STANDARD PLANS FOR HIGHWAY BRIDGES

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
Washington, D.C.
October 1964

REPRINT 1978

Volume
1. CONCRETE SUPERSTRUCTURES
2. STRUCTURAL STEEL SUPERSTRUCTURES
3. TIMBER BRIDGES
4. TYPICAL CONTINUOUS BRIDGES
5. TYPICAL PEDESTRIAN BRIDGES

Volume V

TYPICAL PEDESTRIAN BRIDGES

SPECIFICATIONS

cifications and manuals for design, materials, and struction are included in the following:

ublished by American Association of State Highway fficials, 917 National Press Building, Washington 4, .C.

a) "Standard Specifications for Highway Bridges", adopted 1961 ($5.00) and all tentative revisions thereto.
b) "Standard Specifications for Highway Materials and Methods of Sampling and Testing" (2 volumes) adopted 1961 ($8.00)

ublished by the American Institute of Steel Construction, 101 Park Avenue, New York 17, New York

a) "Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings", adopted 1961, Revised 1963
b) "Manual of Steel Construction", Sixth Edition, 1963 ($7.00)
c) "Design Manual for Orthotropic Steel Plate Deck Bridges", 1963 ($10.00)

ublished by American Society for Testing Materials, 916 Race Street, Philadelphia 3, Pennsylvania
ASTM Standards" 1964

ublished by American Standards Association, 70 East 5th Street, New York 17, New York
"Surface Roughness Waviness and Lay", (ASA Standard B46.1—1955) ($1.50)

ublished by American Welding Society, Inc., 345 ast 47th Street, New York 17, New York
"Specfications for Welded Highway and Railway Bridges", Sixth Edition, 1963 ($4.50)

ublished by Douglas Fir Plywood Association, 1119 A" Street, Tacoma 2, Washington
"DFPA Specification No. SS-8, Stressed Skin Panels", 1962

ublished by National Lumber Manufacturers Association, Washington 6, D.C.
"National Design Specification for Stress-Grade Lumber and Its Fastenings", 1962 (50 cents)

INTRODUCTION

GENERAL

The 1964 Volume V, Typical Pedestrian Bridges, as herein presented is an extension of the 1962 four-volume edition of the Standard Plans for Highway Bridges. Volumes I, II and III represented a substantial revision of the 1953 and 1956 editions, and Volume IV was an expansion thereof. Volumes I and II contain superstructure plans for simple concrete and steel bridges; Volume III contains substructure and superstructure plans for timber bridges; Volumes IV and V present complete detailed plans for typical four-span continuous highway bridges and typical pedestrian bridges respectively.

These plans are intended to serve as a useful guide to State, county, and local highway departments in the development of suitable and economical highway bridges on primary, secondary and urban highways, and pedestrian bridges across the Interstate or other dual highway systems. The plans should be particularly valuable to the smaller highway departments with limited engineering staffs.

An effort has been made to give sufficiently complete information on all plans so that they will approach contract drawings as nearly as practicable. For any given location, however, it must be expected that the requirements imposed by the site conditions will necessitate modification of the plans, since they have been developed for right-angle crossings, level roadway grades and typical walkway vertical curves only.

TYPICAL PEDESTRIAN BRIDGES

This volume has been developed to provide typical examples of pedestrian bridges across the Interstate or other dual highway systems. The roadway crossing selected, two 3-lane roadways with a 40-foot median, was considered to be average and, therefore, the plans can be used as presented in a considerable number of cases. All examples shown have been selected on the basis of aesthetics and economic suitability.

To improve the educational value, calculations for several of the more difficult to design pedestrian bridges have been included with the design plans. It is hoped that the inclusion of these calculations will serve as examples and encourage the use of the most suitable bridge for the location for which it is intended.

The inclusion of two-span continuous bridges has been the direct result of a recent study which showed a favorable economical comparison between two-span and four-span continuous bridges. Removing the shoulder pier creates an impression of openness, which greatly enhances the aesthetic qualities of the bridge. In addition, it increases sight distance, increases safety and improves maintenance conditions.

It is assumed that normally the pedestrian bridge can be placed in a cut section or ramp on earth embankment. For those bridges that require stairs or ramps in extremely limited space, stairway and ramp alternates have been detailed on sheets 454 and 455 and can be adapted for most all bridges.

Appendix A presents a rather complete design criteria for use in the design of pedestrian bridges. It is felt that the design requirements for these bridges should be more liberal than for highway bridges, therefore, the design criteria reflects a considerable number of exceptions to the AASHO Specifications.

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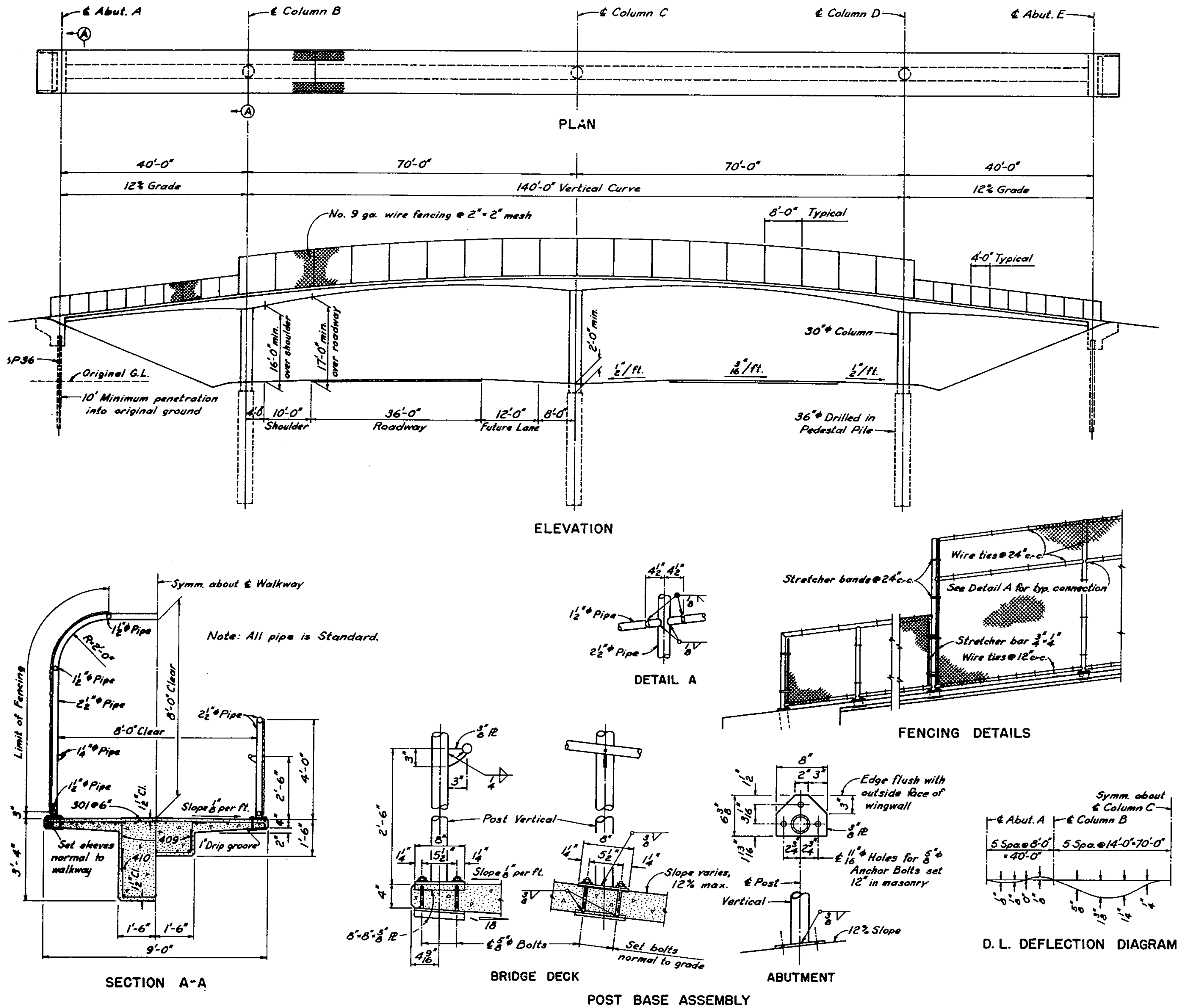

GENERAL NOTES

Design Criteria: Design Criteria for Pedestrian Bridges Over Interstate or Comparable Highways, Appendix A.

Design Specifications: AASHO Standard Specifications for Highway Bridges, 1961, with tentative revisions to 1963 except as modified or amplified by the Design Criteria.

Concrete: All concrete shall be Class A(AE) with a minimum 28 day f'_c = 4000 psi. The air entraining agent shall meet with the approval of the engineer.

Reinforcing Steel: Reinforcing steel shall be deformed bars of hard, intermediate or rail-steel grade conforming to ASTM A-15 or A-16. All bending dimensions are from out to out of the bar. Dimensions shown on the plans from the reinforcing steel to the outside edge of the concrete are all clear dimensions. Bar sizes are designated on the plans by numbers. The first digit of the three digit marks and the first two digits of the four digit marks indicate size of the bar.

Galvanized Fence and Supports: Chain link fence fabric conforming to ASTM A-392 shall be No. 9 gage wire woven in a 2 inch mesh and galvanized after weaving with a Class II zinc coating. Galvanized pipe rail and fence supports shall conform to ASTM A-53 Grade B, Type S or E. ASTM A-36 steel shall be used for all hardware, and after fabrication, shall be galvanized in accordance with ASTM A-123 and A-153. All welding and erection damage to the galvanizing shall be repaired with material conforming to Military Specification MIL-P-21035 (Ships).

Pedestal Piles: The drilled hole shall be cleaned of all loose material and the pedestal pile poured in compact coarse and fine sand, medium stiff clay or better.

Lateral Soil Pressure: Design has been based on an allowable lateral soil pressure of 300 Lbs. per square foot per foot of depth.

Soil Pressure: The required allowable soil pressure to use the stub abutments as detailed but without piles is 1 Ton per square foot.

SUMMARY OF QUANTITIES

Item	Unit	Quantity
Excavation for Structures	C.Y.	※ 30
36"Φ Pedestal Pile	L.F.	※ 70
Steel H-Pile	L.F.	※ 78
Class A(AE) Concrete	C.Y.	96
Reinforcing Steel	Lb.	• 18,320
Fencing (Including supports)	L.F.	224

※ Estimated
• Includes wt. of spiral spacers.

U. S. DEPARTMENT OF COMMERCE
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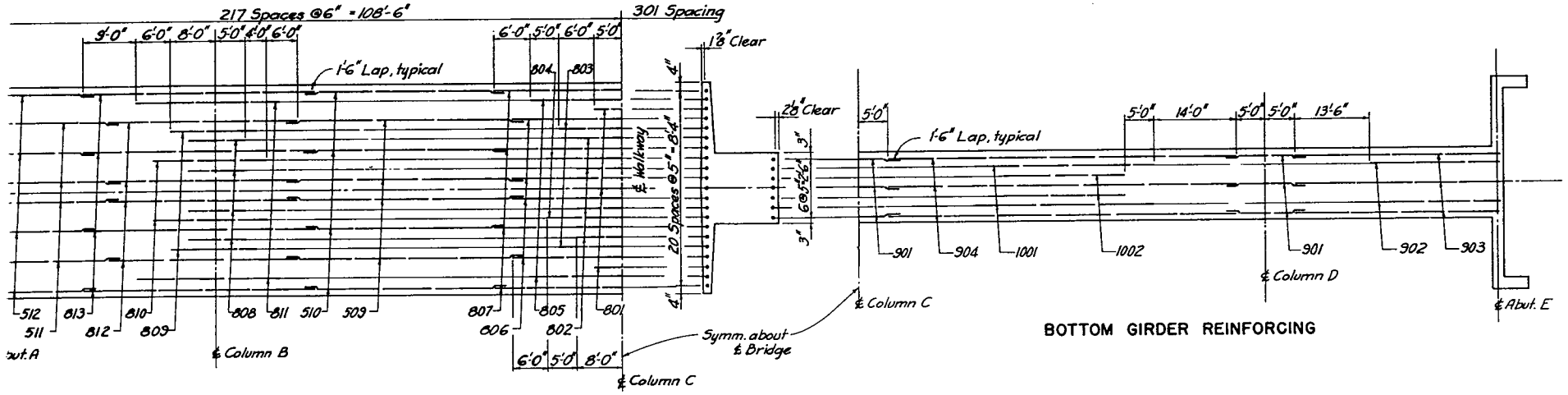
WASHINGTON, D. C.

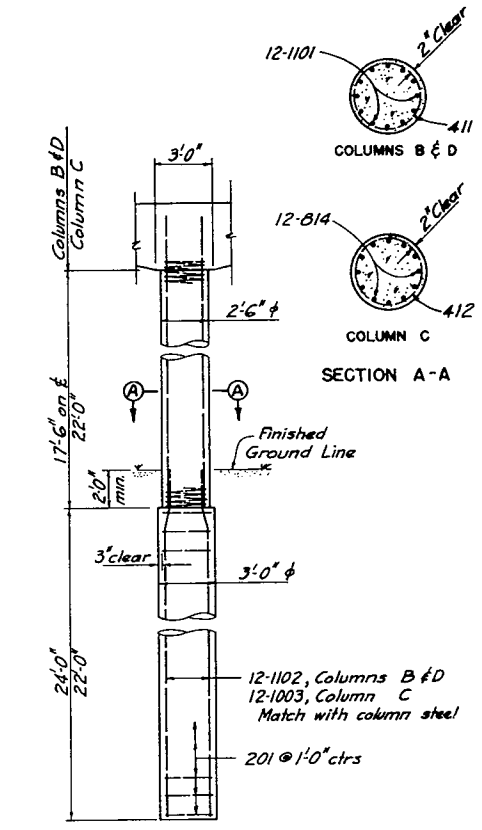
TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

FOUR SPAN CONCRETE T-BEAM BRIDGE
SPANS = 40-70-70-40 = 220 FT.
GENERAL PLAN AND ELEVATION

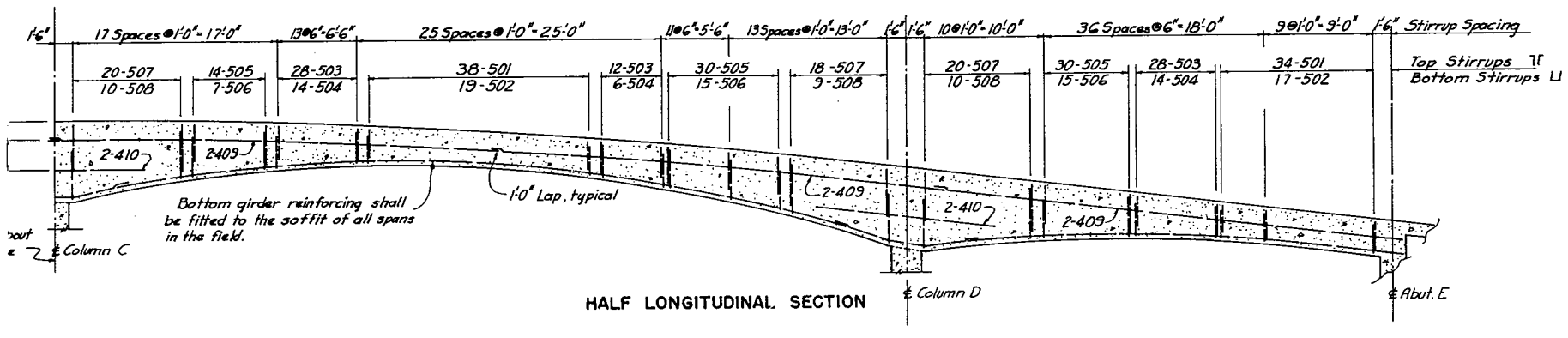
RECOMMENDED S. B. [signature] Chief Design Branch	RECOMMENDED [signature] Director of Engineering	OCTOBER 1964
RECOMMENDED [signature] Chief Bridge Division	APPROVED [signature] Administrator	SHEET NO. 101



TOP GIRDER REINFORCING



COLUMN DETAILS



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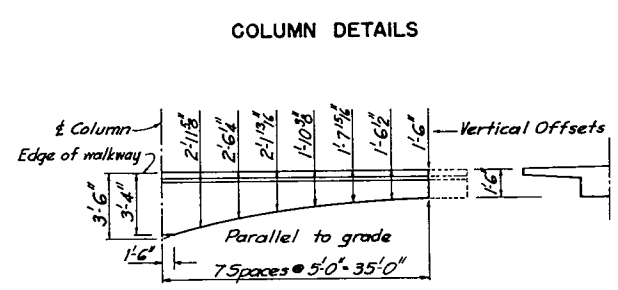


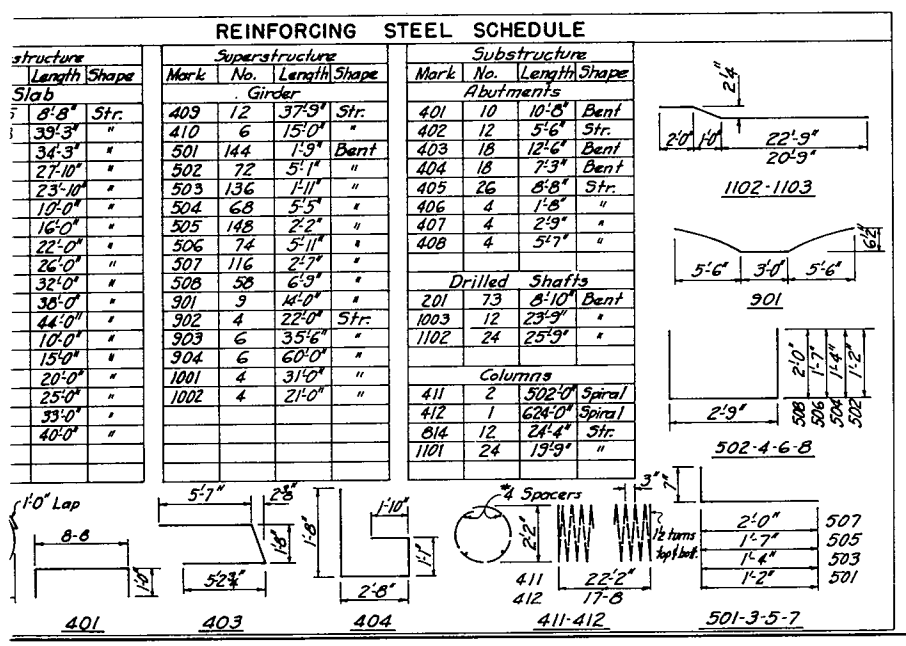
GIRDER GEOMETRICS

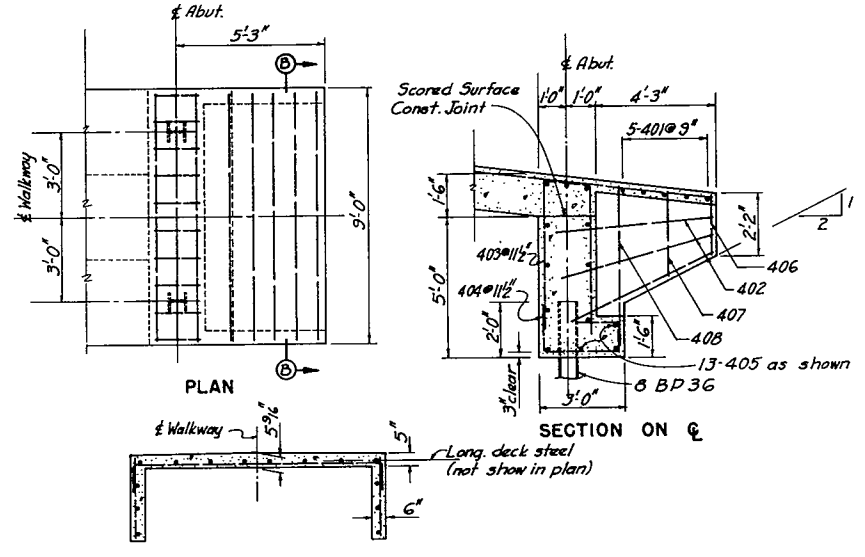
REINFORCING STEEL SCHEDULE

Superstructure Slab Mark	No.	Length	Shape
8'-8"			Str.
39'-3"			"
34'-3"			"
27'-10"			"
23'-10"			"
10'-0"			"
16'-0"			"
22'-0"			"
26'-0"			"
32'-0"			"
38'-0"			"
44'-0"			"
10'-0"			"
15'-0"			"
20'-0"			"
25'-0"			"
33'-0"			"
40'-0"			"

Superstructure Girder Mark	No.	Length	Shape
409	12	37'-9"	Str.
410	6	15'-0"	"
501	144	1'-9"	Bent
502	72	5'-1"	"
503	136	1'-11"	"
504	68	5'-5"	"
505	148	2'-2"	"
506	74	5'-11"	"
507	116	2'-7"	"
508	58	6'-3"	"
901	9	14'-0"	"
902	4	22'-0"	Str.
903	6	35'-6"	"
904	6	60'-0"	"
1001	4	31'-0"	"
1002	4	21'-0"	"

Substructure Abutments Mark	No.	Length	Shape
401	10	10'-8"	Bent
402	12	5'-6"	Str.
403	18	12'-6"	Bent
404	18	7'-3"	Bent
405	26	8'-8"	Str.
406	4	1'-8"	"
407	4	2'-9"	"
408	4	5'-7"	"
Drilled Shafts			
201	73	8'-10"	Bent
1003	12	23'-9"	"
1102	24	25'-9"	"
Columns			
411	2	502'-0"	Spiral
412	1	624'-0"	Spiral
814	12	24'-4"	Str.
1101	24	19'-9"	"





ABUTMENT DETAILS

DO NOT SCALE

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BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

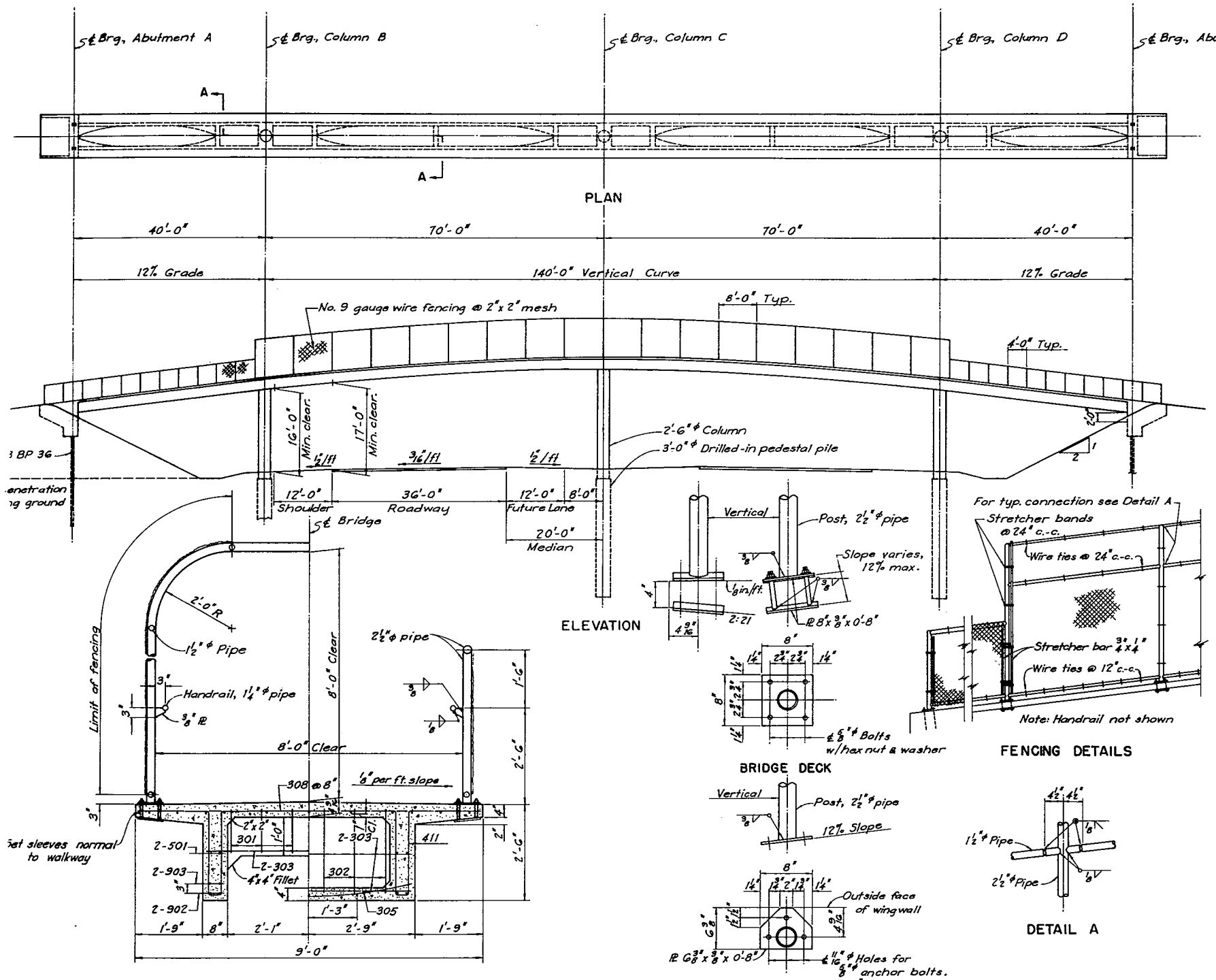
TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

FOUR SPAN CONCRETE T-BEAM BRIDGE
SPANS = 40-70-70-40 = 220 FT.
DETAILS

RECOMMENDED: S. B. Larsen, Chief Design Branch
RECOMMENDED: [illegible], Chief Bridge Division
RECOMMENDED: [illegible], Director of Engineering
APPROVED: [illegible], Administrator

OCTOBER 1964

SHEET NO. 102



GENERAL NOTES

Design Criteria: Design Criteria for Pedestrian Bridges over Interstate or Comparable Highways, Appendix A.

Design Specifications: AASHO Standard Specifications for Highway Bridges, 1961, with tentative revisions to 1963 except as modified or amplified by the Design Criteria.

Concrete: Concrete shall be Class A(AE) with a minimum 28 day $f'_c = 4,000$ psi. The air entraining agent shall meet with the approval of the engineer.

Reinforcing Steel: Reinforcing steel shall be deformed bars of intermediate, hard, or rail grade conforming to ASTM Specification A15 or A16.
Bar sizes are designated on the plans by numbers. The first digit in the three digit marks and the first two digits in the four digit marks indicate the size of the bar.
Dimensions shown on the plans from the reinforcing steel to the outside edge of concrete are all clear dimensions.
All bending dimensions are from out to out of the bar.

Pedestal Piles: The drilled hole shall be cleaned of all loose material and the pedestal pile poured in compact coarse and fine sand, medium stiff clay, or better.

Lateral Soil Pressure: Design has been based on an allowable lateral soil pressure of 300 psf per foot of depth.

Soil Pressure: The required allowable soil pressure for the stub abutments as detailed but without piles is one ton per square foot.

Galvanized Fence and Supports: Chain link fence fabric shall be No. 9 gauge wire, conforming to ASTM A 392, woven in a 2 inch mesh and galvanized after weaving with a Class II zinc coating. Galvanized steel pipe rails and fence supports shall conform to ASTM A 53 Grade B, Type S or E. ASTM A36 steel shall be used for all hardware, and after fabrication shall be galvanized in accordance with ASTM A 123 and A 153. All welding and erection damage to the galvanizing shall be repaired with material that conforms to Military Specification MIL-P-21035 (SHIPS).

SUMMARY OF QUANTITIES

Item	Unit	Quantity
Excavation for Structures	C.Y.	※ 40
36" ϕ Pedestal Pile	L.F.	※ 74.5
Steel H-Pile	L.F.	※ 80
Class A(AE) Concrete	C.Y.	86.6
Reinforcing Steel	Lb.	18,550
Fencing (Including Supports)	L.F.	232

※ Estimated

D. L. DEFLECTION DIAGRAM

U. S. DEPARTMENT OF COMMERCE
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TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

FOUR SPAN CONCRETE OPEN BOX GIRDER BRIDGE
SPANS = 40 - 70 - 70 - 40 = 220 FT.
GENERAL PLAN AND ELEVATION

RECOMMENDED S. B. Lauer, Chief Design Branch	RECOMMENDED L. M. Williams, Director of Engineering	OCTOBER 1964
RECOMMENDED E. Keilman, Chief Bridge Division	APPROVED Thos. H. MacDonald, Administrator	SHEET NO. 111

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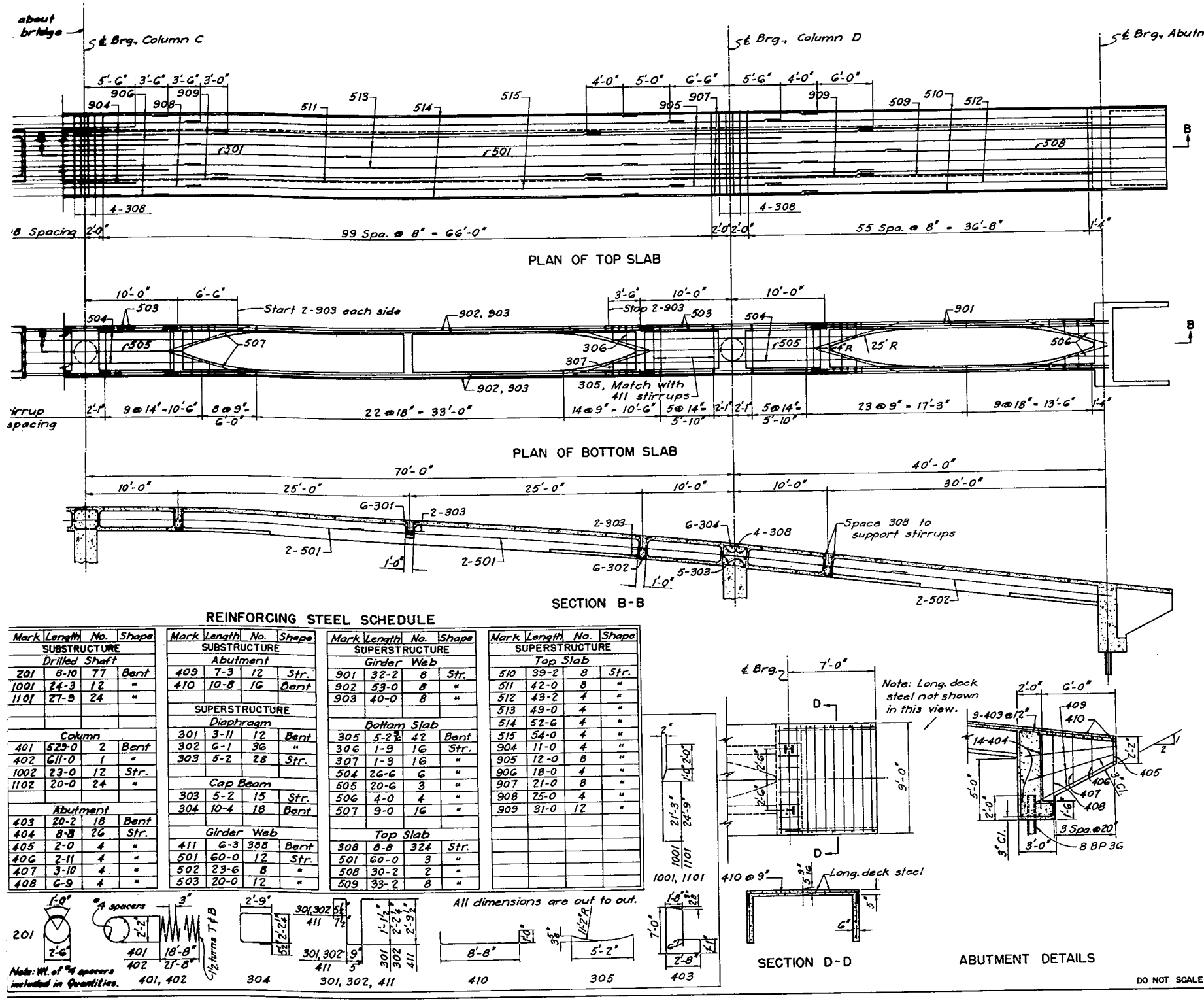

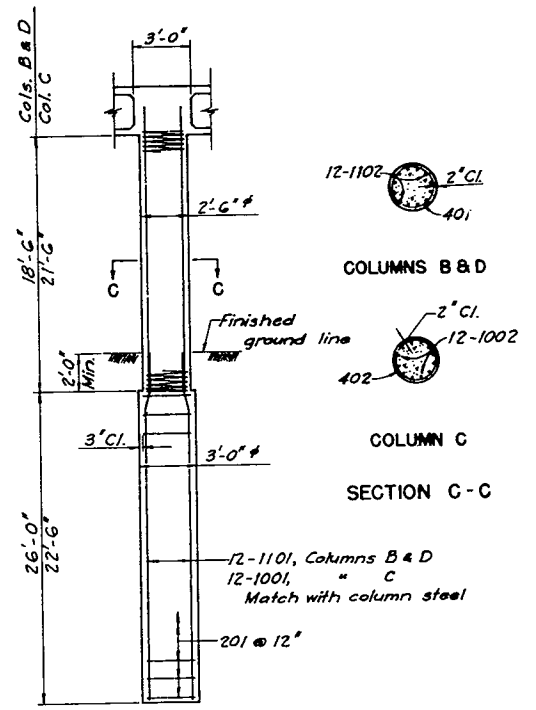

COLUMN DETAILS

REINFORCING STEEL SCHEDULE

Mark	Length	No.	Shape
SUBSTRUCTURE			
Drilled Shaft			
201	8-10	77	Bent
1001	24-3	12	"
1101	27-9	24	"
Column			
401	629-0	2	Bent
402	611-0	1	"
1002	23-0	12	Str.
1102	20-0	24	"
Abutment			
403	20-2	18	Bent
404	8-8	26	Str.
405	2-0	4	"
406	2-11	4	"
407	3-10	4	"
408	6-9	4	"

Mark	Length	No.	Shape
SUBSTRUCTURE			
Abutment			
409	7-3	12	Str.
410	10-8	16	Bent
SUPERSTRUCTURE			
Diaphragm			
301	3-11	12	Bent
302	6-1	36	"
303	5-2	28	Str.
Cap Beam			
303	5-2	15	Str.
304	10-4	18	Bent
Girder Web			
411	6-3	388	Bent
501	60-0	12	Str.
502	23-6	8	"
503	20-0	12	"

Mark	Length	No.	Shape
SUPERSTRUCTURE			
Girder Web			
901	32-2	8	Str.
902	53-0	8	"
903	40-0	8	"
Bottom Slab			
305	5-2½	42	Bent
306	1-9	16	Str.
307	1-3	16	"
504	26-6	6	"
505	20-6	3	"
506	4-0	4	"
507	9-0	16	"
Top Slab			
308	8-8	324	Str.
501	60-0	3	"
508	30-2	2	"
509	33-2	8	"

Mark	Length	No.	Shape
SUPERSTRUCTURE			
Top Slab			
510	39-2	8	Str.
511	42-0	8	"
512	43-2	4	"
513	49-0	4	"
514	52-6	4	"
515	54-0	4	"
904	11-0	4	"
905	12-0	8	"
906	18-0	4	"
907	21-0	8	"
908	25-0	4	"
909	31-0	12	"

SECTION D-D

ABUTMENT DETAILS

DO NOT SCALE

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TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

FOUR SPAN CONCRETE OPEN BOX GIRDER BRIDGE

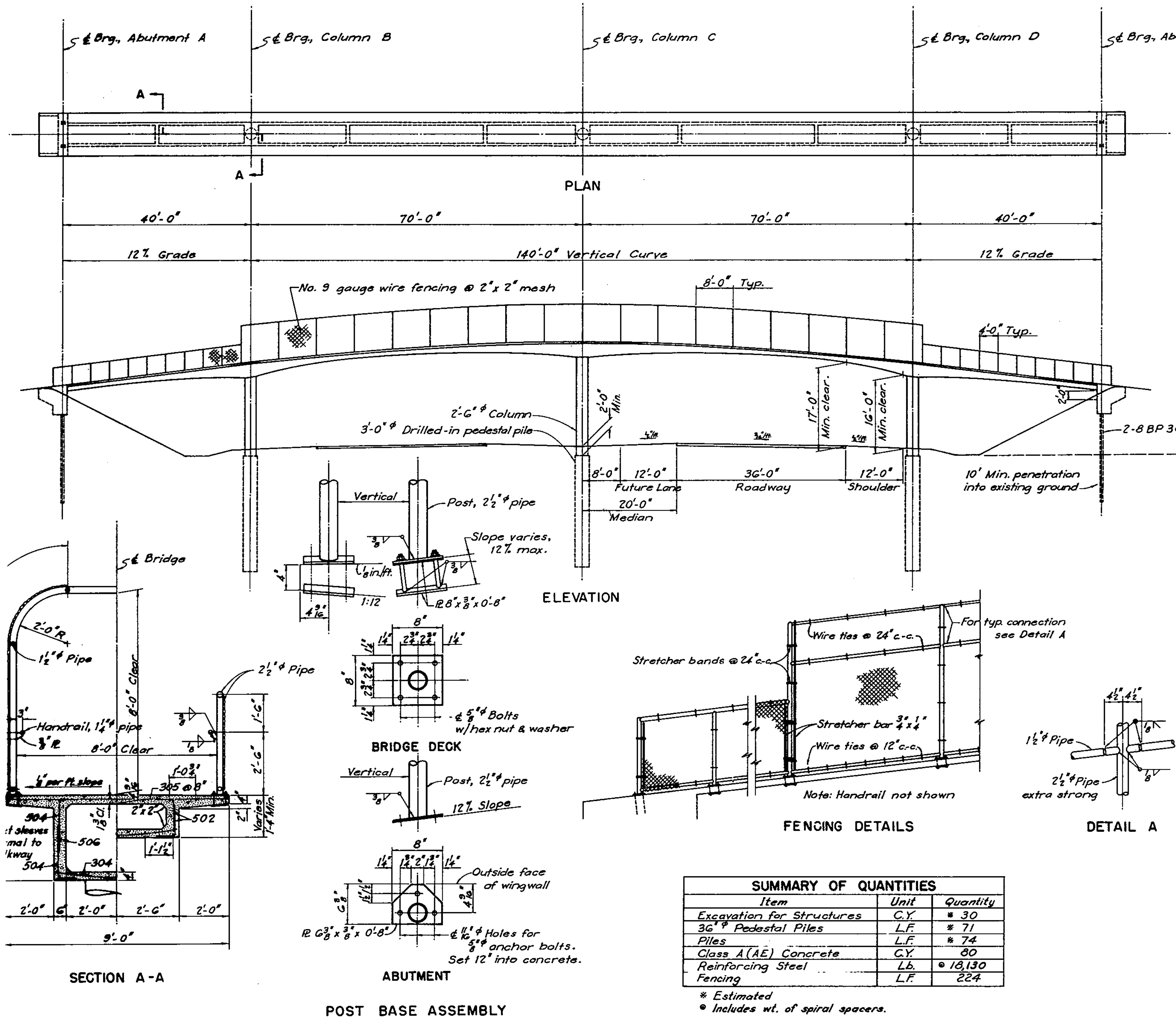
SPANS = 40 - 70 - 70 - 40 = 220 FT.

DETAILS

RECOMMENDED: S. B. Lamm, Chief Design Branch
RECOMMENDED: [illegible], Chief Bridge Division
RECOMMENDED: [illegible], Director of Engineering
APPROVED: [illegible], Administrator

OCTOBER 1964

SHEET NO. 112



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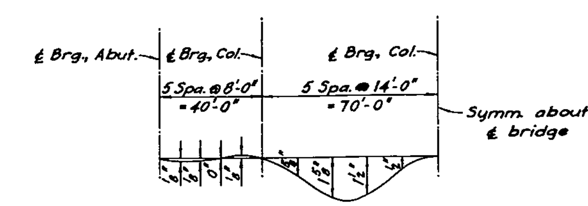

D. L. DEFLECTION DIAGRAM

SUMMARY OF QUANTITIES

Item	Unit	Quantity
Excavation for Structures	C.Y.	* 30
36"⌀ Pedestal Piles	L.F.	* 71
Piles	L.F.	* 74
Class A(AE) Concrete	C.Y.	80
Reinforcing Steel	Lb.	● 18,130
Fencing	L.F.	224

* Estimated
● Includes wt. of spiral spacers.

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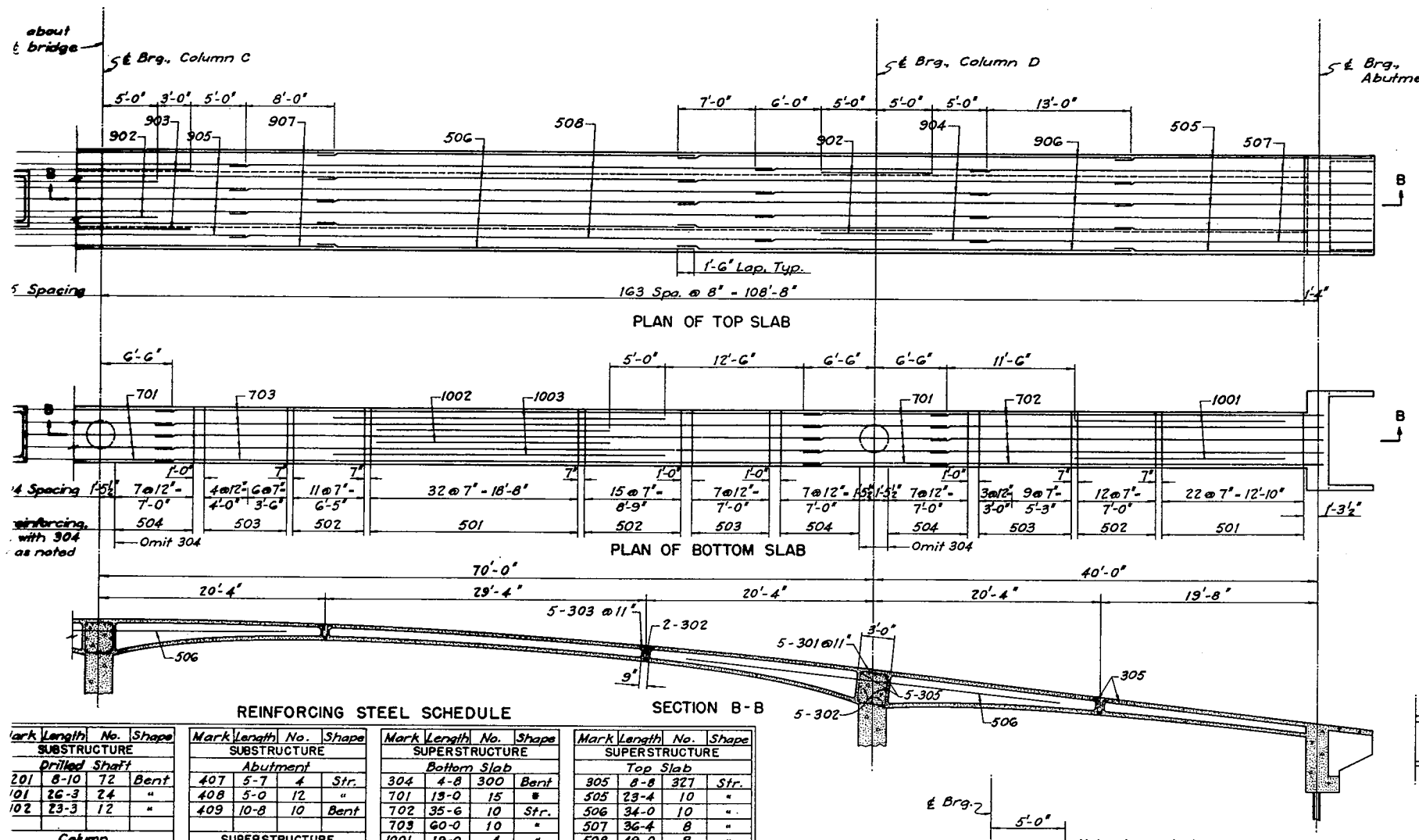
TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

FOUR SPAN CONCRETE BOX GIRDER BRIDGE
SPANS = 40-70-70-40 = 220 FT.
GENERAL PLAN AND ELEVATION

RECOMMENDED S. B. Lauer, Chief Design Branch	RECOMMENDED [signature], Director of Engineering	OCTOBER 1964
RECOMMENDED [signature], Chief Bridge Division	APPROVED [signature], Administrator	SHEET NO. 121



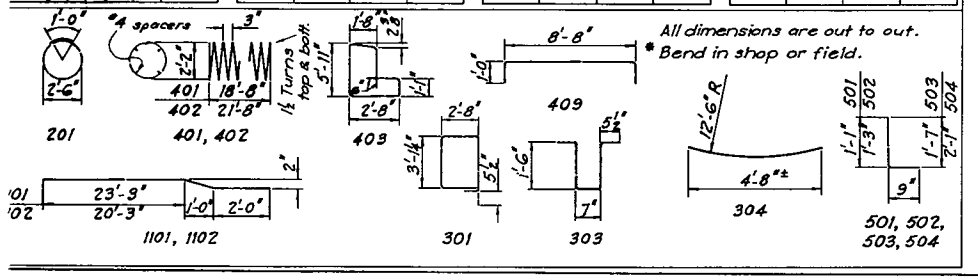
REINFORCING STEEL SCHEDULE

Mark	Length	No.	Shape
SUBSTRUCTURE			
Drilled Shaft			
201	8-10	72	Bent
101	26-3	24	"
102	23-3	12	"
Column			
101	529-0	2	Bent
102	611-0	1	"
01	[illegible]	12	Str.
03	[illegible]	24	"
Abutment			
03	18-0	18	Bent
04	8-8	26	Str.
05	1-10	4	"
06	2-9	4	"

Mark	Length	No.	Shape
SUBSTRUCTURE			
Abutment			
407	5-7	4	Str.
408	5-0	12	"
409	10-8	10	Bent
SUPERSTRUCTURE			
Cap Beam			
301	12-0	15	Bent
302	4-8	15	Str.
Diaphragm			
302	4-8	12	Str.
303	4-6	30	Bent

Mark	Length	No.	Shape
SUPERSTRUCTURE			
Bottom Slab			
304	4-8	300	Bent
701	13-0	15	"
702	35-6	10	Str.
703	60-0	10	"
1001	19-0	4	"
1002	21-0	4	"
1003	30-0	4	"
Girder Web			
501	1-10	448	Bent
502	2-0	328	"
503	2-4	256	"
504	2-10	192	"
506	34-0	6	Str.

Mark	Length	No.	Shape
SUPERSTRUCTURE			
Top Slab			
305	8-8	327	Str.
505	23-4	10	"
506	34-0	10	"
507	36-4	8	"
508	49-0	8	"
902	10-0	6	"
903	16-0	2	"
904	21-0	8	"
905	26-0	4	"
906	41-0	10	"
907	42-0	5	"



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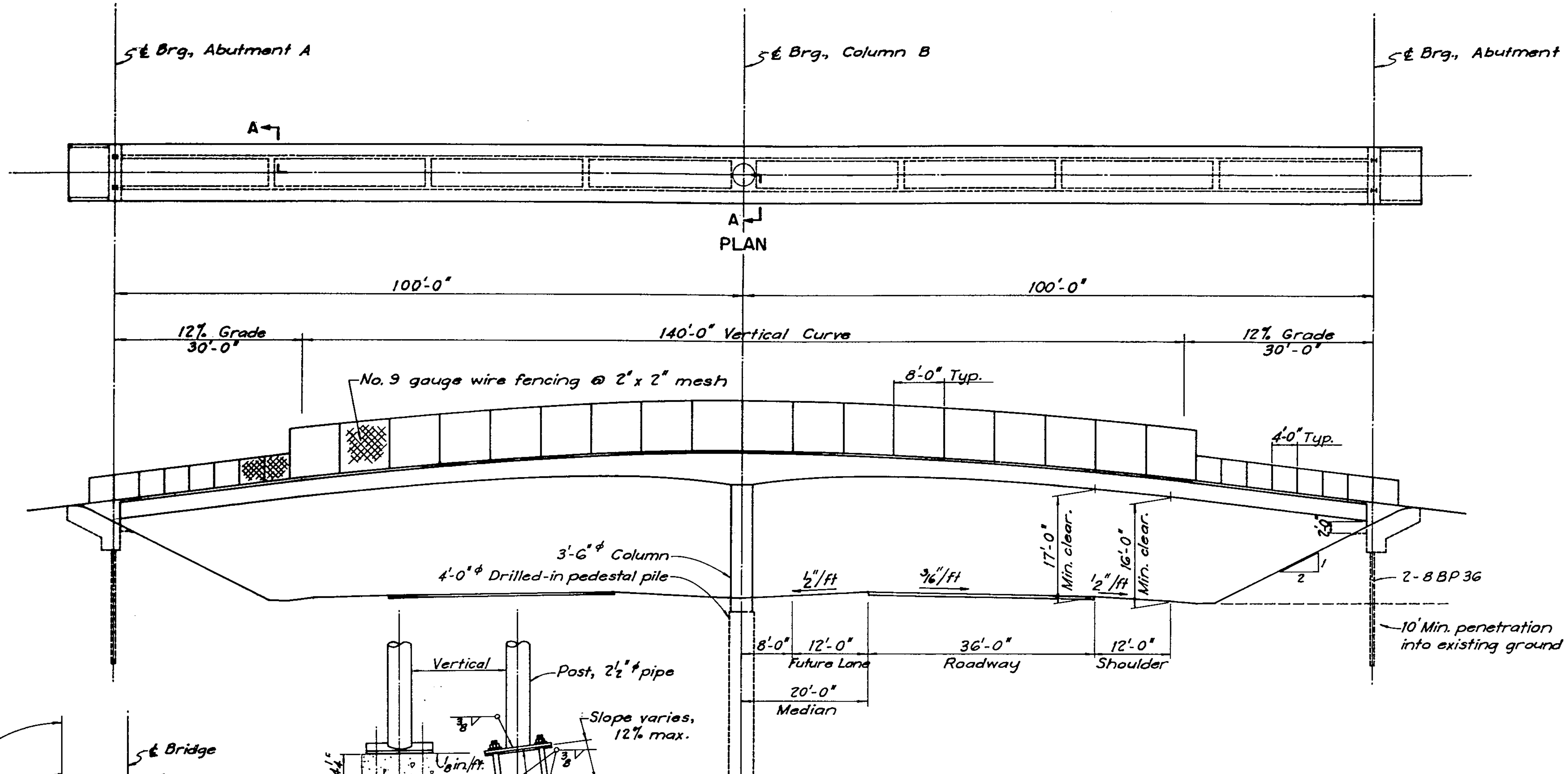
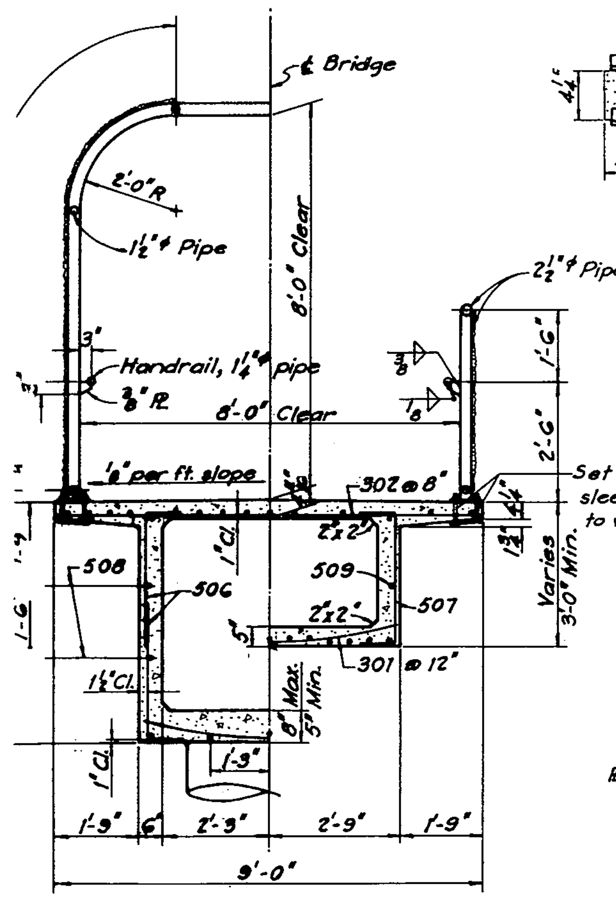
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

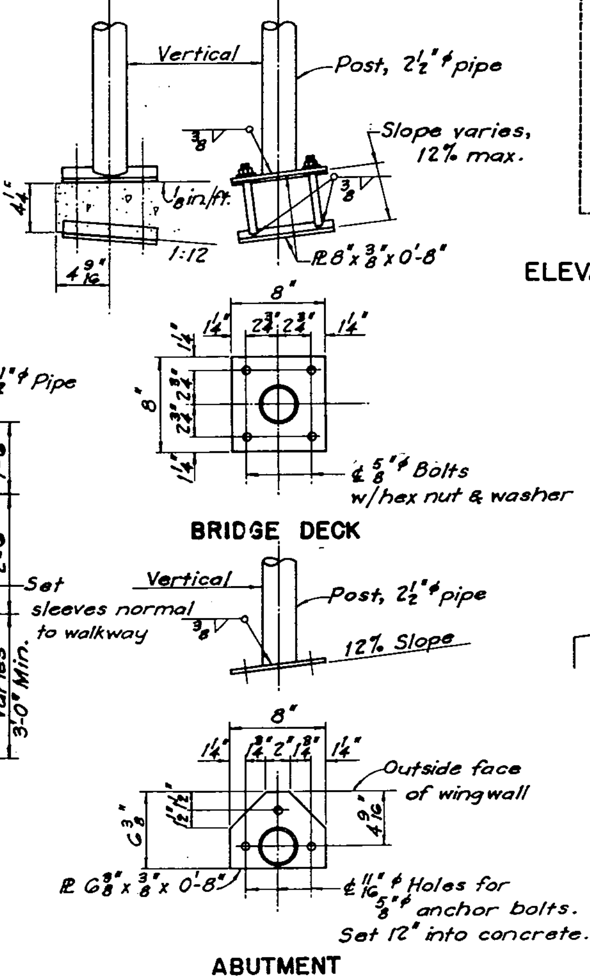
FOUR SPAN CONCRETE BOX GIRDER BRIDGE
SPANS = 40 - 70 - 70 - 40 = 220 FT.
DETAILS

RECOMMENDED S. B. Lauren, Chief Design Branch	RECOMMENDED C. M. Williams, Director of Engineering	OCTOBER 1964
RECOMMENDED [illegible], Chief Bridge Division	APPROVED [illegible], Administrator	SHEET NO. 122

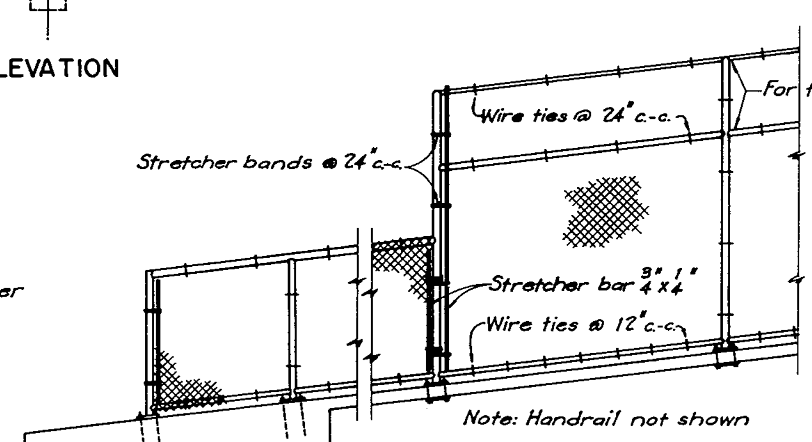




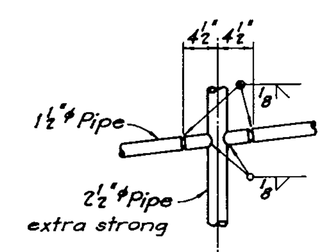
SECTION A-A



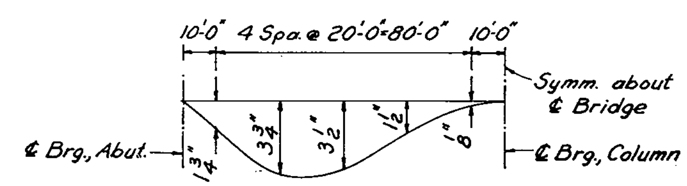
POST BASE ASSEMBLY



FENCING DETAILS



DETAIL A



D.L. DEFLECTION DIAGRAM

GENERAL NOTES

Design Criteria: Design Criteria for Pedestrian Bridges over Interstate or Comparable Highways, Appendix A.

Design Specifications: AASHO Standard Specifications for Highway Bridges, 1961, with tentative revisions to 1963 except as modified or amplified by the Design Criteria.

Concrete: Concrete shall be Class A(AE) with a minimum 28 day $f'_c = 4{,}000$ psi. The air entraining agent shall meet with the approval of the engineer.

Reinforcing Steel: Reinforcing steel shall be deformed bars of intermediate, hard, or rail grade conforming to ASTM Specification A15 or A16.
Bar sizes are designated on the plans by numbers. The first digit in the three digit marks and the first two digits in the four digit marks indicate the size of the bar.
Dimensions shown on the plans from the reinforcing steel to the outside edge of concrete are all clear dimensions.
All bending dimensions are from out to out of the bar.

Pedestal Piles: The drilled hole shall be cleaned of all loose material and the pedestal pile poured in compact coarse and fine sand, medium stiff clay, or better.

Lateral Soil Pressure: Design has been based on an allowable lateral soil pressure of 300 psf per foot of depth.

Soil Pressure: The required allowable soil pressure for the stub abutments as detailed but without piles is one ton per square foot.

Galvanized Fence and Supports: Chain link fence fabric shall be No. 9 gauge wire, conforming to ASTM A 392, woven in a 2 inch mesh and galvanized after weaving with a Class II zinc coating. Galvanized steel pipe rails and fence supports shall conform to ASTM A53 Grade B, Type S or E. ASTM A36 steel shall be used for all hardware, and after fabrication shall be galvanized in accordance with ASTM A123 and A153. All welding and erection damage to the galvanizing shall be repaired with material that conforms to Military Specification MIL-P-21035 (SHIPS).

SUMMARY OF QUANTITIES		
Item	Unit	Quantity
Excavation for Structures	C.Y.	※ 40
48"ϕ Pedestal Pile	L.F.	※ 26
Piles	L.F.	※ 80
Class A(AE) Concrete	C.Y.	91
Reinforcing Steel	Lb.	⊙ 21,810
Fencing (Including Supports)	L.F.	208

※ Estimated
⊙ Includes wt. of spiral spacers.

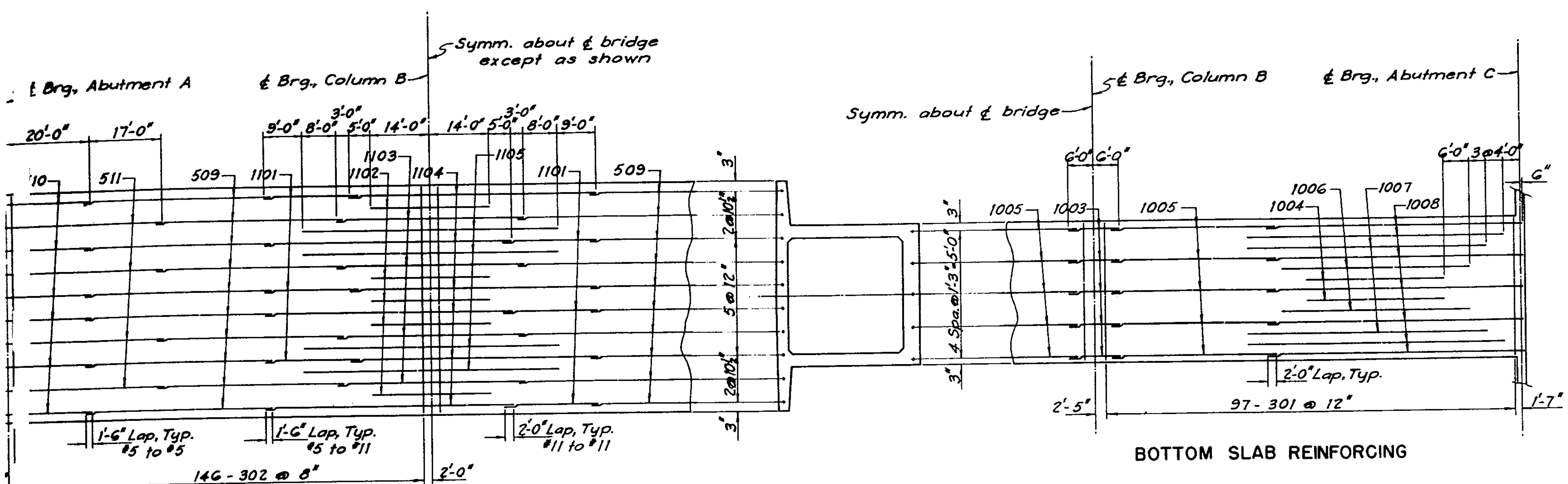
U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

TWO SPAN CONCRETE BOX GIRDER BRIDGE
SPANS = 100 - 100 = 200 FT.
GENERAL PLAN AND ELEVATION

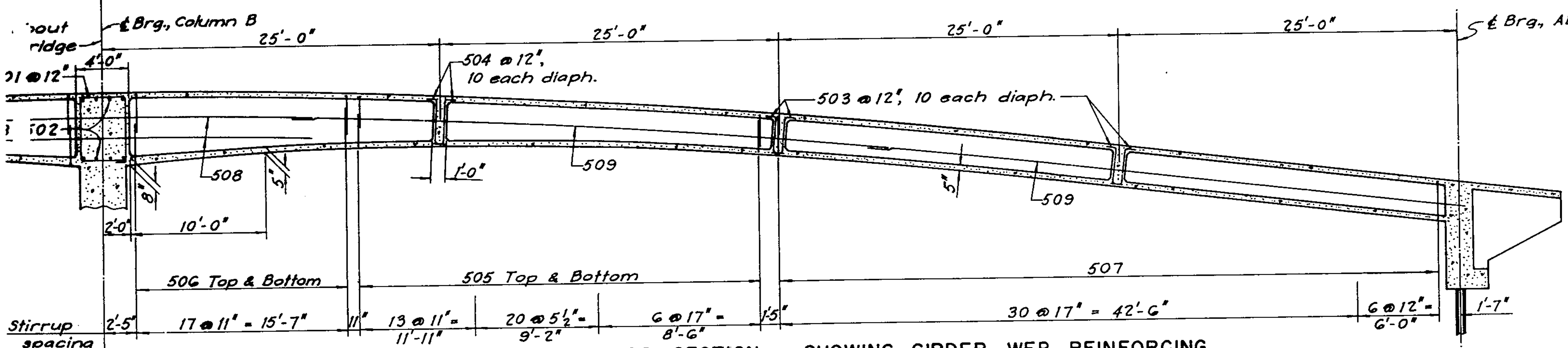
RECOMMENDED S. B. Lau... Chief Design Branch	RECOMMENDED [signature] Director of Engineering	OCTOBER 1964
RECOMMENDED [signature] Chief Bridge Division	APPROVED [signature] Administrator	SHEET NO. 131

DO NOT SCALE

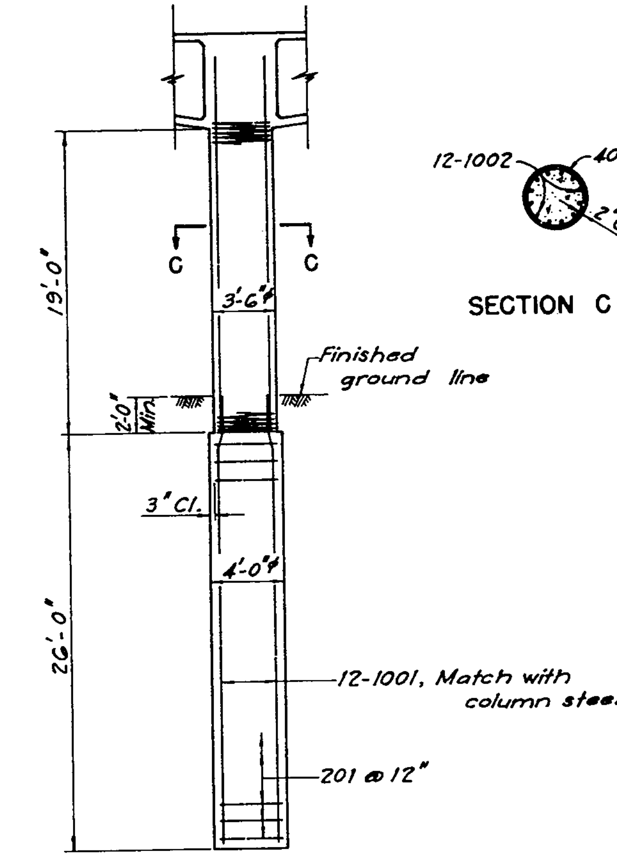


TOP SLAB REINFORCING

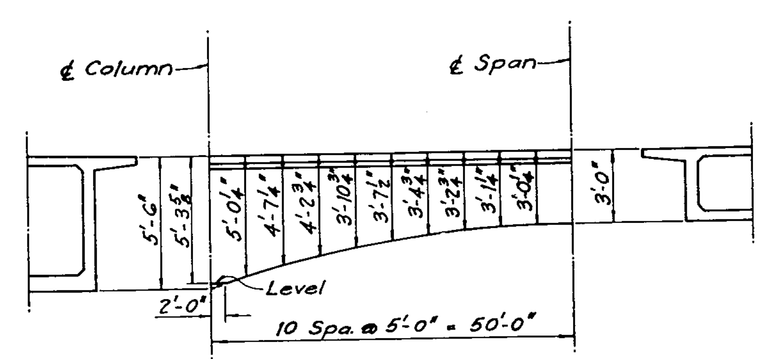
BOTTOM SLAB REINFORCING



HALF LONGITUDINAL CROSS SECTION - SHOWING GIRDER WEB REINFORCING



COLUMN DETAILS



GIRDER GEOMETRICS

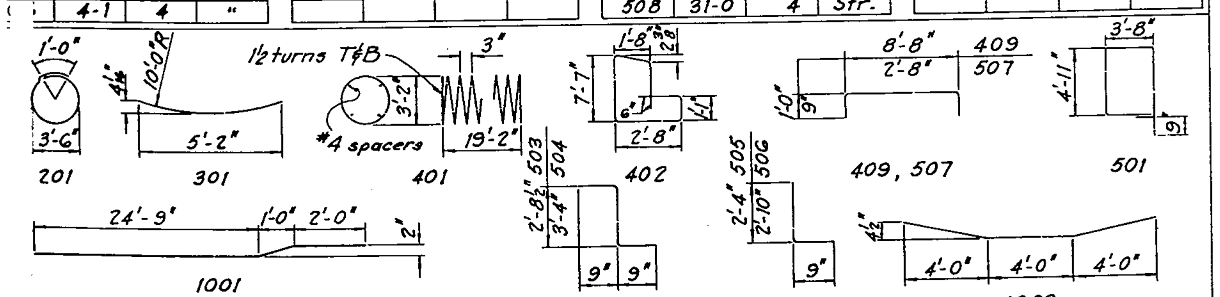
REINFORCING STEEL SCHEDULE

Mark	Length	No.	Shape
SUBSTRUCTURE			
Drilled Shaft			
[illegible]	12-0	26	Bent
01	27-9	12	"
Column			
[illegible]	794-0	1	Bent
2	23-0	12	Str.
Abutment			
[illegible]	21-4	18	Bent
3	8-8	28	Str.
04	1-11	4	"
05	3-0	4	"
[illegible]	4-1	4	"

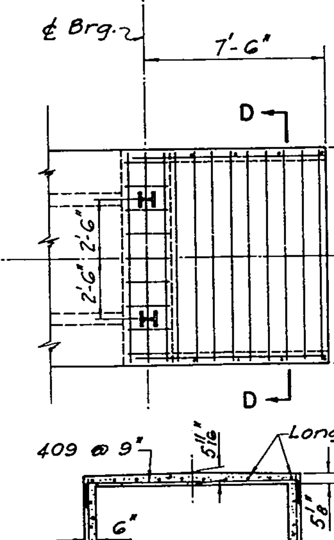
Mark	Length	No.	Shape
SUBSTRUCTURE			
Abutment			
407	7-4	4	Str.
408	8-0	12	"
409	10-8	18	Bent
SUPERSTRUCTURE			
Cap Beam			
501	17-11	5	Bent
502	5-2	8	Str.
Diaphragm			
503	4-2½	40	Bent
504	4-10	20	"

Mark	Length	No.	Shape
SUPERSTRUCTURE			
Bottom Slab			
301	5-2 13/16	194	Bent
1003	12-0	5	"
1004	32-0	4	Str.
1005	38-6	10	"
1006	44-0	4	"
1007	56-0	4	"
1008	60-0	14	"
Girder Web			
505	3-1	320	Bent
506	3-7	144	"
507	4-2	148	"
508	31-0	4	Str.

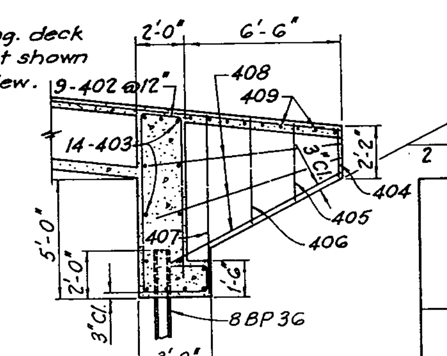
Mark	Length	No.	Shape
SUPERSTRUCTURE			
Girder Web			
509	44-0	8	Str.
Top Slab			
302	8-8	292	Str.
509	44-0	20	"
510	27-6	12	"
511	44-8	8	"
1101	22-0	6	"
1102	28-0	5	"
1103	44-0	4	"
1104	58-0	6	"
1105	60-0	4	"



All dimensions are out to out



SECTION D-D



ABUTMENT DETAILS

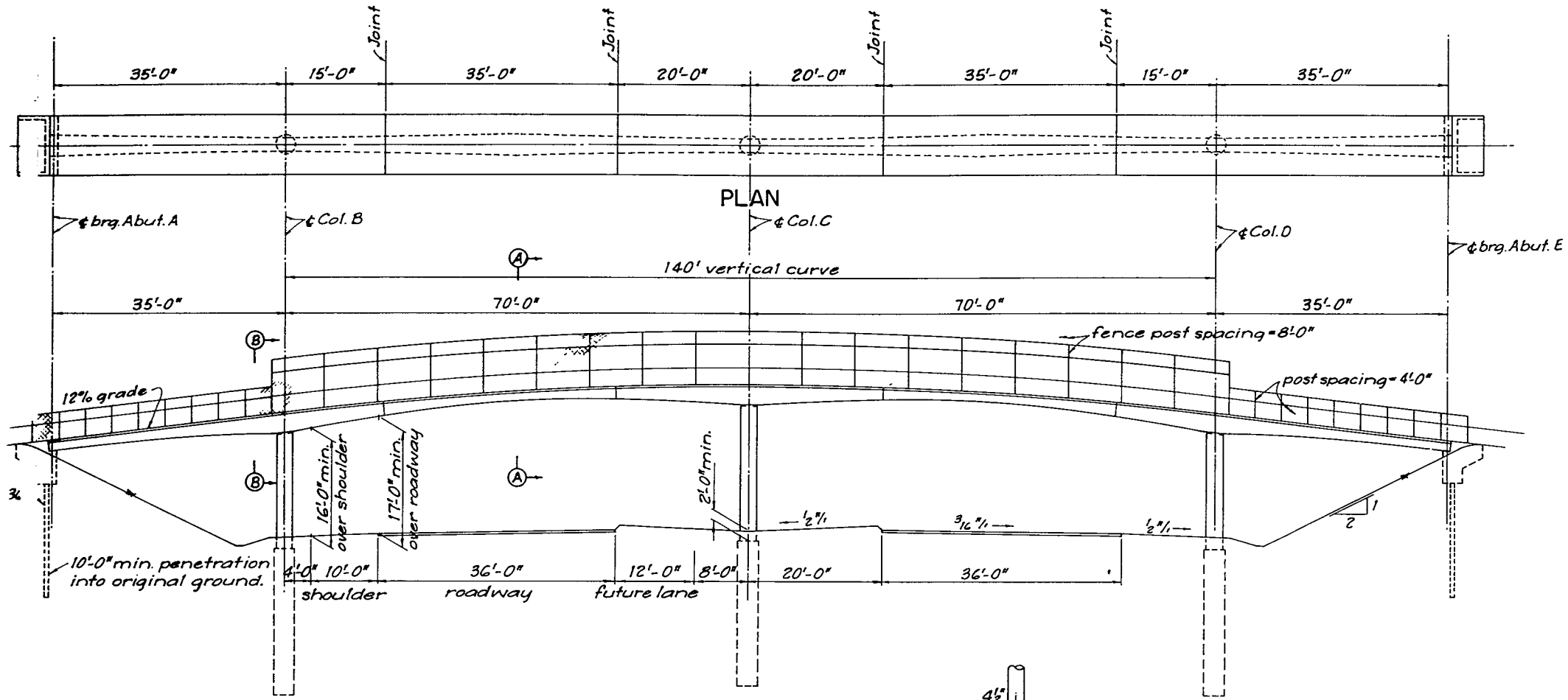
DO NOT SCALE

U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

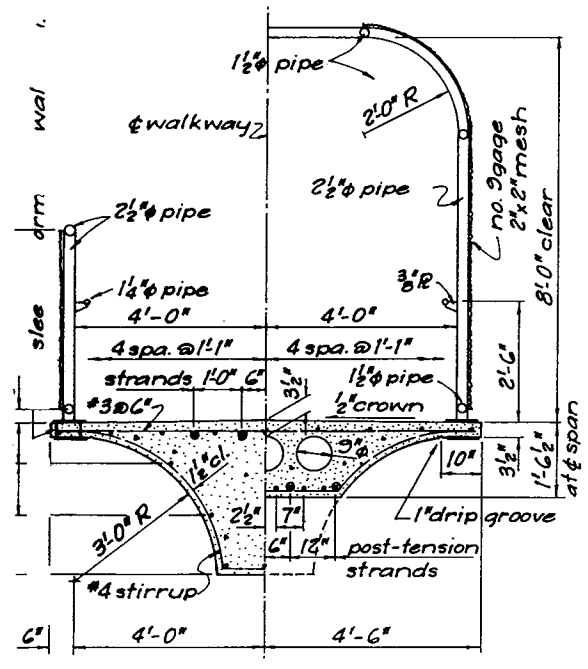
TWO SPAN CONCRETE BOX GIRDER BRIDGE
SPANS = 100 - 100 = 200 FT.
DETAILS

RECOMMENDED S. B. Lauer, Chief Design Branch	RECOMMENDED [signature], Director of Engineering	OCTOBER 1964
RECOMMENDED [signature], Chief Bridge Division	APPROVED [signature], Administrator	SHEET NO. 132



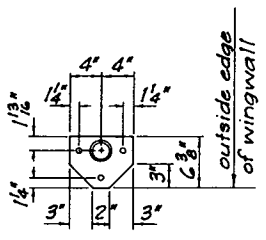
PLAN

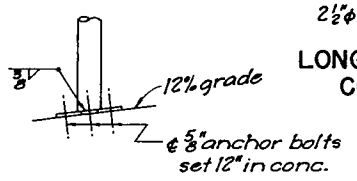
ELEVATION



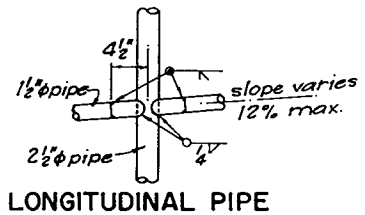
HALF SECTION A-A

SECTION B-B

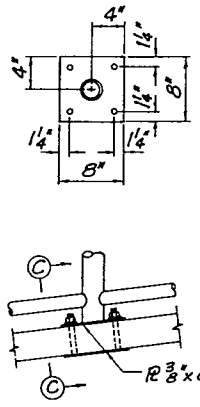


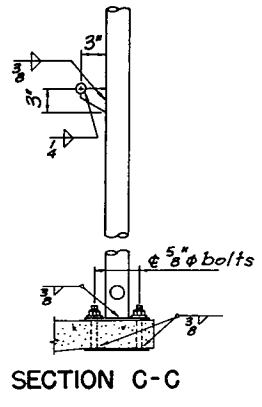


ABUTMENT POST CONNECTION



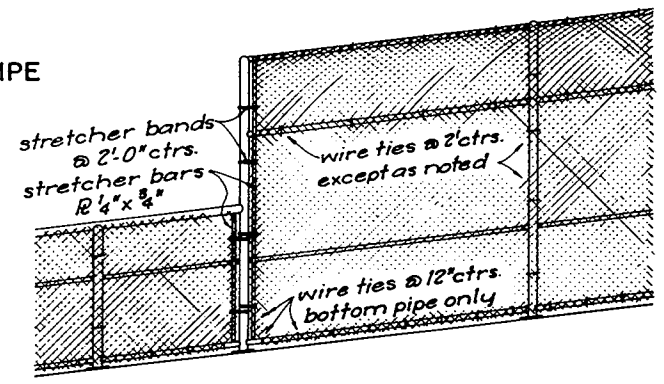
LONGITUDINAL PIPE CONNECTION





SECTION C-C

POST CONNECTION



FENCE DETAILS

GENERAL NOTES

DESIGN CRITERIA: "Design Criteria for Pedestrian Bridges over Interstate or Comparable Highways", Appendix A.

DESIGN SPECIFICATIONS: AASHO Standard Specifications for Highway Bridges, 1961, with tentative revisions to 1963 except as modified or amplified by the design criteria.

CONCRETE: Class A (AE) concrete shall be used; the air entraining agent shall meet with the approval of the engineer. Substructure concrete shall have a minimum 28 day compression strength, f'_c = 4,000 psi. Superstructure concrete shall have a minimum 28 day compression strength, f'_c = 4,000 psi, with a minimum strength at the time of post-tensioning of 3,200 psi. The design mix shall be as approved by the engineer.

EPOXY JOINT FILLER: The epoxy grout mix to be placed between abutting precast members shall meet with the approval of the engineer.

REINFORCING STEEL: Reinforcing steel shall be deformed bars of intermediate, hard, or rail grades conforming with ASTM A-15 or A-16. Dimensions shown on the plans from the reinforcing steel to the outside edge of the concrete are all clear dimensions. All bending dimensions are from "out to out" of bars. Bar sizes are designated on the plans by numbers. The first digit in the three digit marks, and the first two digits in the four digit marks indicate the size of the bar.

POST-TENSIONING STEEL: The proposed type of tendons to be used for post-tensioning, all necessary details including those for end anchorages, methods to be employed, and proceedures to be followed, shall be as appoved by the engineer. All tendons shall be placed so that their center of gravity will be at the position shown on the plans. After the segmental members are in position, post-tensioning shall be performed by simultaneous jacking at each end. The required relaxed post-tension force shall be obtained by applying an initial tensile force of sufficient magnitude to allow for all subsequent losses, including those for elastic deformation, shrinkage, creep, friction and efficiency of end anchorage. After securing the end anchorages, all tendons shall be pressure grouted in their conduits.

GALVANIZED FENCE AND SUPPORTS: Chain link fence fabric conforming to ASTM A-392 shall be no. 9 gage wire woven into a 2" mesh and galvanized after weaving with a Class II zinc coating. Galvanized steel pipe rails and fence supports shall conform to ASTM A-53, Grade B, Type E or S. ASTM A-36 steel shall be used for all structural steel and hardware, and after fabrication, shall be galvanized in accordance with ASTM A123 and A-153. All welding and erection damage to the galvanizing shall be repaired with material that conforms to Military Specification MIL-P-21035 (SHIPS).

PEDESTAL PILES: The drilled hole shall be cleaned of all loose material and the pedestal pile poured in compact coarse and fine sand, medium stiff clay, or better.

SOIL PRESSURE: The minimum soil pressure capacity for the use of stub abutments without piles is 1 ton per sq. ft.

LATERAL SOIL PRESSURE: The design has been based on an allowable lateral soil pressure of 300 psf per foot of depth.

SUMMARY OF QUANTITIES

ITEM	Unit	Superstr.	Substr.	Total
Excavation for Structure	C.Y.		※ 30	30
Class A (AE) Concrete	C.Y.	75	21	96
Reinforcing Steel	LBS.	7100	3690	10,790
Prestressing Steel	L.S.			Lump Sum
Pedestal Piles	L.F.		※ 66	66
Steel Piles	L.F.		※ 76	76
Fencing	L.F.	216		216

※ Estimated

DO NOT SCALE

U. S DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

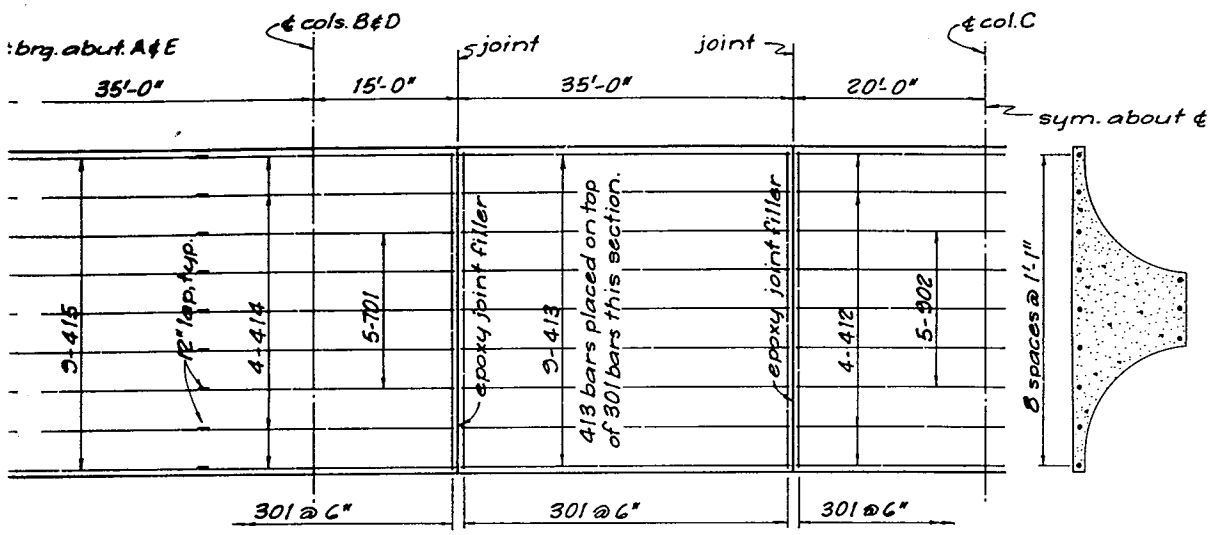
TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

FOUR SPAN PRECAST PRESTRESSED CONCRETE BRIDGE
SPANS = 35 - 70 - 70 - 35 = 210 FT.
GENERAL PLAN AND ELEVATION

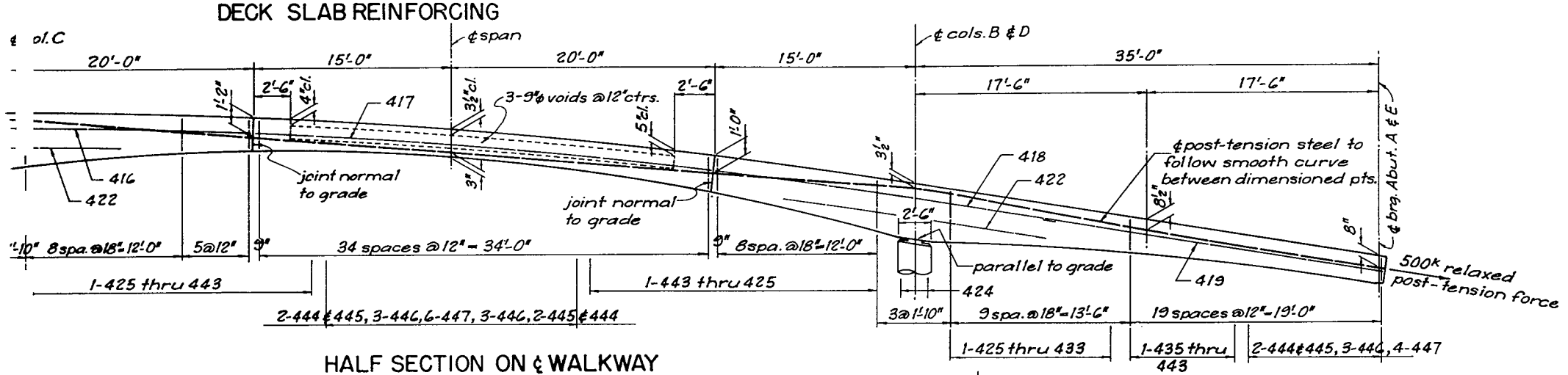
RECOMMENDED S. B. Lauer, Chief Design Branch
RECOMMENDED [illegible], Chief Bridge Division
RECOMMENDED [illegible], Director of Engineering
APPROVED [illegible], Administrator

OCTOBER 1964

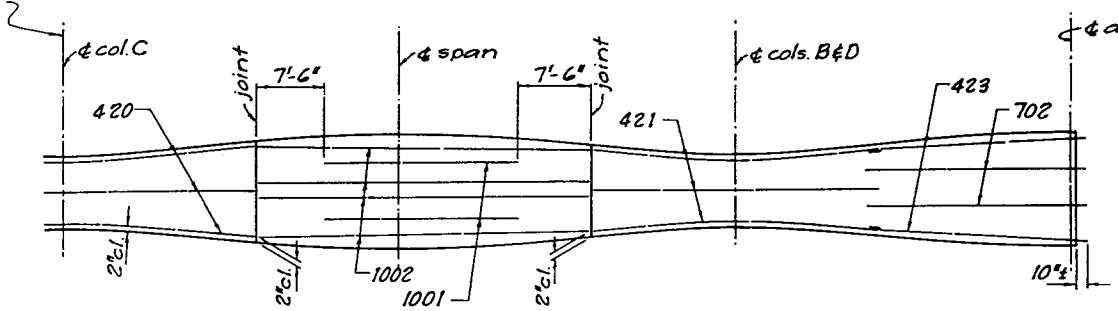
SHEET NO. 201



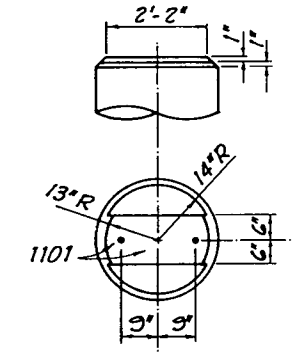
DECK SLAB REINFORCING



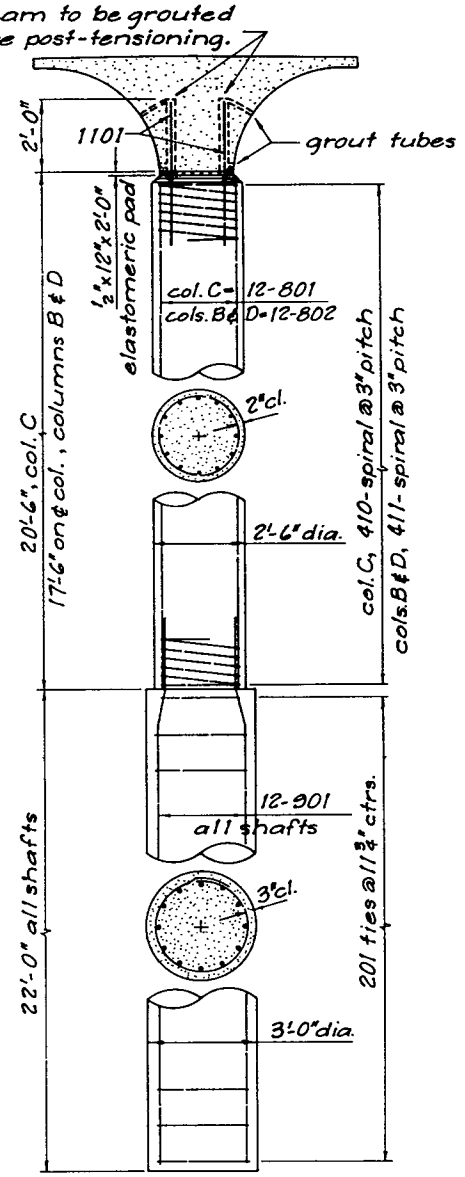
HALF SECTION ON ℄ WALKWAY



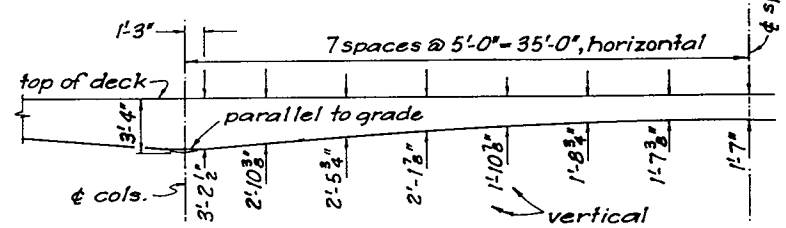
BOTTOM GIRDER REINFORCING



DETAIL AT TOP OF COLUMN



COLUMN DETAILS



GIRDER GEOMETRY ON ℄ WALKWAY

Bearing Notes:
1. reinforcing shall not have deformations.
2. turns of the spiral at the top and bottom spiral unit and at all splices. Four #4 bars deformations shall be provided as

spacers for each spiral unit. These bars shall be equally spaced around the periphery of the coil. The weight of spacers shall be added to the weight of the spirals and paid for as reinforcing.

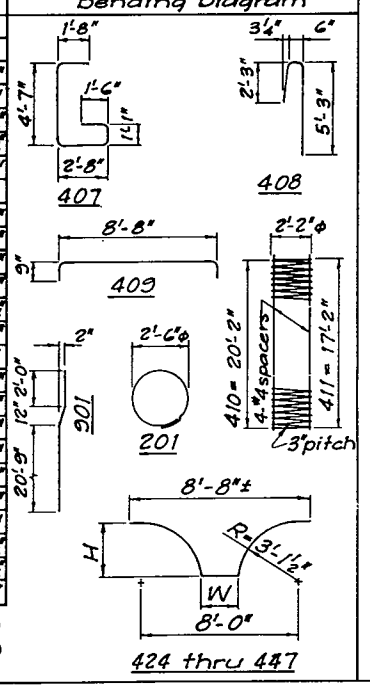
REINFORCING STEEL SCHEDULE

Mk.	No.	Length	Shape
STRUCTURE			
ment			
	8'-8"	Str.	
	5'-9"		
8	4'-2"		
4	9'-0"		
	2'-5"		
	1'-10"	Str.	
v	11'-6"	Bent	
20	8'-0"	Bent	
7	10'-2"	Bent	
olumns			
6	4'-0"	Str.	
2	20'-9"	Str.	
	17'-3"	Str.	
	569'-0"	Spiral	
	488'-0"	Spiral	
shafts			
	23'-9"	Bent	
	9'-0"	Bent	

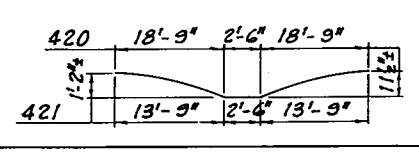
Mark	No.	Length	Shape
SUPERSTRUCTURE			
Deck Slab			
412	4	39'-8"	Str.
413	18	34'-8"	
414	8	27'-0"	
415	18	25'-10"	
301	420	8'-8"	
701	10	27'-0"	
302	5	39'-8"	Str.
Girder *			
416	2	39'-8"	Str.
417	4	34'-8"	
418	4	30'-0"	
419	4	21'-2"	Str.
420	3	40'-0"	Bent
421	6	30'-0"	Bent
422	6	10'-0"	Str.
423	4	23'-0"	
702	4	23'-0"	
1001	4	20'-0"	
1002	8	34'-9"	Str.

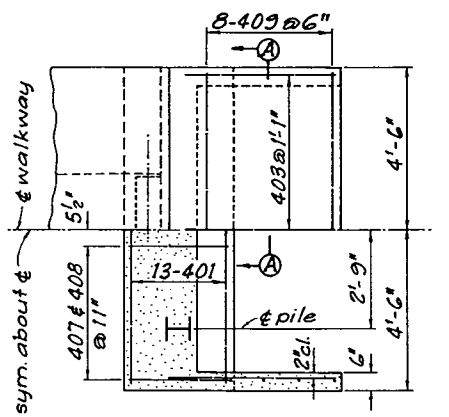
Mark	No.	Length	H	W
Stirrups				
424	6	11'-11"	2'-11"	1'-9"
425	6	11'-7"	2'-9"	1'-9½"
426	6	11'-4"	2'-7¼"	1'-10"
427	6	11'-2"	2'-6"	1'-10⅝"
428	6	10'-11"	2'-4⅝"	1'-11¼"
429	6	10'-9"	2'-3"	2'-0"
430	6	10'-7"	2'-1⅛"	2'-1"
431	6	10'-5"	2'-0⅛"	2'-1⅜"
432	6	10'-4"	1'-11"	2'-2¾"
433	6	10'-2"	1'-10"	2'-3½"
434	4	10'-2"	1'-9½"	2'-4¼"
435	6	10'-1"	1'-9"	2'-4¾"
436	6	10'-0"	1'-8¾"	2'-5¼"
437	6	9'-11"	1'-7⅝"	2'-6"
438	6	9'-11"	1'-7"	2'-6⅝"
439	6	9'-10"	1'-6⅜"	2'-7¼"
440	6	9'-10"	1'-6¼"	2'-7⅝"
441	6	9'-9"	1'-5⅝"	2'-8¼"
442	6	9'-9"	1'-5⅛"	2'-8⅝"
443	6	9'-8"	1'-5"	2'-9¼"
444	12	9'-8"	1'-4⅞"	2'-10"
445	12	9'-7"	1'-4"	2'-10½"
446	18	9'-6"	1'-3⅛"	2'-11⅛"
447	20	9'-5"	1'-3"	3'-0"

Bending Diagram

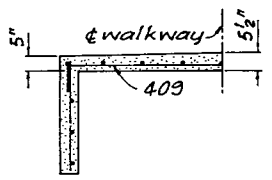


*Reinforcing in the bottom of girders may be bent in the field.

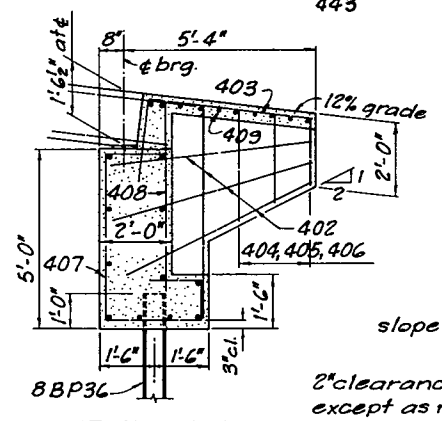




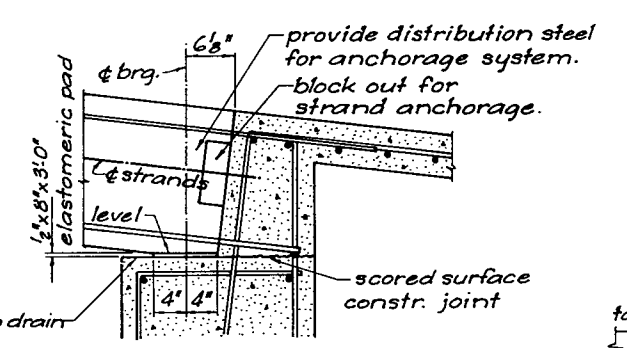
ABUTMENT PLAN AND SECTION



SECTION A-A



ABUTMENT ELEVATION



ABUTMENT SEAT DETAIL

2" clearance to bars except as noted.

DO NOT SCALE

U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

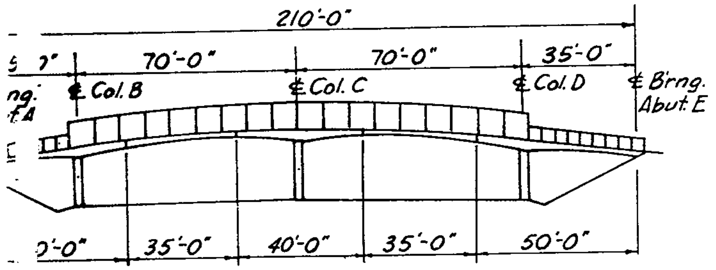
FOUR SPAN PRECAST PRESTRESSED CONCRETE BRIDGE

SPANS = 35 - 70 - 70 - 35 = 210 FT.
DETAILS

RECOMMENDED S. B. Lauer, Chief Design Branch
RECOMMENDED _____, Chief Bridge Division
RECOMMENDED _____, Director of Engineering
APPROVED _____, Administrator

OCTOBER 1964

SHEET NO. 202



DATA:
-70'-35' Spans = 210'
way
ost-tensioned beams

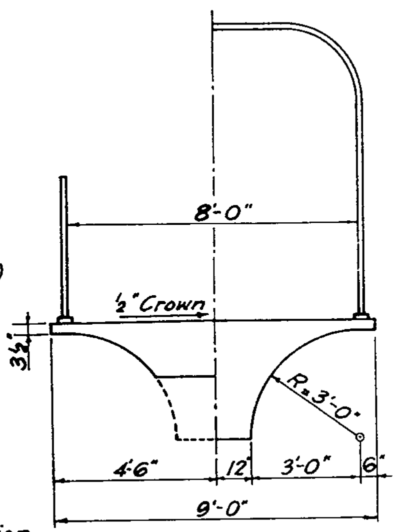
cations:
ith revisions to 1963 as
by Design Criteria.

Allowable Stresses:
$f_c = 20{,}000$ psi
$f'_c = 4{,}000$ psi
$f_c = 1{,}600$ psi
ssing steel f'_s - AASHO-Art. 2.4.33(J)
osses * $f_{se} = 0.7f'_s - 25{,}000$ psi

ad:
30 p.l.f.
50 p.l.f.

d:
sf distributed over 8'
oO p.l.f.

-luding losses due to friction



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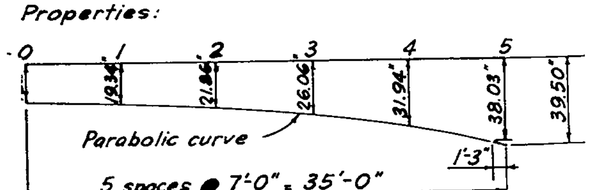


½" crown not included in beam depth.

-tion 0 (d = 18.50")

	A	y	Ay	Ay² + I_o
	378	16.75	6,340	106,500
	1440	7.50	10,800	108,000
	-614	6.15	-3,780	-32,800
	1204	11.10	13,360	181,700

$-1204(11.10)^2 = -148{,}300$

$I = 33{,}400$

-11.10 = 7.40"; $k_1 = I/c_tA = 3.7"$

$k_2 = I/c_bA = 2.5"$

"voids at c.g. concrete
A = 1013 $I = 32{,}400$
$k_1 = 4.3"$
$k_2 = 2.9"$

-tion 5 (d = 39.50")*

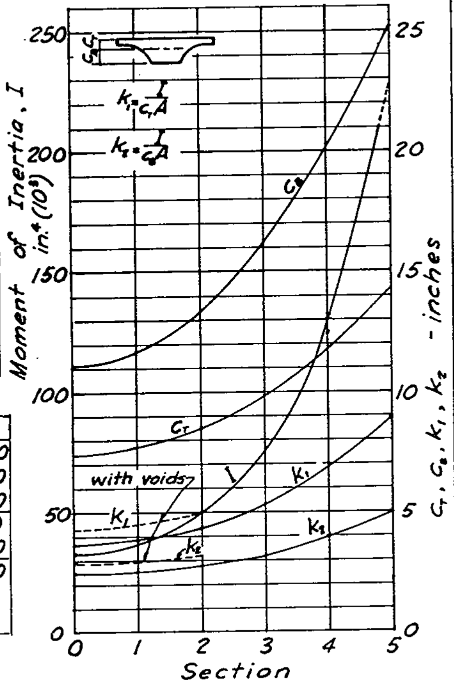
	A	y	Ay	Ay² + I_o
	378	37.75	14,270	538,900
	3456	18.0	62,210	1,493,000
	-2036	15.28	-31,110	-659,800
	1798	25.23	45,370	1,372,100

$-1798(25.23)^2 = -1{,}144{,}600$

$I = 227{,}500$

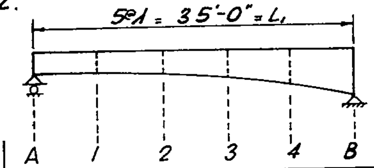
27" $k_1 = 8.9"$
23" $K_2 = 5.0"$

ordance with AASHO
(10)



Beam Properties:
Reference: Elastic Properties of Beams by Newmark's Numerical Procedure, from ASCE Transactions, 1943, Paper No. 2202.

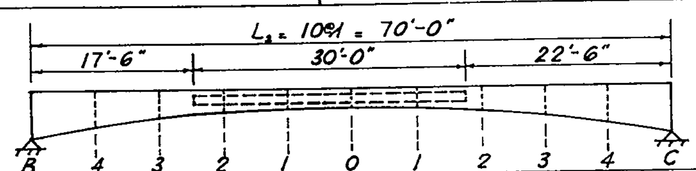
$I_o = I_{min.} = 32.4$
$\lambda = L_1/5$



	Common Factors	A	1	2	3	4	B
Moment of Inertia, I x10³		33.4	37.2	49.8	76.2	131.0	227.5
I' = I/I_o		1.03	1.15	1.54	2.35	4.04	7.02
Moment, M		0	1	2	3	4	5
Angle Change (M/EI')	1/EI_o	0	-0.87	-1.30	-1.28	-0.99	-0.71
Average Slope	λ/EI_o		2.19	1.32	0.02	-1.26	-2.25
Deflection, Δ	λ²/EI_o	0	2.19	3.51	3.53	2.27	0
Angle Change at End	λ/EI_o				$\frac{1}{6}[2(0.71)+0.99]$ =		-0.40
End Slope, θ_BA	λ/EI_o						-2.65

Stiffness, k (M/θ)			$K_{BA} = 5EI_o/2.65\lambda = 9.43EI_o/L_1$
FEM_BA for infl. line (Δλ/θ_BA)	P·L	0	.165 | .265 | .266 | .171 | 0
FEM_BA for uniform load			ΣFEM/5 = .173 $w_uL_1^2$
Haunch D.L. Ratio ($w_h/w_e - w_u$)		0	.051 | .182 | .418 | .687 | 1.00
Haunch FEM_BA	w_hL	0	.008 | .048 | .111 | .118 | 0
FEM_BA for haunch load			ΣFEM/5 = .057 $w_hL_1^2$

$I_o = 32.4$
$\lambda = L_2/10$



	B	4	3	2	1	0	1	2	3	4	C
assumed I	227.5	131.0	76.2	48.8	36.2	32.4	36.2	48.8	76.2	131.0	227.5
I' = I/I_o	7.02	4.04	2.35	1.51	1.12	1.00	1.12	1.51	2.35	4.04	7.02
M	0	1	2	3	4	5	6	7	8	9	10
M/EI'	0	-0.25	-0.85	-1.99	-3.57	-5.00	-5.36	-4.64	-3.40	-2.23	-1.43
Ave. Slope		11.38	11.13	10.28	8.29	4.72	-0.28	-5.64	-10.28	-13.68	-15.91
Defl. Δ	0	11.38	22.51	32.79	41.08	45.80	45.52	39.88	29.60	15.92	0
Angle Ch.	0.04 = ⅙ x 0.25								$-\frac{1}{6}[2(1.43)+2.23]$ =		-0.85
End Slope	11.42										-16.76

Carry-Over, c (B to C) = (C to B) = 11.42/16.76 = 0.681

Stiffness, $K_{BC} = K_{CB} = M_{BC}/(\theta_{CB} - c\theta_{BC}) = 10EI_o/[16.76-(0.681 \times 11.42)]\lambda = 11.14EI_o/L_2$

FEM at B or C for influence line = $K_{BC}(\Delta_{CB} - c_{BC}\Delta_{BC})/10\lambda^2$

Uniform Load

$K_{BC}\Delta_{CB}/10\lambda^2$	0	.178	.330	.445	.508	.510	.458	.366	.251	.127	0
$-K_{BC}C_{BC}\Delta_{BC}/10\lambda^2$	0	-.086	-.171	-.249	-.312	-.348	-.346	-.303	-.225	-.121	0
FEM_BC	0	.092	.159	.196	.196	.162	.112	.063	.026	.006	0

FEM for uniform load = Σ FEM_BC/10 = $0.101 w_uL_2^2$

Haunch Load

D.L. Ratio	1.00	.764	.560	.150	.036	0	.038	.150	.560	.764	1.00
FEM_BC	0	.070	.089	.029	.007	0	.004	.009	.015	.005	0

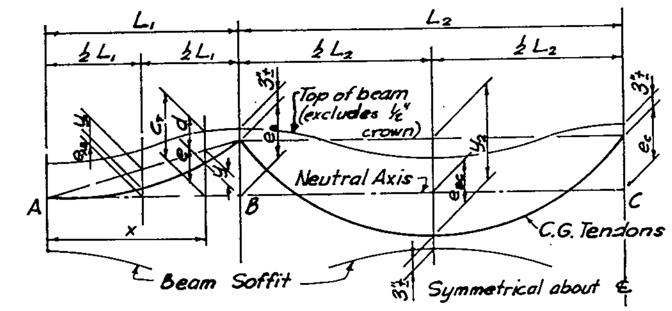
FEM for haunch load = Σ FEM_BC/10 = $0.023 w_hL_2^2$

Alternate Procedure for Determining Beam Properties:
Comparable values for the above beam constants may be obtained from "Handbook of Frame Constants", distributed by the Portland Cement Association. Also, the PCA publication "Continuous Concrete Bridges" explains how the handbook tables may be adapted to the type of beam used in this design.

Moment Distribution

Joint	B		C		D		B		C		D	
Member	BA	BC	CB	CD	DC	DE	BA	BC	CB	CD	DC	DE
Rel. Stiffness	9.43	5.57	5.57	5.57	5.57	9.43						
Distr. Factor	.63	.37	.50	.50	.37	.63						
C.O. Factor	0	-.681	-.681	-.681	-.681	0						
D.F. x C.O.F.	0	-.25	-.34	-.34	-.25	0	0	-.25	-.34	-.34	-.25	0
	Span AB loaded						Span BC loaded					
FEM	-100	0	0	0	0	0	0	+100	-100	0	0	0
			+25						-25			
		-9			-9			+43			+43	
			+2	+2					-11	-11		
		-1			-1			+7			+7	
	-100	-10	+27	+2	-10	0			-2	-2		
Distribution	+69	+41	-15	-14	+4	+6		+1			+1	
Moment	-31	+31	+12	-12	-6	+6	0	+151	-138	-13	+51	0
Distribution							-95	-56	+75	+76	-19	-32
Moment							-95	+95	-63	+63	+32	-32

Post-Tensioning Geometrics of Tendon Profile



Tendon Eccentricities:
Tendon clearances at B, C and midspan BC are approx. 3".
$\therefore e_B = e_C = c_t - 3" \cong 11"$; $e_{BC} = c_b - 3" \cong 8"$
At midspan AB, e_{AB} is selected by trial and error to balance the external loads. Let $e_{AB} = 1"$

Equations:
$y_1 = \frac{1}{2}e_B - e_{AB} = \frac{1}{2}(11) - 1 = 4\frac{1}{2}"$; $y_2 = \frac{1}{2}e_B + \frac{1}{2}e_C + e_{BC} = \frac{1}{2}(11) + \frac{1}{2}(11) + 8 = 19"$
$d = c_t - x/L_1 e_B + y$ in span AB; $d = c_t - e_B + y$ in span BC
$e = c_t \pm d$; $y = y_1(1 - \frac{4}{L^2}x'^2)$

Prestressing Loads:
The equivalent uniform load, w_p, due to eccentricity of final prestress force P kips
in span AB, $w_p = 8y_1P/L_1^2 = 8(4.5)P/(35)^2(12) = 2.44P$ plf
in span BC, $w_p = 8y_2P/L_2^2 = 8(19)P/(70)^2(12) = 2.58P$ plf

Moments:

	A	B	C	D	E
FEM		-100			
Moment		-31	+12	-6	
FEM		+100	-100		
Moment		-95	-63	+32	
FEM			+100	-100	
Moment		+32	-63	-95	
FEM				+100	
Moment		-6	+12	-31	

Dead Load Moment

	AL₁	B	℄L₂	C	D
Load	1.33 kpf	.82 kpf	1.13 kpf	.82 kpf	.62 kpf
FEM		-43 | +93		-93 | +93	+93 | +43
Haunch Moment	+23*	-74	+118*	-107	-74
FEM		-282 | +559		-559 | +559	-559 | +282
Uniform Moment	+196*	-456	+692*	-636	-456
Total DL Moment	+219* -212 +7	-530	+810* -637 +173	-743	-530

*simple span mom.

Live Load Moment (Unif. Loading = 0.48 kpf)

	AL₁	B	C	℄L₂	D
FEM		-102	+238		-238
L.L. Moment	+71* +18 +89	+44	-138	+294* -185 +109	-232
FEM		-102 | +238	-238		+102
L.L. Moment		-264	-126		+38
FEM		+238	-238 | +238		-238
L.L. Moment		-150	-300		-150

Prestress Moment

	B	C	D
Load	2.44P #/'	2.58P #/'	2.44P #/'
FEM	+517P | -1277P	+1277P | -1277P	+1277P | -517P
Moment	+994P'#	+1486P'#	+994P'#
" (Pe')	+11.9P"K	+17.8P"K	+11.9P"K
e' = M÷P	11.9"	17.8"	11.9"

e' is the effective tendon eccentricity due to secondary prestress moments (Pe'-Pe) produced in a continuous beam having a non-concordant tendon profile. e' values at intermediate span points are the ordinates of the M/P diagram of the span loaded as above. These e' values may also be found as the algebraic sum of the primary and secondary eccentricities [e+(e'-e)]. See "Design of Prestressed Concrete Structures" by T.Y. Lin - Chapt. 10

In the adjoining figure, the neutral axis is shown as a straight line.
The curve of the post-tensioning steel in spans AB and BC may be assumed parabolic with respect to the chords that connect the ends of the curves.

Notation:
c_t = vertical distance from top of beam to neutral axis.
d = vertical distance from top of beam to c.g. of tendons.
y' = vertical distance from c.g. tendons to chord of parabola.
y_1, y_2 = y at midspan AB and BC respectively.
e = tendon eccentricity; + below (- above) Neutral Axis
x' = horizontal distance from ℄ span to point

Prestress Force
The minimum final prestress force, P, required to maintain zero tension stress in the concrete:
(a) in the top fibers under negative moment, $P = M \div (e' - k_1)$
(b) in the bottom fibers under positive moment, $P = M \div (e' + k_2)$

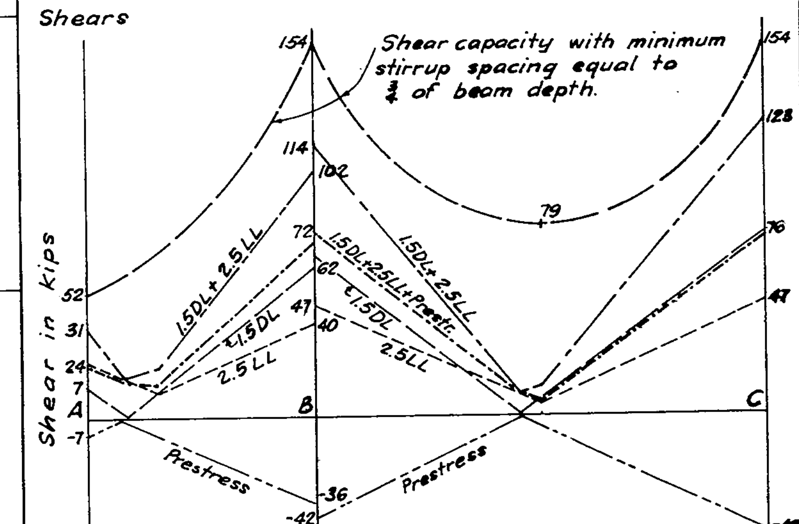
Point		Span AB 1	2	4	B	Span BC 4	1	0	1	4	C
e	(in)	+0.7	-0.1	-5.9	-11.0	-4.2	+7.2	+8.0	+7.2	-4.2	-11.0
e'-e	"	-0.2	-0.4	-0.7	-0.9	-1.5	-3.2	-3.8	-4.4	-6.2	-6.8
e'	"	+0.5	-0.5	-6.6	-11.9	-5.7	+4.0	+4.2	+2.8	-10.4	-17.8
k₁	"	3.9	4.4	6.9	8.9	6.9	4.5	4.3	4.5	6.9	8.9
k₂	"	2.6	2.8	4.0	5.0	4.0	3.0	2.9	3.0	4.0	5.0
e'-k₁	"	-3.4	-4.9	-13.5	-20.8	-12.6	-0.5	-0.1	-1.7	-17.3	-26.7
e'+k₂	"	+3.1	+2.3	-2.6	-6.9	-1.7	+7.0	+7.1	+5.8	-6.4	-12.8
M_DL	(FT-K)	+43	+7	-270	-530	-238	+162	+173	+120	-409	-743
-M_LL max.	"	-46	-93	-186	-264	-144	-	-	-	-179	-300
+M_LL max.	"	+56	+89	+83	-	-	+88	+109	+106	-	-
-M_D+L	"	-3	-86	-456	-794	-382	-	-	-	-588	-1043
+M_D+L	"	+99	+96	-	-	-	+250	+282	+226	-	
P = -M÷(e'-k₁)	(K)	11	211	406	457	363	-	-	-	407	469
P = +M÷(e'+k₂)	-	384	500	-	-	-	439	477	468	-	-

With P = 500K, concrete stresses under service loads in psi are:

	1	2	4	B	4	1	0	1	4	C
f_c (top)	+598	+618	+115	+54	+154	+689	+789	+756	+143	+50
f_c (bottom)	+112	0	+644	+676	+576	+157	+53	+56	+596	+681

Concrete stresses under initial prestress plus dead load are not critical.
Ultimate moment capacity of structure for 1.5DL + 2.5LL is satisfactory.
Friction losses will not be excessive when tensioned from both ends.

Shears



U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

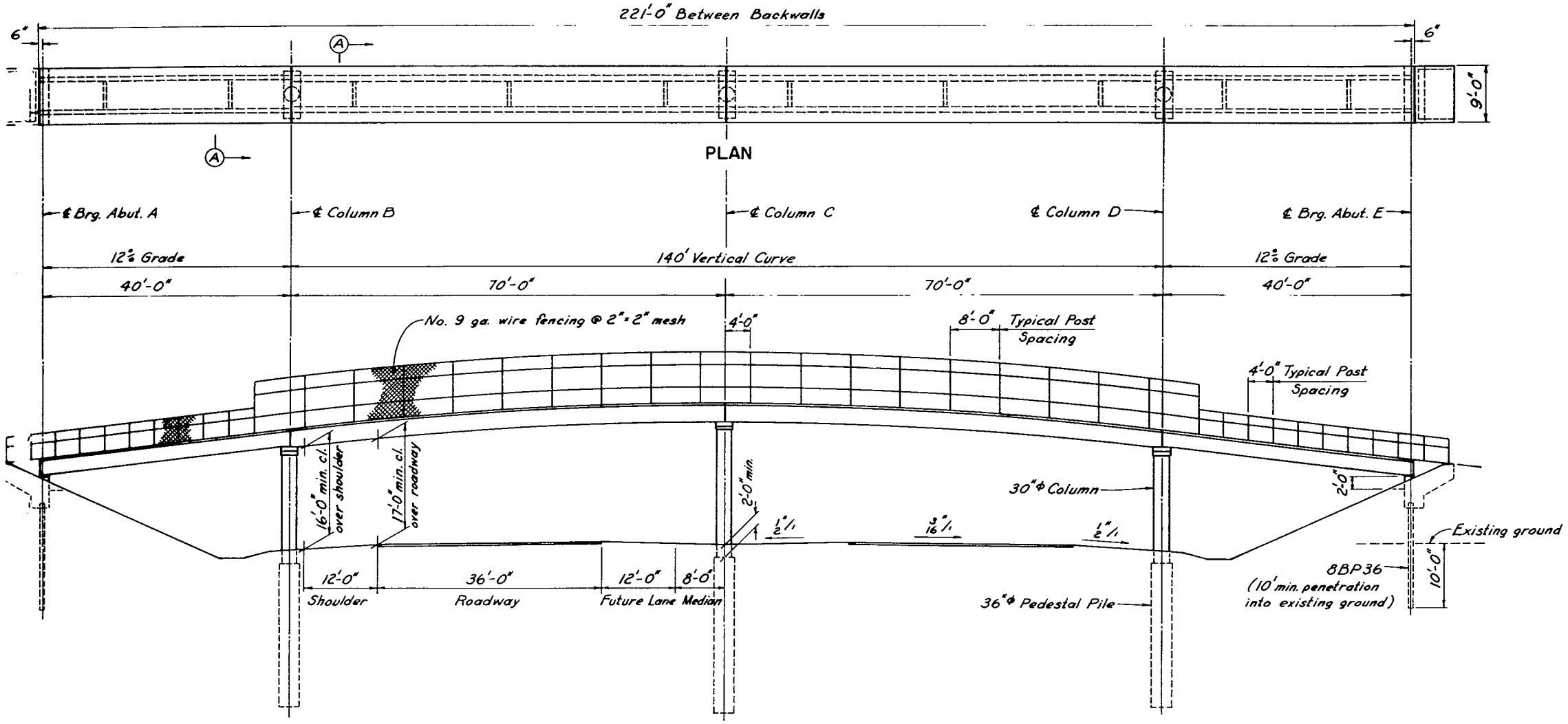
TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

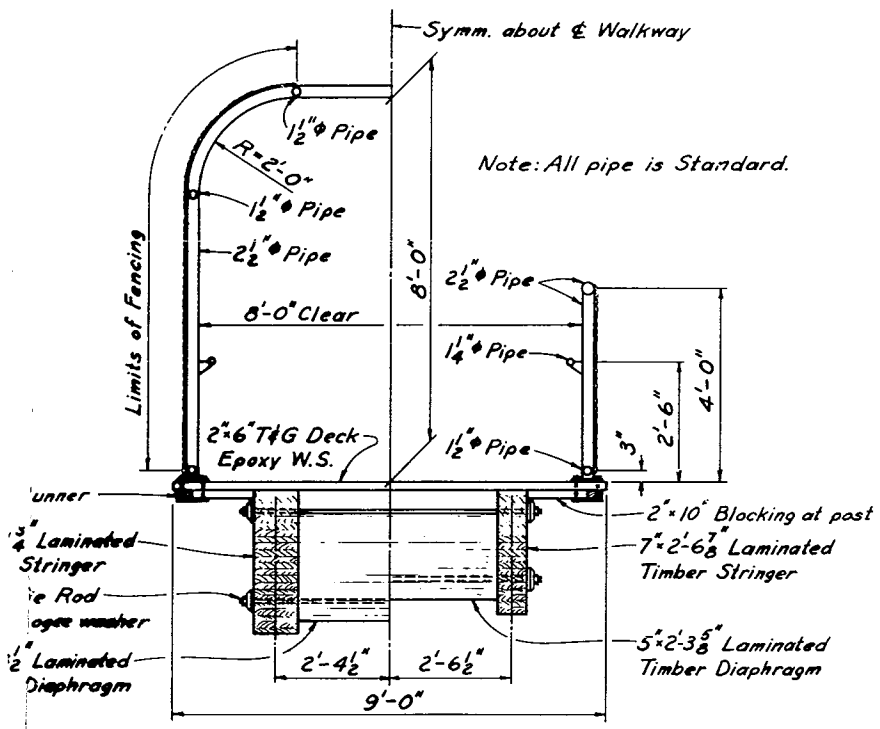
FOUR SPAN PRECAST PRESTRESSED CONCRETE BRIDGE
SPANS = 35-70-70-35 = 210 FT.
DESIGN CALCULATIONS

RECOMMENDED S. B. Lauer — Chief Design Branch
RECOMMENDED [signature] — Director of Engineering
RECOMMENDED [signature] — Chief Bridge Division
APPROVED [signature] — Administrator

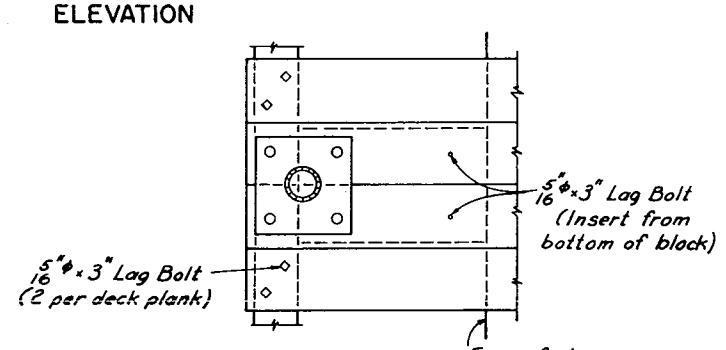
OCTOBER 1964

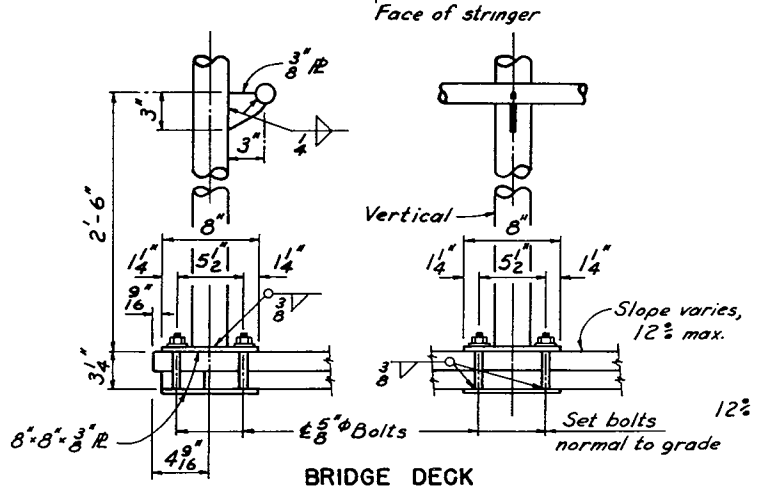
SHEET NO. 203



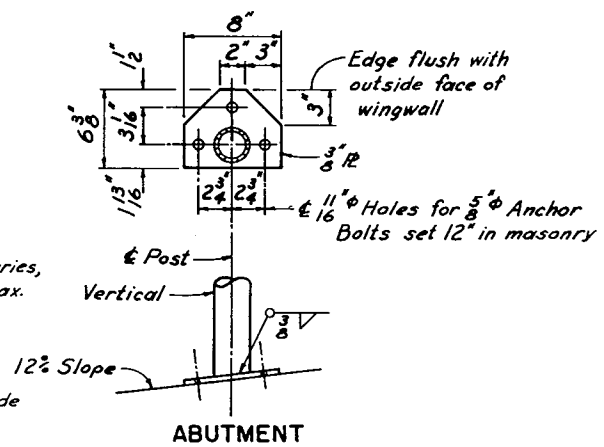


SECTION A-A





BRIDGE DECK



ABUTMENT

POST BASE ASSEMBLY

SUMMARY OF QUANTITIES

Item	Unit	Quantity
Excavation for Structures	C.Y.	40 *
36"ϕ Pedestal Pile	L.F.	71 *
Steel H-Pile	L.F.	68 *
Class A(AE) Concrete	C.Y.	25
Reinforcing Steel	Lb.	5780
Structural Metal	Lb.	870
Glued Laminated Timber		Lump
T&G Timber Deck ⊛	mbm	5.00
Wearing Surface	S.Y.	221
Fencing (including supports)	L.F.	232

*Estimated ⊛Includes runner and blocking

GENERAL NOTES

Design Criteria: "Design Criteria for Pedestrian Bridges Over Interstate or Comparable Highways," Appendix A.

Design Specifications: AASHO Standard Specifications for Highway Bridges, 1961, with tentative revisions to 1963, except as modified or amplified by the Design Criteria.
NLMA National Design Specifications for Stress-Grade Lumber and Its Fastenings, 1962.

Concrete: Concrete shall be Class A(AE) with a minimum 28 day $f'_c = 4000$ psi. The air entraining agent shall meet with the approval of the engineer.

Reinforcing Steel: Reinforcing steel shall be deformed bars of hard, intermediate or rail steel grade conforming to ASTM A-15 or A-16. Dimensions shown on the plans from the reinforcing steel to the outside edge of the concrete are all clear dimensions.

Treated Timber: Stringers shall be structural glued laminated timber of grade or combination of grades providing a minimum allowable "f" of 2000 psi and "c" of 1500 psi under wet conditions of use. Painting of timber is optional.

Adhesives: Adhesives for structural glued members shall be of exterior type conforming to Military Specification MIL-A-397B or MIL-A-5534A. Adhesives used shall be compatible with preservative treatment.

Preservative Treatment: All lumber shall be pressure impregnated with an approved preservative suitable to the species of wood, adhesives and painting requirements. Preservatives and amount of retention shall conform to current AASHO Standard Specifications for Highway Bridges. All cuts, abrasions and holes bored after treatment shall be treated in an approved manner.

Appearance: Stringers and diaphragms shall be of Architectural Appearance Grade.

Hardware: All hardware shall be galvanized in accordance with ASTM A-153.

Galvanized Fence and Supports: Chain link fence fabric conforming to ASTM A-392 shall be No. 9 gage wire woven in a 2 inch mesh and galvanized after weaving with a Class II zinc coating. Galvanized pipe rail and fence supports shall conform to ASTM A-53 Grade B, Type S or E.

Pedestal Piles: The drilled hole shall be cleaned of all loose material and the pedestal pile poured in compact coarse and fine sand, medium stiff clay or better.

Lateral Soil Pressure: Design has been based on an allowable lateral soil pressure of 300 psf per foot of depth.

Soil Pressure: The required allowable soil pressure to use the stub abutment as detailed but without piles is 1 Ton per square foot.

Epoxy Wearing Surface: A Type C coal-tar modified epoxy resin system with a surface aggregate application shall be used for the wearing surface. The system and aggregate shall conform to Interim Guide Specification AASHO M200-631. The deck surface shall be clean and free of foreign material detrimental to bond of the epoxy resin. The rate of application of epoxy resin shall be in accordance with the manufacturer's recommendations. Application of the aggregate shall be at a rate of not less than 15 lbs. per square yard.

DO NOT SCALE

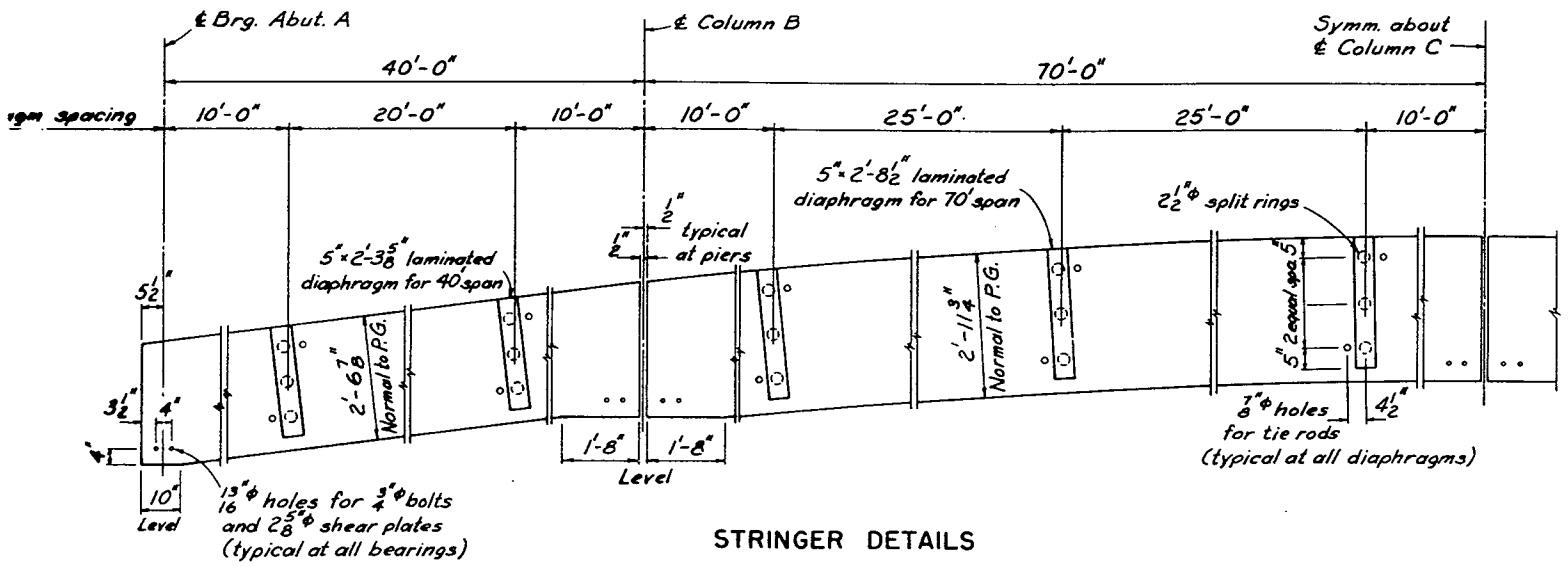
U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

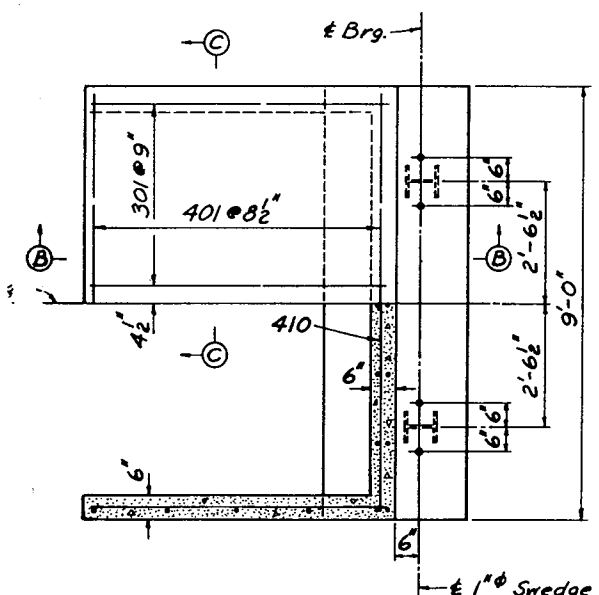
FOUR SPAN LAMINATED TIMBER BRIDGE
SPANS = 40-70-70-40 = 220 FT.
GENERAL PLAN AND ELEVATION

RECOMMENDED S. B. Lauer, Chief Design Branch
RECOMMENDED E. E. Carlson, Chief Bridge Division
RECOMMENDED C. M. Williams, Director of Engineering
APPROVED Rex M. Whitton, Administrator

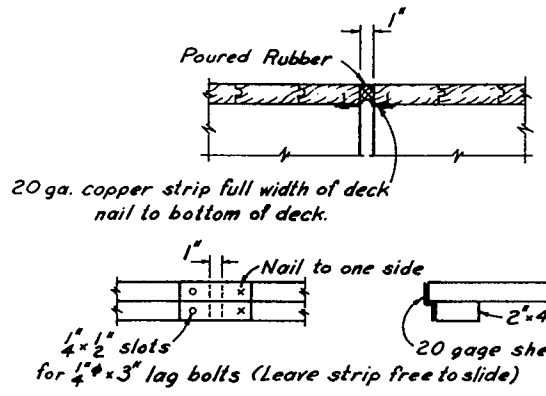
OCTOBER 1964
SHEET NO. 301



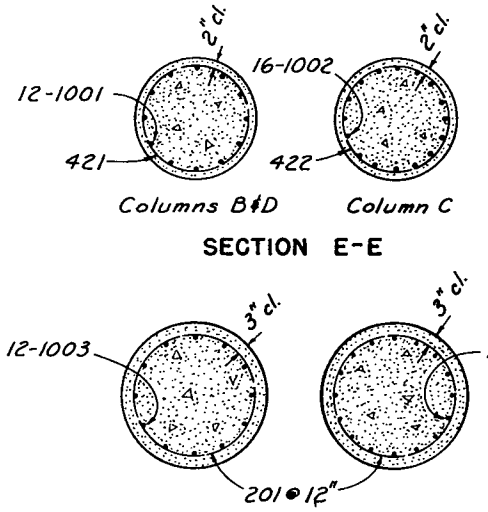
STRINGER DETAILS



PART PLAN — PART SECTION

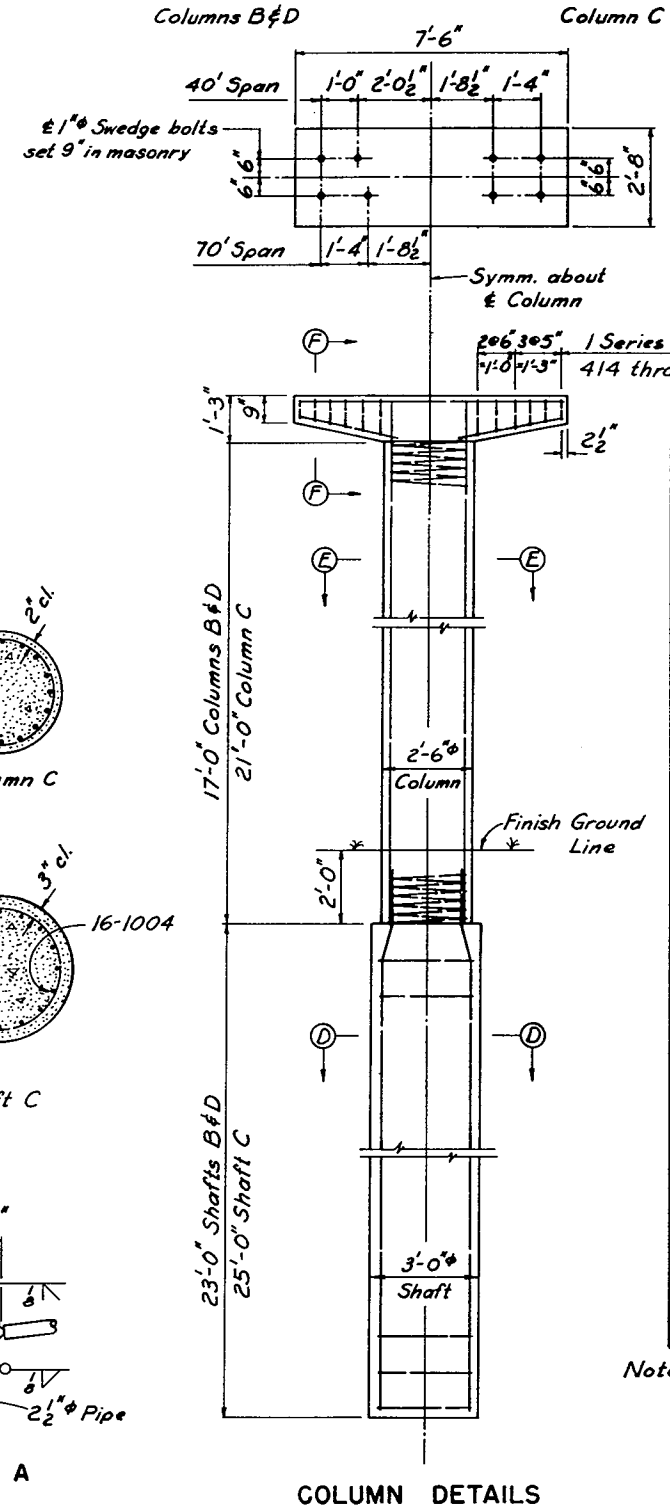


DECK JOINT DETAIL AT PIER

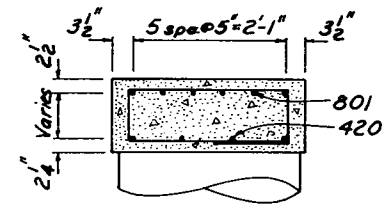


SECTION E-E

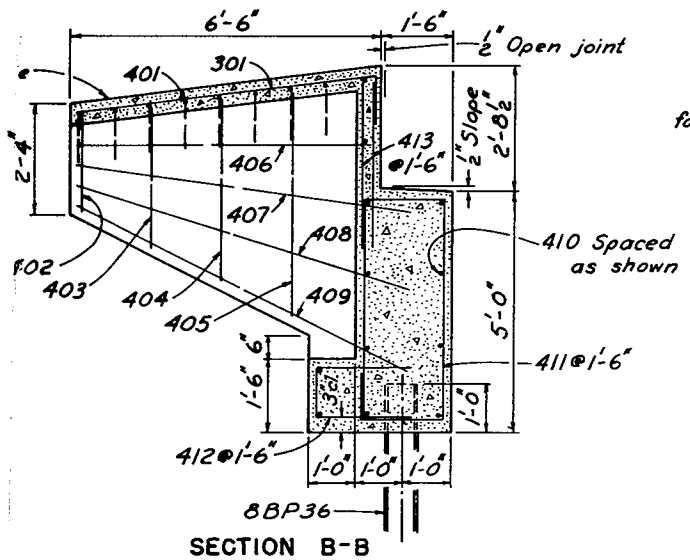
SECTION D-D



COLUMN DETAILS

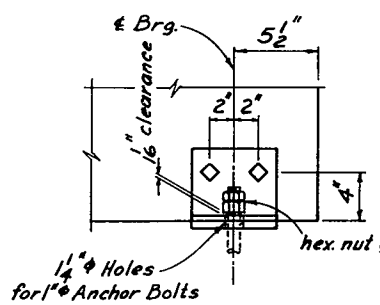


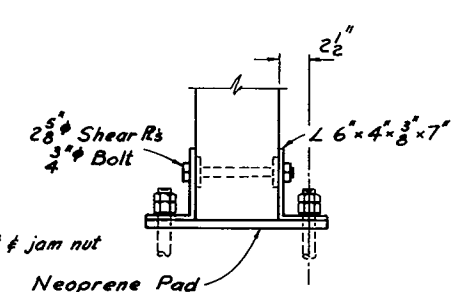
SECTION F-F



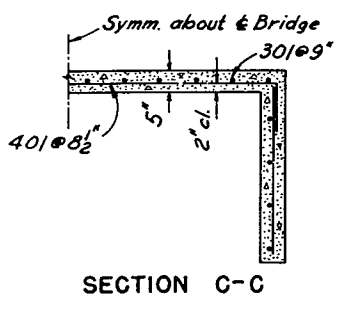
SECTION B-B

ABUTMENT DETAILS

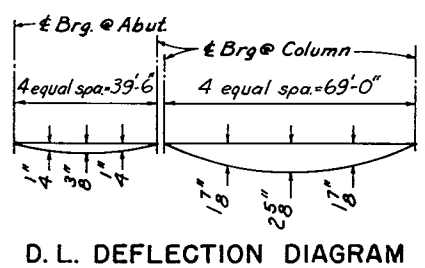




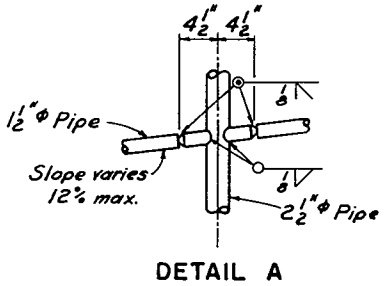
BEARING DETAILS



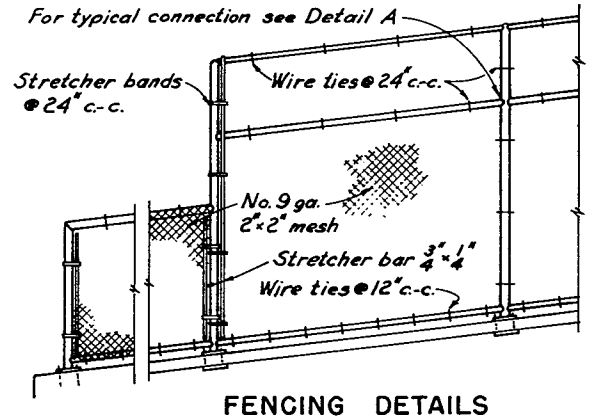
SECTION C-C



D. L. DEFLECTION DIAGRAM



DETAIL A

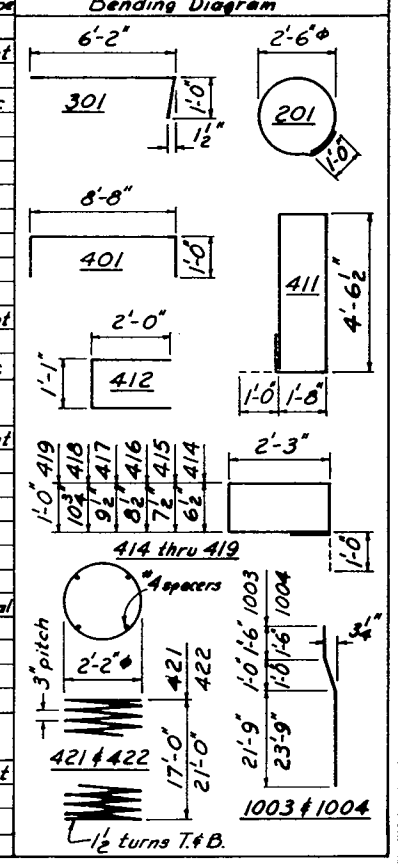


FENCING DETAILS

REINFORCING STEEL SCHEDULE

Mark	No.	Length	Shape
Abutment			
301	24	7'-2"	Bent
401	18	10'-8"	"
402	4	2'-1"	Str.
403	4	3'-1"	"
404	4	4'-0"	"
405	4	4'-11"	"
406	4	6'-2"	"
407	4	7'-2"	"
408	4	7'-4"	"
409	4	7'-9"	"
410	22	8'-8"	"
411	14	13'-5"	Bent
412	14	5'-1"	"
413	28	3'-9"	Str.
Column			
414	6	6'-7"	Bent
415	6	6'-9"	"
416	6	6'-11"	"
417	6	7'-1"	"
418	6	7'-3"	"
419	6	7'-6"	"
420	24	3'-0"	Str.
421	2	483'	Spiral
422	1	592'	"
801	18	7'-2"	Str.
1001	24	18'-0"	"
1002	16	22'-0"	"
Pedestal Pile			
201	71	8'-10"	Bent
1003	24	24'-3"	"
1004	16	26'-3"	"

Bending Diagram



Note: All bending dimensions are "Out to Out" of the bar. Bar sizes are designated on the plans by numbers. The first digit of the three digit marks and the first two digits of the four digit marks indicate size of the bar.

Weight of the #4 spacers shall be added to the weight of the spirals and paid for as reinforcing.

DO NOT SCALE

U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

FOUR SPAN LAMINATED TIMBER BRIDGE

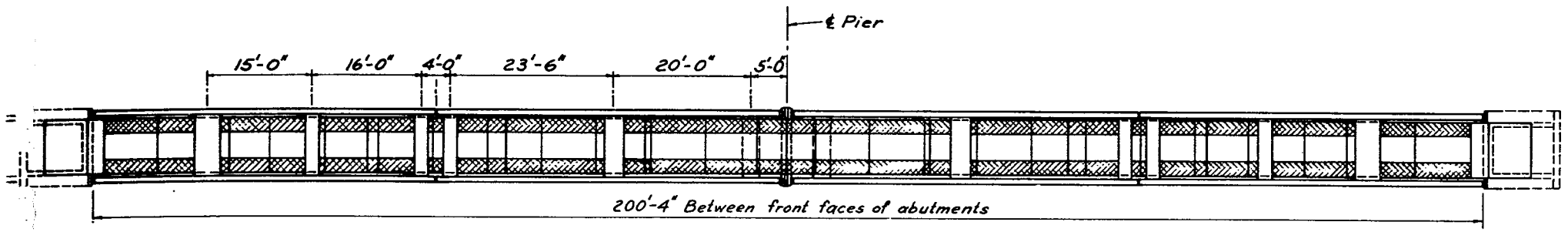
SPANS = 40-70-70-40 = 220 FT.

DETAILS

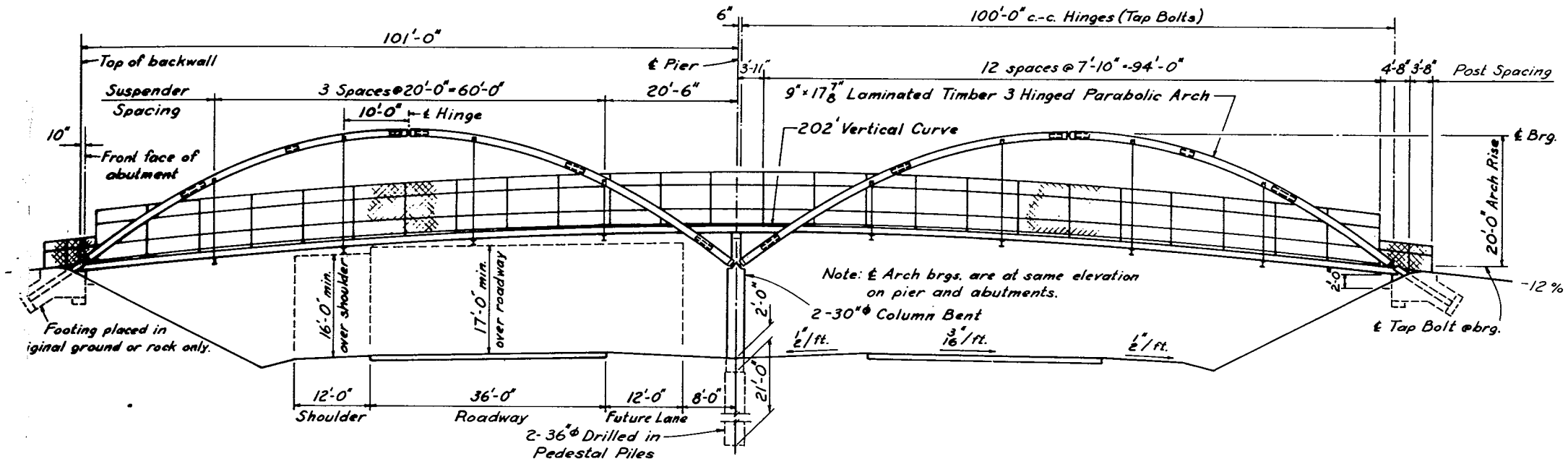
RECOMMENDED S. B. Sauer, Chief Design Branch
RECOMMENDED [illegible], Chief Bridge Division
RECOMMENDED [illegible], Director of Engineering
APPROVED [illegible], Administrator

OCTOBER 1964

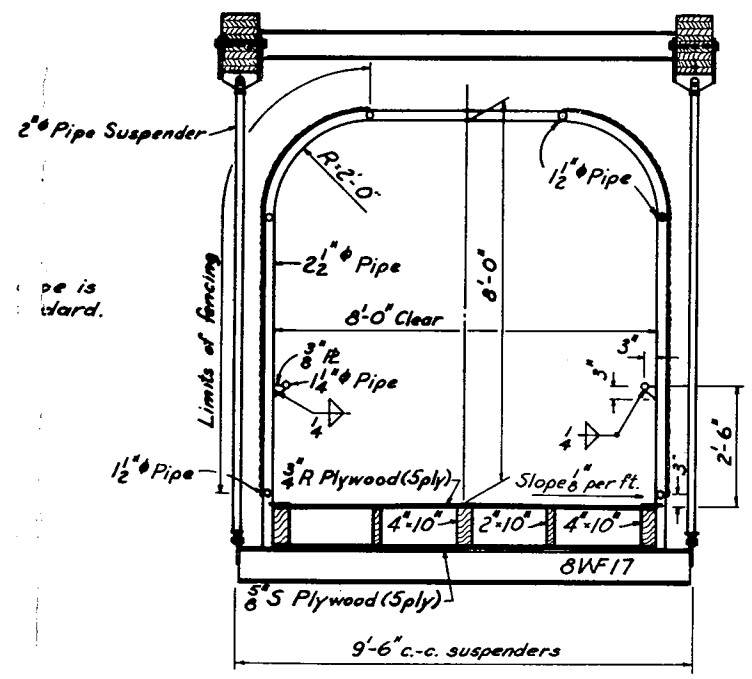
SHEET NO. 302



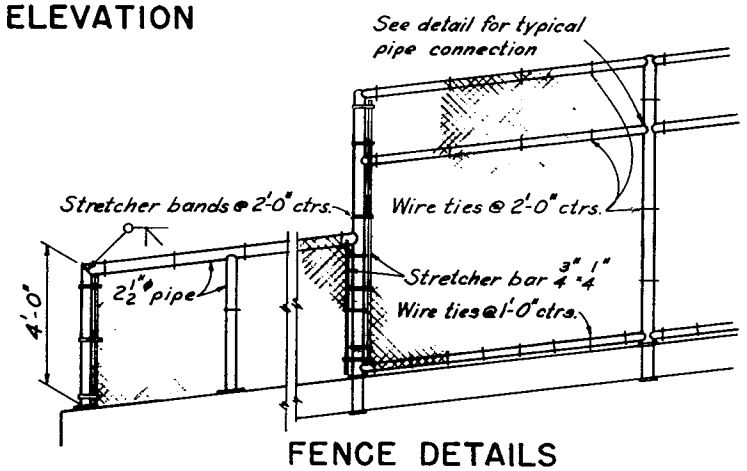
PLAN



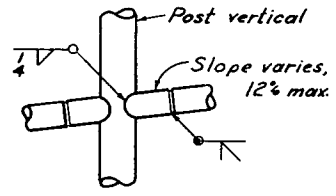
ELEVATION



TYPICAL SECTION



FENCE DETAILS



TYPICAL PIPE CONNECTION

DECK WEARING SURFACE NOTE

A Type C, coal tar modified epoxy resin system, with a surface aggregate application shall be used for the wearing surface. The epoxy resin system and aggregate shall conform to Interim Guide Specification AASHO M200-63 I. The deck surface shall be clean and free of any foreign matter detrimental to bond with the epoxy resin. Rate of application of the epoxy resin shall be according to the manufacturer's recommendations. Application of the aggregate shall be at a rate of not less than 15 pounds per square yard.

TIMBER NOTES

Adhesives: Adhesives for structural glued members shall be of exterior type conforming to Military Specification MIL-A-397B or MIL-A-5534A. Adhesives used shall be compatible with preservative treatment.

Preservative Treatment: All lumber shall be pressure impregnated with an approved preservative suitable to the species of wood, adhesives, and painting requirements. Preservatives and amount of retention shall conform to the current AASHO M-133 and AASHO Standard Specifications for Highway Bridges. All cuts, abrasions, and holes bored after treatment shall be treated in an approved manner.

Appearance: Arch ribs shall be of Architectural Appearance Grade. Deck panels and arch diaphragms shall be of Utility Appearance Grade, but shall have excess glue removed from exposed surfaces.

Paint: Painting is optional.

DO NOT SCALE

GENERAL NOTES

Design Criteria: Design Criteria for Pedestrian Bridges over Interstate or Comparable Highways, Appendix A.

Design Specifications: AASHO Standard Specifications for Highway Bridges, 1961, with tentative revisions to 1963, except as modified or amplified by the Design Criteria.
NLMA National Design Specification for Stress-Grade Lumber and Its Fastenings (1962).
DFPA Specification No. S5-8.

Timber and Plywood: Arch ribs shall be structural glued laminated timber of grade or combination of grades providing a minimum allowable "f" of 2000 psi. and "c" of 1500 psi. under wet conditions of use. Deck panels and arch diaphragms shall be structural glued units of DFPA Exterior Grade A-C plywood and framing members providing a minimum allowable "f" of 1500 psi.

Structural Steel: All structural steel shall conform to ASTM A-36, and shall be galvanized after fabrication in accordance with ASTM A-123.

Concrete: Concrete shall be Class A(AE) with a minimum 28 day compressive strength "f'_c" = 4000 psi. The air entraining agent shall meet with the approval of the engineer.

Reinforcing Steel: Reinforcing steel shall be deformed bars of hard, intermediate or rail-steel grade conforming to ASTM Specification A15 or A16. Dimensions shown on the plans from the reinforcing steel to the outside edge of the concrete are all clear dimensions. Bar sizes are designated on the plans by the first digit of the three digit marks.

Suspender Pipe: Suspender pipe shall be galvanized and shall conform to ASTM A-53 Grade B, Type S or E.

Hardware: All hardware shall be galvanized in accordance with ASTM A-153.

Galvanized Fence and Supports: Chain link fence fabric conforming to ASTM A-392 shall be No. 9 gage wire woven in a 2 inch mesh and galvanized after weaving with a Class II zinc coating. Galvanized pipe rail and fence supports shall conform to ASTM A-53 Grade B, Type S or E.

Pedestal Pile: The drilled hole shall be cleaned of all loose material and the pedestal pile poured in compact coarse and fine sand, medium stiff clay or better.

Lateral Soil Pressure: Design has been based on an allowable lateral soil pressure of 300 psf. per foot of depth.

Soil Pressure: The maximum allowable soil pressure for design of abutment footings has been assumed to be 1 ton per square foot.

SUMMARY OF QUANTITIES

Item	Units	Quantity
Excavation for Structures (Estimated)	Cu. Yd.	60
36"ϕ Pedestal Pile (Estimated)	Lin. Ft.	42
Class A(AE) Concrete	Cu. Yd.	19
Reinforcing Steel	Lb.	4760
Structural Steel (Deck support beams)	Lb.	1350
Structural Metal (Including hardware)	Lb.	2780
Suspender Pipe	Lin. Ft.	189
100' Laminated Arches	Each	4
Prefabricated Deck Units	Each	10
Prefabricated Diaphragm Units (12 required)	Lump	
Fencing (Shear plates and lag bolts excluded)	Lin. Ft.	213
Deck Surfacing	Sq. Yd.	180

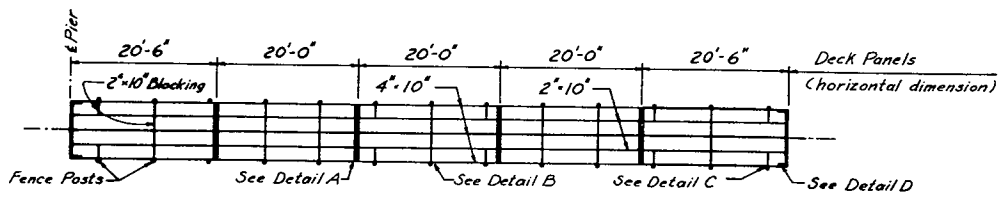
U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

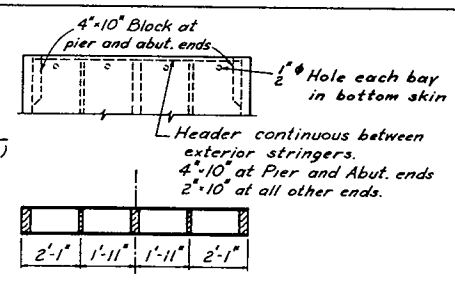
TWO SPAN LAMINATED TIMBER ARCH BRIDGE
100 FT. SPANS - STRESSED SKIN PLYWOOD DECK

GENERAL PLAN AND ELEVATION

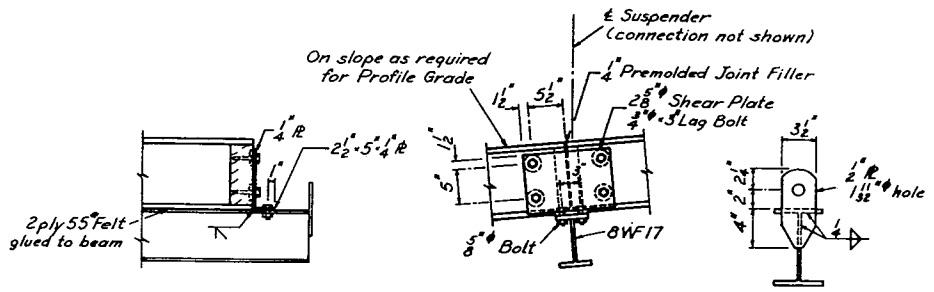
RECOMMENDED S. B. Lauer, Chief Design Branch	RECOMMENDED Director of Engineering	OCTOBER 1964
RECOMMENDED T. E. Erickson, Chief Bridge Division	APPROVED Administrator	SHEET NO. 311



DECK FRAMING DIAGRAM

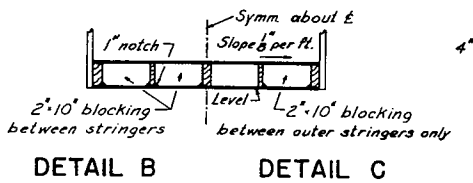


DETAIL A

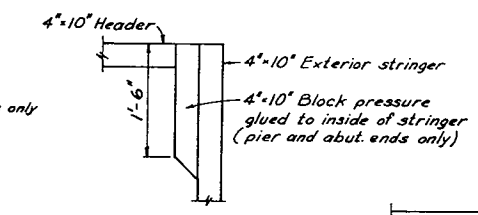


DECK CONNECTION AT FLOORBEAM

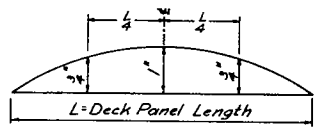
DETAIL AT SUSPENDER



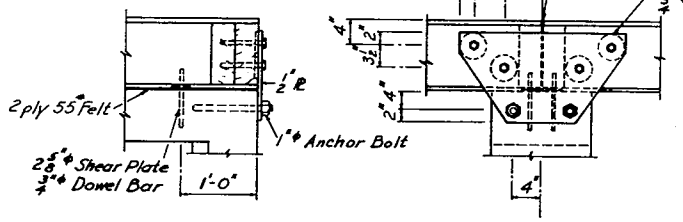
DETAIL B DETAIL C



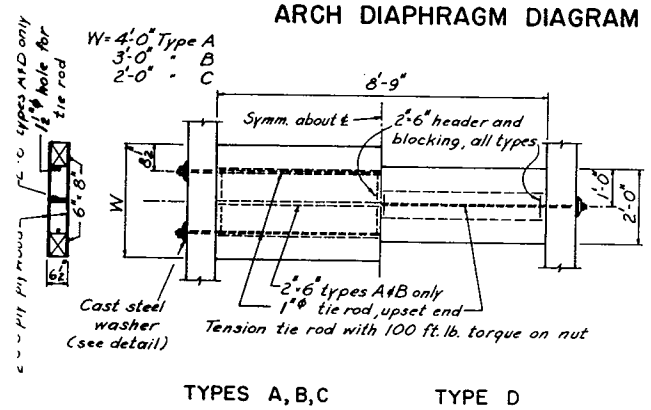
DETAIL D



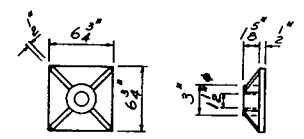
DECK CAMBER DIAGRAM



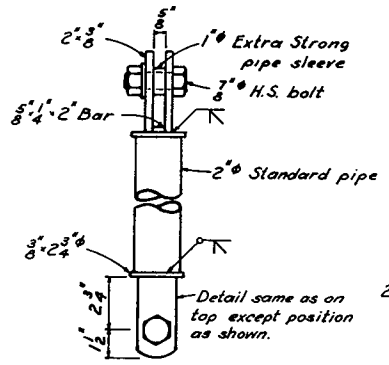
DECK CONNECTION AT PIER



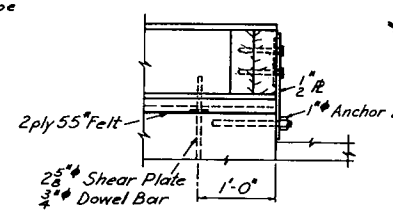
DIAPHRAGM DETAILS

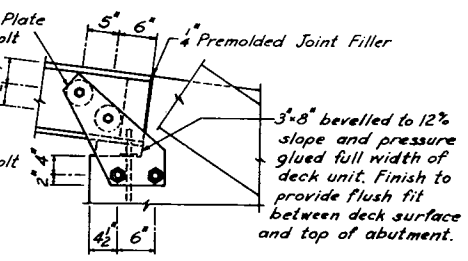


CAST STEEL WASHER

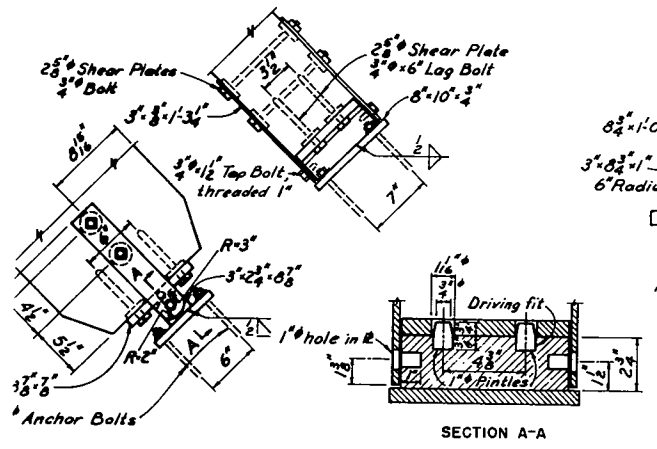


SUSPENDER DETAIL

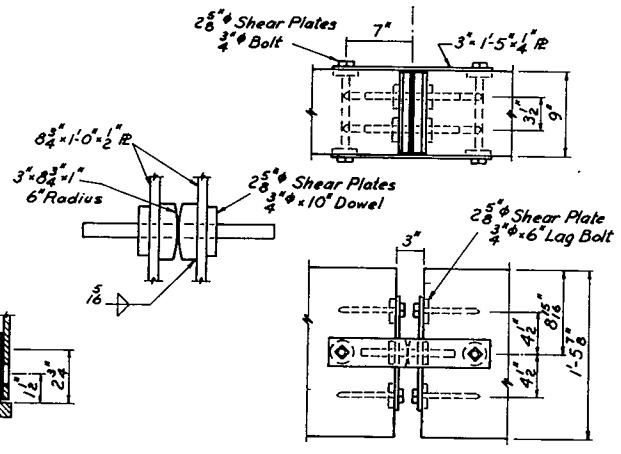




DECK CONNECTION AT ABUTMENT

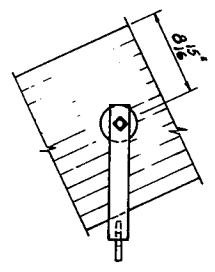


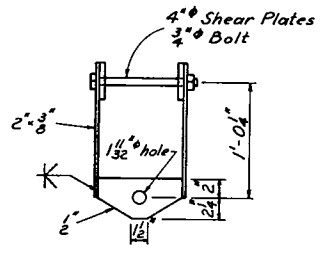
AT PIER AND ABUTMENT



AT CENTER OF SPAN

ARCH HINGE DETAILS





HANGER DETAIL

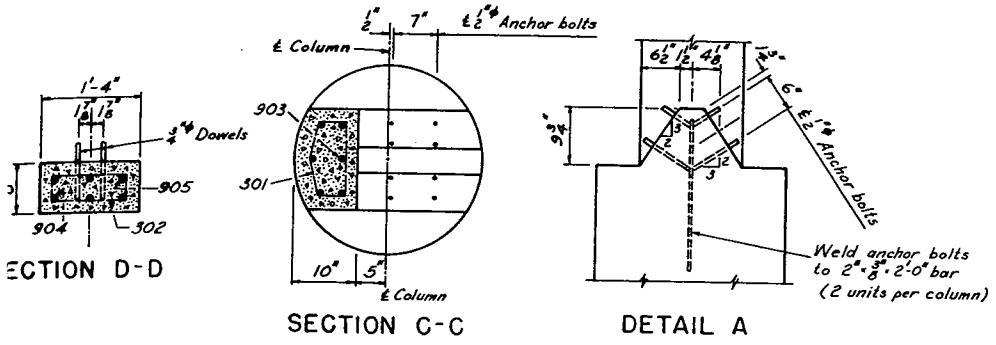
DO NOT SCALE

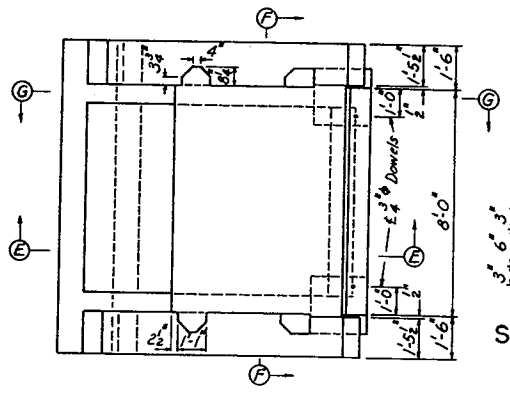
U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

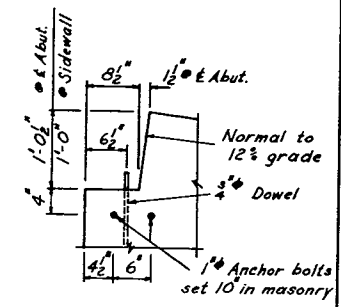
TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

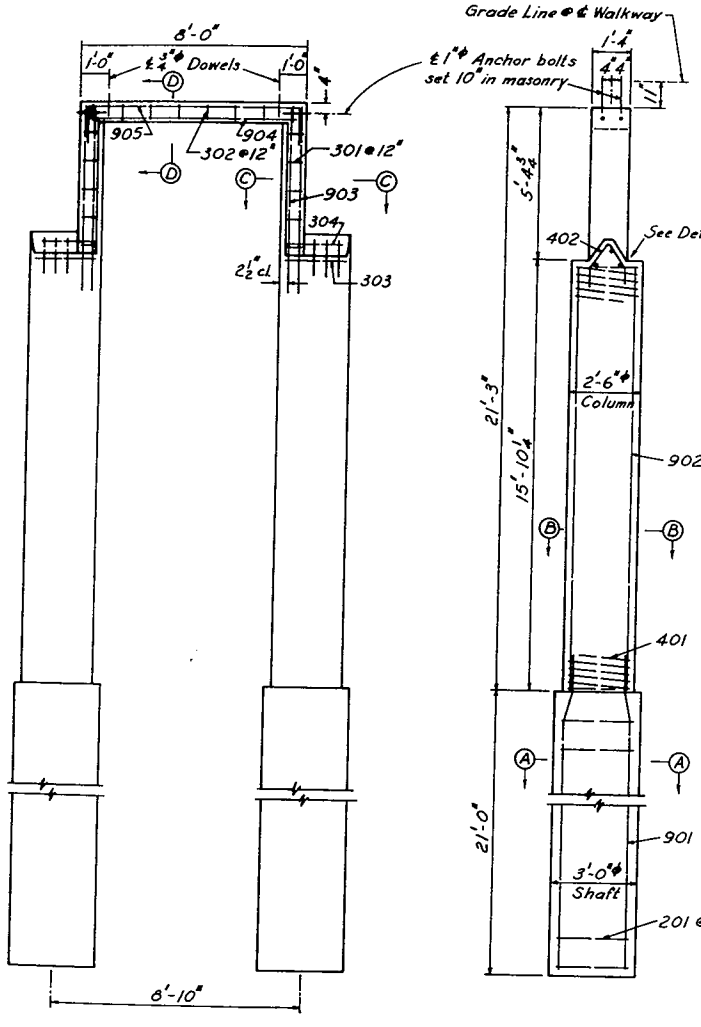
TWO SPAN LAMINATED TIMBER ARCH BRIDGE
100 FT. SPANS - STRESSED SKIN PLYWOOD DECK
DETAILS

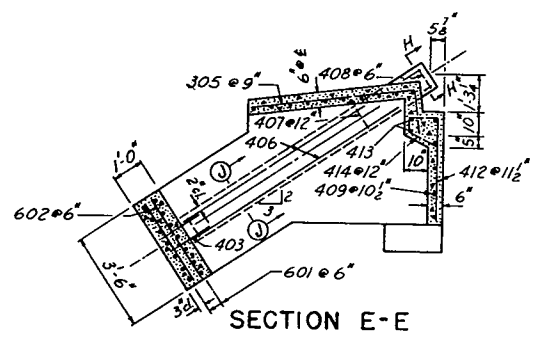
RECOMMENDED S. B. Larsen, Chief Design Branch	RECOMMENDED, Director of Engineering	OCTOBER 1964
RECOMMENDED, Chief Bridge Division	APPROVED, Administrator	SHEET NO. 312

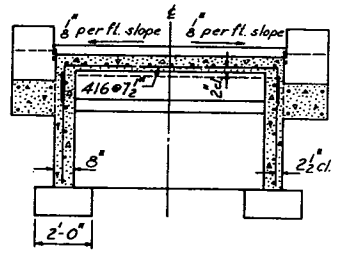


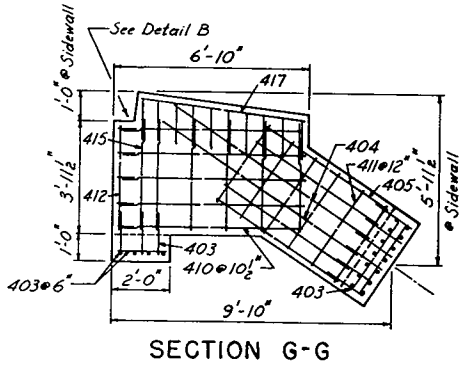


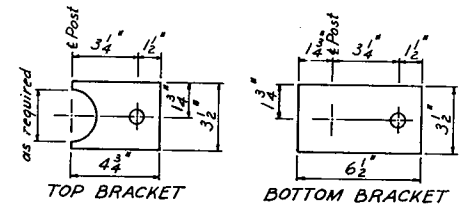


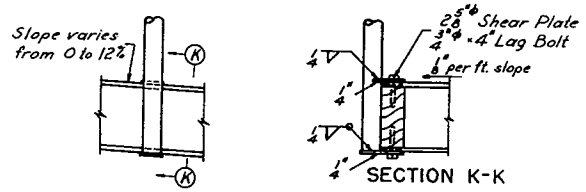


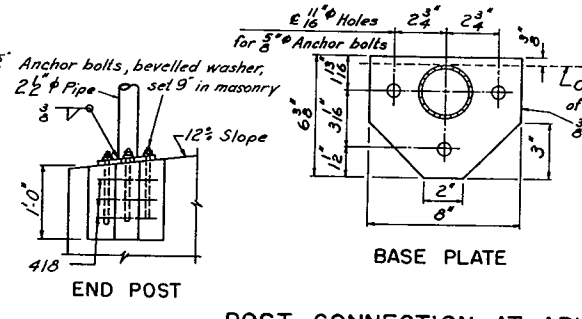










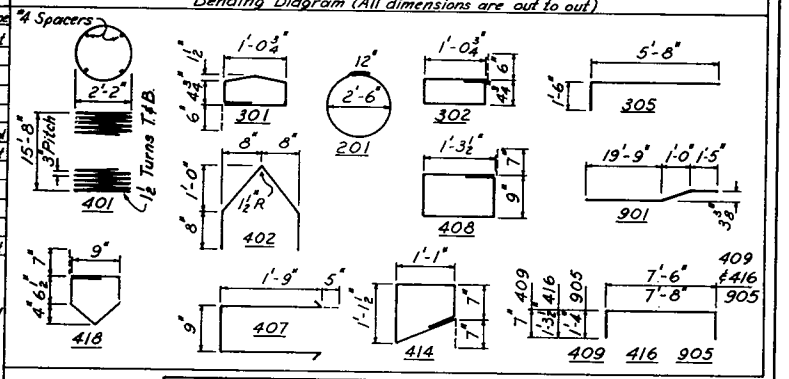


REINFORCING STEEL SCHEDULE

Abutments				Pier			
Mark	No.	Length	Shape	Mark	No.	Length	Shape
305	18	7'-2	Bent	201	42	8'-10"	Bent
403	84	1'-6"	Str.	301	10	3'-5"	-
404	8	8'-6	-	302	8	3'-5"	-
405	16	5'-3"	-	303	4	2'-0"	Str.
406	16	10'-0	-	304	2	1'-9"	-
407	36	5'-1"	Bent	401	2	447	Spiral
408	8	4'-8	-	402	6	3'-9"	Bent
409	10	8'-8	-	901	32	22'-2"	-
410	20	6'-6"	Str.	902	32	15'-8"	Str.
411	24	3'-2"	-	903	12	6'-8"	-
412	18	3'-7"	-	904	3	7'-8"	-
413	4	7'-6"	-	905	3	10'-4"	Bent
414	14	4'-7"	Bent				
415	24	3'-9"	Str.				
416	20	10'-1"	Bent				
417	4	5'-8"	Str.				
418	24	3'-5"	Bent				
601	28	10'-6	Str.				
602	88	3'-0"	-				

Note: Weight of #4 spacers shall be added to spiral and paid for as reinforcing.

Bending Diagram (All dimensions are out to out)



DO NOT SCALE

U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

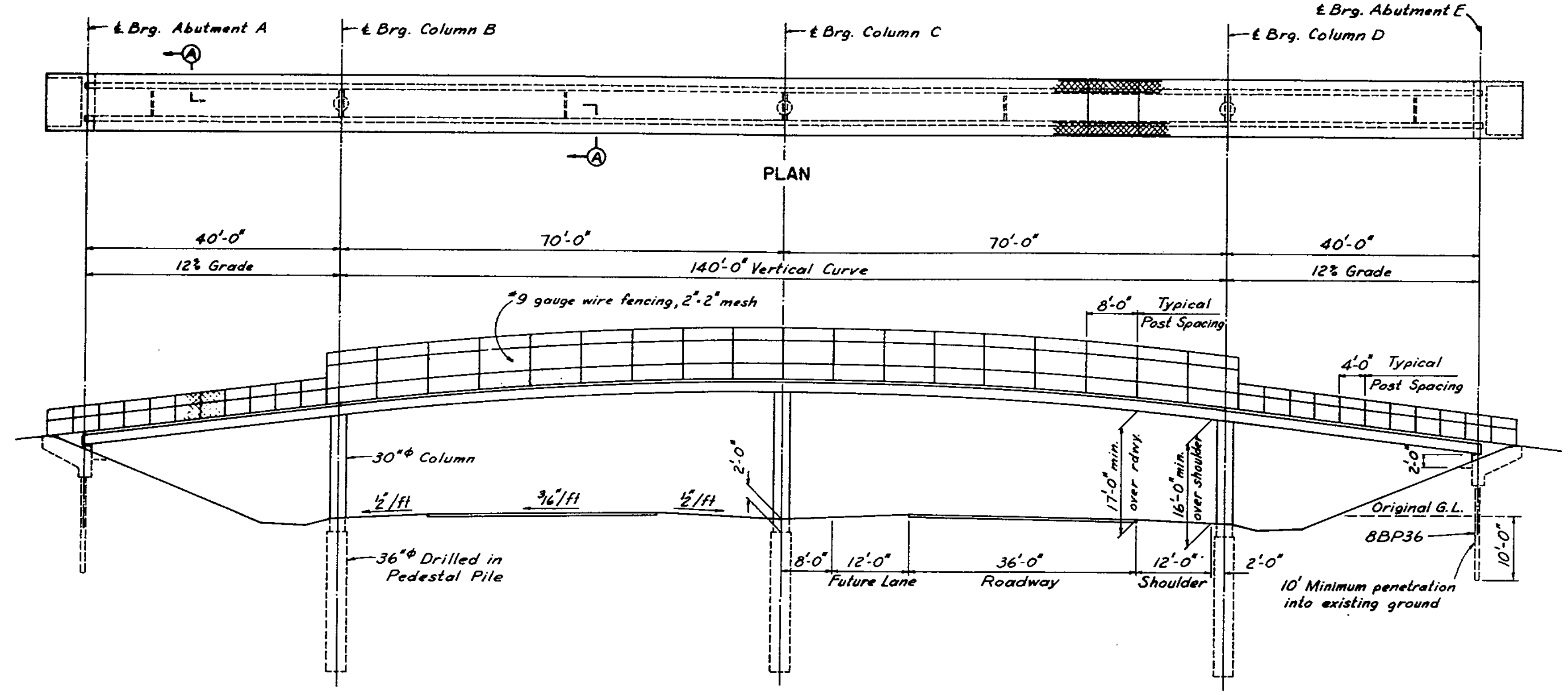
TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

TWO SPAN LAMINATED TIMBER ARCH BRIDGE
100 FT. SPANS - STRESSED SKIN PLYWOOD DECK
DETAILS

RECOMMENDED S. B. Larsen, Chief Design Branch
RECOMMENDED L. L. Erickson, Chief Bridge Division
RECOMMENDED Bert W. Williams, Director of Engineering
APPROVED Rex M. Whitton, Administrator

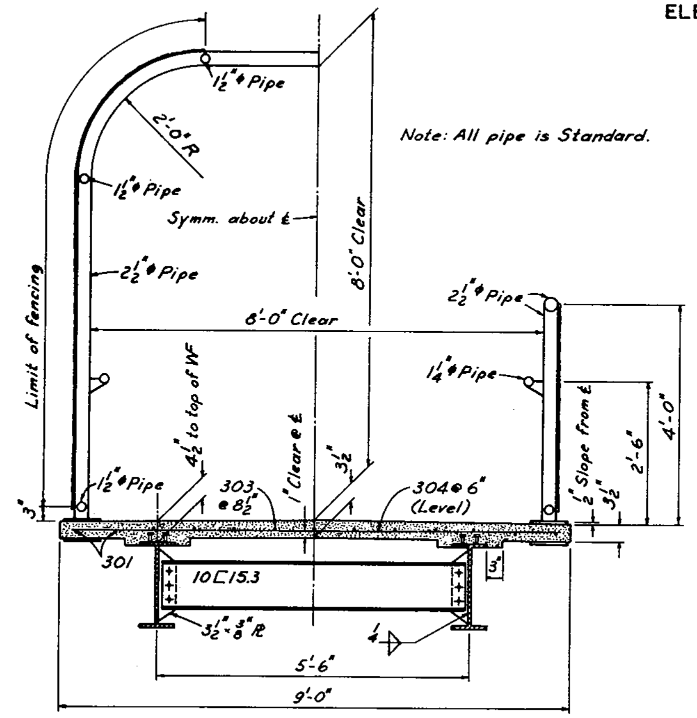
OCTOBER 1964

SHEET NO. 313



PLAN

ELEVATION



Note: All pipe is Standard.

SECTION A-A

SUMMARY OF QUANTITIES		
Item	Unit	Quantity
Excavation for Structures	Cu. Yd.	* 30
36"φ Pedestal Pile	Lin. Ft.	* 66
Steel H-Pile	Lin. Ft.	* 68
Class A (AE) Concrete	Cu. Yd.	46
Reinforcing Steel	Lb.	⊕ 6790
Structural Steel	Lb.	22950
Fencing (including supports)	Lin. Ft.	232

* Estimated
⊕ Includes wt. of spiral spacers.

GENERAL NOTES

Design Criteria: Design Criteria for Pedestrian Bridges over Interstate or Comparable Highways, Appendix A.

Design Specifications: AASHO Standard Specifications for Highway Bridges, 1961, with tentative revisions to 1963 except as modified or amplified by the Design Criteria.

Structural Steel: All structural steel shall conform to ASTM A-36. Field connections shall be bolted, using ⅞"φ high strength bolts. Bolts, nuts and washers shall conform to ASTM A-325. Beam splices shall be subpunched and reamed assembled in the shop to proper camber. Beams shall be cambered for dead load deflection, and where required, for additional permanent camber due to vertical curvature.

Concrete: Concrete shall be Class A(AE) with a minimum 28 day f'_c = 4000 psi. The air entraining agent shall meet with approval of the engineer. When pouring the deck, a retarder shall be used that will delay final set until the entire deck is in place.

Reinforcing Steel: Reinforcing steel shall be deformed bars of hard, intermediate or rail-steel grade conforming to ASTM A-15 or A-16. Dimensions shown on the plans from the reinforcing steel to the outside edge of the concrete are all clear dimensions. Bar sizes are designated on the plans by the first digit of the three digit marks.

Paint: All structural steel shall be given one shop coat of red lead iron oxide paint. Bolted joints and surfaces in contact with concrete shall not be painted. Surfaces inaccessible after erection shall be given three coats of paint in the shop. Field paint shall consist of one coat of tinted red lead iron oxide paint followed by one finish coat of aluminum or light green paint. All paint and workmanship shall conform to AASHO Specifications and Special Provisions for Paint. See Appendix B.

Welding: All welding shall conform to the Specifications for Welded Highway and Railway Bridges of the American Welding Society, AWS D2.0-63.

Galvanized Fence and Supports: Chain link fence fabric conforming to ASTM A-392 shall be No. 9 gauge wire woven in a 2 inch mesh and galvanized after weaving with a Class II zinc coating. Galvanized steel pipe rail and fence supports shall conform to ASTM A-53 Grade B, Type S or E. ASTM A-36 steel shall be used for all structural steel and hardware, and after fabrication shall be galvanized in accordance with ASTM A-123 and A-153.

Footing Pressure: The required allowable soil pressure to use the stub abutment as detailed but without piles is 1 Ton per square foot.

Pedestal Pile: The drilled hole shall be cleaned of all loose material and the pedestal pile poured in compact coarse and fine sand, medium stiff clay or better.

Lateral Soil Pressure: Design has been based on an allowable lateral soil pressure of 300 Lb. per square foot per foot of depth.

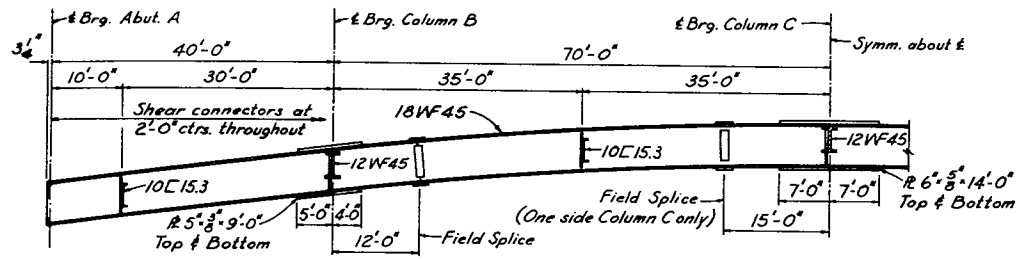
DO NOT SCALE

U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

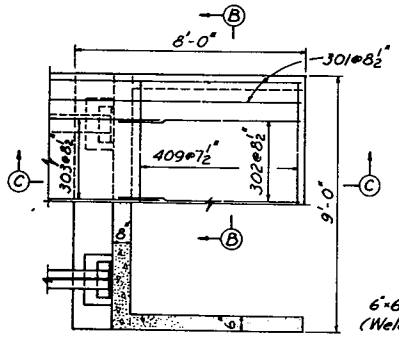
TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

FOUR SPAN STEEL BEAM BRIDGE
SPANS = 40-70-70-40 = 220 FT.
GENERAL PLAN AND ELEVATION

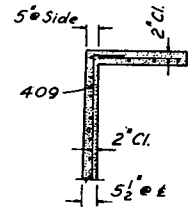
RECOMMENDED S. B. [signature] Chief Design Branch	RECOMMENDED [signature] Director of Engineering	OCTOBER 1964
RECOMMENDED [signature] Chief Bridge Division	APPROVED [signature] Administrator	SHEET NO. 401



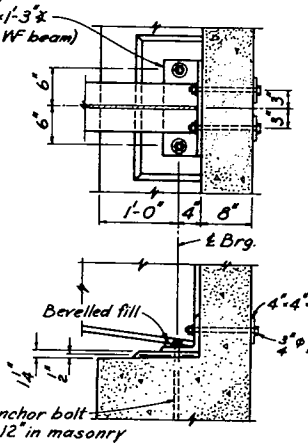
BEAM DETAILS



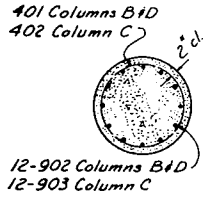
HALF PLAN HALF SECTION



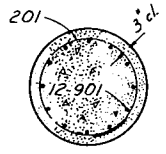
SECTION B-B



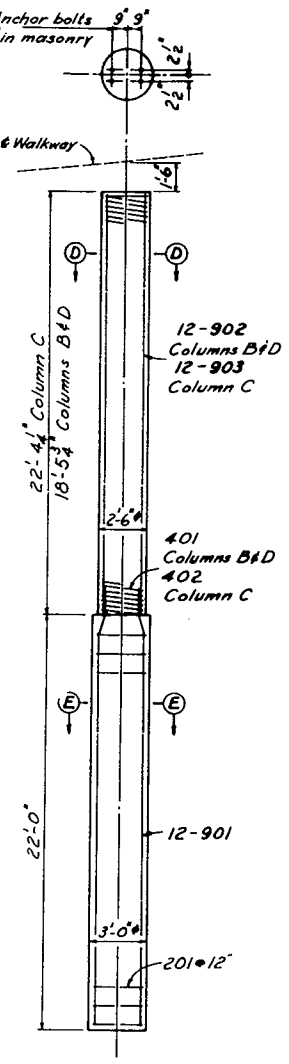
BEARING DETAIL



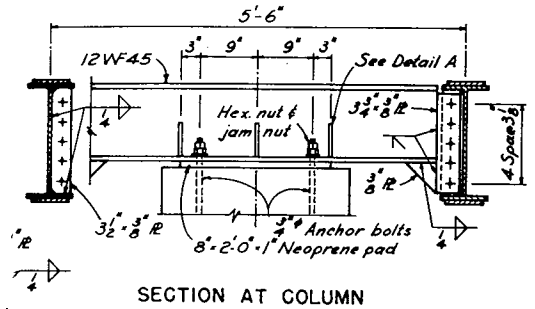
SECTION D-D



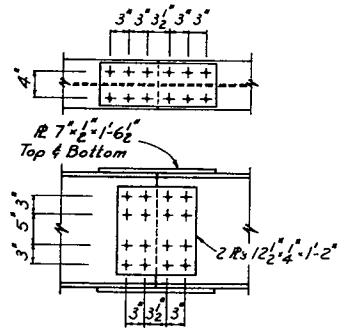
SECTION E-E



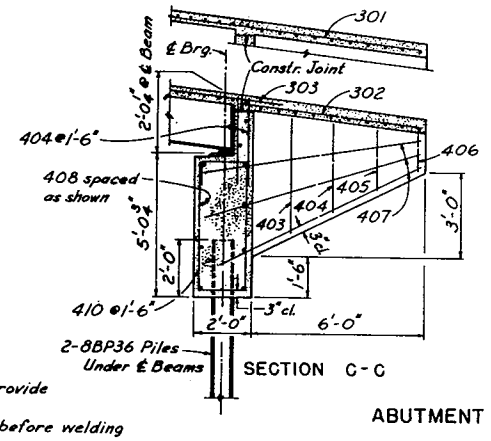
PIER DETAILS



SECTION AT COLUMN

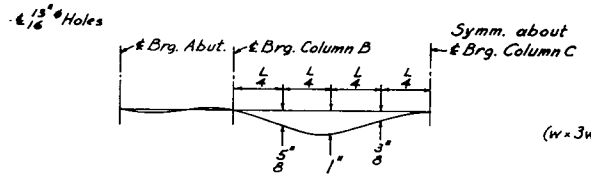


FIELD SPLICE

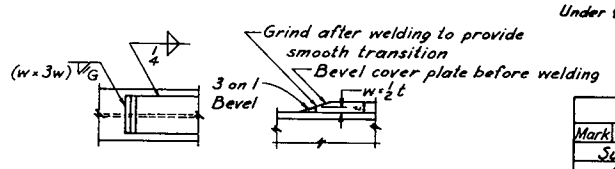


SECTION C-C

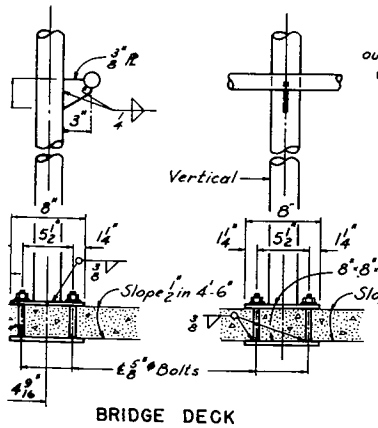
ABUTMENT DETAILS



DEAD LOAD DEFLECTION

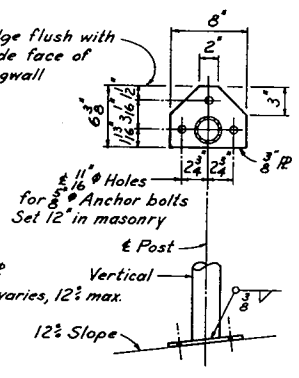


COVER PLATE WELDING DETAIL

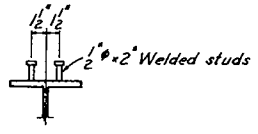


BRIDGE DECK

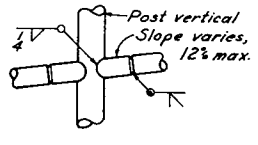
POST BASE ASSEMBLY



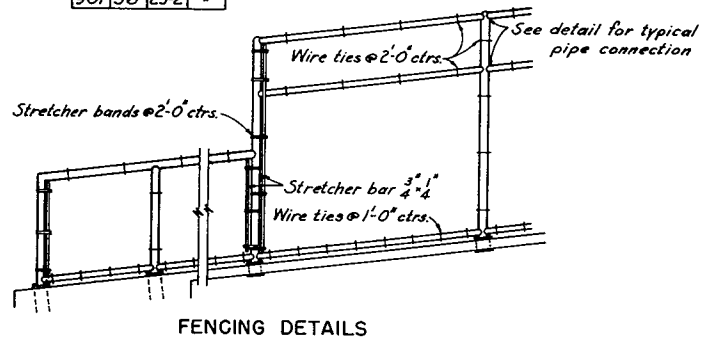
ABUTMENT



SHEAR CONNECTOR DETAIL



TYPICAL PIPE CONNECTION

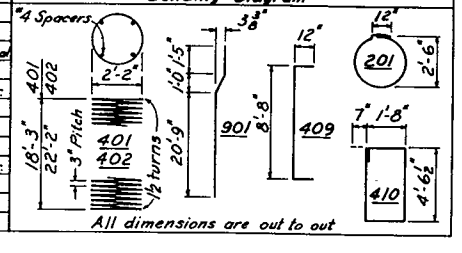


FENCING DETAILS

REINFORCING STEEL SCHEDULE

Mark	No.	Length	Shape	Mark	No.	Length	Shape
Substructure				Substructure			
Abutment				Column			
302	18	6'-4	Str.	401	2	51'8"	Spiral
403	4	4'-1"	"	402	1	62'4"	"
404	32	3'-2"	"	902	24	18'-3"	Str.
405	4	2'-3"	"	903	12	22'-2"	"
406	4	1'-4"	"	Superstructure			
407	12	7'-8"	"	Deck Slab			
408	20	8'-8"	"	301	24	39'-9"	Str.
409	20	10'-8"	Bent	303	54	36'-0"	"
410	14	13'-0"	"	304	444	6'-8"	"
Pedestal Pile							
201	66	8'-10	Bent				
901	36	23'-2	"				

Bending Diagram



All dimensions are out to out

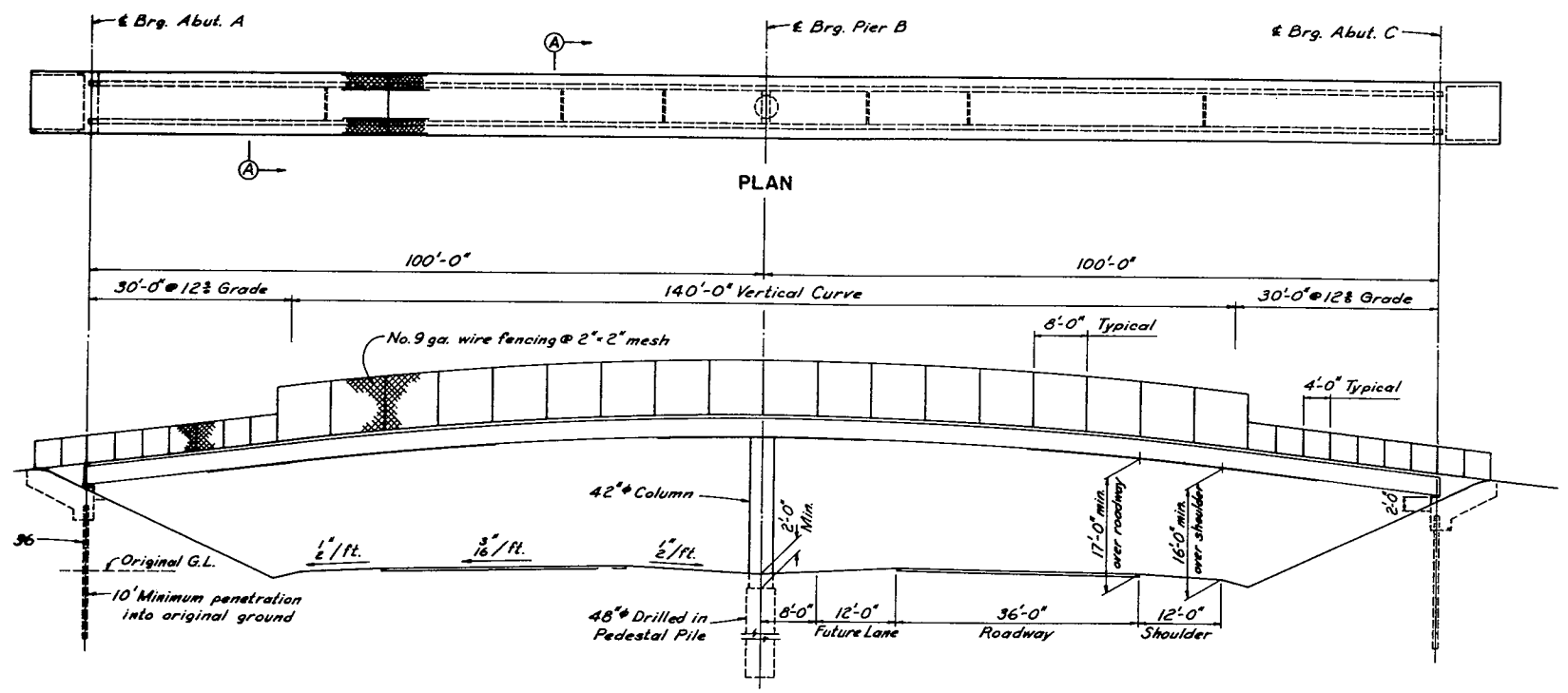
DO NOT SCALE

U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

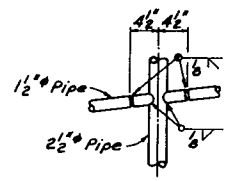
TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

FOUR SPAN STEEL BEAM BRIDGE
SPANS = 40-70-70-40 = 220 FT.
DETAILS

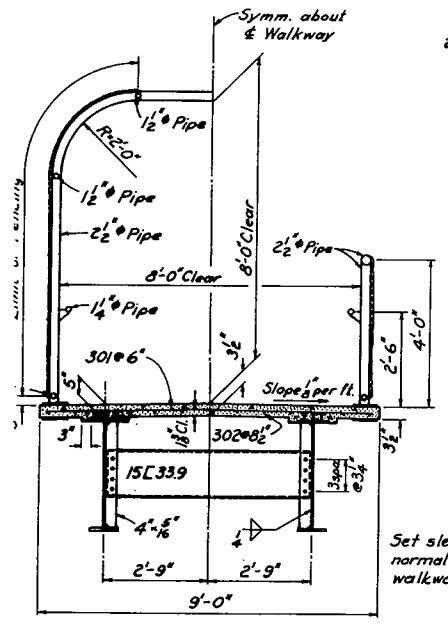
RECOMMENDED S. B. Loveless, Chief Design Branch	RECOMMENDED Director of Engineering	OCTOBER 1964
RECOMMENDED Chief Bridge Division	APPROVED Administrator	SHEET NO. 402



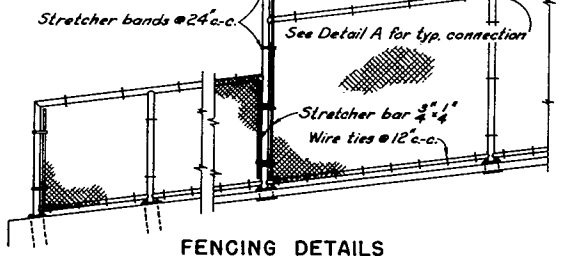
ELEVATION



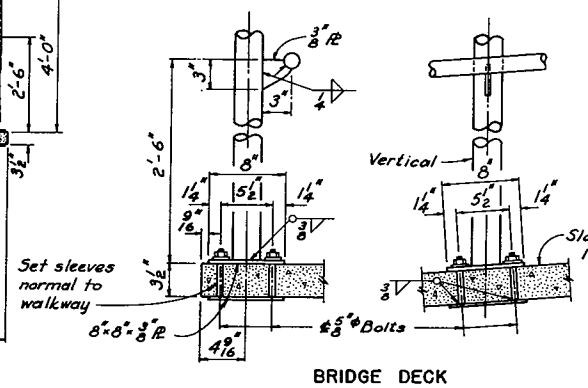
DETAIL A



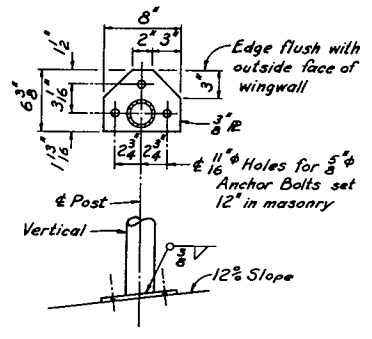
SECTION A-A



FENCING DETAILS



BRIDGE DECK

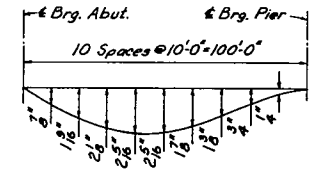


ABUTMENT

POST BASE ASSEMBLY

SUMMARY OF QUANTITIES		
Item	Unit	Quantity
Excavation for Structures	C.Y.	※ 40
48" ϕ Pedestal Pile	L.F.	※ 28
Steel H-Pile	L.F.	※ 78
Class A(AE) Concrete	C.Y.	43
Reinforcing Steel	Lb.	⊙ 5600
Structural Steel	Lb.	29720
Fencing (Including supports)	L.F.	216

※ Estimated
⊙ Includes wt. of spiral spacers.



D.L. DEFLECTION DIAGRAM

GENERAL NOTES

Design Criteria: Design Criteria for Pedestrian Bridges Over Interstate or Comparable Highways, Appendix A.

Design Specifications: AASHO Standard Specifications for Highway Bridges, 1961, with tentative revisions to 1963 except as modified or amplified by the Design Criteria.

Structural Steel: All structural steel shall conform to ASTM A-36. Field connections shall be bolted using ⅞ inch diameter high strength bolts, nuts and washers conforming to ASTM A-325. Girder splices shall be subpunched and reamed assembled in the shop to proper camber. Girders shall be cambered for dead load deflection and where required, for additional permanent camber due to vertical curvature.

Concrete: All concrete shall be Class A(AE) with a minimum 28 day f'_c = 4000 psi. The air entraining agent shall meet with the approval of the engineer. When pouring the deck a retarder shall be used to delay the final set until the entire deck is in place.

Reinforcing Steel: Reinforcing steel shall be deformed intermediate hard or rail grade in accordance with ASTM A-15 or A-16. All bending dimensions are from out to out of the bar. Dimensions shown on the plans from the reinforcing steel to the outside edge of the concrete are all clear dimensions. Bar sizes are designated on the plans by numbers. The first digit of the three digit marks and the first two digits of the four digit marks indicate size of the bar.

Paint: All structural steel shall be given one shop coat of red lead iron oxide paint. Bolted joints and surfaces in contact with concrete shall not be painted. Surfaces inaccessible after erection shall be given three coats of paint in the shop. Field paint shall consist of one coat of tinted red lead iron oxide paint followed by one finish coat of aluminum or light green paint. All paint and workmanship shall conform to AASHO Specifications and Special Provisions for Paint, Appendix B.

Welding: All welding shall conform to the Specifications for Welded Highway and Railway Bridges of the American Welding Society – AWS D2.0-63.

Galvanized Fence and Supports: Chain link fence fabric conforming to ASTM A-392 shall be No. 9 gage wire woven in a 2 inch mesh and galvanized after weaving with a Class II zinc coating. Galvanized pipe rail and fence supports shall conform to ASTM A-53 Grade B, Type S or E. ASTM A-36 steel shall be used for all hardware, and after fabrication, shall be galvanized in accordance with ASTM A-123 and A-153. All welding and erection damage to the galvanizing shall be repaired with material conforming to Military Specification MIL-P-21035 (Ships).

Pedestal Piles: The drilled hole shall be cleaned of all loose material and the pedestal pile poured in compact coarse and fine sand, medium stiff clay or better.

Lateral Soil Pressure: Design has been based on an allowable lateral soil pressure of 300 Lbs. per square foot per foot of depth.

Soil Pressure: The required allowable soil pressure to use the stub abutments as detailed but without piles is 1 Ton per square foot.

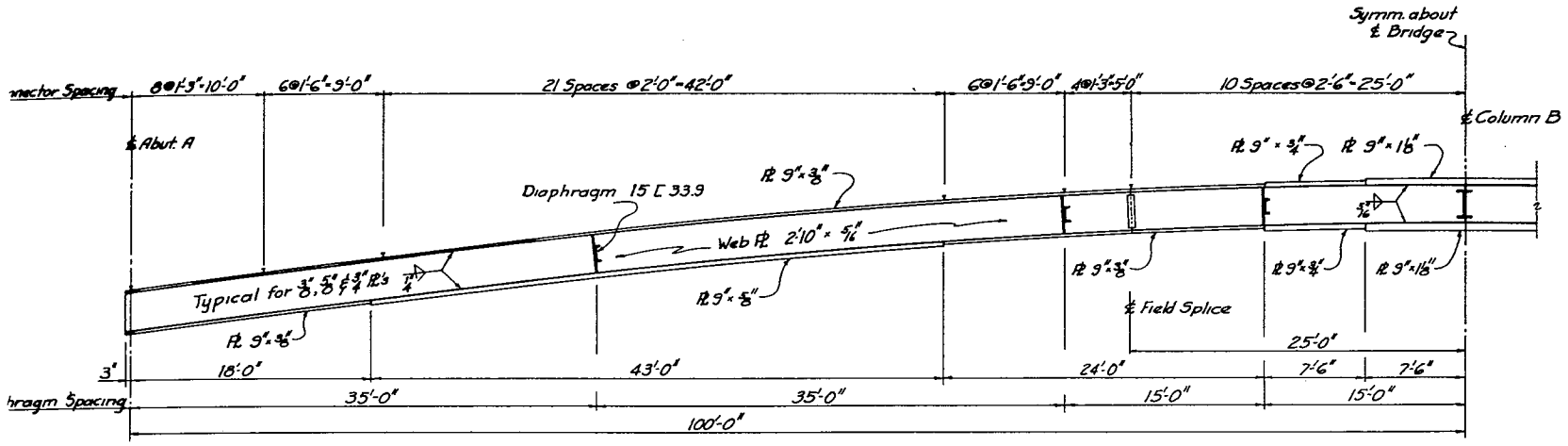
DO NOT SCALE

U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

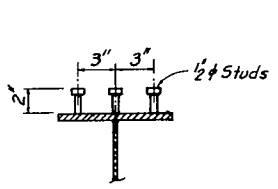
TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

TWO SPAN WELDED GIRDER BRIDGE
SPANS = 100 - 100 = 200 FT.
GENERAL PLAN AND ELEVATION

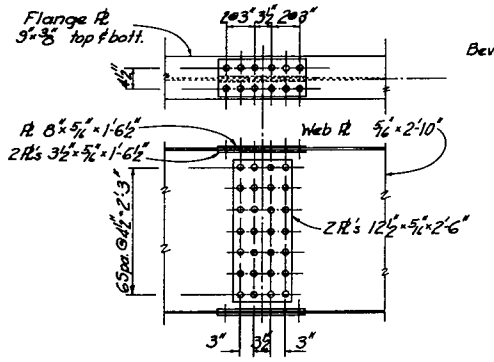
RECOMMENDED S. B. Jansen, Chief Design Branch	RECOMMENDED Director of Engineering	OCTOBER 1964
RECOMMENDED Chief Bridge Division	APPROVED Administrator	SHEET NO. 411



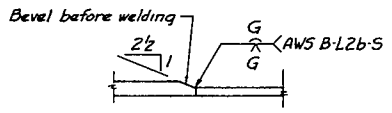
GIRDER DETAILS



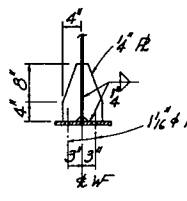
SHEAR CONNECTOR DETAIL



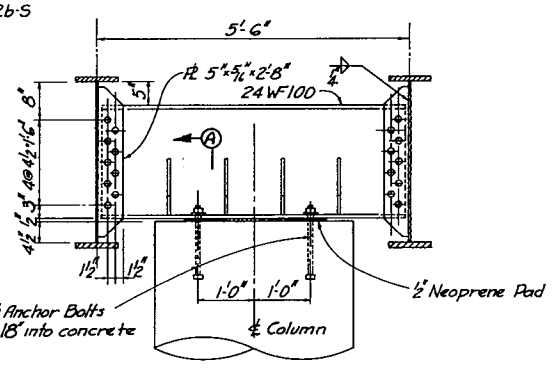
FIELD SPLICE



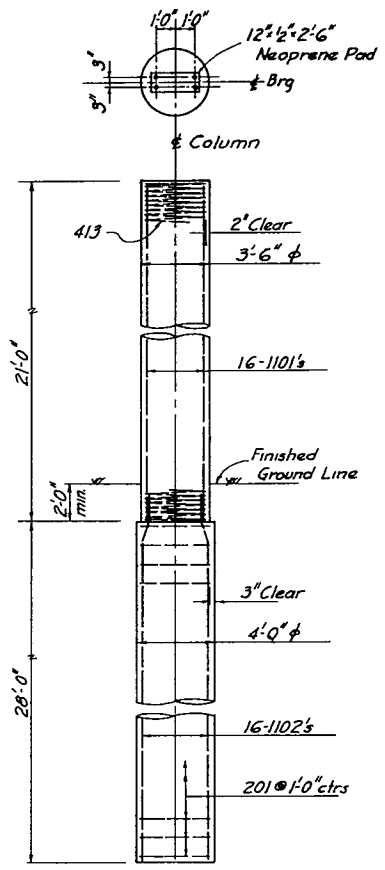
SHOP FLANGE SPLICE



SECTION A-A

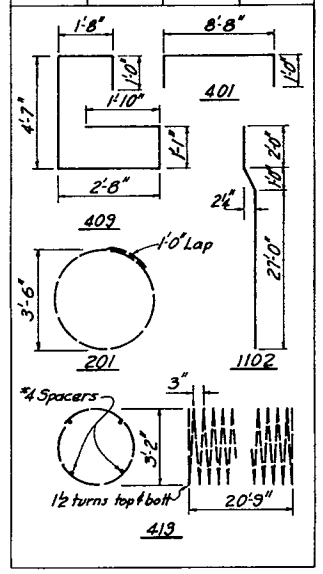


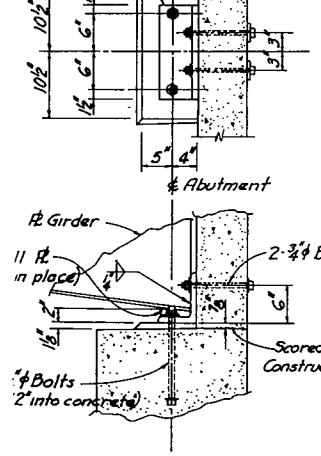
CAP BEAM DETAIL



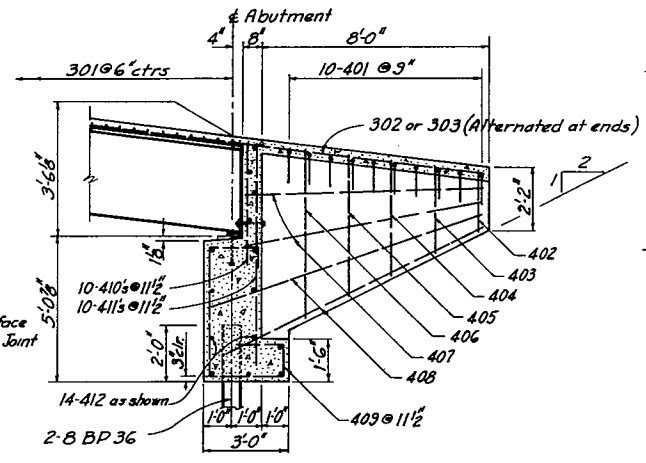
COLUMN DETAILS

REINFORCING STEEL SCHEDULE			
Mark	No.	Length	Shape
Deck			
301	401	8'-8"	Str.
302	91	30'-0"	"
303	13	15'-0"	"
Abutment			
401	20	10'-8"	Bent
402	4	2'-2"	Str.
403	4	3'-1"	"
404	4	4'-0"	"
405	4	4'-11"	"
406	4	5'-10"	"
407	8	8'-4"	"
408	8	10'-0"	"
409	20	12'-10"	Bent
410	20	4'-6"	Str.
411	20	8'-0"	"
412	28	8'-8"	"
Column			
413	1	858'-0"	Spiral
1101	16	20'-10"	Str.
Drilled Shaft			
201	29	12'-0"	Bent
1102	16	30'-0"	Bent





ABUTMENT BEARING DETAILS



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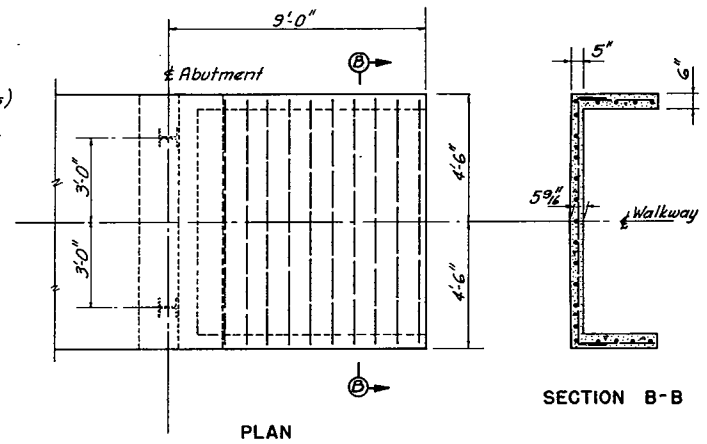


PLAN

SECTION B-B

ABUTMENT DETAILS

DO NOT SCALE

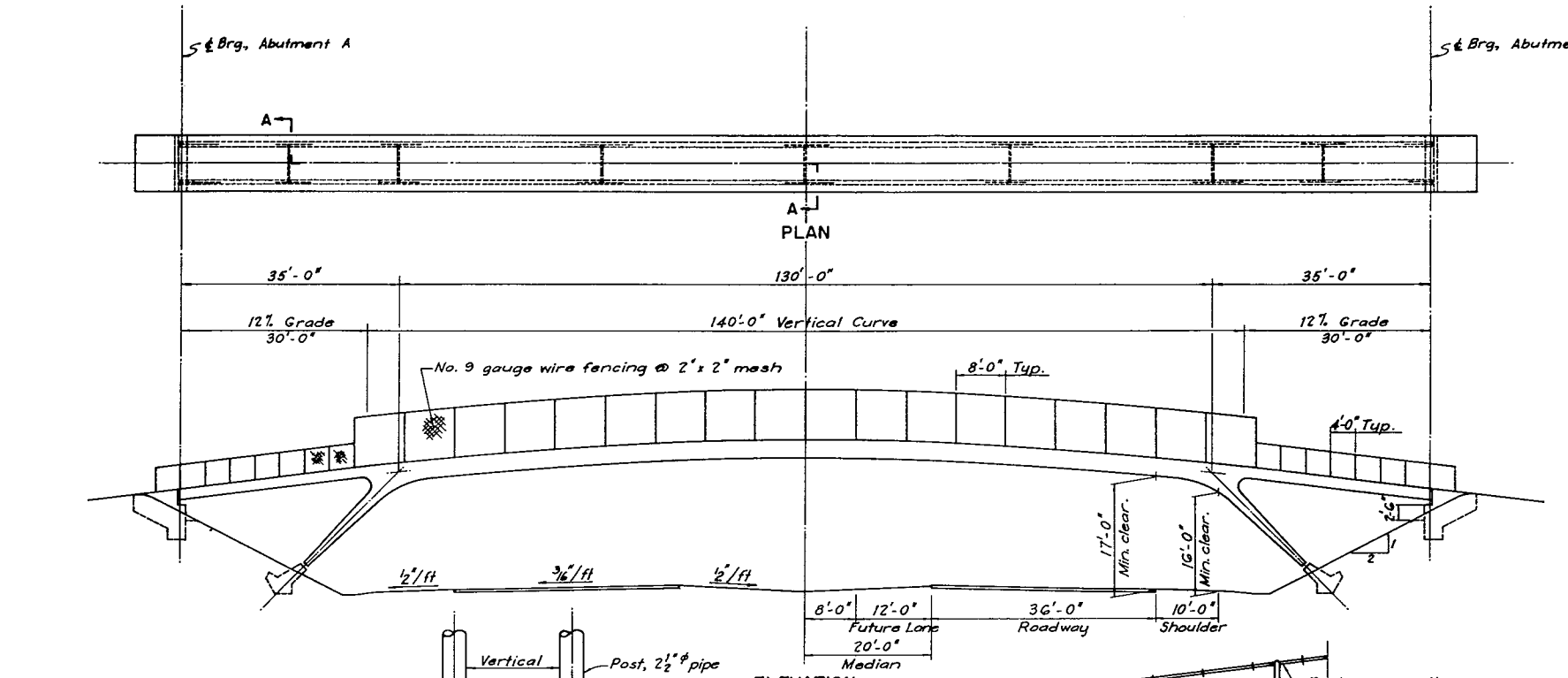
U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

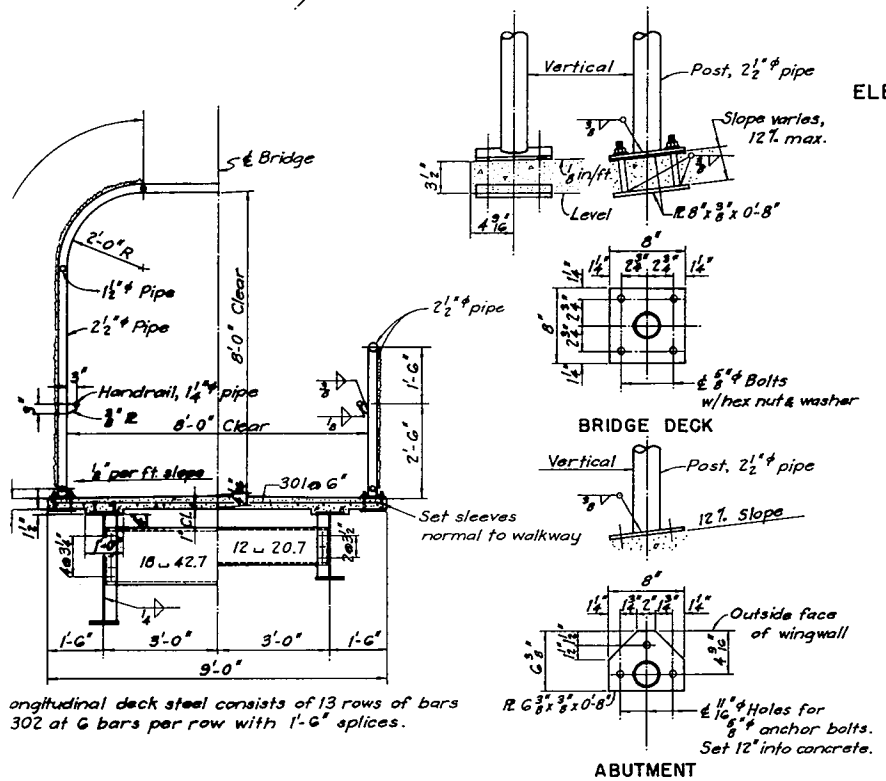
TWO SPAN WELDED GIRDER BRIDGE

SPANS = 100-100 = 200 FT.
DETAILS

RECOMMENDED S. B. Larsen, Chief Design Branch	RECOMMENDED [illegible], Director of Engineering	OCTOBER 1964
RECOMMENDED [illegible], Chief Bridge Division	APPROVED [illegible], Administrator	SHEET NO. 412



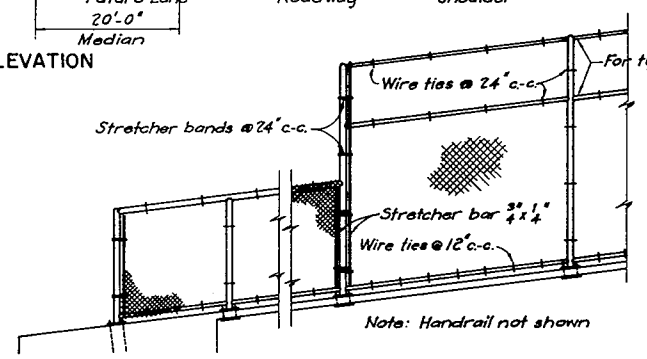
ELEVATION



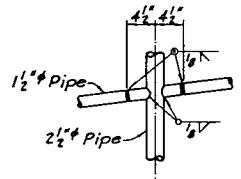
Longitudinal deck steel consists of 13 rows of bars 302 at 6 bars per row with 1'-6" splices.

SECTION A-A

POST BASE ASSEMBLY



FENCING DETAILS



DETAIL A

SUMMARY OF QUANTITIES		
Item	Unit	Quantity
Excavation for Structures (Estimated)	C.Y.	85
Class A (AE) Concrete	C.Y.	38
Reinforcing Steel	Lb.	3400
Structural Steel	Lb.	39050
Structural Metal	Lb.	1240
Fencing (Including Supports)	L.F.	208

DO NOT SCALE

GENERAL NOTES

Design Criteria: Design Criteria for Pedestrian Bridges over Interstate or Comparable Highways, Appendix A.

Design Specifications: AASHO Standard Specifications for Highway Bridges, 1961, with tentative revisions to 1963 except as modified or amplified by the Design Criteria.

Structural Steel: All structural steel shall be A-36 conforming to ASTM A-36. Field connections shall be with ⅞ inch diam. H.S. bolts. The high strength steel bolts, nuts, and washers in the girder splices shall conform to ASTM A-325. Girder splices shall be subpunched and reamed assembled in the shop to proper camber. Girders shall be cambered for dead load deflection, and vertical curvature. Top flanges shall be laterally supported until the concrete deck has set.

Concrete: Concrete shall be Class A(AE) with a minimum 28 day f'_c = 4000 psi. The air entraining agent shall meet with the approval of the engineer. When pouring the deck, a retarder shall be used that will delay the final set until the entire deck is in place.

Reinforcing Steel: Reinforcing steel shall be deformed bars of hard, intermediate, or rail grade conforming to ASTM Specifications A-15 or A-16.
Bar sizes are designated on the plans by numbers. The first digit in the three digit marks and the first two digits in the four digit marks indicate the size of the bar.
Dimensions shown on the plans from the reinforcing steel to the outside edge of the concrete are all clear dimensions. All bending dimensions are from out to out of the bar.

Paint: All structural steel shall be given one shop coat of red lead iron oxide paint. Bolted joints and surfaces in contact with concrete shall not be painted. Surfaces inaccessible after erection shall be given three coats of paint in the shop. Field paint shall consist of one coat of tinted red lead iron oxide paint followed by one finish coat of aluminum or light green paint. All paint and workmanship shall conform to AASHO Specifications and Special Provisions for Paint. See Appendix B.

Welding: All welding shall conform to the Specifications for Welded Highway and Railway Bridges of the American Welding Society, AWS D2.0-63.

Galvanized Fence and Supports: Chain link fence fabric shall be No. 9 gauge wire conforming to ASTM A-392 woven in a 2 inch mesh and galvanized after weaving with a Class II zinc coating. Galvanized steel pipe rail and fence supports shall conform to ASTM A-53 Grade B, Type S or E. ASTM A-36 steel shall be used for all structural steel and hardware, and after fabrication shall be galvanized in accordance with ASTM A-123 and A-153.

Footing Pressure: The maximum allowable soil pressure for design of stub abutments has been assumed to be 1 ton psf. The maximum allowable rock pressure for design of frame abutments has been assumed to be 5 tsf.

Pedestal Piles: The drilled hole shall be cleaned of all loose material and the pedestal pile poured in compact coarse and fine sand, medium stiff clay, or better.

Lateral Soil Pressure: Design has been based on an allowable lateral soil pressure of 300 psf per foot of depth.

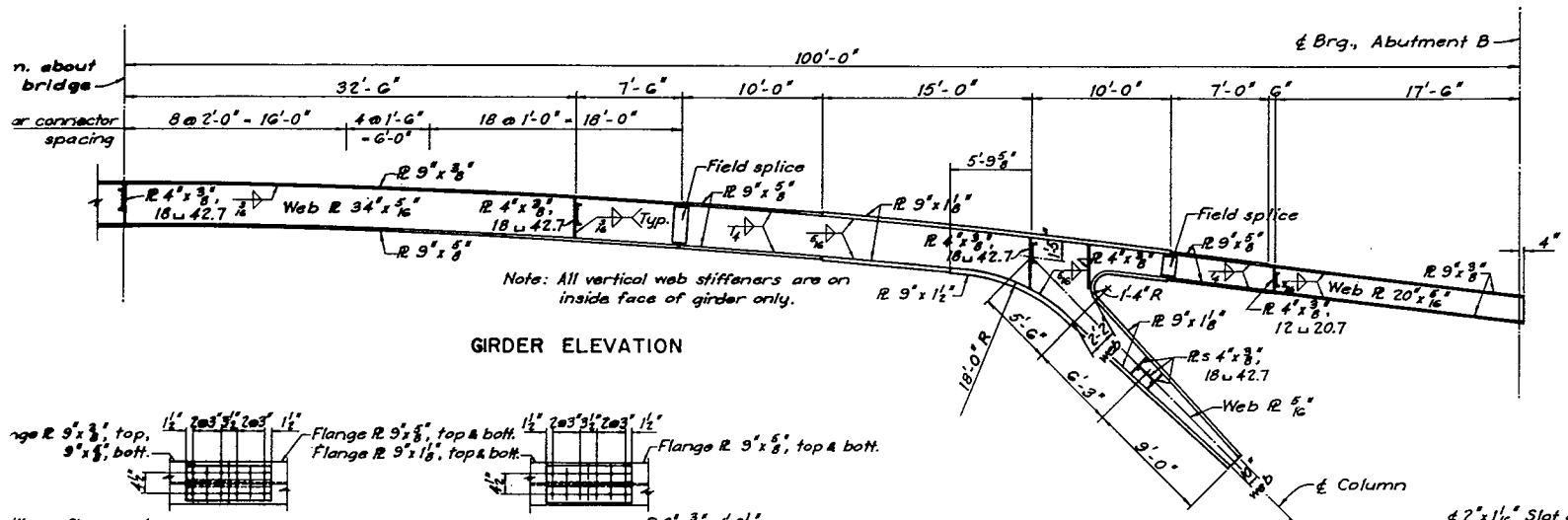
U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

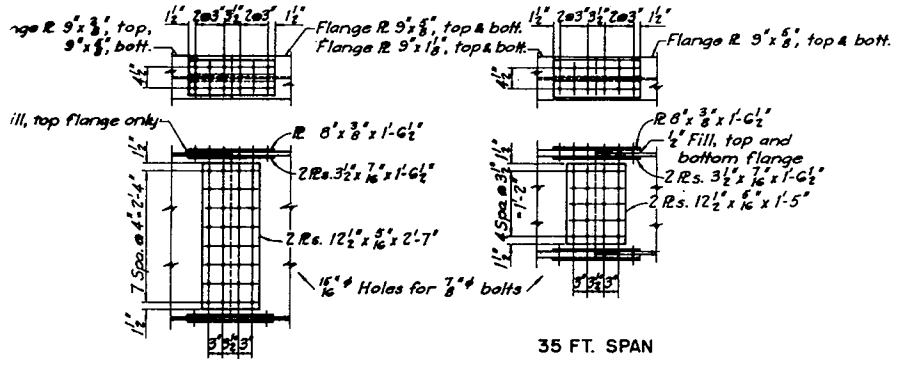
WELDED GIRDER RIGID FRAME BRIDGE
WITH CONCRETE DECK

GENERAL PLAN AND ELEVATION

RECOMMENDED S. B. Barnes, Chief Design Branch	RECOMMENDED, Director of Engineering	OCTOBER 1964
RECOMMENDED, Chief Bridge Division	APPROVED, Administrator	SHEET NO. 421



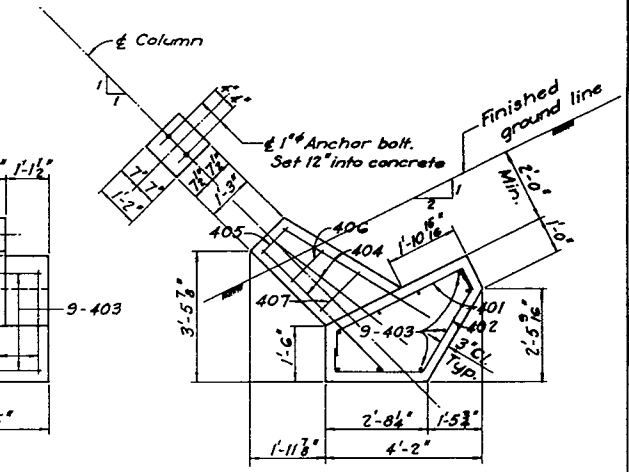
GIRDER ELEVATION



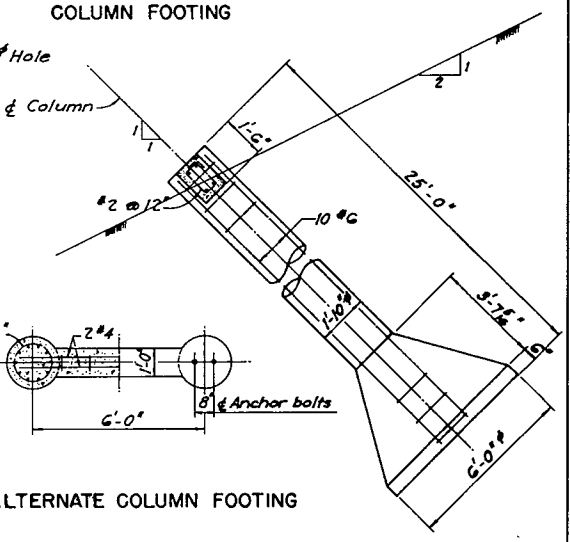
130 FT. SPAN

35 FT. SPAN

FIELD SPLICE DETAILS

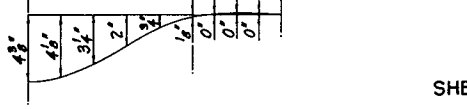


COLUMN FOOTING



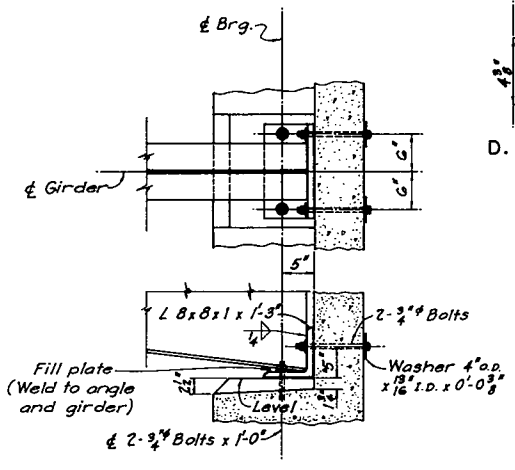
ALTERNATE COLUMN FOOTING

COLUMN BEARING DETAILS

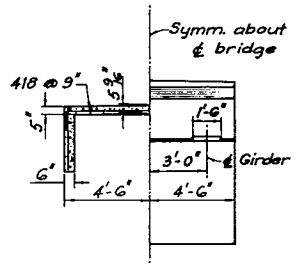


D. L. DEFLECTION DIAGRAM

SHEAR CONNECTOR DETAILS

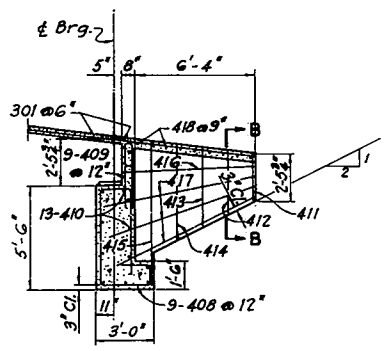


ABUTMENT BEARING DETAILS



HALF SECTION B-B

HALF ELEVATION



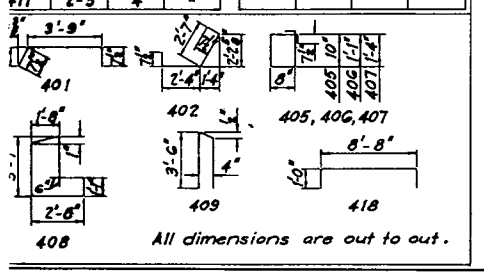
ABUTMENT DETAILS

DO NOT SCALE

REINFORCING STEEL SCHEDULE

Mark	Length	No.	Shape
SUBSTRUCTURE			
Column Footing			
401	5-0	16	Bent
402	6-2	16	"
403	2-11	36	Str.
404	4-6	24	"
405	4-3	4	Bent
406	4-9	4	"
407	5-3	4	"
Abutment			
408	16-5	18	Bent
409	7-4	18	"
410	8-8	26	Str.
411	2-3	4	"

Mark	Length	No.	Shape
SUBSTRUCTURE			
Abutment			
412	3-1	4	Str.
413	3-11	4	"
414	4-9	4	"
415	7-4	4	"
416	6-9	4	"
417	8-0	12	"
418	10-8	18	Bent
SUPERSTRUCTURE			
Deck Slab			
301	8-8	403	Str.
302	37-0	78	"



All dimensions are out to out.

U. S. DEPARTMENT OF COMMERCE
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TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

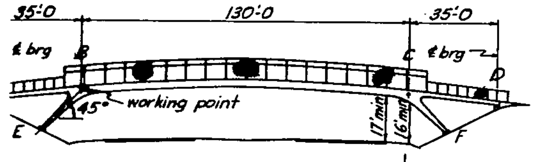
WELDED GIRDER RIGID FRAME BRIDGE
WITH CONCRETE DECK

DETAILS

RECOMMENDED S. B. Lauer, Chief Design Branch
RECOMMENDED Director of Engineering
RECOMMENDED Chief Bridge Division
APPROVED Administrator

OCTOBER 1964

SHEET NO. 422

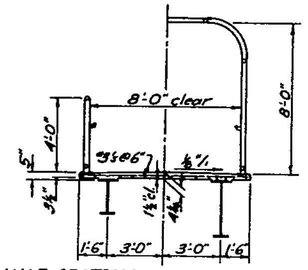


DATA:
: 35'-130'-35' = 200'
lear walkway
te deck

ications:
HO with revisions to 1963
modified by the "Design
eria."

allowable stresses:

steel	f_s = 20,000 psi	
forcing	f_s = 20,000 psi	
rete	f'_c = 4,000 psi	
	f_c = 1,600 psi	
	n = 8	

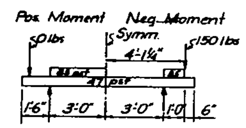


HALF SECTION IN END SPAN — HALF SECTION NEAR MIDSPAN

ESIGN:

load	85 psf
d load	47 psf

umed concentrated fence
d for pos. mom. = 0 lbs
for neg. mom. = 150 lbs



Properties:

Positive Moment
d = 2.37"; A_s = 0.22"/'

rea	y	A·y	I_o + Ay²
22 = 1.76	2.37	4.17	9.90
70 = 8.40	0.35	2.94	1.37
A = 10.16		Σ = 7.11	11.27

ȳ = 0.70" −4.98
I = 6.29 in⁴

Negative Moment
d = 1.56"; A_s = 0.22"/'

Area	y	A·y	I_o + Ay²
8 × 0.22 = 1.76	1.56	2.74	4.28
12 × 0.54 = 6.49	0.27	1.75	0.63
ΣA = 8.25		Σ = 4.49	4.91

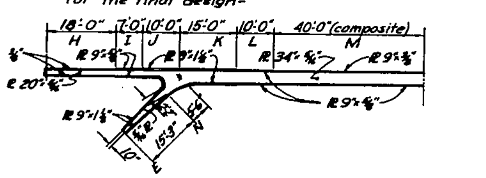
ȳ = 0.54" −2.42
I = 2.49 in⁴

oment: +541'# ; −266'#

	Conc.	Steel		Conc.	Steel
Mod.	9.00 in³	0.47 in³	S.M.	4.61 in³	0.31 in³
es (f = M/S)	723 psi	13,810 psi	stress	692 psi	10,300 psi

gitudinal steel @ 67% = 0.67 × 0.22 = 0.147 "/' - use #3's @ 8" c-c

ESIGN: By trial the following sections have been selected for the final design-



girder
on-composite structure-

ncrete deck	224 #/'
anges (2@ = 20%)	40
eb (1@ = 36%)	36
isc. metal	12
dead load per ft. =	312 #/'

On composite structure-

Fence (½ × 55 #/')	28 #/'
Live load (4' × 60 #/')	240 #/'

Properties:

section	web	top flg.	bot. flg.	I_{web}	$I_{flanges}$	I	S.M.
H	20" × 5/16"	9" × 5/8"	9" × 5/8"	208	703	911 in⁴	88 in³
I	"	9" × 5/8"	9" × 5/8"	208	1191	1399 in⁴	131 in³
J	"	9" × 1⅛"	9" × 1⅛"	208	2260	2468 in⁴	222 in³
K	34" × 5/16"	9" × 1⅛"	9" × 1⅛"	1022	6250	7272 in⁴	400 in³
L	"	9" × 5/8"	9" × 5/8"	1022	3380	4402 in⁴	250 in³
M	"	9" × 3/8"	9" × 5/8"	see	below		
N	26" × 5/16"	9" × 1⅛"	9" × 1⅛"	458	3725	4183 in⁴	296 in³
E	10" × 5/16"	9" × 1⅛"	9" × 1⅛"	26	628	654 in⁴	107 in³

ion M-
n-composite:

rea	y	A·y	I_o + Ay²
3.38	4.81	16.25	78
5.63	39.31	221.00	8690
0.62	22.00	234.00	6172
9.63		ΣAy = 471.25	14 940

ȳ = 24.00" −11 310
I = 3630 in⁴

steel) = 187 in³
steel) = 236 in³

Composite:

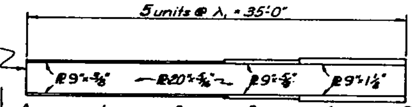
	Area	y	A·y	I_o + Ay²
steel	19.63		471	14940
concrete	18.38	1.75	32	75
ΣA =	38.01		ΣAy 503	15 015

ȳ = 13.25" −6 650
I = 8365 in⁴

S.M. (top of conc.) = 5050 in³
S.M. (top of steel) = 970 in³
S.M. (bot. of steel) = 318 in³

BEAM PROPERTIES: General reference- Elastic Properties of Beams by Newmark's Numerical Procedure, from ASCE Transactions, 1943, paper #2202, page 1171, except that the "assumed I" is determined by straight line variation of length λ/2 each side of a change of section.

A. END SPAN-



	Common Factor	A		1		2		3		4		B
"Assumed I"		911		911		911		1364		2390		2468
I' = I/I₁	I_{01}	1		1		1		1.50		2.62		2.71
Moment, M	$M_B/5$	0		1		2		3		4		5
Angle change = −M/EI	$M_B/5EI_{01}$	0		−1		−2		−2		−1.53		−1.85
Assumed ave. slope, θ	$M_B\lambda_1/5EI_{01}$		5		4		2		0		−1.53	
Trial defl., y	$M_B\lambda_1^2/5EI_{01}$	0		5		9		11		11		9.47
Linear Corr.	"	0		−1.89		−3.80		−5.69		−7.58		−9.47
Deflection, y	"	0		3.11		5.20		5.31		3.42		0
End slope, θ	$M_B\lambda_1/5EI_{01}$											3.42 + ⅙(2 × 1.85 + 1.53) = 4.29

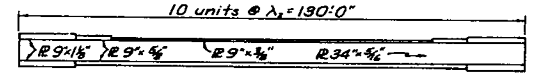
Stiffness factor, $k = M_B/\theta_B = M_B \cdot \frac{5EI_1}{4.29 M_B \lambda} = 5.83\ EI_1/L_1$

F.E.M. coef. for $M_B = P \cdot y/\theta_B$

F.E.M. coef.	PL	0	0.145	0.242	0.248	0.159	0

F.E.M. @ B due to uniform load = ΣF.E.M. ÷ 5 = 0.159 wL₁²

B. INTERIOR SPAN-
Non-composite



	B	1	2	3	4	5	6	7	8	9	C
I	7272	6281	3957	3630	3630	3630	3630	3630	3957	6281	7272
I'	1	.865	.544	.5	.5	.5	.5	.5	.544	.865	1
M	0	1	2	3	4	5	6	7	8	9	10
−M/EI'	0	−1.16	−3.68	−6	−8	−10	−12	−14	−14.70	−10.40	−10
θ		30.97	29.81	26.13	20.13	12.13	2.13	−9.87	−23.88	−38.57	−48.97
y	0	30.97	60.78	86.90	107.03	119.16	121.29	111.42	87.54	48.97	0

end θ: 31.16 = 30.97 + ⅙(2.0 + 1.16); 48.97 + ⅙(2 × 10 + 10.4) = 54.04

Carry over factor = 31.16/54.04 = 0.575; $k = \frac{1}{\theta_C - C.O. \cdot \theta_B} = 2.77\ EI_2/L_2$
F.E.M. coef. = k(y − C.O. × y')

	B	1	2	3	4	5	6	7	8	9	C
k·y	0	.0858	.1683	.2405	.2965	.3305	.3360	.3085	.2425	.1356	0
k·C.O.·y'	0	−.0779	−.1395	−.1774	−.1930	−.1900	−.1709	−.1385	−.0968	−.0493	0
F.E.M. ÷ PL	0	.0079	.0288	.0631	.1035	.1405	.1651	.1700	.1457	.0863	0

F.E.M. due to uniform load = ΣF.E.M. ÷ 10 = 0.091 wL₂²

Composite

	B	1	2	3	4	5	6	7	8	9	C
I	7272	6281	6692	8365	8365	8365	8365	8365	6692	6281	7272
I'	1	.864	.919	1.15	1.15	1.15	1.15	1.15	.919	.864	1
M	0	1	2	3	4	5	6	7	8	9	10
−M/EI'	0	−1.16	−2.18	−2.61	−3.48	−4.34	−5.21	−6.09	−8.71	−10.40	−10
θ		15.57	14.40	12.23	9.61	6.14	1.80	−3.42	−9.50	−18.22	−28.61
y	0	15.57	29.97	42.20	51.81	57.95	59.75	56.33	46.83	28.61	0

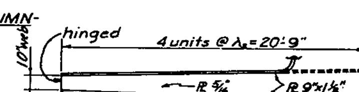
end θ: 15.76 = 15.57 + ½(2.0 + 1.16); 28.61 + ½(2 × 10 + 10.40) = 33.68

C.O. = 15.76/33.68 = 0.468; $k = \frac{1}{\theta_C - C.O. \cdot \theta_B} = 3.80\ EI_2/L_2$

	B	1	2	3	4	5	6	7	8	9	C
k·y	0	.058	.114	.160	.197	.220	.227	.214	.178	.109	0
k·C.O.·y'	0	−.051	−.083	−.100	−.106	−.103	−.092	−.075	−.053	−.028	0
F.E.M. ÷ PL	0	.007	.031	.060	.091	.117	.135	.139	.125	.081	0

F.E.M. due to uniform load = ΣF.E.M. ÷ 10 = 0.079 wL₂²

C. SLOPING COLUMN-



	E		1		2		3		B
I	654		1497		2721		4345		6413
I'	1		2.285		4.16		6.64		9.80
M	0		1		2		3		4
−M/EI'	0		−.438		−.48		−.452		−.408
θ		.681		.243		.237		.687	
y	0		.681		.924		.687		0
end θ					0.687 + ½(2 × .408 + .452) = .898				

$k = M_B/\theta_B = 17.8\ EI_3/L_3$

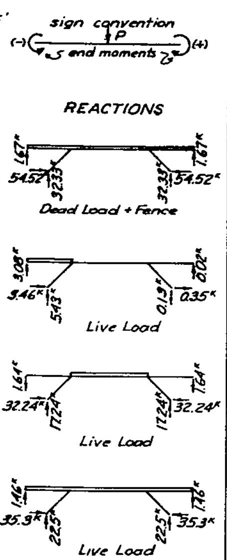
MOMENT DISTRIBUTION: (sidesway prevented)
Non-composite structure-
Fixed end moments

$M^F_{BA} = M^F_{CD} = 0.159 \times 0.312 \times 35^2 = 60.7^{K'}$
$M^F_{BC} = M^F_{CB} = 0.091 \times 0.312 \times 130^2 = 479.8^{K'}$

Joint		B			C	
Member	BA	BE	BC	CB	CF	CD
Stiffness	12.6E	46.8E	13.0E	13.0E	46.8E	12.6E
Dist. factor	0.17	0.65	0.18	0.18	0.65	0.17
Carry Over	0	0	0.575	0.575	0	0
C.O. × D.F.	—	—	−0.103	−0.103	—	—
F.E.M.	+60.7	0	−479.8	+479.8	0	−60.7
			−43.1	+43.1		
			−4.4	+4.4		
Σ	+60.7	0	−527.3	+527.3	0	−60.7
Distribution	+79.4	+303.3	+84.0	−84.0	−303.3	−79.4
Moment	+140.1	+303.3	−443.4	+443.4	−303.3	−140.1

Composite structure-

Joint		B			C	
Member	BA	BE	BC	CB	CF	CD
Stiffness	12.6E	46.8E	17.8E	17.8E	46.8E	12.6E
Dist. factor	0.16	0.61	0.23	0.23	0.61	0.16
Carry Over	0	0	0.468	0.468	0	0
C.O. × D.F.	—	—	−0.108	−0.108	—	—
F.E.M.	+100			−10.8		
			+11.7			
Σ	+100	0	+11.7	−10.8	0	0
Distribution	−16.2	−61.7	−23.3	+2.48	+6.58	+1.73
Moments	+83.8	−61.7	−22.13	−8.32	+6.58	+1.73
F.E.M.			−100	+10.8		
			−11.7			
Σ	0	0	−101.17	+10.8	0	0
Distribution	+16.2	+61.7	+23.3	−2.48	−6.58	−1.73
Moments	+16.2	+61.7	−77.87	+8.32	−6.58	−1.73



MOMENTS:

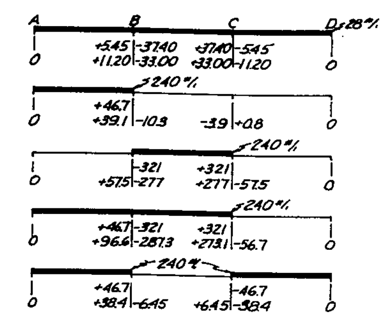
Fence loads:
Fixed end moments:
End moments: (kip-feet)

Live load:
Fixed end moments:
End moments: (kip-feet)

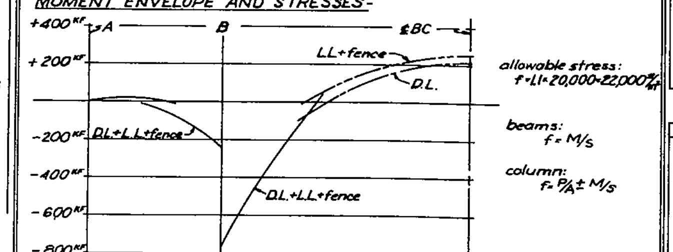
Live load:
Fixed end moments:
End moments: (kip-feet)

Live load:
Fixed end moments:
End moments: (kip-feet)

Live load:
Fixed end moments:
End moments: (kip



MOMENT ENVELOPE AND STRESSES-



allowable stress:
f = LL = 20,000 − 22,000 #/in²

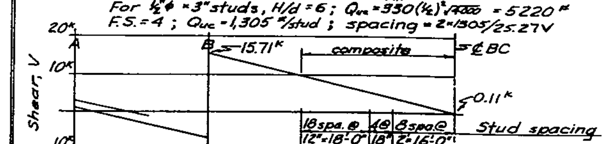
beams:
f = M/S

column:
f = P/A ± M/S

Location	Moment	Stresses: Concrete	top flg.	bot. flg.
BA	248ᴷᶠ	—	+13,000	−13,000
BC	760ᴷᶠ	—	+22,800	−22,800
℄ BC	D.L. = 220ᴷᶠ	—	−14,100	+11,200
	L.L. + fence = 260ᴷᶠ	−618	−3,200	+9,800
	total	−618	−17,300	+21,000
N	M = 399ᴷᶠ	—	+16,200	−16,200
	P = 96.1ᴷ	—	−5,260	−5,260
			+10,940	−21,460

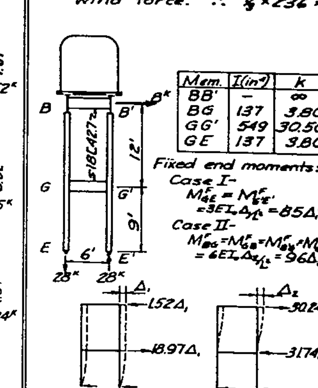
+ = tension
− = compression

LIVE LOAD SHEAR DIAGRAM:
Shear connector spacing (AASHO 1.9.5)
$S = Vm/I = V(211.0/8365) = 25.27\ V$ #/in
For ½"φ × 3" studs, H/d = 6; $Q_{uc} = 330(½)^2\sqrt{4000} = 5220$ #
F.S. = 4; Q_{uc} = 1,305 #/stud; spacing = 2 × 1305/25.27V



SLOPING BENT ANALYSIS:
Loads-
Wind − 45 psf
fence 0.12 × 45 × 8' = 43 #/'
superstructure 1.30 × 45 × 3.3' = 193 #/'
236 #/'

From a horizontal flexure analysis with the sloping bents and superstructure acting together, the bents take approximately ⅓ and abutments ⅔ of the total wind force. ∴ ⅓ × 236 × 100' = 8 ᴷ/bent



Mem.	I (in⁴)	k
BB'	—	∞
BG	137	3.80
GG'	549	30.50
GE	137	3.80

Fixed end moments:
Case I-
$M^F_{GE} = M^F_{G'E'} = 3EI_2\Delta_1/L^2 = 8.5\Delta_1^{K'}$
Case II-
$M^F_{BG} = M^F_{GB} = M^F_{B'G'} = M^F_{G'B'} = 6EI_2\Delta_2/L^2 = 9.6\Delta_2^{K'}$

Moment distribution:

Joint	B		G		
Member	BB'	BG	GB	GG'	GE
dist. factor	1	0	0.1	0.8	0.1
Case I					
F.E.M.					−8.5
dist.			+8.5	+68.0	+8.5
C.O.		+4.3		+34.0	
dist.	−4.3	0	−3.4	−27.2	−3.4
C.O.		−1.7		−13.6	
dist.	+1.7	0	+1.4	+11.0	+1.4
Moment	−2.6	+2.6	+6.5	+72.0	−78.5
Case II					
F.E.M.		−9.6	−9.6		
dist.	+9.6	0	+9.6	+76.8	+9.6
C.O.		+4.8		+38.4	
dist.	−4.8	0	−3.8	−30.8	−3.8
C.O.		−1.9		−15.4	
dist.	+1.9	0	+1.5	+12.4	+1.5
C.O.		+0.7		+6.2	
dist.	−0.7	0	−0.6	−5.0	−0.6
Moment	+92.4	−92.4	−89.5	+82.6	+6.7

−1.52Δ₁ + 30.24Δ₂ = 8ᴷ
18.97Δ₁ − 31.74Δ₂ = 0
∴ Δ₁ = 0.483"; Δ₂ = 0.288"

	BB'	BG	GB	GG'	GE
I × 0.483	−1.3	+1.3	+3.1	+34.8	−37.9
II × 0.288	+26.6	−26.6	−25.7	+23.8	+1.9
Moments	+25.3	−25.3	−22.6	+58.6	−36.0

Stress at point G':
P = 28 + 61.4 = 89.4ᴷ
M_L = 9/20.75 × 325.1 = 141ᴷᶠ
M_T = 36.0ᴷᶠ

$f = P/A \pm M_L c/I_{(long)} \pm M_T c/I_{(tran)}$

$\therefore f_{G'} = \frac{89.4}{26.35} + \frac{141 \times 12 \times 16.3''}{2356} + \frac{36 \times 12 \times 4.5}{137}$

f = 24.99 ksi < 1.45 × 20 = 29 ksi

DEAD LOAD DEFLECTIONS:
A. Deflection of joints B & C due to axial loads-

Shortening of center span: $\delta_B = \frac{PL}{AE} = \frac{54.52^K \times 65' \times 12}{19.63 \times 29{,}000} = 0.0747''$

Shortening of column leg: $\delta_L = \frac{61.4^K \times 20.75 \times 12}{26.8 \times 29{,}000} = 0.0197''$

downward defl. - positive

$\Delta_{VB} = \delta_B + 1.4\delta_L = +0.1042''$

B. End span deflection-
Dead load = 312 #/' on non-composite section

	Factor	A	1	2	3	4	B
Moment	kip feet	0	+2.58	−10.14	−38.16	−81.48	−140.10
−M/EI	K·ft/EI₀₁		−2.58	+10.14	−25.40	+91.10	
θ	K·ft·λ₁/EI₀₁		−20.40	−22.98	−12.84	+12.56	+43.66
y	K·ft·λ₁²/EI₀₁	0	−20.40	−43.38	−56.22	−43.66	
y	inches	0	−0.0653	−0.1385	−0.1800	−0.1400	0
Fence load = 28 #/' on composite section							
y	inches	0	−0.0032	−0.0079	−0.0115	−0.0095	0
Deflection due to Δ_{VB}							
y	inches	0	+0.0208	+0.0417	+0.0625	+0.0833	+0.1042
Σy	inches	0	−0.0477	−0.1048	−0.1290	−0.0661	+0.1042
y		0	~	−⅛"	−⅛"	~	+⅛"

C. Center span deflection (sym. about ℄)
Dead load = 312 #/' on non-composite section

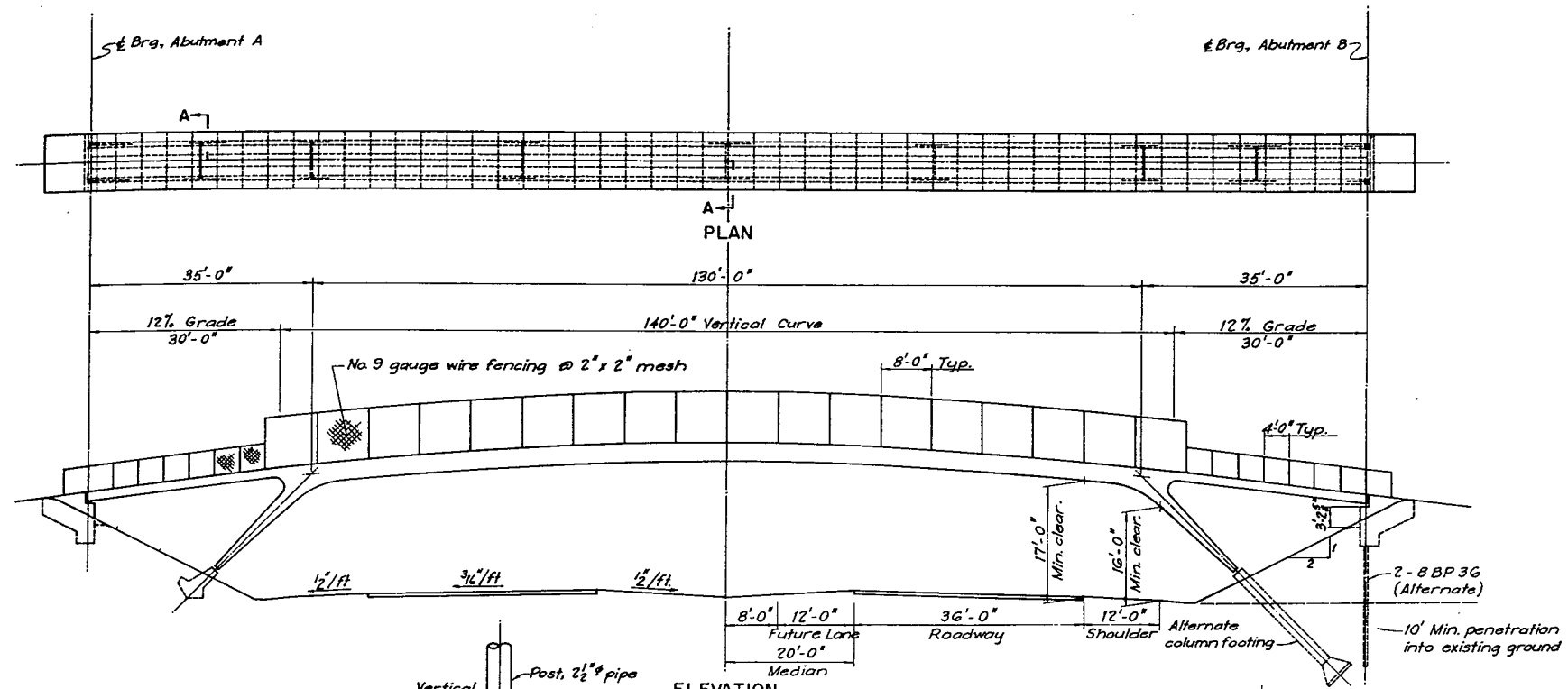
	Factor	B	1	2	3	4	℄
Moment	k·ft	−443.40	−205.40	−22.40	+110.60	+189.60	+215.60
−M/EI	k·ft/EI₀₂	+443.40	+237.60	+41.20	−221.20	−379.20	−431.20
θ	k·ft·λ₂/EI₀₂		+537.20	+774.80	+816.00	+594.80	+215.60
y	k·ft·λ₂²/EI₀₂	0	+537.20	+1312.00	+2128.00	+2722.80	+2938.40
y	inches	0	+0.74"	+1.82"	+2.94"	+3.78"	+4.06"
Fence load = 28 #/' on composite section							
y	inches	0	+0.05"	+0.12"	+0.19"	+0.23"	+0.25"
Deflection due to Δ_{VB}							
y	inches	+0.1042	+0.1042	+0.1042	+0.1042	+0.1042	+0.1042
Σy	inches	+0.1042	+0.8942	+2.0442	+3.2342	+4.1142	+4.4142
y		+⅛"	+⅞"	+2"	+3¼"	+4⅛"	+4⅜"

U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

WELDED GIRDER RIGID FRAME BRIDGE
WITH CONCRETE DECK
DESIGN CALCULATIONS

RECOMMENDED S. B. [signature] Chief Design Branch	RECOMMENDED [signature] Director of Engineering	OCTOBER 1964
RECOMMENDED [signature] Chief Bridge Division	APPROVED [signature] Administrator	SHEET NO. 423



GENERAL NOTES

Design Criteria: Design Criteria for Pedestrian Bridges over Interstate or Comparable Highways, Appendix A.

Design Specifications: AASHO Standard Specifications for Highway Bridges, 1961, with tentative revisions to 1963 except as modified or amplified by the Design Criteria.

Structural Steel: All structural steel shall be A-36 conforming to ASTM A-36. Field connections shall be with ⅞ inch diameter bolts. The high strength steel bolts, nuts, and washers in the girder splices shall conform to ASTM A-325. Girder splices shall be subpunched and reamed assembled in the shop to proper camber. Girders shall be cambered for dead load deflection, and where required, for additional permanent camber due to vertical curvature.

Concrete: Concrete shall be Class A(AE) with a minimum 28 day f'_c=4000psi. The air entraining agent shall meet with the approval of the engineer. When pouring the deck, a retarder shall be used that will delay the final set until the entire deck is in place.

Reinforcing Steel: Reinforcing steel shall be deformed bars of intermediate, hard, or rail grade conforming to ASTM Specifications A-15 or A-16.
Bar sizes are designated on the plans by numbers. The first digit in the three digit marks and the first two digits in the four digit marks indicate the size of the bar.
Dimensions shown on the plans from the reinforcing steel to the outside edge of the concrete are all clear dimensions. All bending dimensions are from out to out of the bar.

Paint: All structural steel shall be given one shop coat of red lead iron oxide paint. Bolted joints and surfaces in contact with concrete shall not be painted. Surfaces inaccessible after erection shall be given three coats of paint in the shop. Field paint shall consist of one coat of tinted red lead iron oxide paint followed by one finish coat of aluminum or light green paint. All paint and workmanship shall conform to AASHO Specifications and Special Provisions for Paint. See Appendix B.

Welding: All welding shall conform to the Specifications for Welded Highway and Railway Bridges of the American Welding Society, AWS D2.0-63.

Galvanized Fence and Supports: Chain link fence fabric shall be No. 9 gauge wire conforming to ASTM A-392 woven in a 2 inch mesh and galvanized after weaving with a Class II zinc coating. Galvanized steel pipe rail and fence supports shall conform to ASTM A-53 Grade B, Type S or E. ASTM A-36 steel shall be used for all structural steel and hardware, and after fabrication shall be galvanized in accordance with ASTM A-123 and A-153. All welding and erection damage to the galvanizing shall be repaired with material conforming to Military Specification MIL-P-21035 (Ships).

Footing Pressure: The maximum allowable soil pressure for design of stub abutments has been assumed to be 1 ton psf. The maximum allowable rock pressure for design of frame abutments has been assumed to be 5 tpsf.

Pedestal Piles: The drilled hole shall be cleaned of all loose material and the pedestal pile poured in compact coarse and fine sand, medium stiff clay, or better.

Lateral Soil Pressure: Design has been based on an allowable lateral soil pressure of 300 psf per foot of depth.

SUMMARY OF QUANTITIES		
Item	*Unit*	*Quantity*
Excavation for Structures (Estimated)	C.Y.	85
Class A(AE) Concrete	C.Y.	17
Reinforcing Steel	Lb.	1140
Structural Steel	Lb.	48660
Structural Metal	Lb.	1050
Fencing (Including Supports)	L.F.	208
Epoxy Wearing Surface	S.Y.	202

DO NOT SCALE

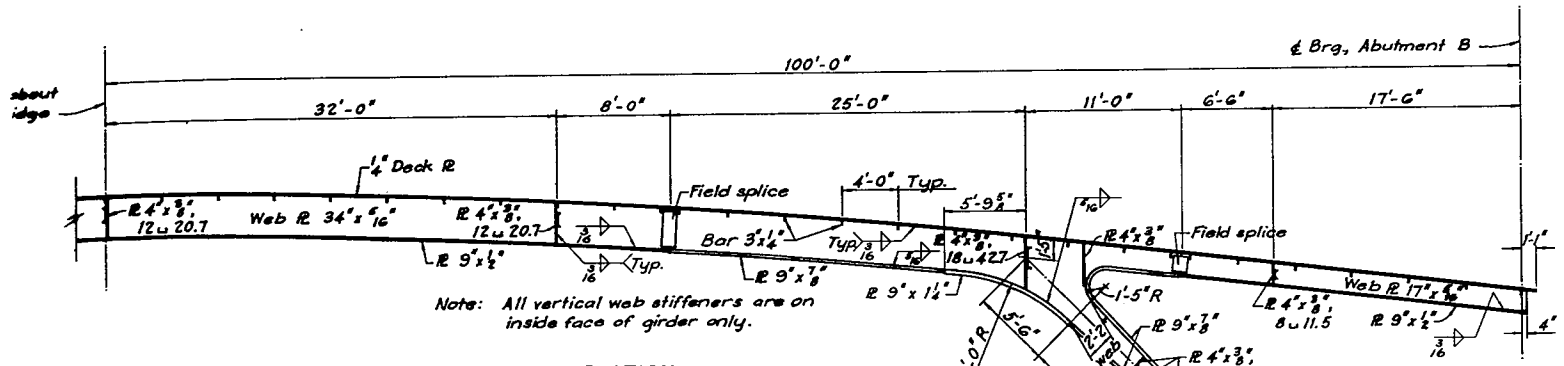
U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

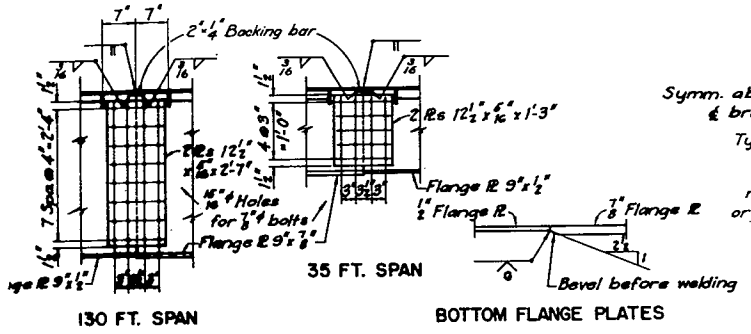
WELDED GIRDER RIGID FRAME BRIDGE
WITH ORTHOTROPIC PLATE DECK

GENERAL PLAN AND ELEVATION

RECOMMENDED S.B. Barnett, Chief Design Branch
RECOMMENDED [illegible], Chief Bridge Division
RECOMMENDED [illegible], Director of Engineering
APPROVED [illegible], Administrator
OCTOBER 1964
SHEET NO. 431



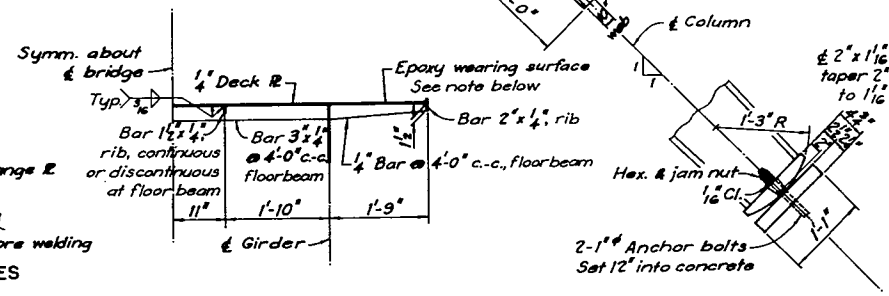
GIRDER ELEVATION



130 FT. SPAN

BOTTOM FLANGE PLATES

FIELD SPLICE DETAILS



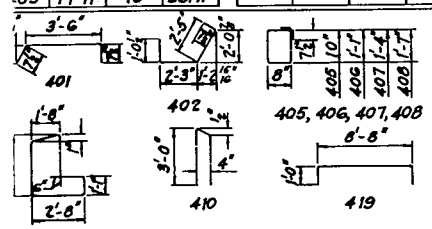
DECK DETAILS

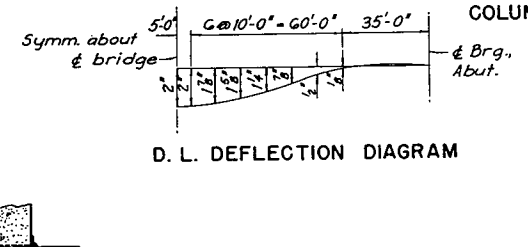
Note: A Type C coal-tar modified epoxy resin system with a surface aggregate application shall be used for the wearing surface. The system and aggregate shall conform to Interim Guide Specification AASHO M 200-631. The deck surface shall be clean and free of foreign material detrimental to bond of the epoxy resin. The rate of application of epoxy resin shall be in accordance with the manufacturer's recommendations. Application of the aggregate shall be at a rate of not less than 15 pounds per square yard.

REINFORCING STEEL SCHEDULE

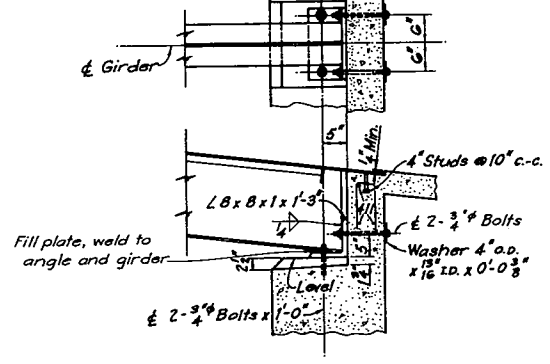
Mark	Length	No.	Shape
SUBSTRUCTURE			
Column Footing			
401	4-9	16	Bent
402	6-4	16	"
403	2-11	36	Str.
404	5-0	24	"
405	4-3	4	Bent
406	4-9	4	"
407	5-3	4	"
408	5-3	4	"
Abutment			
409	17-11	18	Bent

Mark	Length	No.	Shape
SUBSTRUCTURE			
Abutment			
410	6-4	18	Bent
411	8-8	30	Str.
412	2-3	4	"
413	3-1	4	"
414	3-11	4	"
415	4-9	4	"
416	7-4	4	"
417	6-9	22	"
418	8-0	12	"
419	10-8	18	Bent

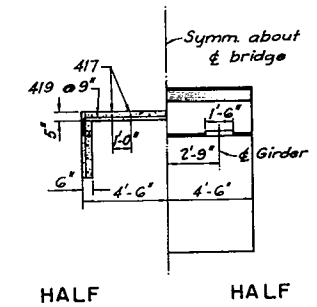




D. L. DEFLECTION DIAGRAM

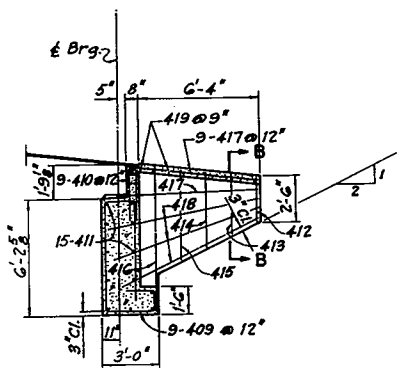


ABUTMENT BEARING DETAILS

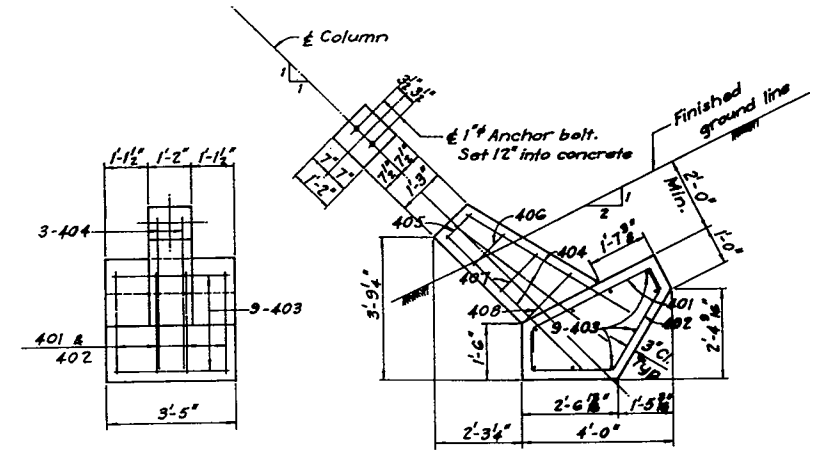


HALF SECTION B-B

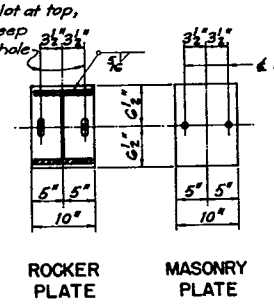
HALF ELEVATION



ABUTMENT DETAILS



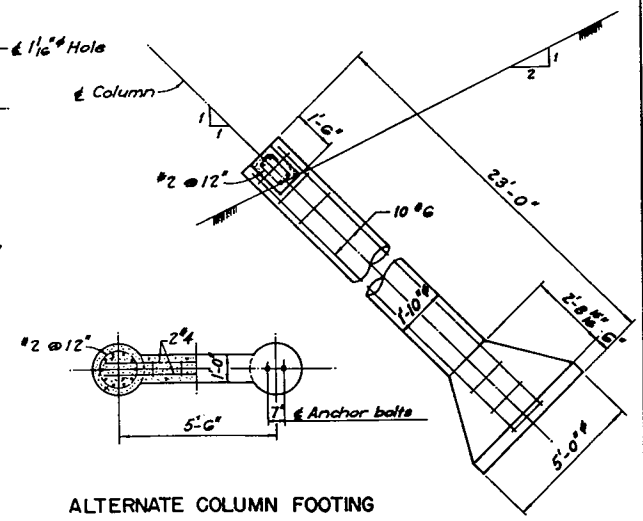
COLUMN FOOTING



ROCKER PLATE

MASONRY PLATE

COLUMN BEARING DETAILS



ALTERNATE COLUMN FOOTING

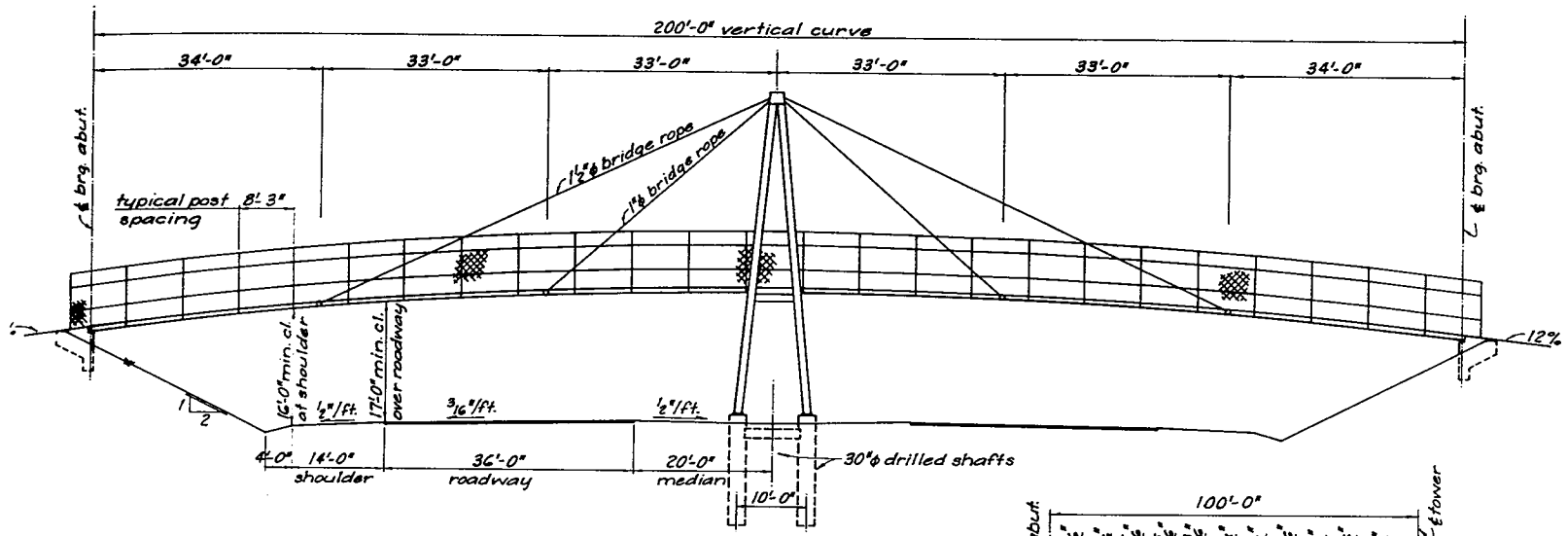
DO NOT SCALE

U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

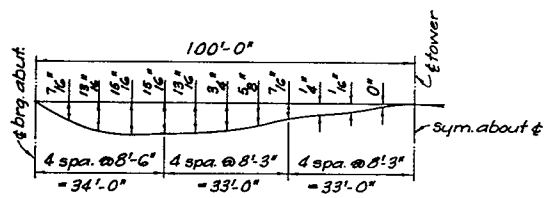
TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

WELDED GIRDER RIGID FRAME BRIDGE
WITH ORTHOTROPIC PLATE DECK
DETAILS

RECOMMENDED S. B. Barnes, Chief Design Branch	RECOMMENDED Bon Williams, Director of Engineering	OCTOBER 1964
RECOMMENDED ____, Chief Bridge Division	APPROVED ____, Administrator	SHEET NO. 432

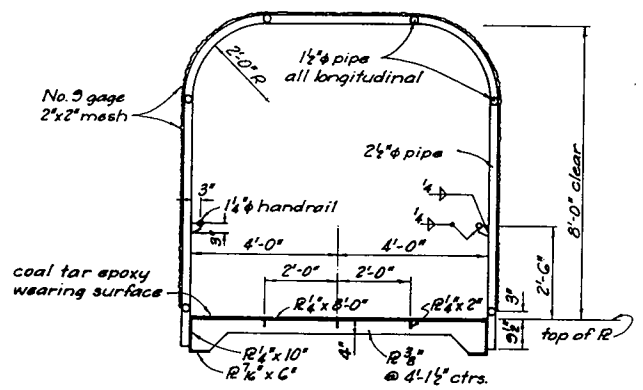


ELEVATION

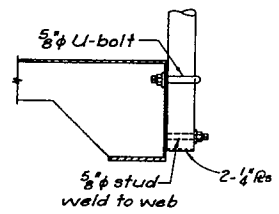


D.L. DEFLECTION DIAGRAM

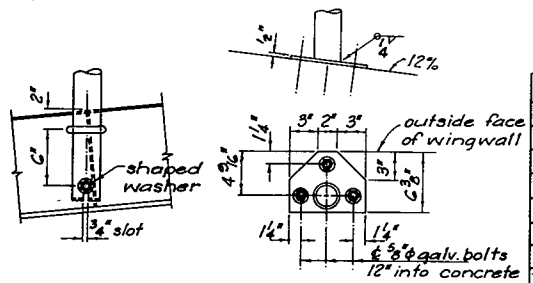
Note: All posts shall be vertical.



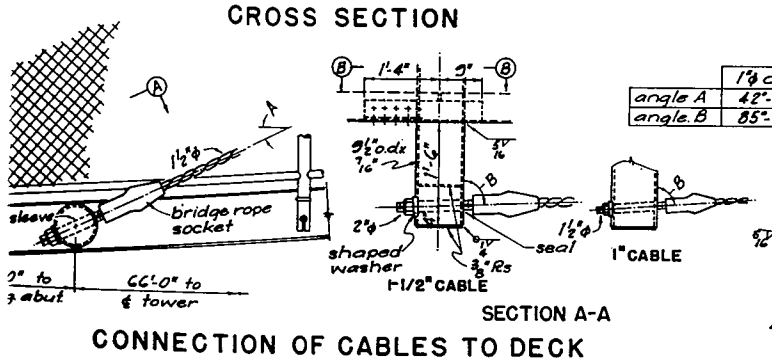
CROSS SECTION



POST CONNECTION



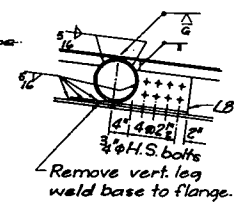
ABUTMENT POST CONNECTION



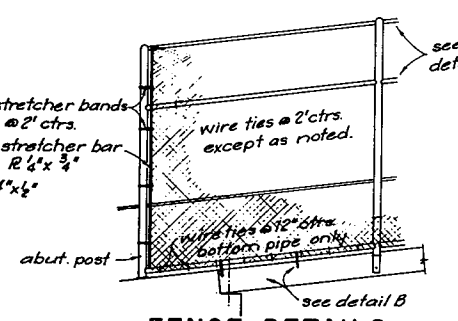
SECTION A-A

CONNECTION OF CABLES TO DECK

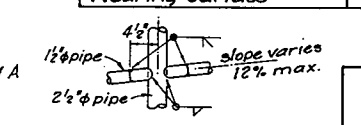
	1"ϕ cable	1½"ϕ cable
angle A	42°-20'	25°-54'
angle B	85°-53'	87°-54'



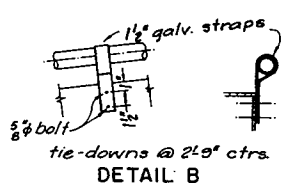
SECTION B-B



FENCE DETAILS



DETAIL A



DETAIL B

DO NOT SCALE

GENERAL NOTES

DESIGN CRITERIA: "Design Criteria for Pedestrian Bridges over Interstate or Comparable Highways." Appendix A

DESIGN SPECIFICATIONS: "AASHO Standard Specifications for Highway Bridges", 1961, with tentative revisions to 1963, except as modified or amplified by the "Design Criteria."

STRUCTURAL STEEL: All structural steel shall be A-36 conforming to ASTM A-36. The steel deck shall be cambered for dead load deflection, and for additional permanent camber (due to vertical curvature).

BRIDGE ROPE: Bridge rope shall be high strength grade conforming to ASTM A-218 with a Class B zinc coating. Bridge rope shall be prestretched to 50% of ultimate rope strength, then released to dead load stress (1¼"ϕ-10k; 1"ϕ-6k). At this load, mark and cut ropes to final geometric lengths. Sockets shall be of sufficient strength to produce failure in rope material.

REINFORCING STEEL: Reinforcing steel shall be deformed bars of intermediate, hard, or rail grade conforming to ASTM A-15 or A-16. Dimensions shown on the plans from the reinforcing steel to the outside edge of the concrete are all clear dimensions. Bar sizes are designated on the plans by the first digit of the bar marks.

CONCRETE: Concrete shall be Class A(AE) with a minimum 28 day f'_c = 4,000 psi. The air-entraining agent shall be as approved by the Engineer.

SOIL PRESSURE: The maximum allowable soil pressure for the design of stub abutments has been assumed to be 1 ton per sq. ft.

PAINT: All structural steel shall be given one shop coat of red lead iron oxide paint. The steel deck surface and surfaces in contact with concrete shall not be painted. Surfaces inaccessible after erection shall be given three coats of paint in the shop. Field paint shall consist of one coat of tinted red lead iron oxide paint followed by one finish coat of aluminum or light green paint. All paint and workmanship shall conform to AASHO Specifications and special provisions for paint (see Appendix B).

GALVANIZED FENCE AND SUPPORTS: Chain link fence fabric, conforming to ASTM A-392, shall be no. 9 gage wire woven in a 2" mesh and galvanized after weaving with a Class II zinc coating. Galvanized steel pipe rail and fence supports shall conform to ASTM A-53, Grade-B, type S or E. ASTM A-36 steel shall be used for all structural steel and hardware, and after fabrication, shall be galvanized in accordance with ASTM A-123 and A-153. All welding and erection damage to the galvanizing shall be repaired with material that conforms to Military Specification MIL-P-21035 (SHIPS).

WELDING: All welding shall conform to the "Specifications for Welded Highway and Railway Bridges" of the American Welding Society - AWS D2.0-63.

PEDESTAL PILES: The drilled holes shall be cleaned of all loose material and the pedestal piles cast in compacted coarse and fine sand, medium stiff clay, or better.

EPOXY WEARING SURFACE: Type C, coal-tar modified system, with a surface aggregate application shall be used for the epoxy wearing surface. The system and the aggregate shall conform to Interim Guide Specification M 200-63I. The deck surface shall be clean and free of foreign material detrimental to the bond of the resin. The rate of application of epoxy resin shall be in accordance with manufacturer's recommendations. Application of the aggregate shall be at a rate of not less than 15 lbs. per sq. ft.

SUMMARY OF QUANTITIES

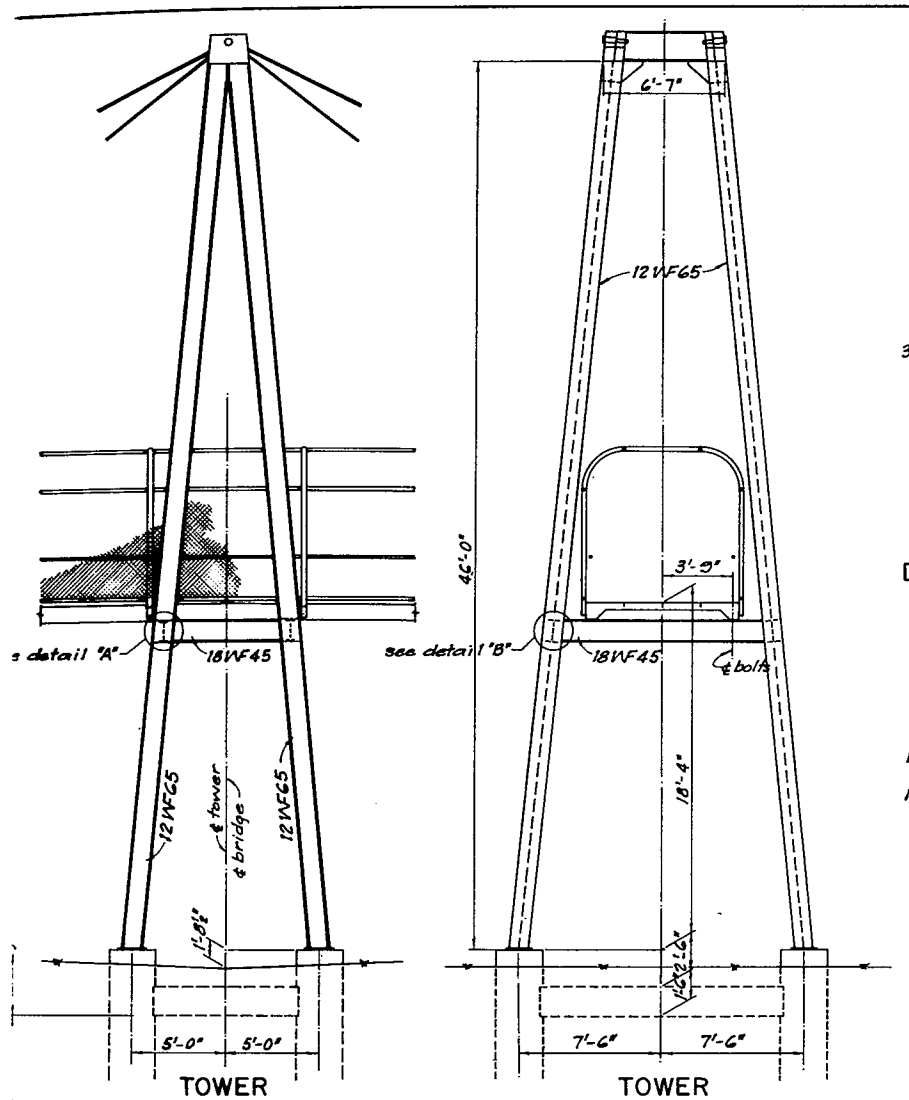
ITEM	UNITS	AMOUNT
Excavation For Struct. (est.)	C.Y.	50
30"ϕ Drilled Shafts	L.F.	48
Structural Steel	Lbs.	43,860
Concrete, Class A (AE)	C.Y.	13
Reinforcing Steel	Lbs.	2,580
Fencing (Including Supports)	L.F.	206.3
1½"ϕ Bridge Rope	L.F.	290
1"ϕ Bridge Rope	L.F.	175
Structural Metal	Lbs.	1,140
Wearing Surface	S.Y.	180

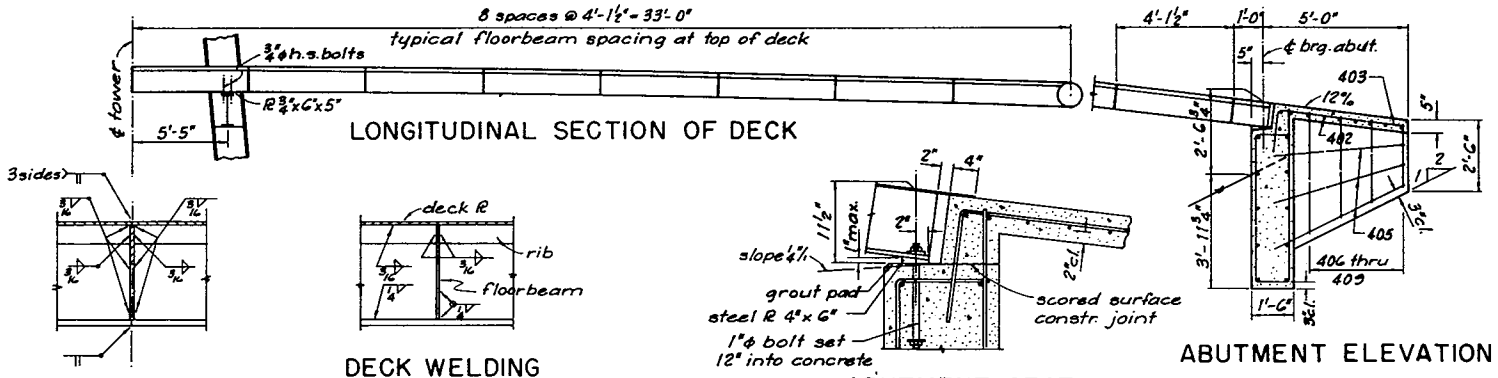
U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

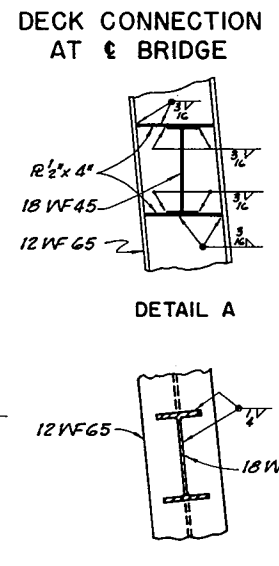
TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

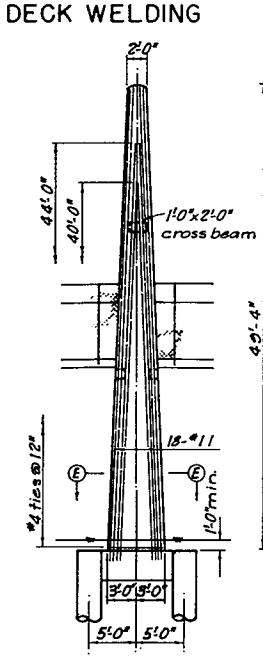
SINGLE TOWER SUSPENSION BRIDGE
SPANS = 100 + 100 = 200 FT.
SUPERSTRUCTURE DETAILS AND GENERAL NOTES

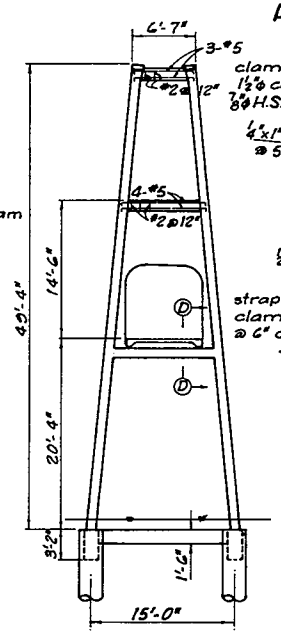
RECOMMENDED: E. B. Lewis, Chief Design Branch	RECOMMENDED: [illegible], Director of Engineering	OCTOBER, 1964
RECOMMENDED: [illegible], Chief Bridge Division	APPROVED: [illegible], Administrator	SHEET NO. 441

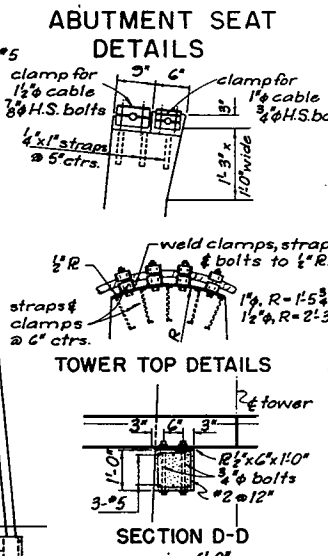


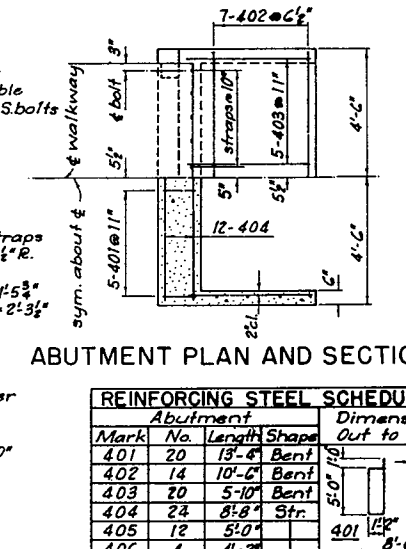






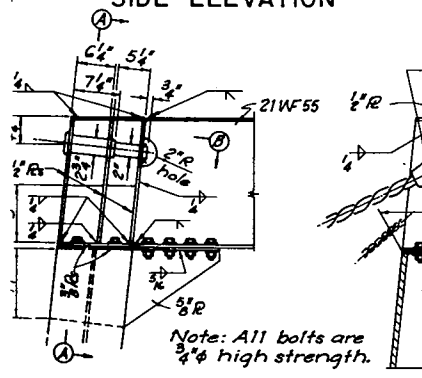


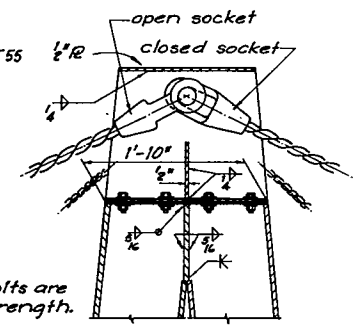


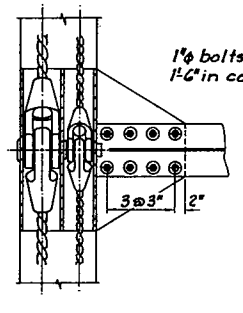


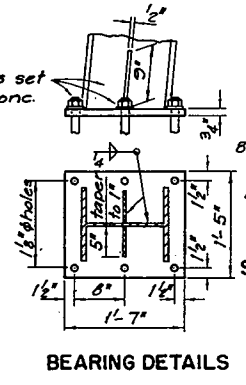
REINFORCING STEEL SCHEDULE				
Abutment				Dimensions Out to Out
Mark	No.	Length	Shape	
401	20	13'-6"	Bent	
402	14	10'-6"	Bent	
403	20	5'-10"	Bent	
404	24	8'-8"	Str.	
405	12	5'-0"		
406	4	4'-2"		
407	4	3'-6"		
408	4	2'-10"		
409	4	2'-2"	Str.	
Pedestal Piles				
801	44	15'-6"	Str.	
501	8	10'-6"	Str.	
201	68	6'-9"	Bent	
202	42	4'-0"	Bent	

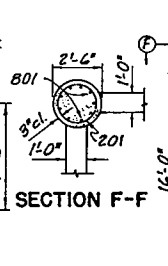
Note: Concrete and reinforcing in the alternate concrete tower are not included in quantities.

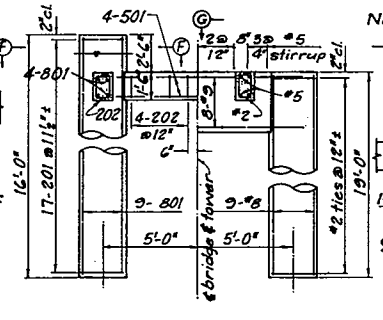


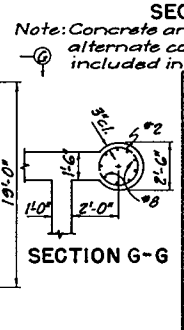












DO NOT SCALE

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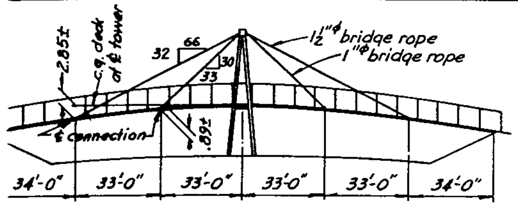
TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

SINGLE TOWER SUSPENSION BRIDGE
SPANS = 100 + 100 = 200 FT
DETAILS

RECOMMENDED: S. B. Jansen, Chief Design Branch
RECOMMENDED: [illegible], Chief Bridge Division
RECOMMENDED: [illegible], Director of Engineering
APPROVED: [illegible], Administrator

OCTOBER, 1964
SHEET NO. 442

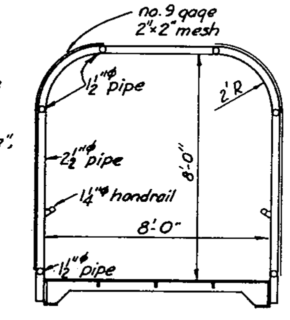


V DATA:
00' spans = 200'
walkway
otropic steel plate

ifications:
ASHO with revisions to 1963
modified by Design Criteria.
ISC "Design Manual for Orthotropic Steel Plate Deck Bridge",
963.

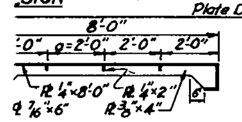
Allowable Stresses:

36 steel	f_s = 20,000 psi
inforcing	f_s = 20,000 psi
ncrete	f_c' = 4,000 psi
	f_c = 1,600 psi



Load:
L = 60 psf, except for
sign of deck elements
here W_{LL} = 85 psf.

SIGN *Reference: AISC "Design Manual for Orthotropic Steel Plate Deck Bridge" 1963



te: *Pg. 97, Section 6.3
k deflection < 1/300 of the
acing, t min. = $0.0065a\sqrt[3]{W_{LL}}$
24", W_{LL} = 85 psf = 0.59 psi
in. = 0.131, Use ¼" ℞

gn: *Pg. 44, Section 3.3
er rib:

d	= 85#/□'×2'	= 17.0
surface	= 5.5#/□'×2'	= 11.0
℞	= 10.2#/□'×2'	= 20.4
	= 1.70#/'	= 1.7
W_R = 16.9#/'		= 203.1

e rib span (Si):
7 $S_{Fl.Bm.}$ = 0.7×49.5 = 34.65#
ve deck width per rib (a_e):
Si/3 = 11.55

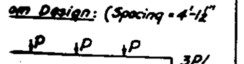
Properties: (try 2"×¼" rib)

A	y	Ay	Ay² + I₀
.89	0.125	0.361	0.0588
.50	1.25	0.625	0.9480
.39	0.291	0.986	1.0068
29'			0.2865
959'		I(in)⁴ =	0.7203

$M = \frac{W_R (Si)^2}{8} = 2{,}540$#"

2,540#"×0.291/0.7203 = 1040 psi
2,540#"×1.959/0.720 = 6900 psi
se ℞ ¼"×2"

am Design: (Spacing = 4'-1½")



reaction = 16.9#/"×49.5" = 836#
$\frac{3P}{2}(L/2) - P(\frac{L}{4}) = \frac{PL}{2} = \frac{1}{2}(836)(8')$
344#' = 40,128#"
deck width per floor beam:
L/3 = 96/3 = 32"

Floorbeam Properties (try 4"×3/8")

	A	y	Ay	Ay² + I₀
32"(¼)	8.00	0.125	1.000	0.1667
4(3/8)	1.50	2.250	3.375	9.5938
	9.50	0.46	4.375	9.7605
c(top) = 0.46"				2.0125
c(Bot.) = 3.79"			I(in)⁴	7.7480

$-f_{(top)}$ = 41,328#"×0.46"/7.748 = 2,350 psi
$+f_{(bot.)}$ = 41,328#"×3.79"/7.748 = 20,200 psi
allowable f_s = 110%×20 ksi = 22000 psi

Use ℞ 3/8"×10" at 4'-1½" ctrs.

Deck Design:
Use ℞ ¼"×10" side plates
℞ 7/16"×6" bottom flange plates

Dead Load:

deck	= 81.6#/'×1'	= 81.6
fl. bm.	= $\frac{490\#/CF}{1728\times4.125}(\frac{3}{8})(4\times96+6\times18)$	= 12.7
ribs	= 3×1.70#/'	= 5.1
webs	= 2×8.50#/'	= 17.0
bot. flgs.	= 2×8.93#/'	= 17.9
fence		= 60.0
epoxy	= 5.5#/S.F.×8'	= 44.0
		238.3

Use W_{DL} = 240#/ft.

Cross-Section Properties

	A	y	Ay	Ay² + I₀
℞ ¼"×10	5.00	5.25	26.25	180
℞ 7/16"×6	5.25	10.47	54.97	580
℞ ¼"×2	1.50	1.25	1.88	3
℞ ¼"×8	24.00	0.13	3.12	0
	35.75	2.41	86.22	763
$c_{(top)}$ = 2.41"				208
$c_{(bot)}$ = 8.28"			I(in)⁴ =	555

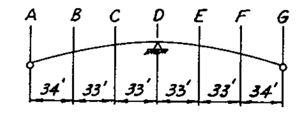
Mom. capacity = 110% $f_s I/c$
Positive $M_{cap.}$ = 123 KF
Negative $M_{cap.}$ = 422 KF

Final Rib Stress (Max.)

$+f_s = 6900 + 22{,}000\left(\frac{2.41-2.25}{2.41}\right) = 8370$ psi

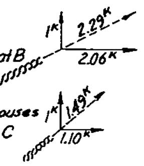
$-f_s = 1040 + 22{,}000\left(\frac{2.41-0.25}{2.41}\right) = 20{,}740$ psi

Basic Structure:



Note:
1K vertical at B
2.06K horizontal at B

1K vertical at C causes
1.10 horizontal at C



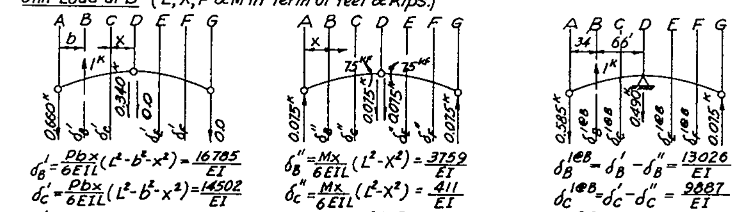
Method of Solution: Reference.."Analysis Of Statically Indeterminate Structures" by Parcel and Moorman, Chapter 3

The structures is considered as a continuous beam on yielding supports at cable connections B, C, E and F. By applying the "principle of consistent distortions", a series of elastic equations are developed from the following general equations:

$$\delta_x = \Delta_x - \delta_x^{1@B} R_B - \delta_x^{1@C} R_C - \delta_x^{1@E} R_E - \delta_x^{1@F} R_F$$

where δ_x = Final vertical deflection at x from cable elongation.
Δ_x = Vertical deflection at x of basic structure due to applied loads.
$\delta_x^{1@B}$ = Vertical deflection at x of basic structure due to unit vertical load @ B.
R_B = Final vertical reaction at B.
$\delta_x^{1@C}$ = Vertical deflection at x of basic structure due to unit vertical load at C.
etc.---

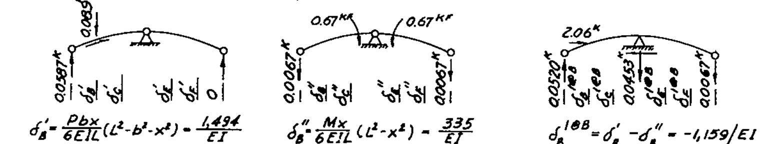
Unit Load at B (L, X, P & M in term of feet & kips.)



$\delta_B' = \frac{Pbx}{6EIL}(L^2-b^2-x^2) = \frac{16785}{EI}$
$\delta_C' = \frac{Pbx}{6EIL}(L^2-b^2-x^2) = \frac{14502}{EI}$
$\delta_E' = 0.$
$\delta_F' = 0$

$\delta_B'' = \frac{Mx}{6EIL}(L^2-X^2) = \frac{3759}{EI}$
$\delta_C'' = \frac{Mx}{6EIL}(L^2-X^2) = \frac{411}{EI}$
$\delta_E'' = \delta_C'' = \frac{4615}{EI}$
$\delta_F'' = \delta_B'' = \frac{3759}{EI}$

$\delta_B^{1@B} = \delta_B' - \delta_B'' = \frac{13026}{EI}$
$\delta_C^{1@B} = \delta_C' - \delta_C'' = \frac{9887}{EI}$
$\delta_E^{1@B} = \delta_E' - \delta_E'' = \frac{4615}{EI}$
$\delta_F^{1@B} = \delta_F' - \delta_F'' = \frac{3759}{EI}$

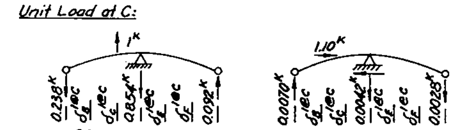


$\delta_B' = \frac{Pbx}{6EIL}(L^2-b^2-x^2) = \frac{1,494}{EI}$
δ_C' = " = −1,291/EI
$\delta_E' = 0$
$\delta_F' = 0$

$\delta_B'' = \frac{Mx}{6EIL}(L^2-x^2) = \frac{335}{EI}$
δ_C'' = " = 411/EI
$\delta_E'' = \delta_C'' = 411/EI$
$\delta_F'' = \delta_B'' = 335/EI$

$\delta_B^{1@B} = \delta_B' - \delta_B'' = -1{,}159/EI$
$\delta_C^{1@B} = \delta_C' - \delta_C'' = -880/EI$
$\delta_E^{1@B} = \delta_E' - \delta_E'' = 411/EI$
$\delta_F^{1@B} = \delta_F' - \delta_F'' = 335/EI$

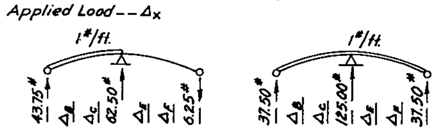
Unit Load at C:



$\delta_B^{1@C} = 9{,}887/EI$
$\delta_C^{1@C} = 10{,}634/EI$
$\delta_E^{1@C} = -5{,}661/EI$
$\delta_F^{1@C} = -4{,}615/EI$

$\delta_B^{1@C} = -289/EI$
$\delta_C^{1@C} = -310/EI$
$\delta_E^{1@C} = 172/EI$
$\delta_F^{1@C} = 140/EI$

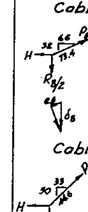
Applied Load -- Δ_x



Δ_B = +831.59/EI
Δ_C = +740.31/EI
Δ_E = −384.62/EI
Δ_F = −313.23/EI

Δ_B = +518.36/EI
Δ_C = +355.69/EI
Δ_E = +355.69/EI
Δ_F = +518.36/EI

Cable Elongation: δ_x
Cable at B (2-1½"φ cables)



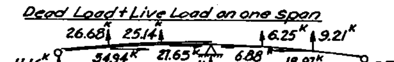
$P_B = (R_B/2)(73.4/32)$
$P_B = 1.15 R_B$
$e_B = \frac{P_B L_B}{AE} = \frac{1.15R_B \times 73.4}{1.08\times21\times10^6}$ =
$e_B = 0.00372 R_B$ '/kip
$\delta_B = e_B(73.4/32) = 0.00853 R_B$ '/kip
Cable at C (2-1"φ cables)
$P_C = 0.743 R_C$
$e_C = 0.00335 R_C$ '/kip
$\delta_C = 0.00498 R_C$ '/kip
Note:
E = 21×10⁶ psi - prestretched cables
E = 29×10⁶ psi -- A36 steel

ELASTIC EQUATIONS (Dead Load of 240#/ft. + Live Load on one span of 480#/ft)

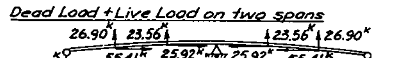
$$EI = 15{,}921\#(in)^2 = 110{,}562.50\,K(ft)^2$$

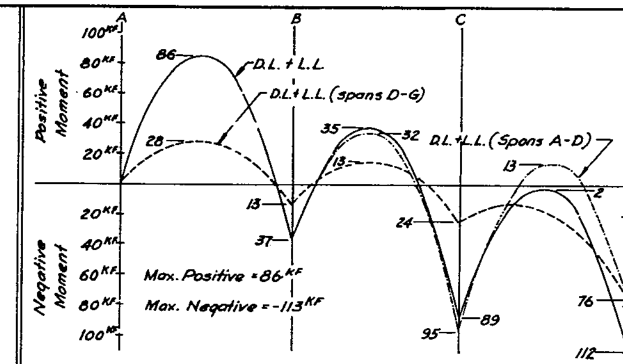
@B → $943R_B$ − 523,569 − R_B(13,026 − 1,159) − R_C(9,887 − 289) − R_E(−4,615 + 140) − R_F(−3,759 + 335)
@C → $551R_C$ = 440,714 − R_B(9,887 − 880) − R_C(10,634 − 310) − R_E(−5,661 + 172) − R_F(−4,615 + 411)
@E → $551R_E$ = −99,252 − R_B(−4,615 + 411) − R_C(−5,661 + 172) − R_E(10,634 − 310) − R_F(9,887 − 880)
@F → $943R_F$ = −25,944 − R_B(−3,759 + 335) − R_C(−4,615 + 140) − R_E(9,887 − 289) − R_F(13,026 − 1,159)
Solving: R_B = 26.68K, R_C = 25.14K, R_E = 6.25K, R_F = 9.21K

Dead Load + Live Load on one span

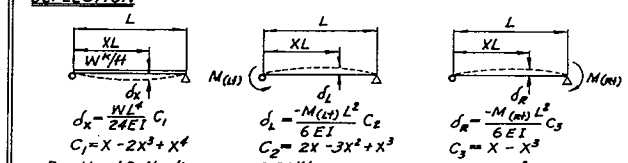


Dead Load + Live Load on two spans





DEFLECTION



$\delta_x = \frac{WL^4}{24EI}C_1$ $\quad C_1 = X - 2X^3 + X^4$

$\delta_L = \frac{-M_{(Lt)}L^2}{6EI}C_2$ $\quad C_2 = 2X - 3X^2 + X^3$

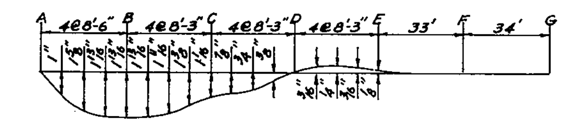
$\delta_R = \frac{-M_{(Rt)}L^2}{6EI}C_3$ $\quad C_3 = X - X^3$

Dead Load Deflection: W = 0.24 K/ft.; EI = 110,562.50 K-Ft²

M = (12KF at B, 30KF at C, 37KF at D)

	A	¼ pt.	½ pt.	¾ pt.	B	¼ pt.	½ pt.	¾ pt.	C	¼ pt.	½ pt.	¾ pt.	D
C_1	0	.222	.313	.222	0	.222	.313	.222	0	.222	.313	.222	0
C_2	0	.328	.375	.234	0	.328	.375	.234	0	.328	.375	.234	0
C_3	0	.234	.375	.328	0	.234	.375	.328	0	.234	.375	.328	0
δ_x	0	+.0268	+.0378	+.0268	0	+.0238	+.0336	+.0238	0	+.0238	+.0336	+.0238	0
δ_L	0	0	0	0	0	−.0065	−.0074	−.0046	0	−.0161	−.0185	−.0115	0
δ_R	0	−.0048	−.0078	−.0069	0	−.0115	−.0185	−.0161	0	−.0142	−.0228	−.0199	0
Cable δ	0	+.0191	+.0383	+.0574	+.0765	+.0672	+.0578	+.0484	+.0391	+.0294	+.0196	+.0022	0
Δ (ft.)	0	+.0411	+.0683	+.0773	+.0765	+.0730	+.0655	+.0515	+.0391	+.0229	+.0119	0	0
Δ (in)	0	+½"	+13/16"	+15/16"	+15/16"	+7/8"	+3/4"	+5/8"	+7/16"	+¼"	+1/8"	0	0

L.L. Deflection:
For L.L. on 2 spans, deflections = 2 × D.L. deflections
For L.L. on 1 span

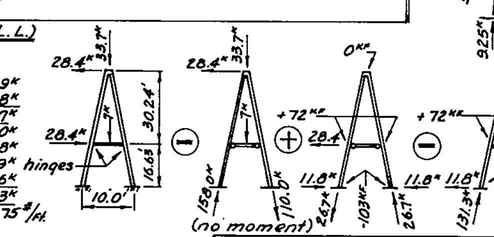


Tower Design (D.L. + L.L.)
D.L+L.L (1 span):

hor. D.L.	=	0
hor. L.L.	1@35.9K =	35.9K
	1@20.8K =	20.8K
H, total 2 sides	=	56.7K
vert. D.L.	2@9.0K =	18.0K
	2@7.9K =	15.8K
vert. L.L.	1@17.9K =	17.9K
	1@15.6K =	15.6K
V, total 2 sides	=	67.3K

R_D = 13.9K; tower D.L. = 75#/ft.

Loads per frame:
H = 28.4K, V = 33.7K, R_D = 7.0K, D.L. = 75#/ft./Leg.



D.L. + L.L.
110% allowable stress
M = 103 KF, P = 131.3K
P/A + M/SM ≤ 22 ksi

$\frac{131.3}{19.11} + \frac{103\times12}{88.0} = 20.95$ ksi

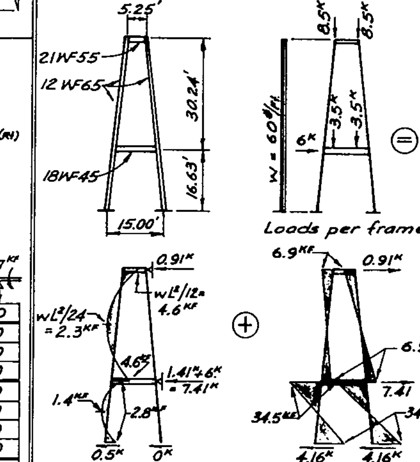
12 WF 65 is o.k.

Final Cable Loads:
Cables at B
Dead Load = 10.3K
D.L. + L.L. (1 span) = 30.7K = 10.6K
D.L. + L.L. (2 spans) = 30.9K
Vert. δ_B (D.L.) = 0.0765K
Breaking strength = 208K

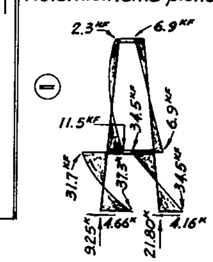
Cables at C
Dead load = 5.8K
D.L. + L.L. (1 span) = 18.7K = 4.6K
D.L. + L.L. (2 spans) = 17.5K
Vert. δ_C (D.L.) = 0.0391K
Breaking strength = 91.4K

Tower Design: (D.L. + Wind) 4-12WF65 legs.
transverse wind @ 45 psf
on deck, 0.90'×5/8×200'×1.3×45#/SF 6.6K
on fence, 8.00'×5/8×200'×0.12×45#/SF 5.4K
total 12.0K
on legs. 2×1.00'×1.3×45#/S.F. (say) 120#/ft.
D.L. of leg 75#/ft./leg.

Basic Assumptions:
1. All joints are rigid.
2. Column bases are considered fixed.
3. Changes in length of all members due to axial loads are neglected.
4. Horizontal members are considered infinitely stiff relative to columns, because their stiffnesses are at least five times that of the columns. (see pg. 217 "Continuous Frames of Reinforced Concrete" by Cross & Morgan.)



Note: Moments plotted on compression face.



D.L. + Wind
145% allowable stress
M = 37.3KF, P = 9.3K
P/A + M/S.M. ≤ 22.0 ksi

$\frac{9.3}{19.11} + \frac{37.3\times12}{29.1} = 15.9$ ksi

∴ 12WF65 is o.k.

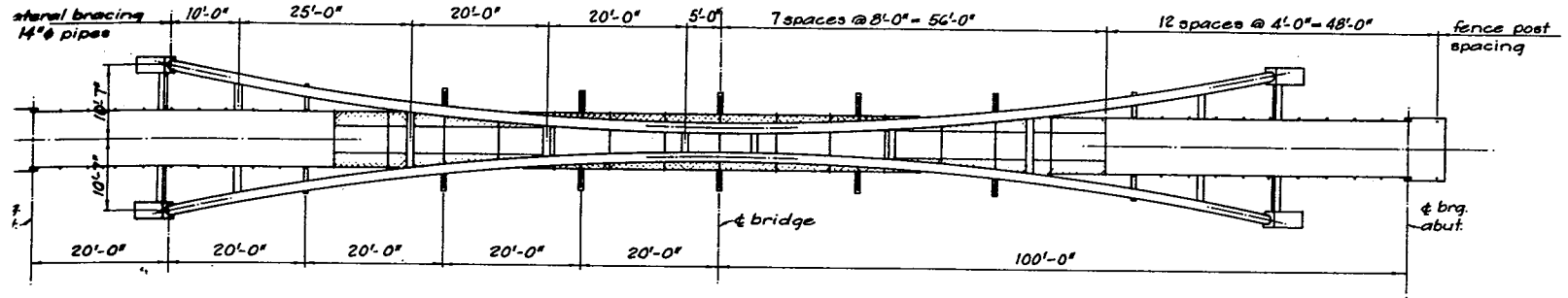
DO NOT SCALE

U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

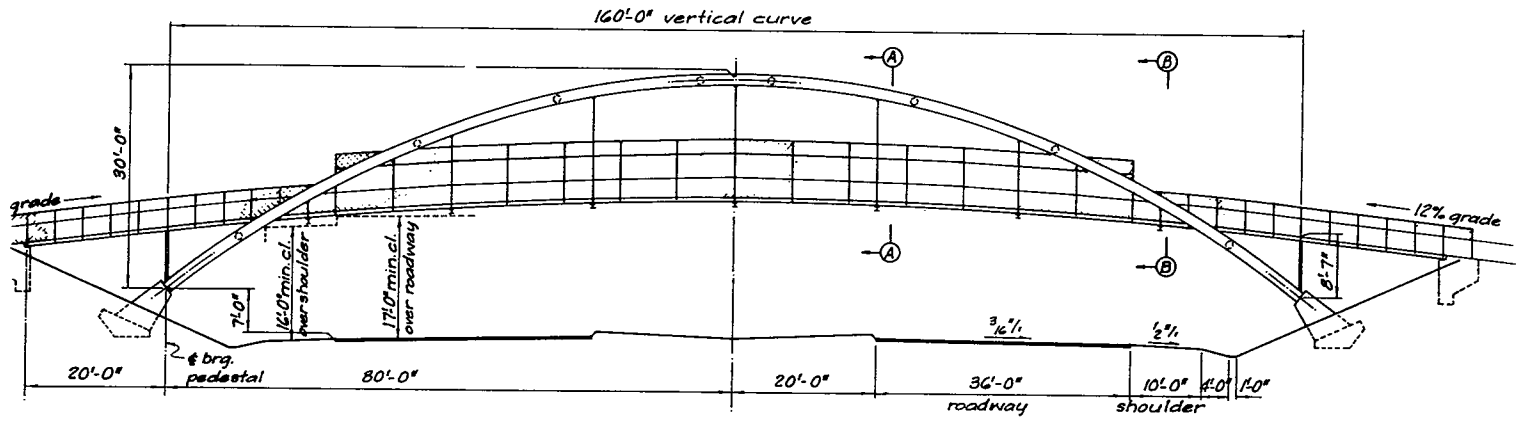
TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

SINGLE TOWER SUSPENSION BRIDGE
DESIGN CALCULATIONS

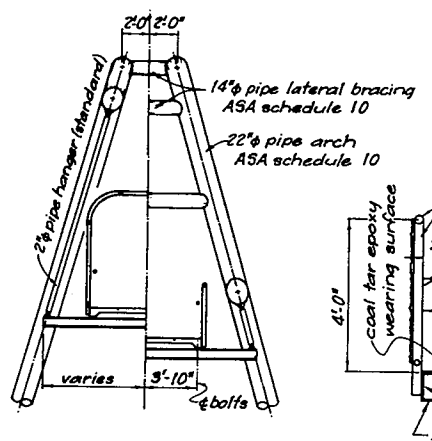
RECOMMENDED S. B. Lauu — Chief Design Branch	RECOMMENDED [signature] — Director of Engineering	OCTOBER 1964
RECOMMENDED [signature] — Chief Bridge Division	APPROVED [signature] — Administrator	SHEET NO. 443



PLAN

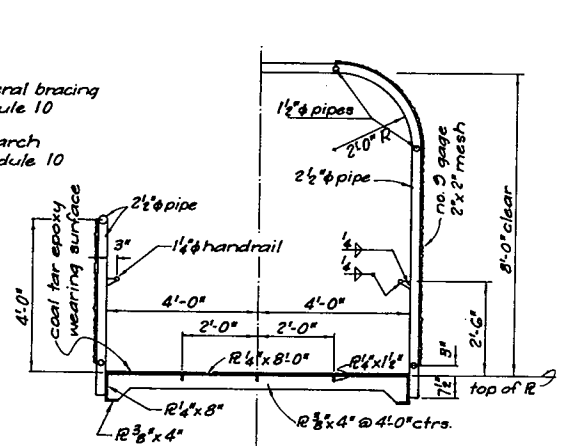


ELEVATION



HALF SECTION A-A

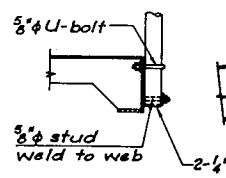
HALF SECTION B-B



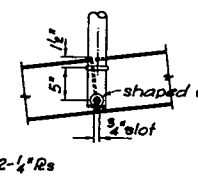
END SPAN MIDSPAN

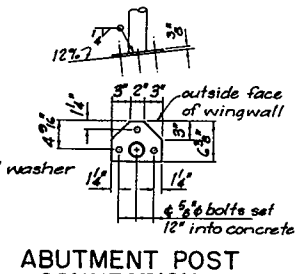
CROSS SECTION

Note: All posts shall be vertical.

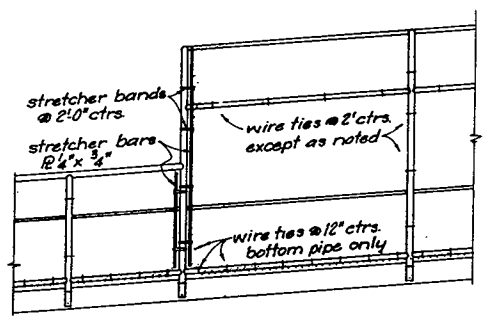


POST CONNECTION

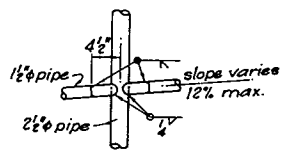




ABUTMENT POST CONNECTION



FENCE DETAILS



LONGITUDINAL PIPE CONNECTION

GENERAL NOTES

DESIGN CRITERIA: "Design Criteria for Pedestrian Bridges over Interstate or Comparable Highways," Appendix A

DESIGN SPECIFICATIONS: "AASHO Standard Specifications for Highway Bridges," 1961, with tentative revisions to 1963, except as modified or amplified by the "Design Criteria."

STRUCTURAL STEEL: All structural steel shall be A-36 conforming to ASTM A-36. The steel deck shall be cambered for dead load deflection, and for additional permanent camber (due to vertical curvature). Steel arch pipes and pipe hangers shall conform to ASTM A-53 Grade B, Type E or S.

CONCRETE: Concrete shall be Class A(AE) with a minimum 28 day f'_c = 4,000 psi. The air-entraining agent shall be as approved by the Engineer.

REINFORCING STEEL: Reinforcing steel shall be deformed bars of intermediate, hard, or rail grade conforming with ASTM A-15 or A-16. Dimensions shown on the plans from the reinforcing steel to the outside edge of the concrete are all clear dimensions. All bending dimensions are from "out-to-out" of the bars. Bar sizes are designated on the plans by numbers. The first digit in the three digit marks, and the first two digits in the four digit marks indicate the size of the bars.

WELDING: All welding shall conform to the "Specifications for Welded Highway and Railway Bridges" of the American Welding Society -- AWS D 2.0-63.

PAINT: All structural steel shall be given one shop coat of red lead iron oxide paint. The steel deck surface and surfaces in contact with concrete shall not be painted. Surfaces inaccessible after erection shall be given three coats of paint in the shop. Field paint shall consist of one coat of tinted red lead iron oxide paint followed by one finish coat of aluminum or light green paint. All paint and workmanship shall conform to AASHO Specifications and special provisions for paint (see Appendix B).

GALVANIZED FENCE AND SUPPORTS: Chain link fence fabric, conforming to ASTM A-392, shall be no. 9 gage wire woven in a 2" mesh and galvanized after weaving with a Class II zinc coating. Galvanized steel pipe rails and fence supports shall conform to ASTM A-53, Grade B, Type E or S. ASTM A-36 steel shall be used for all structural steel and hardware, and after fabrication, shall be galvanized in accordance with ASTM A-123 and A-153. All welding and erection damage to the galvanizing shall be repaired with material that conforms to Military Specification MIL-P-21035 (SHIPS).

ROCK PRESSURE: The maximum allowable rock pressure for design of pedestal footings is assumed to be 5 ton per sq. ft., with a minimum penetration of 6" by the footing.

SOIL PRESSURE: For the design of stub abutments, 1 ton per sq. ft. is assumed to be the maximum allowable soil pressure.

LATERAL SOIL PRESSURE: Design is based on an allowable lateral soil pressure of 300 lbs. per sq. ft. per foot of depth.

ALTERNATE PEDESTAL PILE: The drilled hole shall be cleaned of all loose material and the pile cast in compacted coarse and fine sand, medium stiff clay, or better.

EPOXY WEARING SURFACE: Type C, coal-tar modified system, with a surface aggregate application shall be used for the epoxy wearing surface. The system and aggregate shall conform to Interim Guide Specification AASHO M 200-63I. The deck surface shall be clean and free of foreign material detrimental to the bond of the resin. The rate of application of epoxy resin shall be in accordance with manufacturer's recommendations. Application of the aggregate shall be at a rate of not less than 15 lbs. per sq. yd.

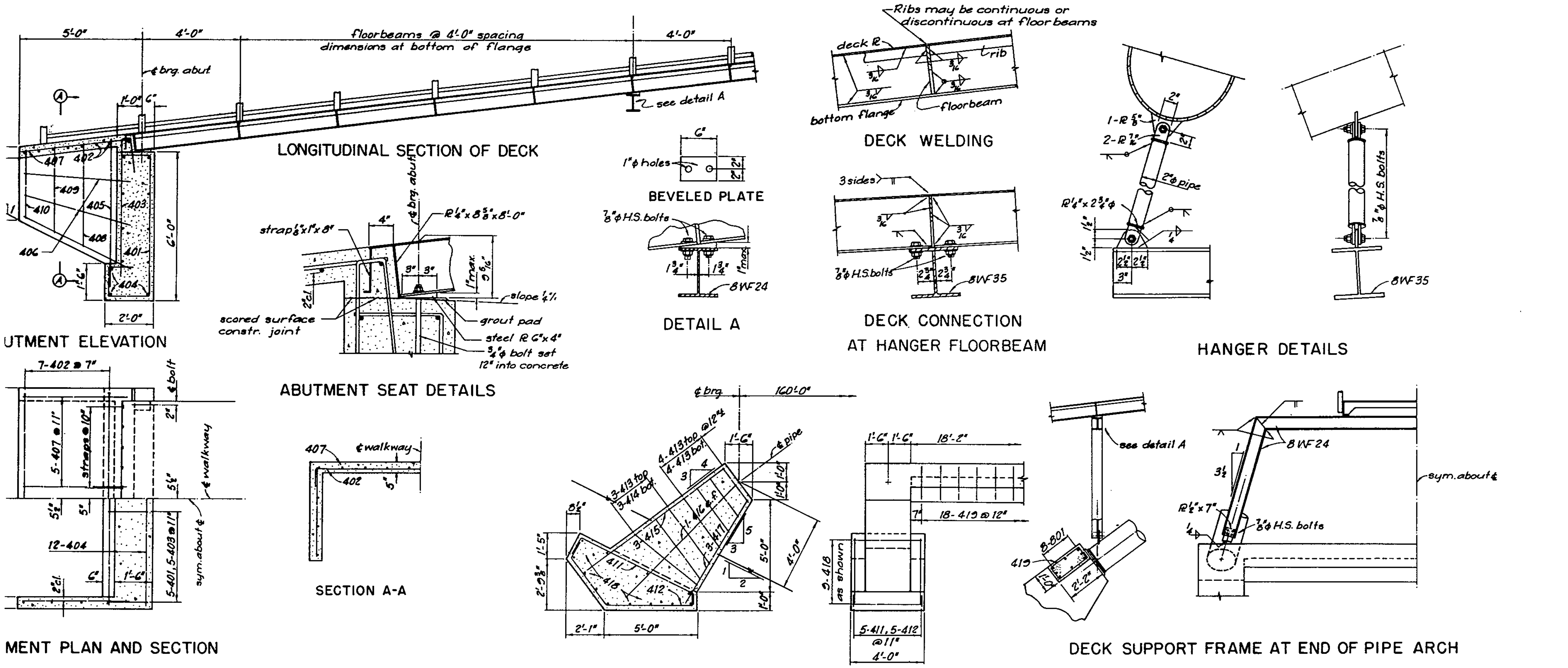
DO NOT SCALE

U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

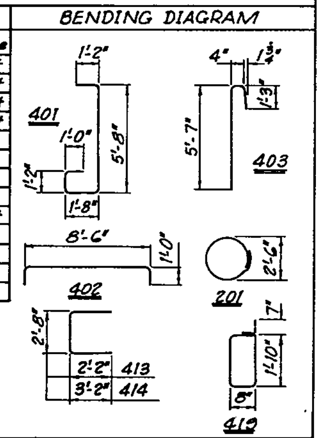
PIPE ARCH BRIDGE
160' ARCH SPAN
GENERAL PLAN AND ELEVATION

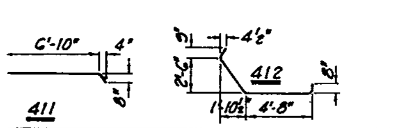
RECOMMENDED S. B. Sauer, Chief Design Branch	RECOMMENDED Director of Engineering	OCTOBER 1964
RECOMMENDED Chief Bridge Division	APPROVED Administrator	SHEET NO. 451

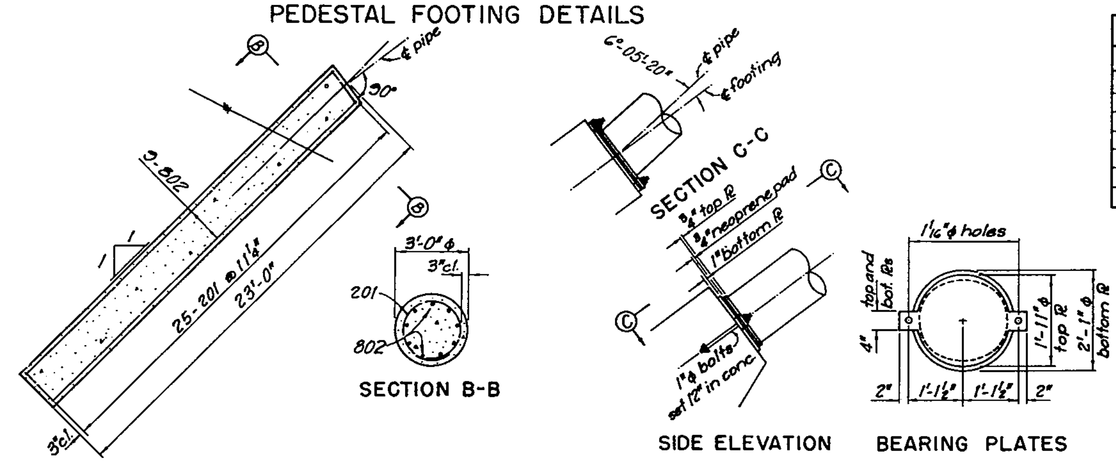


REINFORCING STEEL SCHEDULE						
BUTMENT			PEDESTAL FOOTING			
No.	Length	Shape	Mark	No.	Length	Shape
20	10'-8"	Bent	411	20	8'-7"	Bent
14	10'-6"	Bent	412	20	9'-4"	Bent
20	7'-2"	Bent	413	44	7'-0"	Bent
24	8'-8"	Str.	414	12	9'-0"	Bent
4	6'-0"	"	415	12	9'-0"	Str.
12	4'-9"	"	416	8	8'-6"	"
20	4'-4"	"	417	12	6'-6"	"
4	4'-9"	"	418	48	3'-8"	"
4	3'-6"	"	419	36	5'-7"	Bent
4	2'-9"	"				
DESTAL PILE			801	16	22'-2"	Str.
100	7'-6"	Bent				
36	22'-6"	Str.				

BENDING DIAGRAM







DO NOT SCALE

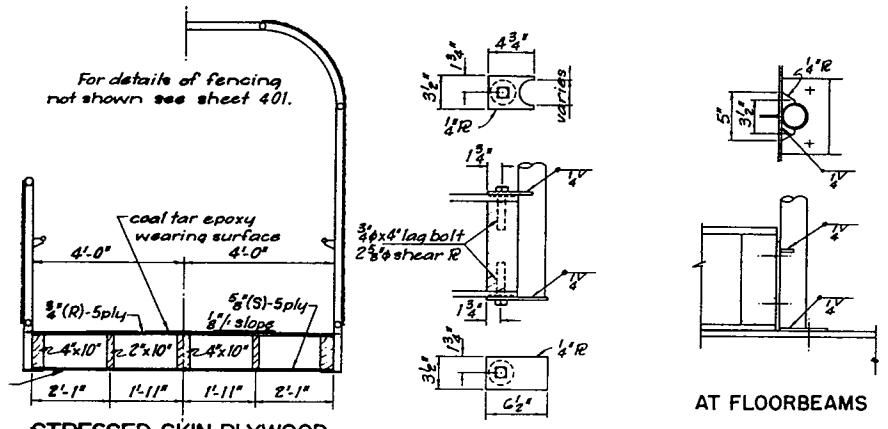
SUMMARY OF QUANTITIES		
Item	Unit	Quantity
Excavation for Structures (Estimated)	C.Y.	115
Class A(AE) Concrete	C.Y.	33
Reinforcing Steel	Lb.	2500
Structural Steel	Lb.	52770
Structural Metal	Lb.	1040
Fencing (Including Supports)	L.F.	208
Epoxy Wearing Surface	S.Y.	180

U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

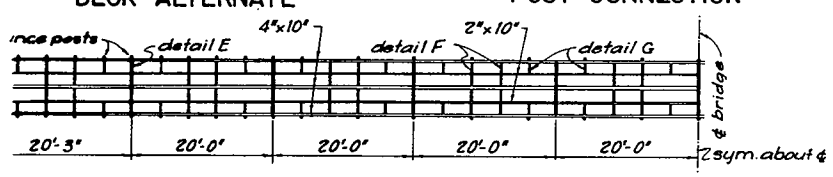
PIPE ARCH BRIDGE
160' ARCH SPAN
DETAILS

RECOMMENDED S. B. [signature] Chief Design Branch	RECOMMENDED [signature] Director of Engineering	OCTOBER 1964
RECOMMENDED [signature] Chief Bridge Division	APPROVED [signature] Administrator	SHEET NO. 452

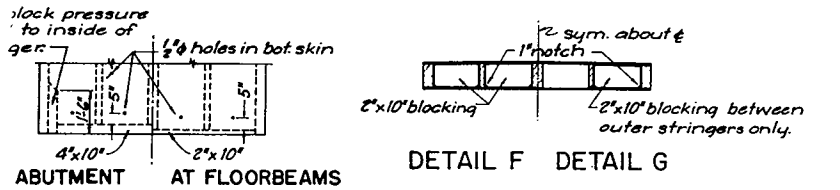


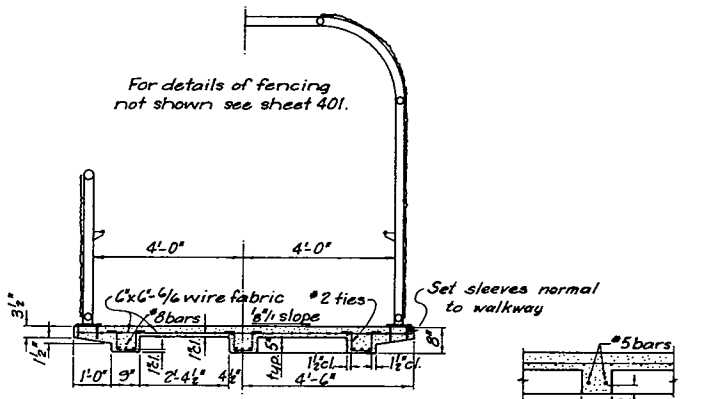
STRESSED SKIN PLYWOOD DECK ALTERNATE

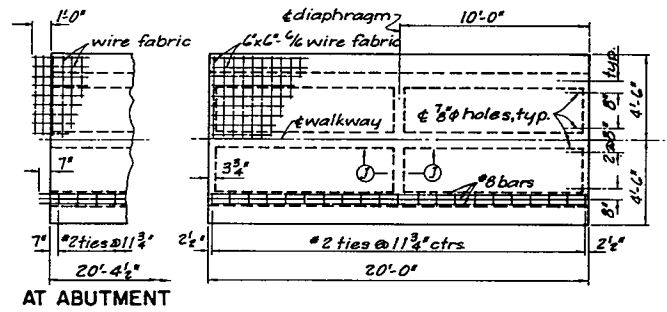
POST CONNECTION

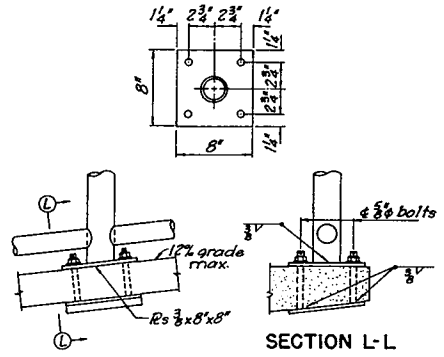


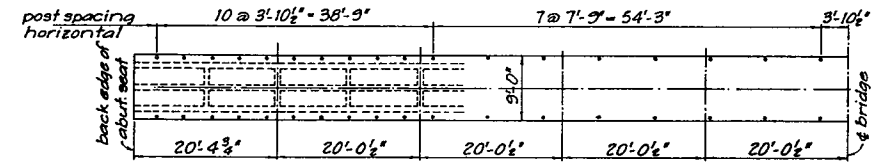
PLYWOOD DECK FRAMING DIAGRAM

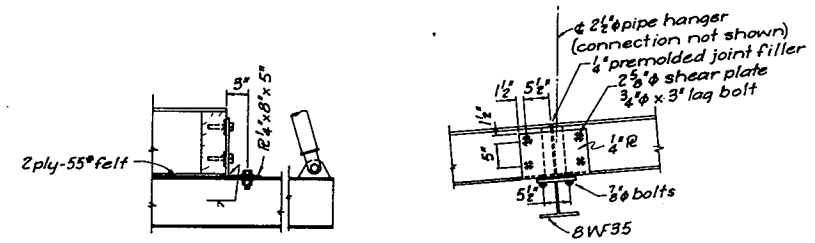


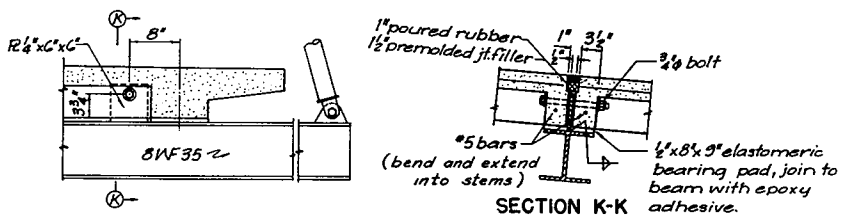


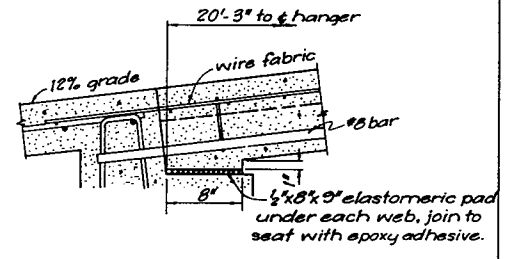


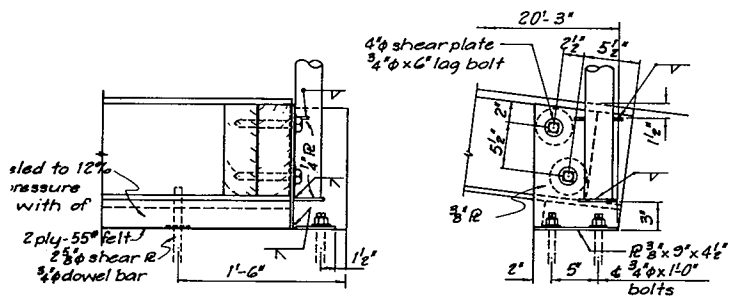












TIMBER NOTES

ADHESIVES: Adhesives for structural glued members shall be of exterior type conforming to Military Specification MIL-A-397B or MIL-A-5534A. Adhesives used shall be compatible with preservative treatment.

PRESERVATIVE TREATMENT: All lumber shall be pressure impregnated with an approved preservative suitable to the species of wood, adhesives, and painting requirements. Preservatives and amount of retention shall conform to the current AASHO M-133 and AASHO Standard Specifications for Highway Bridges. All cuts, abrasions, and holes bored after treatment shall be treated in an approved manner.

APPEARANCE: Deck panels shall be of Utility Appearance Grade, but shall have excess glue removed from exposed surfaces.

PAINTING: Optional

DO NOT SCALE

U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

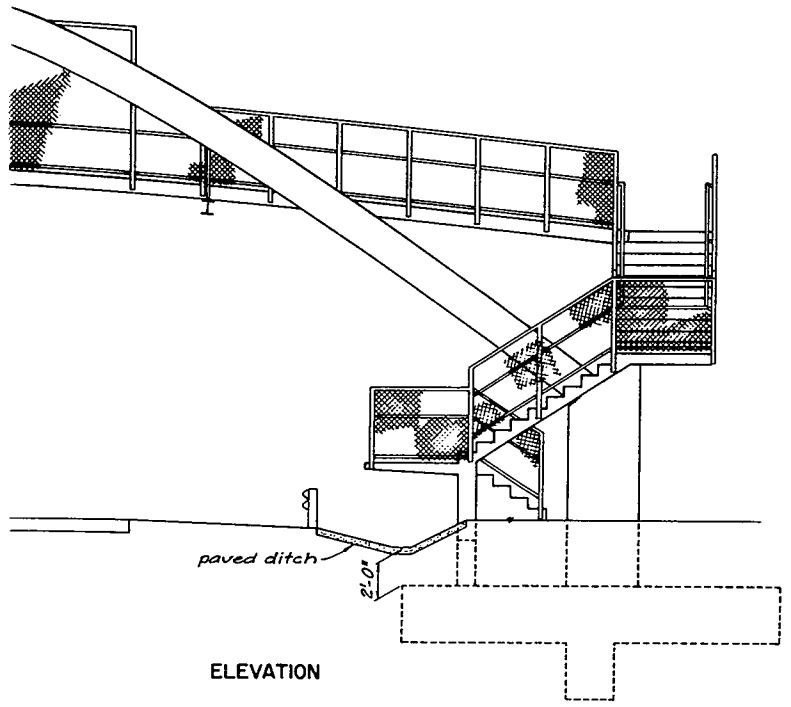
TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

PIPE ARCH BRIDGE
160' ARCH SPAN
ALTERNATE DECKS

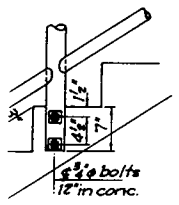
RECOMMENDED S. B. James, Chief Design Branch
RECOMMENDED [illegible], Director of Engineering
RECOMMENDED [illegible], Chief Bridge Division
APPROVED [illegible], Administrator

OCTOBER 1964

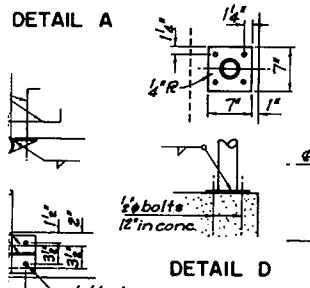
SHEET NO. 453



ELEVATION

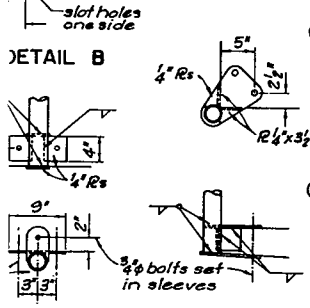


DETAIL A



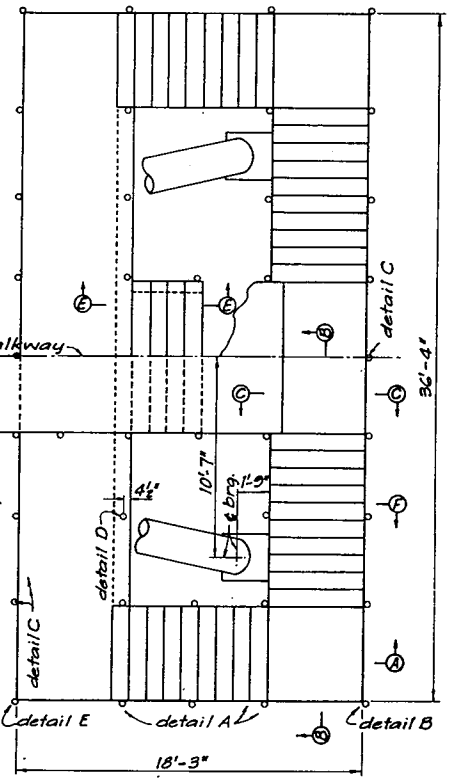
DETAIL D

DETAIL B

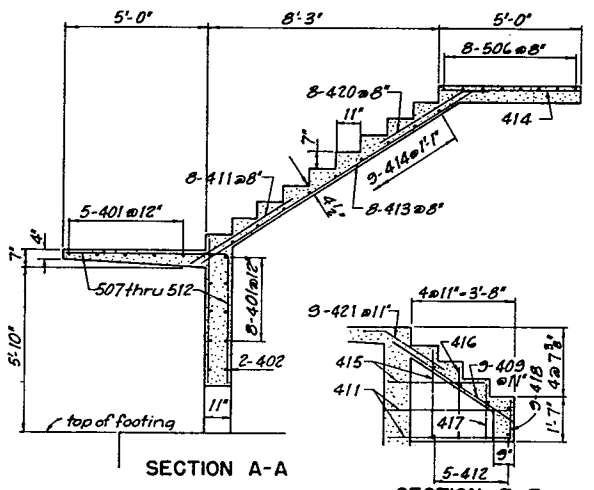


DETAIL C

DETAIL E

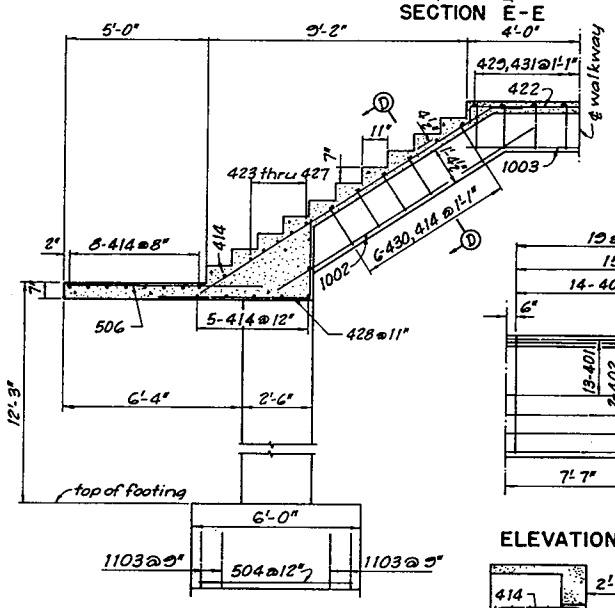


PLAN

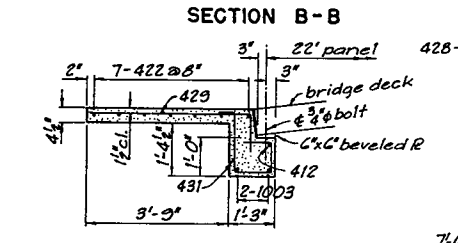


SECTION A-A

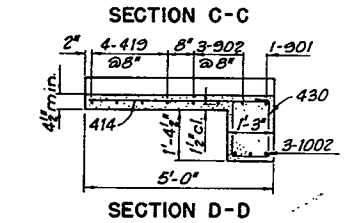
SECTION E-E



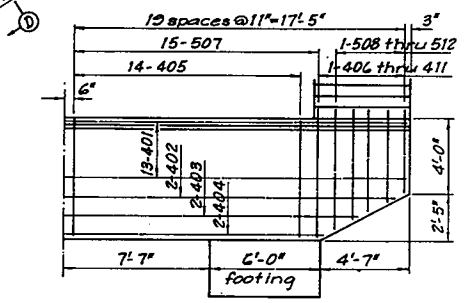
SECTION B-B



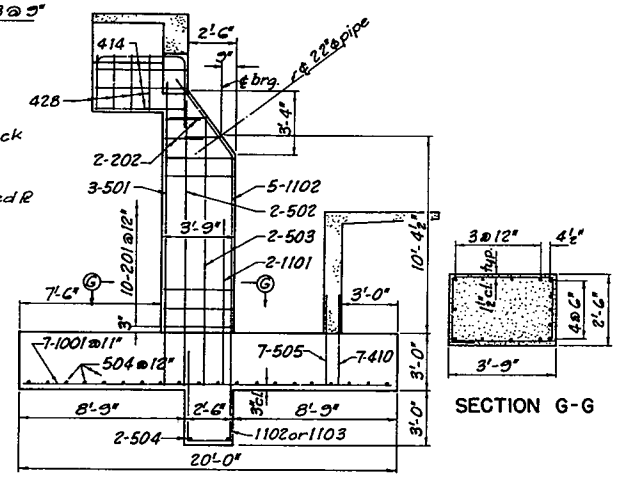
SECTION C-C



SECTION D-D



ELEVATION SHOWING WALL REINFORCING



SECTION F-F

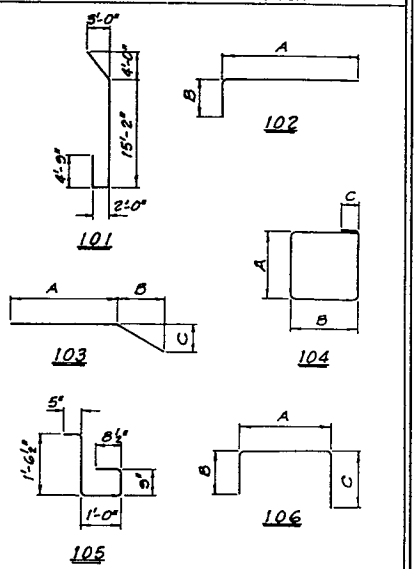
SECTION G-G

DO NOT SCALE

REINFORCING STEEL SCHEDULE

Mark	No.	Length	Mark	No.	Length	Shape	A	B	C
Straight Bars			Bent Bars						
1101	4	9'-0"	1102	10	22'-11"	101			
			1103	8	11'-6"	106	2'-0"	4'-9"	4'-9"
1001	14	13'-6"	902	6	13'-6"	103	12'-5"	11"	7"
1002	6	10'-6"							
1003	2	8'-0"	507	30	11'-8"	102	5'-8"	6'-0"	
			508	2	11'-8"	102	5'-8"	5'-7"	
901	2	11'-8"	509	2	10'-9"	102	5'-8"	5'-1"	
			510	2	10'-3"	102	5'-8"	4'-7"	
501	6	16'-0"	511	2	9'-10"	102	5'-8"	4'-2"	
502	4	15'-0"	512	2	9'-4"	102	5'-8"	3'-8"	
503	4	14'-0"							
504	44	5'-6"	419	8	13'-6"	103	12'-5"	11"	7"
505	14	4'-10"	420	16	6'-6"	103	5'-0"	1'-3 1/2"	11 1/2"
506	16	7'-0"	421	9	8'-3"	103	2'-6"	7 3/8"	5 3/8"
			422	7	9'-0"	106	7'-9"	7 1/4"	7 1/4"
401	13	36'-0"	423	2	11'-2"	106	4'-9"	2'-8"	3'-9"
402	2	34'-6"	424	2	10'-6"	106	4'-9"	2'-4"	3'-5"
403	2	30'-6"	425	2	9'-10"	106	4'-9"	2'-0"	3'-1"
404	2	27'-0"	426	2	9'-2"	106	4'-9"	1'-8"	2'-9"
405	28	6'-1"	427	2	8'-6"	106	4'-9"	1'-4"	2'-5"
406	2	6'-9"	428	8	8'-0"	102	5'-8"	2'-9"	
407	2	6'-3"	429	8	5'-3"	102	4'-3"	1'-0"	
408	2	5'-9"	430	12	5'-6"	104	1'-6"	1'-0"	6"
409	11	5'-4"	431	8	4'-5"	105			
410	16	4'-10"							
411	22	4'-4"	201	20	12'-0"	104	2'-8"	3'-0"	6"
412	6	7'-9"	202	8	5'-11"	106	2'-3"	1'-10"	1'-10"
413	16	11'-3"							
414	62	4'-9"							
415	4	3'-0"							
416	2	2'-6"							
417	2	1'-10"							
418	9	1'-4"							

Bending Diagram
Dimensions are "out to out"



U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

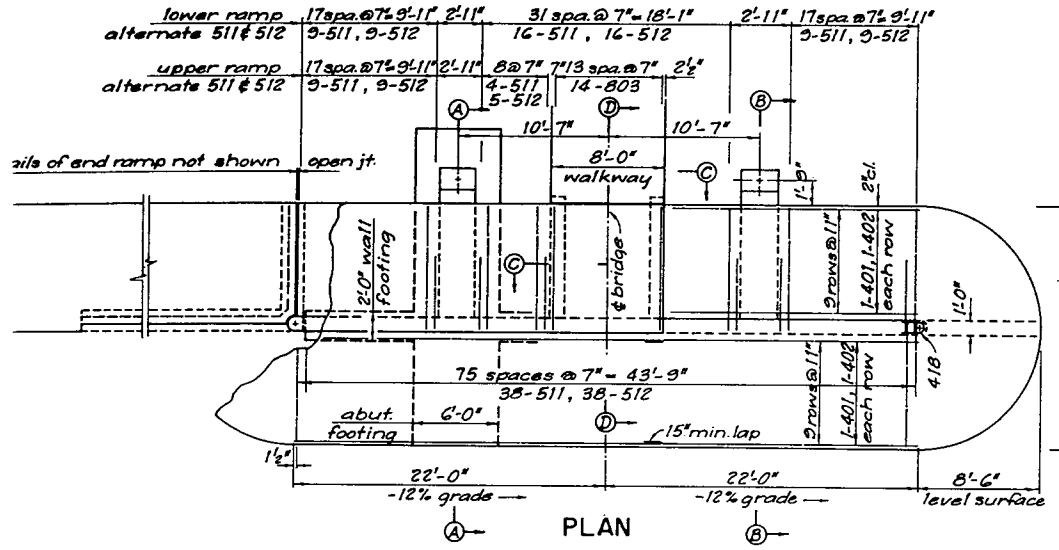
TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

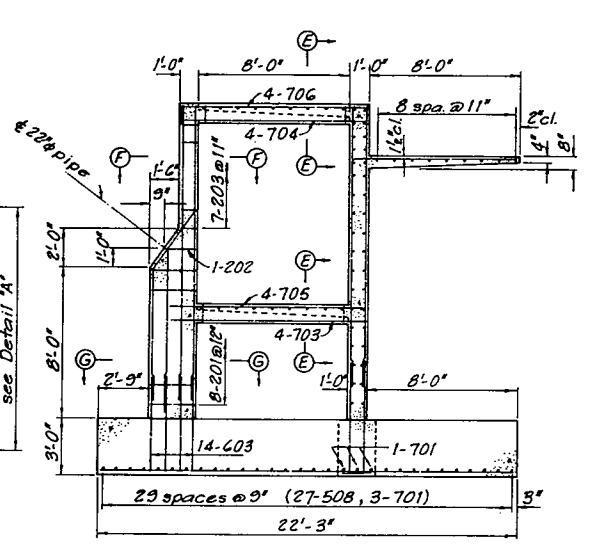
PIPE ARCH BRIDGE
160' ARCH SPAN
STAIRWAY ALTERNATE

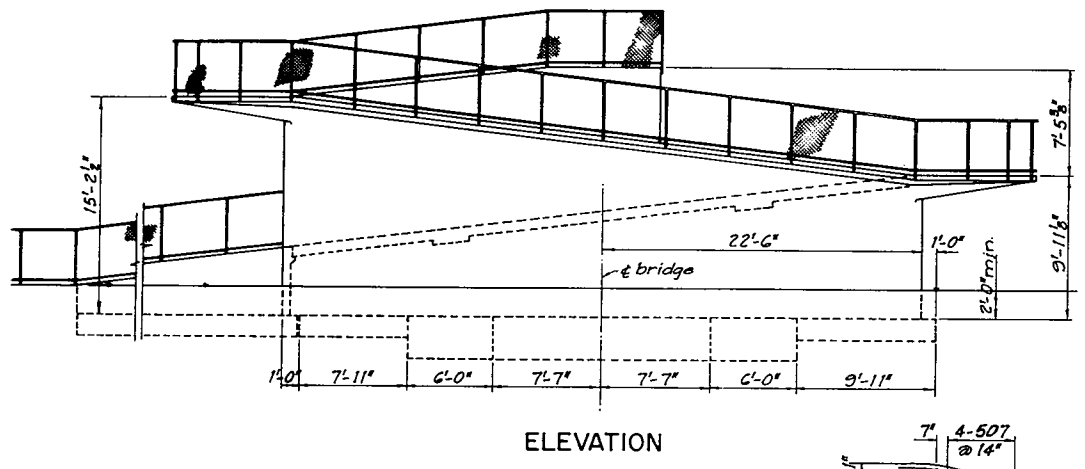
RECOMMENDED S. B. Lauer, Chief Design Branch
RECOMMENDED [illegible], Chief Bridge Division
RECOMMENDED [illegible], Director of Engineering
APPROVED [illegible], Administrator

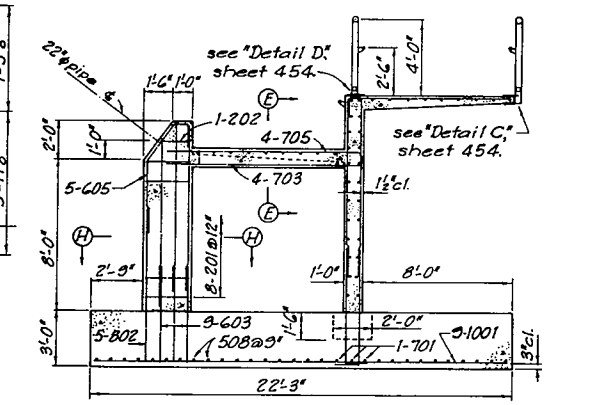
OCTOBER 1964

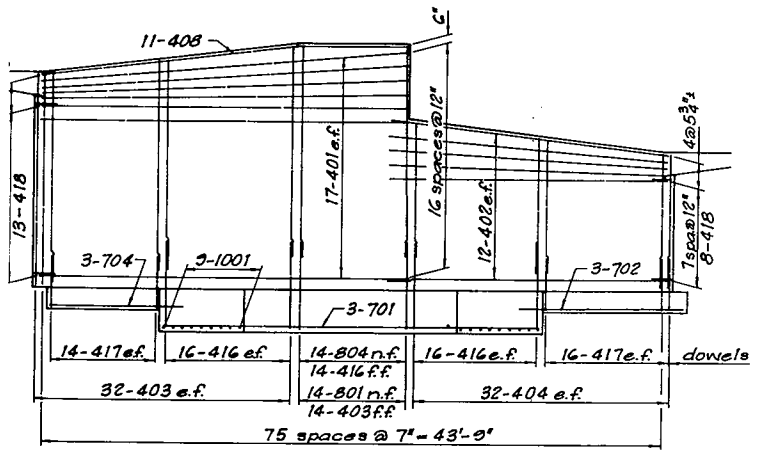
SHEET NO. 454

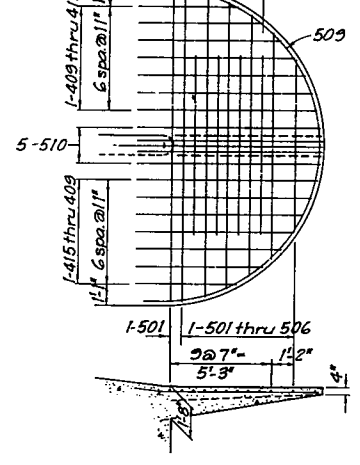


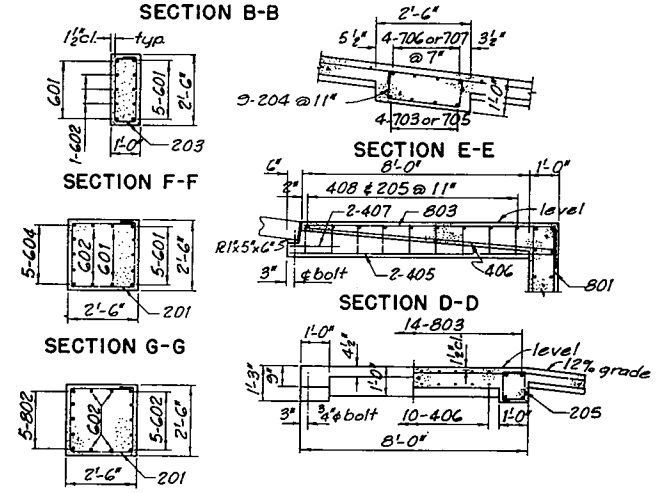










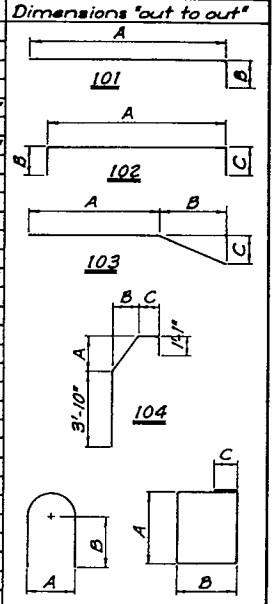


DO NOT SCALE

REINFORCING STEEL SCHEDULE

Mark	No.	Length	Mark	No.	Length	Shape	A	B	C
Straight Bars			Bent Bars						
1001	18	21'-9"	803	14	10'-9"	101	8'-9"	1'-6"	
			804	14	7'-2"	101	6'-0"	1'-2"	
801	14	15'-0"							
802	5	8'-0"	705	8	11'-3"	101	10'-3"	1'-0"	
			706	4	10'-9"	102	8'-9"	1'-0"	1'-0"
701	3	22'-8"							
702	3	11'-6"	604	5	10'-6"	103	6'-9"	8'-0"	2'-9"
703	8	10'-3"	605	5	8'-0"	104	1'-10½"	1'-6"	9"
704	7	9'-6"	509	2	28'-2"	106	16'-8"	1'-0"	
			510	10	9'-9"	103	8'-8"	1'-5⅞"	2⅝"
			511	85	9'-9"	101	8'-9"	1'-0"	
601	7	15'-6"	512	86	6'-2"	101	5'-2"	1'-0"	
602	14	8'-9"							
603	23	5'-0"	408	11	25'-10"	103	18'-0"	7'-0⅝"	11¼"
			409	4	9'-6"	103	8'-0"	1'-5⅞"	2⅝"
501	4	12'-9"	410	4	9'-4"	103	7'-10"	"	"
502	2	16'-0"	411	4	8'-11"	103	7'-5"	"	"
503	2	15'-0"	412	4	8'-4"	103	6'-10"	"	"
504	2	13'-9"	413	4	7'-8"	103	6'-2"	"	"
505	2	12'-0"	414	4	6'-8"	103	5'-2"	"	"
506	2	9'-0"	415	4	5'-4"	103	3'-10"	"	"
507	8	9'-6"	416	78	7'-2"	101	6'-0"	1'-2"	
508	54	5'-8"	417	60	8'-8"	101	3'-6"	1'-2"	
			418	21	3'-0"	106	8"	1'-0"	
			201	16	9'-6"	105	2'-9"	2'-9"	6"
401	52	25'-10"	202	2	8'-0"	105	2'-3"	1'-6"	6"
402	42	19'-2"	203	7	6'-6"	105	2'-3"	9"	6"
403	78	14'-9"	204	27	5'-9"	105	1'-10½"	9"	6"
404	64	9'-6"	205	18	3'-10"	105	11"	9"	6"
405	4	9'-3"							
406	10	8'-9"							
407	4	1'-9"							

Bending Diagram
Dimensions "out to out"



U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

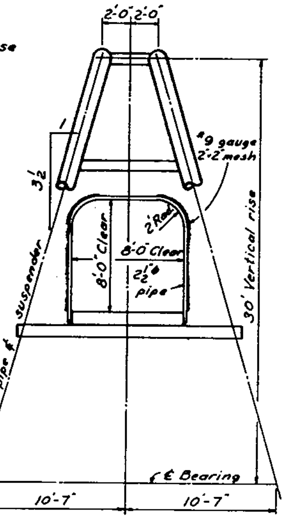
PIPE ARCH BRIDGE
160' ARCH SPAN
RAMP ALTERNATE

RECOMMENDED S. B. Lauren, Chief Design Branch
RECOMMENDED Chief Bridge Division
RECOMMENDED Director of Engineering
APPROVED Administrator
OCTOBER 1964
SHEET NO. 455

ata:
pe Arch, 30' Vertical Rise
Lateral Slope
y 8'-0" between posts
lternates
otropic Steel Plate
ss-Skin Plywood
cast Concrete
ations
with revisions to 1963
ed as per "Design Criteria"
esign Manual for Orthotropic
Plate Deck Bridge, 1963.
Specification No. SS-8

llowable Stresses

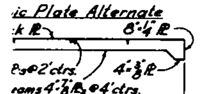
	f_s = 20,000 psi
te	f'_c = 4,000 psi
	f_c = 1,600 psi
tl. A-36	f_s = 20,000 psi
d	f_t = 1,875 psi
	f_c = 1,375 psi
53 Gr. B	f_s = 20,000 psi



ad
esign w = 85 psf
esign
$u = (30 + \frac{3000}{L})(\frac{55-W}{50}) = 46$ psf

sign

ic Plate Alternate



Thickness (AISC, Sec. 6.3, p.97)
ck plate deflection ≤ 1/300 of
ng, min. $t_e = 0.0065 a \sqrt[3]{w}$
4" w = 0.59 psi
n. t_e = 0.131" Use ¼" ℞

n (AISC Manual Sec 3.3, p.44)
Load per rib

ad	85(2.0)	= 170
rface	5.5(2.0)	= 11
℞	10.2(2.0)	= 20.4
		= 1.3
		202.7 plf

Rib Span
S_{FB} = (0.7)(48) = 33.6"

width of deck ℞ per rib
/3 = 33.6/3 = 11.2"

Properties

A	y	Ay	Ay²+Io
80	0.125	0.35	0.044 / 0.070
375	1.0	0.375	0.375
175	0.228	0.725	0.489
			− 0.165
			I = 0.324 in⁴

124/228 = 1.42 in³
124/322 = 0.213 in³
$\frac{202.7(33.6)^2}{8(12)}$ = 2380 in. lbs.
0/1.42 = 1680 psi (comp.)
0/.213 = 11,200 psi (tension)
1½"-¼" Ribs @ 2'-0" ctrs.

Design
tion = 202.7(4) = 810 lb. = P
P P P
½P
$\frac{L}{2}$) − P($\frac{L}{2}$) = $\frac{0.81(96)}{2}$ = 38.8"K

width of deck ℞ per FB
96/3 = 32"

Floorbeam Section Properties

	A	y	Ay	Ay²+Io
32-¼	8.0	0.125	1.0	0.125
4-⅜	1.5	2.25	3.375	2.00 / 7.60
	9.5	0.46	4.375	9.725
				− 2.02
			I =	7.705 in⁴

$SM_T = \frac{7.705}{0.46}$ = 16.75 in³
$SM_B = \frac{7.705}{3.79}$ = 2.03 in³
f_T = 38.8/16.75 = 2.3 ksi (Comp.)
f_B = 38.8/2.03 = 19.1 ksi (Tension) allow. = 22.0 ksi [∴ OK]

Use 4"-⅜" Floorbeam @ 48" ctrs.

20' Main Deck Unit

Use 8"-¼" Side ℞'s
4"-⅜" Bottom Flange ℞'s

Dead Load per ft.

¼·96"	= 81.6
¼(3·1½"+2·8")	= 17.4
⅜(2·4"+¼·4"·8"+¼·4"·1)	= 21.7
Fencing	= 55.0
Wearing Surface	= 44.0
	219.7 plf

Max. Moment
+M = (0.22+8·0.085)20²/8 = 45.0 'K

Section Properties

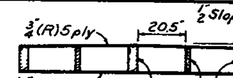
	A	y	Ay	Ay²+Io
96-¼	24.0	0.125	3.0	0.4 / 0.23
1½-¼-3	1.125	1.0	1.125	1.13 / 21.3
8-¼-2	4.0	4.25	17.0	72.25
4-⅜-2	3.0	8.44	25.25	213.0
	32.125	1.44	46.375	308.31
				− 66.81
			I =	241.5 in⁴

S_T = 241.5/1.44 = 167.5 in³
S_B = 241.5/7.185 = 33.6 in³
f_T = 45.0(12)/167.5 = 3.22 ksi (C)
f_B = 45.0(12)/33.6 = 16.07 ksi (T)
$f_{T\text{-}.175}$ = (16.07+3.22)($\frac{.175}{8.625}$) − 3.22 = +0.69 ksi

Check Live Load Deflection
Max. Allow. Δ = L/600 = 0.40"
$\Delta = \frac{5wL^4}{384EI} = \frac{5(0.68)(20)^4(1728)}{384(29000)(241.5)}$
Δ = 0.349" < 0.40" [∴ OK]

Final Rib Stress
f_T = 1.68 + 3.22 = 4.90 ksi
f_B = 11.20 + 0.69 = 11.89 ksi

Stressed Skin Plywood Deck Alternate



Design 20' Span in accordance with DFPA Spec. No. SS-8.
Material: Plywood - Ext. DFPA A-C
Stringers - Construction Grade DF

Live Load Deflection Limit = L/450

Uniform Load

Live Load 85(8)	= 680
Plywood (2.225+1.825)(8)	= 33
2-2×10's	= 9
3-4×10's	= 29
Fencing	= 55
Wearing Surface 5.5(8)	= 44
	850 plf
	106.2 psf

Section Properties

	A∥	y	A∥y	A∥y²+Io
¾ Ply(R) 8×4.5 =	36.0	.375	13.5	5
2-2×10 = 2×15.44 =	30.88	5.5	170	232 / 935
3-4×10 = 3×34.44 =	103.32	5.5	570	777 / 3140
⅝ Ply(S) = 8×4.5 =	36.0	10.56	380	4000
	206.2	5.495	1133.5	9089
				− 6223
		I (8' panel)	=	2866 in⁴

E = 1,760,000

Allow. Live Load based on Δ (℔2.4.5)
$w = \frac{2EI}{15cL^3} = \frac{2(1,760,000)2866}{15(450)(20)^3(2)}$ = 93 psf
actual = 85 psf [∴ OK]

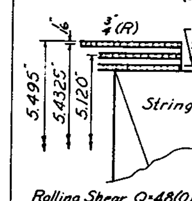
Check Top Skin Deflection (℔2.4.6)
ℓ/450 = 20.5/450 = 0.0456" allow.
$\Delta = \frac{w\ell^4}{4608EI} = \frac{85(20.5)^4}{4608(1,600,000)(.123+.299/20)}$ = 0.0148" < Allow. [∴ OK]

Determine Allow. Stresses (℔2.5.3)
Top Skin b = 38.2"
$\frac{\text{clear span}}{b} = \frac{20.5}{38.2}$ = 0.537
Allow. c∥ = 1375(0.8)(1.1)(0.96) = 1160 psi
Bottom Skin b = 29.1"
$\frac{\text{clear span}}{b} = \frac{20.5}{29.1}$ = 0.705
Allow. t∥ = 1875(0.8)(1.1)(0.86) = 1420 psi

Allow. Load based on bending stress (℔2.5.4)
$w_c = \frac{fI}{6c\ell^2} = \frac{1160(2866)(0.5)}{6(5.495)(20)^2}$ = 126 psf
$w_t = \frac{1420(2866)(0.5)}{6(5.38)(20)^2}$ = 157 psf
Actual = 106.2 psf [∴ OK]

Rolling Shear (℔2.6)

$v = \frac{VQ}{Ib}$ Standard Equation
Converted to an allow. uniform load as determined by rolling shear
$w = \frac{I(\Sigma st)}{2Q\ell}$
w = Allow. Load (psf)
I = I of 4' wide panel (in⁴)
Q = 1st moment of the ∥ plies outside the critical rolling shear plane in 4' wide panel (in³)
ℓ = panel length (ft)
Σst = sum of the glueline widths over each stringer, times allow. rolling shear stress (lbs/in)



Rolling Shear Q = 48(0.125)(5.12+5.4325) = 63.3 in³
Allow. rolling shear stress
s = 72(1.1) = 79.2 psi (Interior stringer)
s = 79.2(0.5) = 39.6 psi (Exterior stringer)
Σst = 1(1.5)79.2 + 1(39.6)(3.5+1.75) = 326.8 lbs/in
$w = \frac{2866(326.8)}{2(2)(63.3)20}$ = 185 psf
actual = 106.2 psf [∴ OK]

Horizontal Shear (℔2.7)
$w = \frac{vIt}{2QL}$
v = allow. shear in stringer (psi) = 100 psi (see ℔2.7.2) This is good for either Hemlock or Douglas Fir stringers.
t = sum of stringer widths (4' panel)
Q = 1st moment of all ∥ grain material above or below neutral axis.

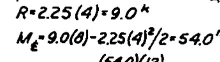
Q = 48(0.375)(5.12) + 6.75(4.745)²/2 = 92.2 + 75.8 = 168.0 in³
$w = \frac{100(2866)(0.5)6.75}{2(168)20}$ = 144 psf
actual = 106.2 psf [∴ OK]

Therefore the section as shown is adequate. The critical load condition is deflection under live load.

Deck Support Beam

Uniform Load
Orthotropic Deck = 900 plf
Stress-Skin Deck = 850 plf
Use 900 plf for design

Uniform Load on WF
900(20)/8 = 2250 plf

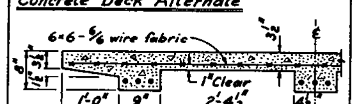


R = 2.25(4) = 9.0ᵏ
$M_℄$ = 9.0(8) − 2.25(4)²/2 = 54.0'ᵏ
Req'd. SM = $\frac{(54.0)(12)}{(20.0)(1.1)}$ = 29.4 in³

Use 8WF35 (SM = 31.1 in³)

(8WF48 Req'd. with concrete deck)

Concrete Deck Alternate

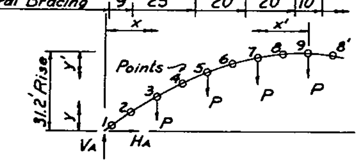


Slab Design
w = 85 + 37.5 = 122.5 psf
Req'd. $A_s = \frac{(0.1225)(2.375)^2(12)/10}{22(.875)1.90}$ = 0.023 in²/ft.
Use 6×6-6/6

20' Unit Design
w = 680 + 480 = 1160 plf
Req'd. $A_s = \frac{(1.16)(20)^2(12)/8}{22(.875)(6.125)}$ = 5.9 in²
Use 3-#8 Rebars each beam

Arch Analysis

Segment No.	1	2	3	4	5	6	7	8	9	8'
Horizontal Length	5	10'	10'	10'	10'	10'	10'	10'	10'	
Lateral Bracing	9'	25'			20'		20'		10'	



Design Loads (Vertical)
Pipe = 60 plf (assumed)
P_{DL} = (0.22)20(0.5) = 2.2ᵏ
Live Load = 46 psf AASHO 1.2.11(A)
P_{LL} = (0.046)4(20) = 3.7ᵏ
Lateral Bracing = 46 plf

Length of Arch
$L = \sqrt{4\bar{y}^2 + \bar{x}^2} + \frac{\bar{x}^2}{2\bar{y}} \log_e \frac{2\bar{y} + \sqrt{4\bar{y}^2 + \bar{x}^2}}{\bar{x}}$
= 101.463 + 102.552 $\log_e$ 2.0484 = 174.99'

Arch Geometry
$p = \frac{31.20374}{(80)^2}$ = 0.00487558
$y' = p(x')^2 = 0.00487558(x')^2$
$\tan\alpha = \frac{dy'}{dx'} = 2p(x') = 0.00975116(x')$

Pt.	x'	y'	x	y	tan α	α	sin α	cos α	Segment Length	Segment Wt.	Brace Wt.
A	80	31.20	0	0	.78009	37°57'30"	.61509	.78846			
1	77.5	29.28	2.5	1.92	.75571	37°04'40"	.60290	.79782	6.27	.38	
2	70	23.89	10	7.31	.68258	34°19'00"	.56377	.82593	12.11	.73	.37
3	60	17.55	20	13.65	.58507	30°19'50"	.50499	.86313	11.59	.69	
4	50	12.19	30	19.01	.48756	25°59'30"	.43824	.89886	11.13	.67	.09
5	40	7.80	40	23.40	.39005	21°18'30"	.36339	.93164	10.73	.64	.09
6	30	4.39	50	26.81	.29253	16°18'20"	.28076	.95978	10.42	.63	.05
7	20	1.95	60	29.25	.19502	11°02'10"	.19143	.98151	10.19	.61	.05
8	10	0.49	70	30.71	.09751	5°34'10"	.09705	.99528	10.05	.60	.03
9	0	0	80	31.20	0	0°	0	1.0	2×5.00	2×.30	2×.03

Σ Segment Length (Total Span) = 2 × 87.49 = 174.98

Live Load

Design Load = P_{LL} × secant arch lateral slope ∡ = 3.7(1.04) = 3.85ᵏ

	Load @ Pt.#3	Load @ Pt.#5	Load @ Pt.#7	Load @ Pt.#9
V_A	3.85(140/160) = 3.369ᵏ	3.85(120/160) = 2.8875ᵏ	3.85(100/160) = 2.406ᵏ	3.85(80/160) = 1.925ᵏ
V_B	0.481ᵏ	0.9625ᵏ	1.444ᵏ	1.925ᵏ

Pt.	Mo	Mo·y·Δℓ	H·y	M3	Mo	Mo·y·Δℓ	H·y	M5	Mo	Mo·y·Δℓ	H·y	M7	Mo	Mo·y·Δℓ	H·y	M9
A																
1	8.42	101	2.92	+ 5.50	7.22	87	5.30	+ 1.92	6.01	72	6.85	− 0.84	4.81	58	7.39	− 2.58
2	33.69	2982	11.11	+22.58	28.88	2557	20.18	+ 8.70	24.06	2130	26.10	− 2.04	19.25	1704	28.14	− 8.89
3	67.38	10660	20.75	+46.63	57.75	9136	37.67	+20.08	48.12	7613	48.73	− 0.61	38.50	6091	52.55	−14.05
4	62.56	13237	28.89	+33.67	86.63	18329	52.47	+34.16	72.19	15274	67.87	+ 4.32	57.75	12219	73.19	−15.44
5	57.75	14500	35.57	+22.18	115.50	29000	64.58	+50.92	96.25	24167	83.54	+12.71	77.00	19410	90.09	−13.09
6	52.94	14804	40.75	+12.19	105.88	29607	74.00	+31.88	120.31	33642	95.71	+24.60	96.25	26914	103.22	− 6.97
7	48.13	14346	44.46	+ 3.67	96.25	28688	80.73	+15.52	144.37	43031	104.42	+39.95	115.50	34426	112.61	+ 2.89
8	43.31	13367	46.68	− 3.37	86.63	26737	84.76	+ 1.87	129.94	40110	109.63	+20.31	134.75	41589	118.23	+16.52
9	38.50	12013	47.42	− 8.92	77.00	24027	86.27	− 9.27	115.50	36040	111.38	+ 4.12	154.00	48054	120.12	+33.88
8'	33.69	10398	46.68	−12.99	67.38	20796	84.76	−17.38	101.07	31194	109.63	− 8.56				
7'	28.87	8605	44.46	−15.59	57.75	17213	80.73	−22.98	86.63	25821	104.42	−17.79				
6'	24.06	6728	40.75	−16.69	48.13	13458	74.00	−25.87	72.19	20186	95.71	−23.52				
5'	19.25	4833	35.57	−16.32	38.50	9667	64.58	−26.08	57.75	14500	83.54	−25.79				
4'	14.43	3053	28.89	−14.46	28.88	6111	52.47	−23.59	43.32	9166	67.87	−24.55				
3'	9.62	1522	20.75	−11.13	19.25	3045	37.67	−18.42	28.88	4569	48.73	−19.85				
2'	4.81	426	11.11	− 6.30	9.63	852	20.18	−10.55	14.44	1278	26.10	−11.66				
1'	1.20	14	2.92	− 1.72	2.41	29	5.30	− 2.89	3.61	43	6.85	− 3.24				
B																
Σ		131589				239339				308836				332876		

$H = \frac{\Sigma M_o \cdot y \cdot \Delta\ell}{\Sigma y^2 \cdot \Delta\ell}$ = 1.52ᵏ; H = 2.76ᵏ; H = 3.57ᵏ; H = 3.85ᵏ

Dead Load

Design Load = Vertical Loads × secant arch lateral slope ∡

Pt.	D.L.	V	Δx	Mo	y	Δℓ	Mo·y·Δℓ	y²·Δℓ	H·y	Mo	V sin α	H cos α	N
A		14.21	2.5										
1	0.40	13.81	7.5	35.5	1.92	6.27	427	23	36.7	−1.2	8.36	15.25	23.61
2	1.14	12.67	10.0	139.1	7.31	12.11	12314	647	139.8	−0.7	7.14	15.79	22.93
3	3.00	9.67	10.0	265.8	13.65	11.59	42050	2159	261.0	+4.8	4.93	16.50	21.43
4	0.79	8.88	10.0	362.5	19.01	11.13	76698	4022	363.5	−1.0	3.89	17.19	21.08
5	3.04	5.84	10.0	451.3	23.40	10.73	113313	5875	447.4	+3.9	2.12	17.81	19.93
6	0.71	5.13	10.0	509.7	26.81	10.43	142526	7497	512.6	−2.9	1.44	18.35	19.79
7	2.97	2.16	10.0	561.0	29.25	10.19	167210	8718	559.3	+1.7	0.41	18.77	19.18
8	0.67	1.49	10.0	582.6	30.71	10.05	179811	9478	587.2	−4.6	0.14	19.03	19.17
½×9	1.49	0		597.5	31.20	5.00	93210	4867	596.5	+1.0	0	19.12	19.12
Σ							827559	43286					

H = Σ Mo·y·Δℓ / Σ y²·Δℓ = 19.12ᵏ

Live Load Deflection Requirements

Deflection at Pt. 5 — Loads: P_3 = 3.85ᵏ, P_5 = 3.85ᵏ, P_7 = 3.37ᵏ, P_9 = 0.48ᵏ
Load from Pt. A to Pt. 8

Pt.	M3	M5	M7	M9	Mo = ΣM	m5	Δℓ	Mo·m5·Δℓ
A								
1	+ 5.50	+ 1.92	− 0.74	− 0.32	+ 6.36	+ 0.50	6.27	20
2	+22.58	+ 8.70	− 1.78	− 1.11	+28.39	+ 2.26	12.11	777
3	+44.63	+20.08	− 0.53	− 1.76	+62.42	+ 5.22	11.59	3776
4	+33.67	+34.16	+ 3.78	− 1.93	+69.68	+ 8.87	11.13	6879
5	+22.18	+50.92	+11.12	− 1.64	+82.58	+13.23	10.73	11723
6	+12.19	+31.88	+21.52	− 0.87	+64.72	+ 8.28	10.42	5584
7	+ 3.67	+15.52	+34.96	+ 0.36	+54.51	+ 4.03	10.19	2239
8	− 3.37	+ 1.87	+17.77	+ 2.06	+18.33	+ 0.49	10.05	90
9	− 8.92	− 9.27	+ 3.60	+ 4.24	−10.35	− 2.41	10.00	249
8'	−12.99	−17.38	− 7.49	+ 2.06	−35.80	− 4.51	10.05	1623
7'	−15.59	−22.98	−15.57	+ 0.36	−53.78	− 5.97	10.19	3272
6'	−16.69	−25.87	−20.58	− 0.87	−64.01	− 6.72	10.42	4482
5'	−16.32	−26.08	−22.57	− 1.64	−66.61	− 6.77	10.73	4839
4'	−14.46	−23.59	−21.48	− 1.93	−61.46	− 6.13	11.13	4193
3'	−11.13	−18.42	−17.37	− 1.76	−48.68	− 4.78	11.59	2697
2'	− 6.30	−10.52	−10.20	− 1.11	−28.13	− 2.74	12.11	933
1'	− 1.72	− 2.89	− 2.84	− 0.32	− 7.77	− 0.75	6.27	37
B								
							Σ =	53,413

$\Delta_{max} = \frac{L}{600} = \frac{\Sigma M_o m_5 \Delta\ell}{EI}$ Req'd. $I = \frac{53{,}413(1728)600}{29{,}000(160)(12)}$ = 1005 in⁴

Use 22"φ Schedule 10 Pipe I = 1010 in⁴
A = 17.08 in² t = ¼" SM = 91.84 in³ r = 7.69" wt. = 58.07 plf

Stress Analysis

Assume: $F_A = 16{,}000 - 0.39(\frac{L_u}{r})^2$ and F_B = 20,000 psi

% Capacity = $\frac{(N/A)}{F_A} + \frac{(M/SM)}{F_B}$ Allow 110%

Pt.	Live Load M	Live Load N	DL+LL M	DL+LL N	Lu (ft.)	FA (ksi)	(N/A)/FA	(M/S)/FB	% Capacity
3	+66.4 / −63.8	7.99 / 14.66	+71.2 / −59.0	29.42 / 36.09	28.71	15.22	10.3 / 12.9	46.5 / 38.6	56.8 / 51.5
4	+70.5 / −75.9	8.71 / 14.75	+69.5 / −76.9	29.79 / 35.83	28.71	15.22	11.4 / 13.8	45.4 / 50.2	56.8 / 64.0
5	+82.6 / −79.0	8.36 / 14.74	+86.5 / −75.1	28.29 / 34.67	21.15	15.58	10.6 / 13.0	56.6 / 49.1	67.2 / 62.1

Note: Size of the arch pipe is controlled by the live load deflection limitation of L/600 and stress in the pipe is not critical.
The stress analysis above is based on dead load of 220 plf for an orthotropic plate deck. If the concrete deck alternate with a dead load of 480 plf is used, the maximum stress in the arch pipe at Pt.#5 will be 75% of capacity.

U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

TYPICAL PEDESTRIAN BRIDGES
(8 FT. WALKWAY)

PIPE ARCH BRIDGE
160' ARCH SPAN
DESIGN CALCULATIONS

RECOMMENDED S. B. [signature] Chief Design Branch
RECOMMENDED [signature] Chief Bridge Division
RECOMMENDED [signature] Director of Engineering
APPROVED [signature] Administrator

OCTOBER 1964
SHEET NO. 456

DO NOT SCALE

APPENDIX A

)ESIGN CRITERIA FOR PEDESTRIAN BRIDGES

:RAL

·ose of this criteria has been to establish basic data for the development signs and the preparation of the final detail drawings included in this This same criteria is suggested as a basis for design and detailing)edestrian bridges.

)UT USED FOR BRIDGES DETAILED HEREIN

ituation has been assumed as follows:

ew 0°

adway

Median=40′–0″ providing width for future additional lanes.
Roadways=2 at 36′–0″.
Shoulder=10′ minimum.

arances

Horizontal clearance from right edge of roadway pavement to face of pier or abutment=12′–0″.
Vertical clearance over roadway and future lanes=17′–0″. Vertical clearance over full width of shoulder=16′–0″.

ure cross section

izontal clearance=8′–0″ between fence posts.
tical clearance=8′–0″.
ce; use chain link mesh mounted outside of posts, and arc inward ially from vertical sides.
umed maximum grade 12 percent.

ERAL CRITERIA

Specifications

cifications. Except where modified or amplified in this criteria, the gn of the structures shall be in accordance with the following specifi-)ns:

'he current "Standard Specifications for Highway Bridges" of the merican Association of State Highway Officials and all Tentative tevisions thereto which have been adopted by the AASHO Bridge 'ommittee.

'he current American Welding Society Specifications for Highway ridges for welded construction.

'he current National Lumber Manufacturers Association Specifica- ons for timber structures.

he current Douglas Fir Plywood Association Specification SS–62D r plywood stressed skin panels.

he current American Institute of Steel Construction "Specifications r the Design, Fabrication, and Erection of Structural Steel for uildings," for welded girder intermediate stiffener design.

ing

ead load—No allowance for a future wearing surface shall be made.

ve load—Sidewalk live load shall be in accordance with AASHO)ecification, Article 1.2.11 (A).

arth pressure—For design, the equivalent fluid pressure shall be irty (30) pounds per cubic foot.

d. Lateral soil pressure for design of pedestal piles shall be assumed as 300 p.s.f. per foot of depth, based on compact coarse and fine sand, medium stiff clay, or better.

e. **Wind loads**—Acting horizontally in any direction.
   (1) Wind pressure (p)=45 p.s.f.
   (2) Supplementary factors for determination of wind pressure on structural members.

Type of Member	*Shape of Member*	
	Flat or Angular	*Cylindrical*
Girders and beams	1. 30	0. 83
Trusses and arches	2. 20	1. 43
Substructures	1. 30	0. 83

   (3) Supplementary factors for determination of wind pressure on chain link fencing as applied to the gross area as seen in elevation=0.12.
   (4) Compliance with Article 1.2.14(c) of AASHO Specifications shall not be necessary.

f. **Ice load**—Ice or snow load shall be applied as follows: The larger of the two loading conditions shall be used.
   Load 1: 3 p.s.f. applied to the entire surface of exposed structural member
   1 p.s.f. applied to the entire area of chain link fence.
   10 p.s.f. applied to the deck area.
   Load 2: 20 p.s.f. applied to the deck area.
   These values are averages and shall be eliminated or increased depending on the geographic location of the proposed design. The above values have been used for these standards.

g. **Thermal loads**—Provisions shall be made for stresses or movements resulting from the following range of temperatures:
   Temperature rise 35° F.
   Temperature fall 45° F.

h. **Shrinkage**—For concrete structures, provision shall be made for the stresses or movements resulting from the shrinkage of concrete.

3. **Group Loads**

The loads described in section 2 shall be combined in groups and all components of the substructure or superstructure shall be designed for the combination producing the maximum effect using normally allowable stresses increased as indicated for the group loads.

Loads	*Percent of normally allowable stress*	*Including Temperature and Shrinkage*
Group 1 DL+LL	110	140
Group 2 DL+Ice+½LL	110	140
Group 3 DL+Wind	145	160
Group 4 DL+Ice+20# Wind	145	160
Group 5 DL+½LL+15# Wind	145	160

4. **Unit Stresses and Material**

a. **Concrete**—shall have a 28-day compressive strength of 4,000 p.s.i.

b. **Steel**
   (1) **Reinforcing steel**
      (a) Allowable unit stresses in accordance with the AASHO Specifications for intermediate, hard, or rail steel grades.
      (b) Reinforcement shall be deformed in accordance with ASTM Specifications A–15 or A–16.
   (2) **Structural Steel**
   Structural carbon steel and high strength low alloy steel shall conform to designation ASTM A–36 and ASTM A–441, respectively.
   (3) **Welding** of structural steel shall be in accordance with the AWS Specifications dated 1963 with exceptions shown on the detail drawings pertaining to cover plate welding.
   (4) **High-tensile strength bolts** shall be ASTM A–325.

c. **Structural Lumber and Plywood**
   (1) **Structural Lumber.** Allowable unit stresses, and fabrication shall conform to the "National Design Specification for Stress-Grade Lumber and Its Fastenings," published by the National Lumber Manufacturers Association, 1962 Edition.
   (2) **Plywood**
      (a) Plywood shall have allowable unit stresses in accordance with Douglas Fir Plywood Association General Design Specification No. 1, "Designs of Plywood Lumber Structural Assemblies," suitably modified as covered therein for conditions and duration of loading.
      (b) Fabrication of plywood lumber stressed skin panels shall be in accordance with Douglas Fir Plywood Association Fabrication Specification No. SS–8, "Fabrication of Plywood Stressed Skin Panels."

B. **Features of Superstructure**

1. **Walkway** width shall be 8′–0″ clear between posts. Minimum thickness of slab shall be 3½″. Minimum concrete cover shall be 1½″ top of slab, 1″ elsewhere. Maximum spacing of main deck reinforcement shall be twice the depth of the deck.
2. **Drainage**—Use ½ inch cross slope on walkway deck wherever possible. No other provision for drainage on the structure shall be made.
3. **1¼ inch diameter pipe handrails** shall be set 2′–6″ above walkway.
4. **Galvanized Fence and Supports**
   a. Use chain-link fence fabric conforming to ASTM A–392, No. 9 gage wire, 2 inch mesh, galvanized after weaving with a Class II zinc coating.
   b. Galvanized steel pipe rails and supports shall conform to ASTM A–53, Grade B, Type E or S. ASTM A–36 steel shall be used for all structural steel and hardware, and after fabrication, shall be galvanized in accordance with ASTM A–123 and A–153.
   c. Damage to galvanizing from field welding and erection handling shall be repaired with material that conforms to Military Specification MIL–P–21035 (SHIPS).
   d. Fence shall have vertical sides, topped with a curved section.

C. **Features of Substructure**

1. **Pier Columns**—Except where otherwise specified use 18 inch diameter or larger spiral columns and where possible, monolithic with the superstructure.
2. **Abutments** shall be monolithic with cast-in-place concrete superstructures and where practical, rigidly connected by monolithic deck slab and fixed shoes to all other superstructures.
3. **Foundation pressures** assumed for the design of the bridges in this volume are as follows:
   a. Rock—5 tons per square foot.
   b. Embankment—1 ton per square foot.
   c. Earth—2 tons per square foot.

EL BEAM AND GIRDER CONSTRUCTION

n Specifications

mber

Rolled beams

(1) Rolled beams shall not be cambered if the amount of dead load deflection and vertical curvature is less than one inch.

(2) The camber called for in the plans shall not be less than that likely to remain in the beam as indicated by the AISC Handbook (p. 1–95).

Welded plate girders shall be cambered throughout their length.

flection

flections due to live load shall be computed and preferably shall not eed 1/600 of the span, the span length being considered as the distance ıter to center of bearing. The moment of inertia of the gross cross- tional area shall be used in computing the deflection of beams and ders.

pth ratios

e ratio of the depth to length of spans preferably shall be not less than 5 for rolled beams and welded girders. For continuous spans, the span gth shall be considered as the distance between dead load points of lection.

ures of Superstructure

aming

Main supporting members

Rolled beams heavier than 194 lbs. shall not be used.

Beam and girder details

(1) Shop splices for welded girders or rolled beams shall be butt welded. Field splices shall be bolted using high strength bolts.

(2) Field splices for continuous beams or girders shall be made at or near the dead load inflection points. Splices shall be so arranged that the individual pieces will be 90 feet or less in length.

(3) Cover plates on rolled beams shall be extended at least 12″ beyond the theoretical cutoff point. The end of the cover plate shall be cut square across and both weld and cover plate tapered, in thickness, to a slope of 3 on 1. For complete details see sheet No. 402 this volume.

(4) Butt welded girder flange splices shall be made where the larger plate has been extended 1 foot beyond the point where the smaller plate would have a stress equal to the allowable stress.

(5) In order to maintain a constant width of girder flange, the thickness of plate shall be varied. The desirable maximum increase in plate size is 50%, this may be exceeded if necessary, but not to exceed 100% at any one splice.

(6) Welded plate bearing stiffeners shall be used where needed on all rolled beams and girders.

(7) Design of intermediate stiffeners for welded girders shall be in accordance with AISC Specifications, Article 1.10.5.

Diaphragms

(1) Spacing of diaphragms shall not exceed 35 feet.

(2) Abutment backwall may be considered as the end diaphragm if monolithic with the superstructure.

xpansion—For structure length of 300 feet or less, expansion may be sorbed by the abutment movement.

earing devices—Except where the magnitude of the reaction or length expansion would render their use impractical, shoes shall be fixed at ıe abutments. Elastomeric pads shall be used on the piers.

V. CAST-IN-PLACE CONCRETE CONSTRUCTION

A. Design Specifications

1. **Deflections**
   Deflections shall be computed for the structure as a unit using the gross moment of inertia of the concrete and steel section and a modulus of elasticity of 1/30 that of steel. Deflections shall be given for full dead load to the nearest ⅛ inch.
2. **Special Design Provisions**
   a. **For superstructure design,** the following assumptions and criteria shall be used:
      (1) **General**—In the region of moment reversal the concrete shall be assumed as not capable of carrying shear and the stirrups shall be designed to carry the entire shear.
      (2) **Box girder construction**
         (a) In distribution of moments, a continuous frame with variable moments of inertia, and monolithic columns shall be used. Columns shall be assumed hinged at the footings. Column action shall be assumed distributed equally to both girders. For relative moment of inertia of the box section, slab and webs shall be used and fillets neglected.
         (b) Girders shall be designed as T-beams neglecting fillets and compression in the web. Slabs preferably shall be of uniform thickness throughout the structure.
         (c) Web thickness at the support shall be determined by shear requirements. Minimum thickness shall be 6 inches. Where greater thickness is required, it shall be by full inch increments and tapered in 12 times the thickness increase but not less than 4 feet. Taper on inside of webs only.
         (d) Vertical stirrups shall be provided as required by diagonal tension but the spacing shall not exceed 1 foot 6 inches.
         (e) Shear keys need not be provided between web and slab. A scored surface shall be used for horizontal construction joints.
      (3) **T-beam constructions**
         (a) In the distribution of moments, a continuous frame of variable moment of inertia and monolithic columns shall be assumed. Columns shall be assumed hinged at the footings.
         (b) Vertical stirrups shall be provided as required by diagonal tension but the spacing shall not exceed 1 foot 6 inches.
   b. **For substructure design,** the following assumptions and criteria shall be used:
      (1) Bents shall be designed for longitudinal forces due to temperature rise of 35 degrees, temperature fall of 45 degrees, and a shrinkage of 0.0002 foot per foot.
      (2) Columns shall be designed as spiral columns.
      (3) Wind load shall be distributed to the bents according to their resistance. For distribution, the superstructure (gross moment of inertia) shall be assumed as a beam supported on unyielding pin supports at the abutment and yielding supports at the piers.
      (4) In the design of the columns and footings, torsional resistance of the superstructure may be considered in the application of eccentric live load.

B. Features of Superstructure

1. **Walkway**
   a. **Box girder construction**
      (1) Top and bottom slabs shall have a minimum thickness of 3½ inches or $\frac{1}{4}$ the clear distance between girder webs, whichever is greater. Thickness shall be in increments of ¼ inch.
      (2) The minimum reinforcement in the top and bottom flanges shall be in accordance with AASHO Article 1.7.11(f) if greater than design requirements. Bottom slab transverse bar spacing shall not exceed 12 inches. Spacing of main (top) slab reinforcement shall not exceed twice the slab thickness.
   b. **T-beam construction**—Omit horizontal construction joint between slab and stem, therefore, AASHO Article 1.7.5 (G) will not apply.
   c. **Combination box girder and T-beam** shall be in accordance with provisions herein for Box Girders.
2. **Framing**
   a. **Construction joints**
      (1) Construction joints shall be indicated on the plans where permissible.
      (2) Transverse joints, when required, shall be placed 0.2 span lengths from the pier on interior spans and 0.25 span lengths from the pier on end spans.
   b. **Box girder details**
      (1) Spacing of diaphragms shall not exceed 35 feet.
      (2) No access into the box section need be provided.
      (3) Drain holes 3 inches in diameter and covered with a copper screen shall be provided at low points of the bottom slab. Drain holes 2 inches in diameter shall be provided at the bottom of each diaphragm.

VI. CONCRETE PRECAST CONSTRUCTION—POST-TENSIONED

This criteria is based on elastic design analysis using the loading conditions as given in Section III, General Criteria. In addition, the members are checked by ultimate strength theory for compliance with load factors of 1.5 DL+2.5 LL.

A. Design Specifications

1. **Materials**
   The steel used in the post-tensioning operations shall be one of the types listed below:
   a. High-tensile wire conforming to ASTM Designation A–421.
   b. High-tensile 7-wire strand conforming to ASTM Designation A–416.
   c. High-tensile alloy bars having a minimum ultimate tensile stress of 145,000 p.s.i.
2. **Unit Stresses**
   a. The required strength of the concrete at time of transfer of prestress shall be not less than 3,200 p.s.i.
   b. For the post-tensioned method the losses of prestress shall be used as 25,000 p.s.i. not including either friction or anchorage slippage losses. These latter two loss factors shall be assigned after the type of post-tensioning system is selected and the drape of the elements is determined.
   c. All other stresses shall be in accordance with the specifications for prestressed concrete, AASHO Article 1.13.
3. **Deflections and Camber**
   a. Camber sections for residual deflection and deck curvature.
   b. For computation of girder deflection due to post-tensioning the modulus of elasticity (Ec) shall be used as 3,400,000 p.s.i.
   c. The drawings shall show the camber as computed for prestress, the dead load deflection and the difference or "residual camber" at the quarter points of all spans, if significant.
4. **Special Design Provisions**
   a. Post-tensioning cables shall be designed for the live load and dead load moments.

. The joints between the precast sections shall be at the dead load points of contraflexure.
. Using a coefficient of friction value of 0.4 on the area of the joint and the post-tensioning force applied, check for possible slippage due to shear.

ures of Superstructure

he ends of the precast sections shall be constructed normal to the profile rade and square with the centerline of bridge.
Ise epoxy mortar in the joints prior to post-tensioning to provide ven distribution of stress between elements. The mortar may be mitted if the adjacent elements have been cast against one another.
ach precast unit shall be reinforced for its dead load.

tures of Substructure

iers

. Columns may be doweled to the precast superstructure sections and assumed monolithic.
). The dowels for the piers shall be grouted prior to post-tensioning of the deck.

lternate or additional substructure details shall be determined at time f design.

AMINATED STRUCTURAL LUMBER CONSTRUCTION—STRINGERS AND ARCHES

ign Specifications

Materials.

.. **Glued-laminated structural timber.** Allowable unit stresses, 2,000 p.s.i. for extreme fiber in bending and 145 p.s.i. for horizontal shear, for wet conditions of use. See Table No. 20 b of the "National Design Specifications for Stress Grade Lumber and Its Fastenings", published by the National Lumber Manufacturers Association for grades of lumber to satisfy the above requirement of unit stresses. Maximum nominal thickness of laminations shall be 2 inches or less as required for curvature.
). **Timber decking.** Allowable unit stress of 1,200 p.s.i. for decking lumber in accordance with Table No. 1 of the above-mentioned specifications.
:. **Stress-skin plywood decking**—See Section XI.
1. **Bearing devices** for stringers shall consist of elastomeric pads, or 18 gauge, zinc coated corrosion resistant sheet steel.

Special Design Provisions

a. Glued-laminated stringers, arches, and diaphragms, and tongue and groove timber decking shall be designed in accordance with formulae found in the "National Design Specifications for Stress-Grade Lumber and Its Fastenings" as published by the National Lumber Manufacturers Association.
b. Stress skin plywood decking—See Section XI.
c. Glued-laminated stringers shall be designed as simple spans.
d. Glued-laminated arches shall be designed as three-hinged arches.
e. Arches shall have a lateral bracing system designed to withstand the wind forces called for in Section III A.2.
f. No field splices are allowed in the glued-laminated stringers or arches.
g. Arch reactions may be resisted by a spread footing placed approximately normal to the line of thrust of the arch.
h. Arch reactions may be resisted by a drilled shaft constructed along the line of thrust at the abutments. Shaft may be belled if necessary.

3. **Deflections and Camber**
   a. Deflections due to live load shall be computed and preferably shall not exceed 1/450 of the span, the span length being considered as the distance center to center of bearings.
   b. Glue-laminated beams and arches shall be manufactured with the camber built-in, to compensate for dead load deflection and vertical curvature.
   c. In computing dead load deflections the value of modulus of elasticity (E) shall be assumed to be 1/2 the value given in the National Lumber Manufacturers Association specifications for glued-laminated structural members (Table No. 20 b).

B. **Features of Superstructure**

1. **Framing**
   a. **Main supporting members**
      (1) Nominal depth and width for glued-laminated stringers and arches shall not be less than 10 and 6 inches respectively.
      (2) The width-to-depth ratio shall preferably vary between 1:3 and 1:5 for stringers (Arche-, 1:2).
   b. **Diaphragms**
      (1) Diaphragms shall be spaced at a maximum spacing of 25 feet.
      (2) Full depth laminated diaphragms shall be used. These shall be anchored in place with split ring connectors and high-strength tie rods. Refer to Bureau of Public Roads publication "Standard Plans for Highway Bridges," Volume V, Sheet No. 302, for details.
2. **Walkway Deck**
   a. **Timber**
      (1) Decking snall be tongue and groove planking or plywood.
      (2) Timber decking, when flat grain, shall be placed so that the face nearest the heartwood is down.
      (3) Stress-skin type plywood decking units may be used—See Section XI.
   b. **Epoxy wearing surface**
      A Type C, Coal tar modified epoxy resin system, with a surface aggregate application shall be used. The epoxy resin system and aggregate shall conform to Interim Guide Specification AASHO M200–63I. The deck surface shall be clean and free of any foreign matter detrimental to bond with the epoxy resin. Rate of application of the epoxy resin shall be according to the manufacturer's recommendations. Application of the aggregate shall be at a rate of not less than 15 pounds per square yard.
3. **Expansion**—Provision for expansion may be made with an open joint between the decking and the abutment backwall.
4. **Bearing devices**—Except where the magnitude of the reaction or expansion would render their use impractical, bearing shall be on elastomeric pads, or 18 gauge, zinc coated, corrosion resistant, sheet steel.

C. **Features of Substructure**

1. **Timber piers**—Consideration should be made for the use of framed timber bents.
2. **Pier or abutment pedestals**—For frame or arch type of superstructures, pedestals shall be as required to fit the frame and shall project a minimum of 2 feet above the surface of the ground. Similar pedestals shall be used for timber bents or columns.

VIII. ORTHOTROPIC PLATE DECKS

A. **Design Specifications**

1. **Camber**—Orthotropic plate decks shall be cambered to compensate for D. L. deflection and to meet the deck curvature requirements shown on the plans.
2. **Deflections**—Deflections due to live load shall be computed and preferably shall not exceed 1/600 of the span, the span length being considered as the distance center to center of supports. The moment of inertia of the gross cross-sectional area shall be used in computing the deflection of orthotropic plate deck sections.
3. **Materials**—All steel shall meet the ASTM Specifications for A–36 steel. The minimum allowable thickness shall be 1/4 inch.

B. **Design and framing** shall be in accordance with the "Design Manual for Orthotropic Steel Plate Deck Bridge," published by the American Institute of Steel Construction, 1963.

1. **Deck plate**—The thickness of the deck plate shall be determined by the allowable deflection under the live loading, which shall not exceed 1/300 of the spacing of the deck plate supports, unless a greater thickness is required for the deck plate acting as the flange of the ribs, floor beams, or main structural members of the bridge, or for corrosion resistance.

   The plate thickness, t_p, satisfying the above condition may be determined by the formula:

   $$t_p \geq 0.0065\ a\ \sqrt[3]{p}$$

   where a=spacing of the open ribs, or the maximum spacing of the walls of the closed ribs, in inches.
   p=design unit pressure on the top of the steel plate (0.6 p.s.i.).

   Computation of the local flexural stresses in the deck plate satisfying the above allowable deflection provision is not required.
2. **Longitudinal ribs**—shall be designed as continuous over the span length and proportioned on the basis of the moment of inertia of the effective section and the effective span length of the ribs. The longitudinal ribs shall be assumed to be supported by rigid floor beams. The effective spans of a longitudinal rib is assumed to be 0.7 x the floor beam spacing. The effective width of deck plate acting with the longitudinal ribs shall be assumed to be effective span/3 or the center to center spacing of ribs, whichever is less.
3. **Floor beams**—shall be spaced so that the floor beams can act as web stiffeners where fence posts are connected to the deck. Floor beams shall be proportioned on the basis of the moment of inertia of the effective section, assuming the floor beams are simply supported by the webs. The effective width of deck plate acting with the floor beam shall be assumed to be the floor beam span/3.
4. **Gross section**—The gross section requirements shall be determined using the gross moment of inertia of the transverse section. Stresses in the deck plate and longitudinal ribs determined by this section shall be superimposed on stresses determined in Section B2.

C. **Epoxy Wearing Surface**

Type C, coal-tar modified system, with a surface aggregate application shall be used for the epoxy wearing surface. The system and aggregate shall conform to Interim Guide Specification AASHO M200–63I. The deck surface shall be sand blasted and cleaned of foreign material detrimental to the bond of the resin. The rate of application of epoxy resin shall be in accordance with manufacturer's recommendations. Application of the aggregate shall be at a rate of not less than 15 pounds per square yard.

D. **Fabrication**

Fabrication of the orthotropic plate deck shall be in accordance with Chapter 7, Section 7.2 of "Design Manual for Orthotropic Steel Plate Deck Bridges" published by the American Institute of Steel Construction, 1963.

IX. STEEL PIPE ARCH CONSTRUCTION

A. Design Specifications

1. **Materials**
   a. Arch pipe shall conform to ASTM A–53 Grade B, electric-resistance welded, Type E or seamless, Type S.
   b. Structural pipe for the lateral bracing system and suspenders shall be the same as above for the arch pipe.
   c. Decking material is covered by other sections of this criteria.
2. **Allowable Stresses for Structural Pipe:** Use those specified for ASTM A–36 structural steel.
3. **Special Design Provisions**
   a. The parabolic shape of the arch shall be used to produce negligible dead load moments.
   b. Design lateral bracing of arch members to resist the wind forces as called for in Section III A.2.e.
   c. Arch reaction shall be resisted by a drilled shaft constructed along the line of thrust at the abutments or by a special footing placed normal to the line of thrust.
   d. Due to the improbability of repeated loadings causing reversals, provisions for alternating stresses, AASHO Article 1.6.5, need not apply.

B. Features of Superstructure

1. **Framing Main Arch Members**
   a. For the arch, the minimum permissible pipe section shall be 12-inches diameter, schedule 20.
   b. At field splices the arch sections shall be closed by plates and connections shall be by butt welding.
2. **Lateral Bracing**
   a. Lateral bracing for the arch shall be pipe struts only, no diagonals. In addition, the arch members should be brought together or nearly so at the center of the span thereby removing any need for diagonals.
   b. Pipe struts shall be approximately ⅝ of the diameter of the arch pipe and shall have a minimum wall thickness of 0.250 inch.
3. **Deck**
   a. Floor beams suspended by pipe hangers shall be used to support the deck units.
   b. Prefabricated deck panels shall be rigidly connected together so as to transfer lateral wind loads to the abutments.

C. Features of Substructure

1. Arch abutments may have drilled shafts along the line of the arch thrust. These drilled shafts may be belled if required and should penetrate natural ground a minimum of 10 feet.
2. Arch abutments may have spread footings placed normal to line of the arch thrust.
3. Arch ribs may be connected to abutments with hinge type bearing devices, or elastomeric bearing pads may be used to provide the necessary hinge action.

X. SUSPENSION TYPE CONSTRUCTION

A. Design Specifications

1. **Materials**
   a. Towers shall be either steel or concrete for which the material specification has been given in Section III A.4.
   b. Cables shall be high strength grade conforming to ASTM A–218 with a class B zinc coating. Cables shall be prestretched to 50% of ultimate strength.
   c. For all wire cables and fittings a safety factor of 3 shall be used.
   d. Orthotropic plate deck is covered in Section VIII.
2. **Special Design Provisions**
   a. Cables shall be straight stays from tower to approximately the third points.
   b. Different size cables may be used to correct for unequal deflections at cable connections.
   c. The deck shall be designed as a beam on elastic supports.
   d. The tower located at centerline of median shall be designed as an A-frame.
   e. The compression on the deck from the cables should be considered in the design of the deck.
3. Live load deflection in any one deck panel between cable connections shall be restricted to 1/600 of the span of the panel.

B. Features of Superstructure

1. Orthotropic plate deck panels shall be welded together so as to transfer the lateral wind loads to the substructure.
2. Cables shall be anchored to the towers.

C. Features of Substructure

1. The spread of the tower footings should be such as to resist the overturning of the loads as specified in Section III A.2.
2. Bearing devices shall be capable of transfer of lateral loads from the wind forces.

XI. PLYWOOD STRESSED SKIN PANEL CONSTRUCTION

A. Design Specifications

1. **Materials**
   a. **Plywood** shall be exterior type, with allowable stresses as given in Douglas Fir Plywood Association General Design Specification #1, "Design of Plywood Lumber Structural Assemblies" for wet conditions of use, Table 7.3.
   b. **Lumber** shall be Douglas fir, Coast Region, of the grades shown on drawings. Unit stresses shall be for wet conditions of use. Unit stresses for lumber in stressed skin panels shall be in accordance with Table 1 of the "National Design Specifications for Stress Grade Lumber and Its Fastenings."
   c. **Glue** shall be of an exterior type and shall conform to one of the following specifications:
      (1) Military Specifications—MIL–A–46051, for room or intermediate temperature setting resin adhesive (phenol, resorcinol, or melamine base).
      (2) Military Specifications—MIL–A–5534A, for high temperature setting resin adhesive (phenol, resorcinol, or melamine base).
   d. **Bearing pads** shall consist of elastomeric pads or 2 ply 55 pound felt.
2. **Special Design Provisions**
   a. Plywood stressed skin panels shall be designed in accordance with DFPA Design Method No. SS–62D.
   Plywood shall be not less than 5/8 inch thick.
3. *Deflection and Camber*
   a. Deflections due to live loads shall be computed and preferably shall not exceed 1/450 of the span, the span length being considered as the distance center to center of bearings.
   b. Stressed skin panels for deck shall be manufactured with ½-inch crov and required camber built in.
   c. In computing dead load deflections, the value of modulus of elastici (E) shall be assumed to be one-half the value given in the Nation Lumber Manufacturers Association Specification for Glued-Laminat Structural Members (Table 20b).

B. Fabrication of plywood stressed skin panels shall be in accordance wi DFPA Fabrication Specification No. SS–8.

1. Splices shall not be permitted in panel stringers.
2. Splices in plywood shall be scarfed joints.
3. Pressure gluing shall be such that will provide a pressure of 100–1 p.s.i. on the glued area. Such pressure must be provided positively, with clamping or other mechanical means. Clamping may start at a point but shall progress to an end or ends.
4. Nail gluing techniques may be used.
5. Dimensional tolerances shall be in accordance with DFPA Fabricati Specification SS–8.

C. Epoxy Wearing Surface

A Type C, coal-tar modified epoxy resin system, with a surface aggreg application shall be used. The epoxy resin system and aggregate sh conform to Interim Guide Specification AASHO M200–63I. The de surface shall be clean and free of any foreign matter detrimental to bond w the epoxy resin. Rate of application of the epoxy resin shall be accordi to the manufacturer's recommendations. Application of the aggreg shall be at a rate of not less than 15 pounds per square yard.

APPENDIX B

SPECIAL PROVISIONS FOR PAINT FOR STRUCTURAL STEEL BRIDGES

The specification for Red Lead Iron Oxide Paint for use as the primer shop coat and the first field coat of paint is given below. A permissible ternate to this paint is the Red Lead Ready-Mixed Paint of the Standard pecifications for Highway Materials Designation AASHO M 72–51, Type or Type II.

(1) **Color.**—The shop coat shall be the color characteristic of a blend of d lead and red iron oxide.
The first field coat shall be tinted brown as required with lamp black in amount not to exceed 1 ounce per gallon of vehicle.

(2) **Pigment.**—The pigment shall be as specified below (requirements ited as percentage by weight):

	Minimum	*Maximum*
Red lead (TT–R–191 Type 1, Grade C)	65.0	----
Aluminum stearate	0.3	0.4
Red iron oxide pigment (85% Fe_2O_3)	15.0	----
Magnesium silicate	----	14.7
Mica, 325-mesh	4.0	6.0

The iron oxide pigment shall be on a siliceous base (not calcium sulfate). e extracted pigment on analysis shall conform to the following quantita- e requirements (stated as percentage by weight):

	Minimum	*Maximum*
True red lead, Pb_3O_4	62.5	----
Iron oxide, Fe_2O_3	12.5	----
Siliceous matter	----	22.0

(3) **Vehicle.**—The vehicle shall consist of raw linseed oil blended with a ceryl phthalate type varnish composed of a linseed oil modified resin gether with the necessary driers and volatile thinners, as follows (require- nts stated as percentage by weight):

	Minimum	*Maximum*
Raw linseed oil	49	--
Alkyd resin solids	16	--
Volatile thinner and dryer	--	35

The raw linseed oil shall conform to the Federal Specification TT–O–369. e alkyd resin shall conform to the Federal Specification TT–R–266 Type All vehicles shall be free from rosin and rosin derivatives. (The test for in under paragraph 11 shall be negative.) The vehicles may contain lition agents such as anti-oxidants and wetting aids.

(4) **Paint quantitative requirements.**—The paint shall meet the quantitative requirements as follows:

	Minimum	*Maximum*
Pigment, percent by weight	68	-----
Nonvolatile vehicle, percent by weight of vehicle	65	-----
Phthalic anhydride, percent by weight of nonvolatile vehicle	7	-----
Oil acids, percent by weight of nonvolatile vehicle	78	-----
Uncombined water, percent by weight of paint	-----	0.5
Coarse particles and "skins" (retained on No. 325 sieve) percent by weight of pigment	-----	2.0
Consistency (Krebs-Stormer, shearing rate 200 r.p.m.):		
Grams	155	225
Equivalent KU	73	86
Weight per gallon, pounds	17.0	-----
Drying time:		
Set to touch, hours	-----	7
Dry through, hours	-----	24
Flash point	30°C.(86°F.)	

(5) **Condition in container.**—The paint shall not show excessive settling in a freshly opened full can, and shall easily be redispersed with a paddle to a smooth, homogeneous state. The paint shall show no curdling, livering, caking, or color separation, and shall be free from lumps and skins.

(6) **Brushing properties.**—The paint as received shall brush easily, possess good leveling properties, and show no running or sagging tendencies when applied at a spreading rate of 500 square feet per gallon to smooth steel vertical surfaces.

(7) **Skinning.**—The paint shall not skin within 48 hours in a three-quarters-filled closed container.

(8) **Appearance.**—The paint shall dry to a smooth uniform finish free from roughness, grit, unevenness, and other surface imperfections. The paint shall show no streaking or separation when flowed on clean glass.

(9) **Sampling.**—Sampling shall be performed in accordance with Method 102 of Federal Specification TT–P–141.

(10) **Storage stability.**—The paint shall show no thickening, curdling, gelling, or hard caking whenstored for 6 months from date of delivery in a full, tightly covered container at a temperature of 21° to 32°C. (70° to 90°F.).

(11) **Testing.**—The paint shall be tested in accordance with applicable methods of Federal Specification TT–P–141 and as hereinafter specified. The following tests shall be conducted in accordance with Federal Specification TT–P–141:

	Test		*Test*
Percentage of pigment	402.1	Weight per gallon	401
True red lead (Pb_3O_4)	716	Set to touch time	406.1
Iron oxide (Fe_2O_3)	711	Condition in container	301
Isolation of vehicle	403.2	Brushing properties	205, 432
Nonvolatile in vehicle	[1] 405.2	Spraying properties	204, 433
Phthalic anhydride	702	Flexibility	[2] 622.1
	Procedure A	Skinning	414.1
Oil acids	703	Rosin and rosin derivatives	[3] 503.1
Uncombined water	408	Flash point	429.3
Consistency (Krebs-Stormer)	428.1	Storage stability	414.2
Coarse particles (and skins)	409		

[1] A gravity convection oven may be used to determine the nonvolatile content of the supercentrifuged vehicle if the procedure outlined in Method 404.1 is modified as follows: Weigh accurately from 0.8 to 1.2 grams of sample (by difference), heat for 1 hour, cool, and weigh. Use the lower value to calculate the percentage of nonvolatile matter.

[2] Apply the paint to flat tin panels with a 0.002-inch (wet-film thickness) doctor blade or by other suitable means. The baking and bending constants shall be as follows: Allow the paint to dry for 18 hours at room temperature. Bake for 5 hours at 75°±2° C. (167°±4° F.). Condition the panel for a minimum of 15 minutes at 25°±0.1° C. (77°±0.2° F.), and then bend at that temperature over ¼-inch mandrel. Examine for cracks in a strong light at 7-diameter magnification.

[3] Make the test on a portion of the isolated vehicle.

(12) **Dry-through time.**—Prepare a panel as in paragraph 1 of Method 406.1. The film shall be considered dry through when it cannot be distorted or removed by the following test: Place the panel in a horizontal position at a height such that when the thumb is placed on the film the arm of the operator is in a straight line from wrist to shoulder. Bear downward on the film with the full area of the ball of the thumb, exerting the maximum pressure of the arm only (not body weight), simultaneously turning the thumb through an angle of 90 degrees in the plane of the film. Examine for loosening, detachment, wrinkling, or other evidence of distortion of the film.

(13) **Appearance of paint coat.**—(a) Examine the prepared panels for brushing and spraying properties. (b) Flow a portion of the paint on a clean glass plate, let dry in a nearly vertical position at room temperature, and examine 4 inches from the top.

(14) **Analysis of pigment; siliceous extender and mica.**—Transfer 0.5 gram of the pigment to a 400-milliliter beaker. Add 15 milliliters of concentrated hydrochloric acid and heat gently until the iron oxide and red lead are completely dissolved. Evaporate to dryness and bake at 105° to 110° C. for 1 hour. Moisten the residue with a few drops of concentrated hydrochloric acid, dilute to 100 milliliters with hot water, boil, filter, and wash with hot water. Transfer the paper and contents to a weighed porcelain crucible, ignite, cool, and weigh; examine the residue microscopically to determine the presence of mica. Calculate the percentage and report this residue as the siliceous extender and mica.

☆ U.S. GOVERNMENT PRINTING OFFICE : 1978 O—277-788