) is small relative to the dominant distance R^* obtained from the hazard calculations. Thus, only the wave-passage and coherency effects are considered in this study.

Typically, these effects are characterized by generating multiple-support artificial ground motions that exhibit the desired wave-passage and coherency effects and have response spectra

consistent with the results from the probabilistic seismic-hazard analysis, with the geometry of the bridge piers, and with the azimuth of the dominant seismic source relative to the bridge axis.

Because these artificial ground motions are to be used with several bridges, it is impossible to accommodate the exact pier layout of each bridge. Instead, we generate ground motions at 17 hypothetical piers on a straight line, with 100-m spacing between adjacent piers. By selecting an appropriate subset of the ground motions generated for these piers, one can approximate the ground motions for any bridge length up to one mile.

If there is significant variation in the foundation conditions of the bridge piers, the anticipated ground-motion amplitudes at the piers will be different as a result of differences in the site-amplification properties of the corresponding soil columns. In addition, the travel times of seismic waves through these soil columns will be different, causing non-uniform time delays in the corresponding ground motions. Because the artificial ground motions from this study are to be used with several bridges, it is impossible to accommodate bridge-specific variations in foundation conditions. This study will assume that all piers have the same foundation conditions (i.e., rock outcrop typical of the northeastern U.S.) and the same elevation.

We consider return periods of 500 and 2500 years and generate three sets of ground motions for each return period.

The formulation used here (based on Abrahamson, 1993, 1998a) has been used in several studies of multiple-support inputs for bridges in California (Geomatrix and ICEC, 1992; Woodward-Clyde, 1994) and in the Central United States (Risk Engineering, 1994).

D.2 ASSUMPTIONS

D.2.1 Wave Passage

The wave-passage effect is caused by non-vertical wave propagation which induces systematic

time delays in the arrival of seismic energy. To capture this effect, one needs the information of the apparent wave velocity in the direction of propagation and the fault-structure geometry. Previous studies of strong motions on seismic arrays indicate that the apparent velocity of S-waves is in the range of 2.0-5.0 km/s (Luco and Sotiropoulos, 1980; Chang, et al, 1986; Abrahamson, 1985, 1992, 1998b). This apparent shear-wave velocity is lower than the crustal shear-wave velocity. For conservative reasons, we use 2.5 km/s as the apparent velocity of the S-waves in the analysis. This value is also used in Abrahamson (1993). The apparent velocity along the bridge axis can be written as

$$V_{\text{Bridge}} = \frac{V}{\sin\theta} \quad (\text{D-1})$$

where V is the apparent velocity in the direction of wave propagation and θ is the angle between the line perpendicular to the bridge axis and the direction of propagation of the incoming waves. Note that the apparent velocity of S-waves is chosen over those of P and surface waves because S-waves generally contain the strongest shaking. It is also assumed that all piers have the same foundation conditions and the same elevation. If these were not the same, additional time shifts would be added to account for the associated differences in wave travel time.

D.2.3 Coherency

The coherency effect can be examined by analyzing the Fourier content of the ground motion inputs. The variations of the Fourier amplitudes of the ground motions are not considered here since the response spectra of all time histories need to be compatible with a given target spectrum. Therefore, only the variations of the Fourier phases will be considered for the coherency effect.

The phase variations can be characterized by the path coherency, which can be written in terms of a complex coherency function for ground motions at two stations i and j as

$$\gamma_{ij}(f, \xi) = \frac{S_{ij}(f)}{\sqrt{S_{ii}(f)S_{jj}(f)}} \quad (D-2)$$

where $S_{ij}(f)$ is the smoothed cross-power spectrum density and $S_{ii}(f)$ and $S_{jj}(f)$ are the smoothed power spectrum densities of the motions at station i and j , respectively; f is the frequency and ξ is the distance between the two stations. Typically the absolute value of the coherency (also called the lagged coherency) is used. Note that the computed coherency depends on the specific smoothing window used. Also note that the lagged coherency does not require a unique apparent wave speed for all frequencies. In order to apply a single wave speed for the wave-passage effect, however, a single plane-wave factor needs to be introduced. The final coherency function, denoted as $\gamma_{pw}(f, \xi)$, is obtained by multiplying $\gamma(f, \xi)$ by a plane-wave correction factor, $h(f, \xi)$, as

$$|\gamma_{pw}(f, \xi)| = |\gamma(f, \xi)h(f, \xi)| \quad (D-3)$$

This study uses the coherency model recently developed by Abrahamson (1998a) from the analysis of seismic-array data. This model takes the form

$$|\gamma_{pw}(f, \xi)| = \left[1 + \left(\frac{f}{a_1 f_c(\xi)} \right)^{n_1} \right]^{-1/2} \left[1 + \left(\frac{f}{a_2 f_c(\xi)} \right)^{n_2} \right]^{-1/2} \quad (D-4)$$

with

$$f_c = \left[1.886 - 2.2209 \ln \left(\frac{4000}{\xi} + 1.5 \right) \right] \quad (D-5)$$

for horizontal motions, and

$$f_c(\xi) = \exp(2.43 - 0.025 \ln(\xi + 1) - 0.048 [\ln(\xi + 1)]^2) \quad (D-6)$$

for vertical motions. The values of the coefficients in Equation D-4 are given in Table D-1. Figures D-1 and D-2 show calculated values of the plane-wave coherency for selected distances. The same coherency model is typically used for both rock and soil, although soil sites tend to exhibit slightly more coherency than rock sites. This model was derived on the basis of data from seismic arrays throughout the world and is consistent with coherency results obtained by Menke et al. (1990) for two seismic arrays in the eastern United States.

D.3. SIMULATION PROCEDURE

The following steps outline the procedure used for generating multiple-support ground motions.

- Step 1. Select initial time histories. The initial time histories are selected to have magnitudes and distances similar to the real events that dominate seismic hazard at the site (see Section D.4.2).
- Step 2. Modify the initial time histories to be compatible with the rock uniform-hazard spectrum for the return period of interest. This step is achieved using a time-domain spectral matching method. A baseline correction is also performed as part of this step. The spectral match is maintained for frequencies of 0.15 Hz and higher (for most artificial motions, this match is maintained down to 0.1 Hz). The reference time history obtained in this step has the same high-frequency energy as eastern-U.S. records because it has been modified to match a target spectrum based on eastern-U.S. attenuation equations. Therefore, no deficit of high-frequency energy is introduced by using western records as the starting points.
- Step 3. Generate time histories (for all piers) that are compatible with the coherency model of Figures D-1 or D-2. We use the procedures developed by Abrahamson, as documented in Appendix A of Abrahamson (1993), to generate these motions. This procedure generates the ground motion at each pier by adding random shifts to the phase angles

of the reference time history (from step 2). The probability distribution of the random shift for a given frequency depends on the coherency model, the frequency, and the distance to the reference pier. In this study, pier 9 is taken as the reference pier for generating motions.

Step 4. Modify each time-history generated in Step 3 to be compatible with the rock uniform-hazard spectrum, using a frequency-domain method.

Step 5. Apply the wave-passage effect by systematically shifting the time-histories of the motions at various supports according to the S-wave velocity and the fault-structure geometry.

These artificial ground motions apply to rock outcrop conditions (i.e., rock at the surface) and assume that site conditions are similar for all piers. In site-specific applications to bridges with non-uniform foundation conditions, an additional step would introduce amplification factors and time delays consistent with the soil column beneath each pier. The amplification effect may be introduced by propagating each time history through the corresponding soil column using SHAKE or a similar program. Depending of the program used, the time delays may be introduced as part of the amplification calculation or may have to be introduced separately.

D.4 APPLICATION

Multiple-support ground motions were generated for return periods of 500 years and 2500 years. Three sets of three-component, 17-pier ground motions were generated for each return period, resulting in a total of 306 artificial time histories.

D.4.1 Target Spectra

The target spectra for horizontal motions are taken as the 84% uniform-hazard spectra given in Table 4-1. These spectra correspond to rock-outcrop motions. In addition, these spectra contain the high-frequency energy typical of ground motions at rock sites in the eastern U.S.

because they were derived using eastern-U.S. attenuation equations. The corresponding vertical spectra are obtained by applying the EPRI (1993) frequency-dependent vertical/horizontal (V/H) ratios. The EPRI ratios, as described in Section 5, were modified so that the transition from $V/H=0.7$ to $V/H=1$ occurs between frequencies of 10 and 30 Hz. This modification leads to more realistic spectral shapes and adds a slight conservatism relative to the EPRI (1993) recommendations. The vertical and horizontal spectral shapes are given in Figure D-3.

D.4.2 Selection of Real Ground Motion Records

The first step in the selection of real ground motion records to use as inputs to the time-history simulation is to determine the magnitudes and distances that dominate seismic hazard for the return periods of interest. To this effect, we compute the dominant magnitudes and distances M^* and R^* from the seismic-hazard de-aggregation results presented in Section 4, using the procedure by McGuire (1995). The resulting values are shown in Table D-2. Based on the dominant magnitudes and distances for 1 Hz, we select magnitude ranges of 5.7-6.3 for the 500-year return period and 6.3 to 6.7 for the 2500-year return period. The selected distance range is 15-30 km for both return periods.

We searched the strong-motion databases for California (Silva, personal communication), CEUS (EPRI, 1993), and worldwide (NOAA-NGDC) for ground-motion records in these magnitude-distance ranges on rock site conditions. Six records (three for each return period) were selected on the basis of their durations, time-domain appearance, and spectral shapes. Table D-3 lists these records and their characteristics.

D.4.3 Wave Passage

In this study, seismic waves are equally likely to come from any direction for any of the New York City bridges of interest. θ is conservatively chosen as 90 degrees (corresponding to waves that travel parallel to the bridge axis). Once the wave velocity is known, the wave-passage effect is introduced by applying systematic time shifts to the coherency-compatible

time histories. It is assumed that the waves propagate in the direction from pier 1 to pier 17. Pier 9 is taken as the reference pier.

D.5 RESULTS

The artificial time histories were generated using the procedure outlined above, and were checked for consistency with the coherency model, for consistency with the target response spectra, and for overall appearance. Figures D-4 through D-9 show the acceleration (top), velocity (middle), and displacement (bottom) time histories for the three components of motion at pier 9 (the reference pier). One figure is provided for each combination of return period and ground-motion set. Figures D-10 through D-13 show the acceleration, velocity, and displacement time histories at piers 5, 9, and 13 for the longitudinal and vertical components of ground-motion set 1. The corresponding displacement time histories of all the piers are shown in Figures D-14 through D-17. All amplitudes in Figures D-4 through D-17 have unit of cm/sec^2 , cm/sec , and cm . Figures D-18 through D-35 compare the response spectra (for 5% damping) from the artificial motions the corresponding target spectra and indicate a good match down to frequencies of 0.15 to 0.1 Hz. Figure D-36 shows the coherency calculated from the artificial time histories in ground-motion set 500_01L. The calculated coherency is consistent with the horizontal coherency model in Figure D-1.

All artificial multiple-support acceleration time histories are provided in digital form for use as inputs to time-domain dynamic structural analyses. These time histories may be used as a whole set (i.e., for all 17 piers) or in any appropriate subset. The name of each time-history file is of the form

RRRR_SSCPP.acc

where RRRR indicates return period (500 or 2500), SS indicates ground-motion set (01, 02, or 03), C indicates component of motion (L for longitudinal, T for transverse¹, and V for vertical

¹ Our naming of the two horizontal components as longitudinal and transverse is arbitrary because there is no fundamental difference between the two (i.e., we could have switched the two

[up]), and PP indicates pier number (01 through 17). The time histories for each RRRR_SSC combination are stored in a separate sub-directory. The duration of each time history is the shortest of 50 sec and the duration of the real time history used as seed. The time step in all time histories is 0.005 sec

D.6 NOTES ON THE USE OF THE ARTIFICIAL GROUND MOTIONS

Interpolation between Pier Motions. Interpolation of time histories to obtain the time history at the exact location of a bridge support has three detrimental effects, as follows: (1) reduction of incoherent (high-frequency) portions of the motion (because interpolation may be viewed as a weighted averaging), and (2) reduction in the coherent (low-frequency) portion of the motion (due to averaging of two sinusoids that are not in phase as a result of wave-passage effects), and (3) alteration of the phase spectrum, thereby affecting the match to the target spectrum.

Although neither of these effects is very large, it is recommended that the time histories not be interpolated. Instead, the engineer should select a subset of the time histories provided, such that their spacing provides the closest possible match to the footprint of the bridge under consideration.

If it becomes necessary to generate artificial ground motions with time delays and coherency that match the geometry of a specific bridge, the preferred approach is to generate site-specific motions for that bridge geometry by repeating Steps 3 through 5 in Section D.3.

Application to Short Spans. For bridges with short total spans, one can use a subset of the

components because longitudinal and transverse time histories are intended to match the same target spectrum and to have the same duration and other characteristics). The longitudinal and transverse components are not identical to each other, however, because they use different horizontal components from the same record as input. It would be perfectly legitimate, but is not required, to run an additional analysis where the longitudinal and transverse components are reversed. It is true that the two analyses may produce different demands; this is the price paid for not running a large suite of artificial ground motions, which would allow the calculation of statistically stable estimate of the average seismic demand. It would have been more accurate, but somewhat cumbersome, to name them horizontal-1 and horizontal-2.

piers, as indicated in Appendix D. It is desirable, but not required, to include Pier 9 (the reference pier) in this subset.

Simultaneity of components. The three components of motion represent simultaneous motions. They should be entered as such for software that accepts multi-component inputs.

Use of Ground-Motion Sets Each pier ground motion in each ground-motion set matches the corresponding target spectrum. Therefore, it is not necessary to use all three ground-motion sets. It is preferable, but it may not always be practical, to run all three ground-motion sets and use the average of the absolute value of the response from all three runs.

D.6 REFERENCES

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TABLE D-1

COEFFICIENTS OF COHERENCY MODEL

Coefficient	Horizontal	Vertical
a_1	1.647	3.15
a_2	1.01	1.0
n_1	7.02	4.95
$n_2(\xi)$	$5.1 - 0.51 \ln(\xi+10)$	1.685

TABLE D-2
DOMINANT MAGNITUDES AND DISTANCES

Return Period (yrs)	Frequency (Hz)	M*	R*
2500	1	6.5	22.5
	PGA	5.2	12.5
500	1	6.0	22.5
	PGA	5.1	18

TABLE D-3
REAL GROUND MOTION RECORDS USED AS REFERENCE GROUND MOTIONS

500-year Return Period

Set	Earthquake	Magnitude	Station	Distance (km)
01	1984 Morgan Hill	6.2 M	Gilroy Station 1	16
02	1987 Whittier Narrows	6.0 M	Mount Wilson	21
03	1980 Mammoth Lakes	6.3 M	Long Valley Dam, left abutment	16

2500-year Return Period

Set	Earthquake	Magnitude	Station	Distance (km)
01	1994 Northridge	6.7 M	Vasquez Rocks Park	24
02	1994 Northridge	6.7 M	CDMG Sta. 24047	24
03	1985 Nahanni, Canada	6.4 m _s	Site 3	22

Plane-wave coherency for Horizontal Motions

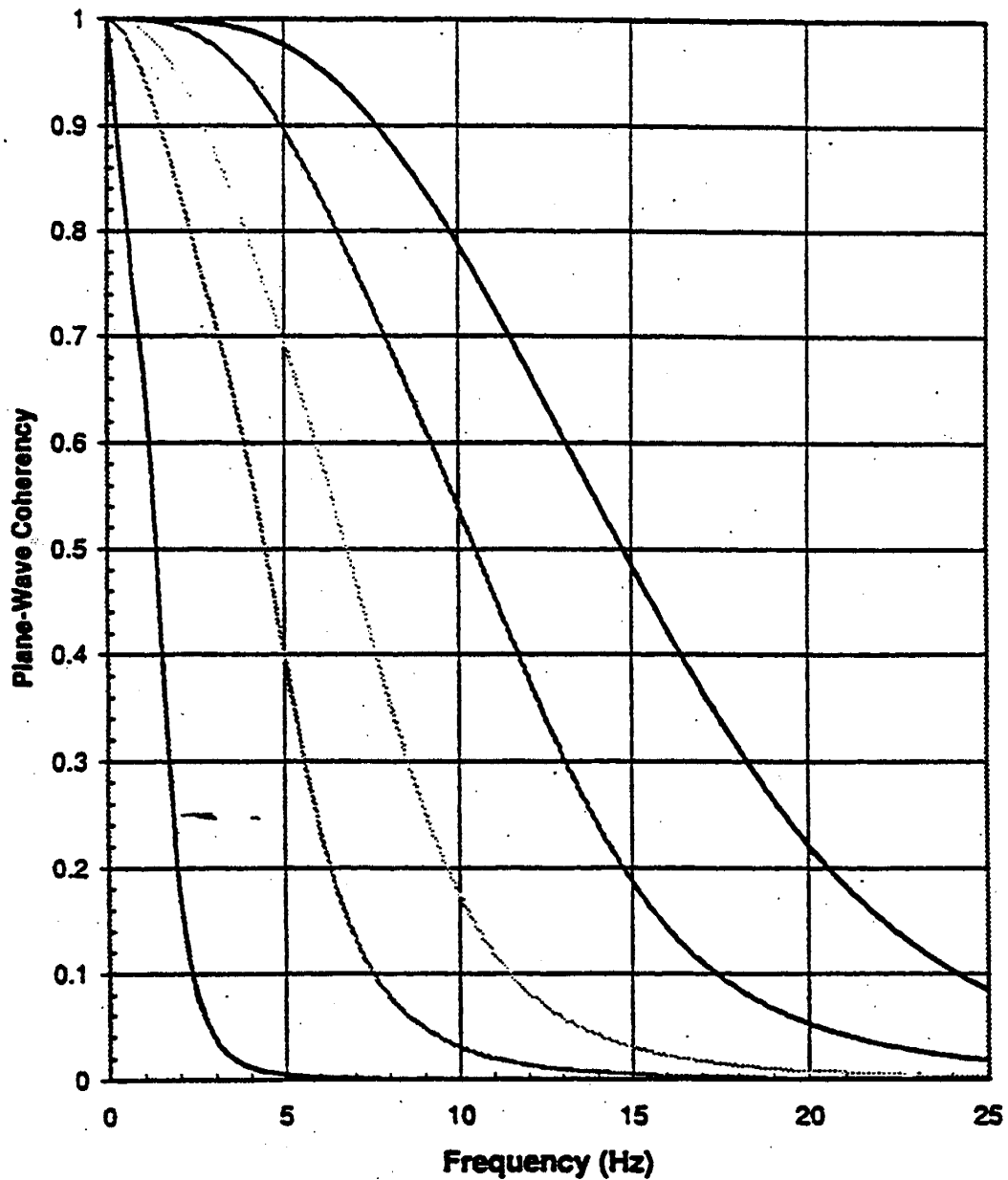


Figure D-1. Plane-wave coherency model for horizontal motions. Curves shown for separation distances of 10m (highest), 50m, 200m, 500m, and 2000m (lowest).

Plane-wave coherency for Vertical Motions

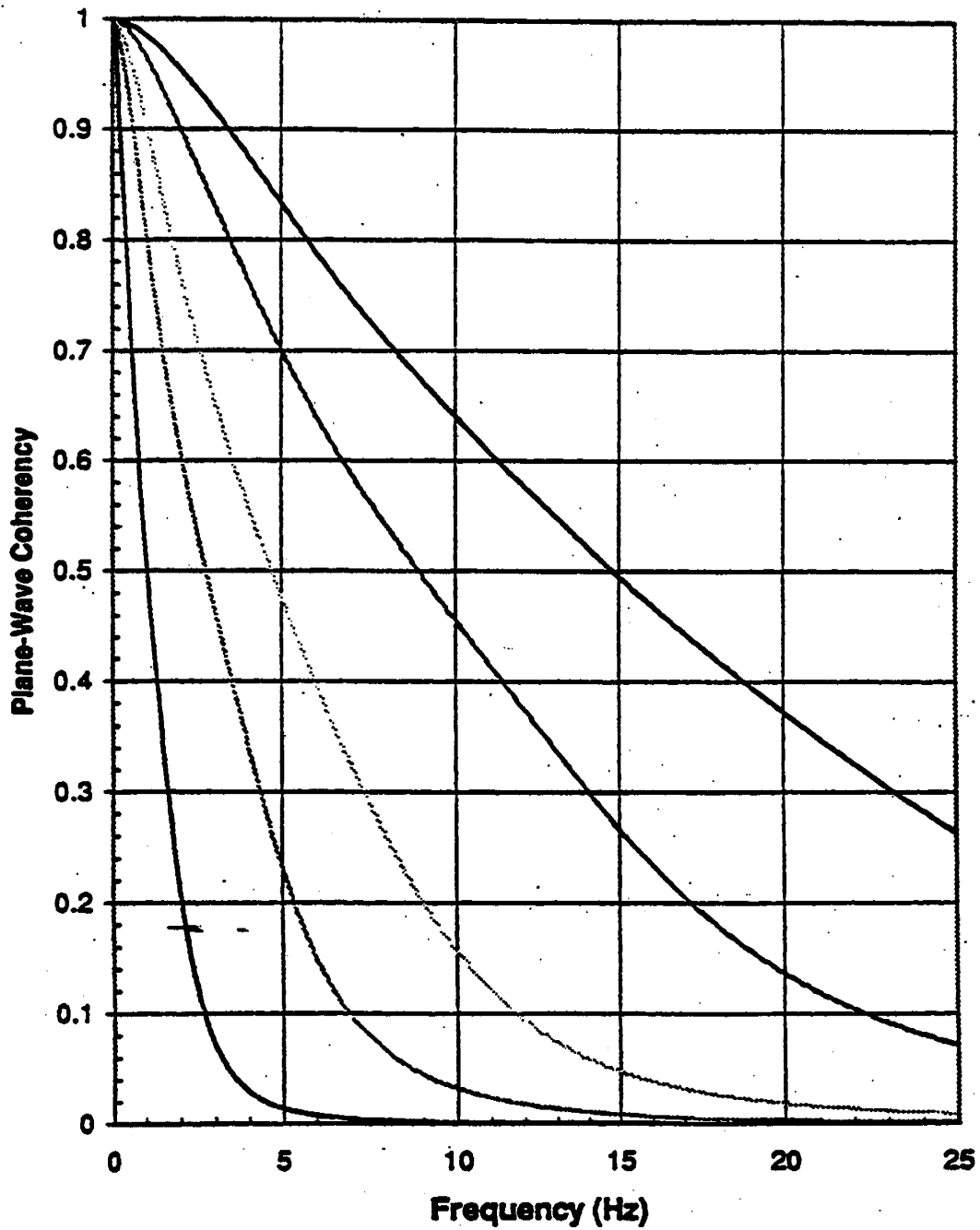


Figure D-2. Plane-wave coherency model for vertical motions. Curves shown for separation distances of 10m (highest), 50m, 200m, 500m, and 2000m (lowest)

Target Spectra

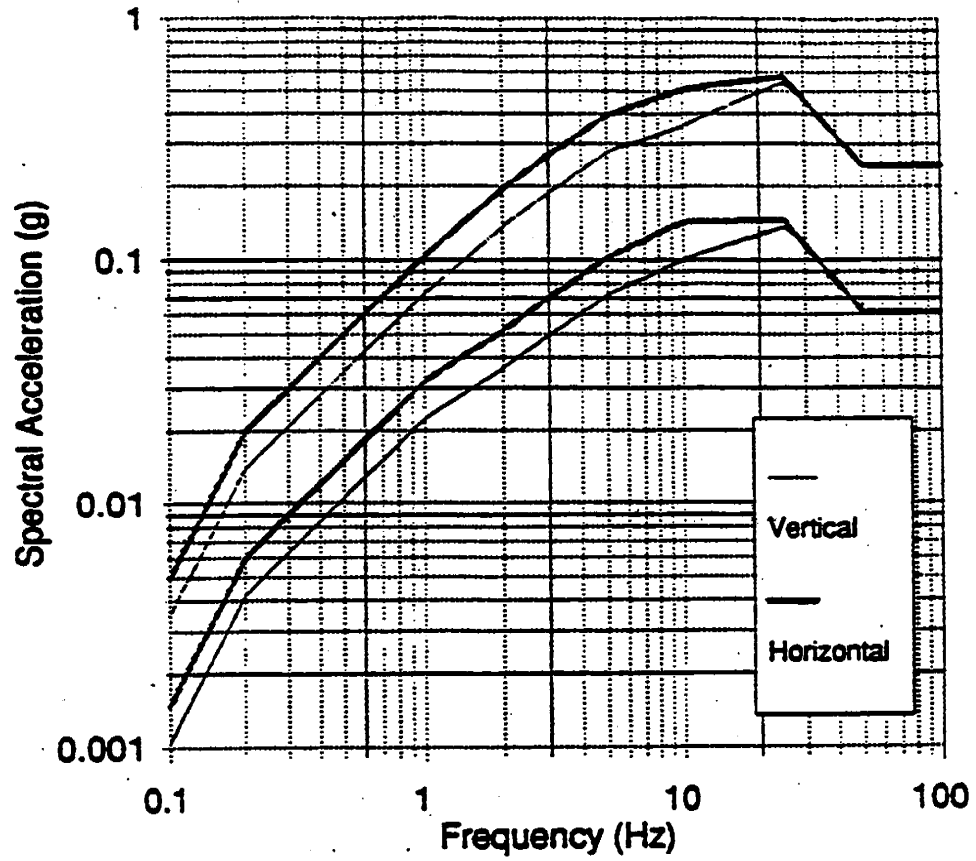


Figure D-3. Target spectra for return periods of 2500 years(top) and 500 years (bottom).

Appendix E

3.24.3.2 Change **Metric Expression** for **Parameter E** to read as follows:

$$(1.22 + 0.06S) \leq 2.13\text{m}$$

and **Metric Units**

Change mm to m