REVISED

DESIGN STUDY REPORT

BURLINGTON BRIDGE REPLACEMENT

DES MOINES COUNTY, IOWA; HENDERSON COUNTY, ILLINOIS

MISSISSIPPI RIVER BRIDGE

between

BURLINGTON, IOWA AND GULFPORT, ILLINOIS

Prepared for

IOWA DEPARTMENT OF TRANSPORTATION

HIGHWAY DIVISION

AND

ILLINOIS DEPARTMENT OF TRANSPORTATION

MAY 31, 1985

By

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ENGINEERS - ARCHITECTS - PLANNERS

ST. LOUIS, MISSOURI

FOREWORD

This report is submitted in fulfillment of an Agreement dated October 17, 1984, as amended by an Extra Work Order dated February 28, 1985, and a Supplemental Agreement dated April 23, 1985, between the Highway Division of the Iowa Department of Transportation, Ames, Iowa and the Illinois Department of Transportation, Springfield, Illinois, and Sverdrup & Parcel and Associates, Inc., Engineers - Architects - Planners, St. Louis, Missouri.

ACKNOWLEDGEMENTS

Appreciation is expressed for cooperation received from the administrative and technical staffs of the States of Iowa and Illinois Departments of Transportation, the Federal Highway Administration, the U.S. Army Corps of Engineers, Rock Island District and the Bridge Branch of the U.S. Coast Guard, District II during the studies and the preparation of this report.

Appreciation is also expressed to the personnel of the City of Burlington, Village of Gulfport, the Henderson County Drainage District No. 1, and the utility companies for their considerate cooperation during the field investigation phase of the work.

SUMMARY

This Design Study Report presents the investigations made for the replacement of the US 34 MacArthur Bridge over the Mississippi River connecting Burlington, Iowa with Gulfport, Illinois. The study area includes all roadway and structures from Central Avenue in Iowa to the Burlington Northern Railroad bridge in Illinois. The replacement river bridge is set either 75 ft downstream and parallel to the existing structure (South Alignment) or upstream of the facility on a curve (North Alignment).

The project is divided into three sections for the study of both alignments. These are the Iowa Approach up to the first land pier for the river bridge, the river bridge from the first land pier in Iowa to the abutment on the river side of the Henderson County Levee, and the Illinois Approach from the river bridge abutment to the Burlington Northern Railroad overpass.

The selection of a structural concept for the river bridge has little or no affect on the construction cost of roadway items, demolition and structures for the Iowa or the Illinois Approaches.

For the river crossing, four concepts for the channel span--a truss, a tied arch, and cable-stayed girders in steel and in concrete--were investigated for a 480-ft span on the south alignment and a 515-ft span on the north alignment. In addition, cable-stayed girders in steel and concrete were studied for a 660-ft channel span on the south alignment. Multiple girders in steel and prestressed concrete were evaluated for the approaches to these channel span schemes.

If first cost is the prime consideration, a simple span truss with multiple girders in steel or prestressed concrete would be recommended for the north alignment. The construction cost of the three segments for this alignment is:

Iowa Approach	$ 4,681,000
River Bridge	26,543,000
Illinois Approach	3,286,000
Total	$34,510,000

If the cost of maintenance, constructability, aesthetics, gradients and design speeds are considered foremost, a 515':282' assymmetrical cable-stayed concrete girder with multiple girders on the north alignment may be selected. The construction cost for the three segments for that river bridge and approaches is:

Iowa Approach	$ 4,681,000
River Bridge	26,616,000
Illinois Approach	3,303,000
Total	$34,600,000

The riverfront Connector A is recommended in the State of Illinois, based on cost, and its selection is reflected in the roadway construction costs of the Illinois Approach.

TABLE OF CONTENTS

FIGURES

TABLES OF COST ESTIMATES

TABLES OF COST ESTIMATES

(continued)

TABLES OF COST ESTIMATES

(continued)

PLATES

(Bound at end of Report)

PLATES (cont'd.)

PLATES (cont'd.)

I INTRODUCTION

A. PROJECT LOCATION

The States of Iowa and Illinois have agreed to replace the existing US 34 bridge (MacArthur Bridge) over the Mississippi River between Burlington, Iowa and Gulfport, Illinois. Two locations for the replacement river bridge were investigated. These are 75-ft downstream and parallel to the existing bridge and upstream on a curved alignment (see Figure 2).

B. BRIDGE SITE TOPOGRAPHY

The river at the proposed bridge alignments is about 2,000 ft bank to bank. Normal pool elevation is 519.4 with maximum high water of record being elevation 533.0.

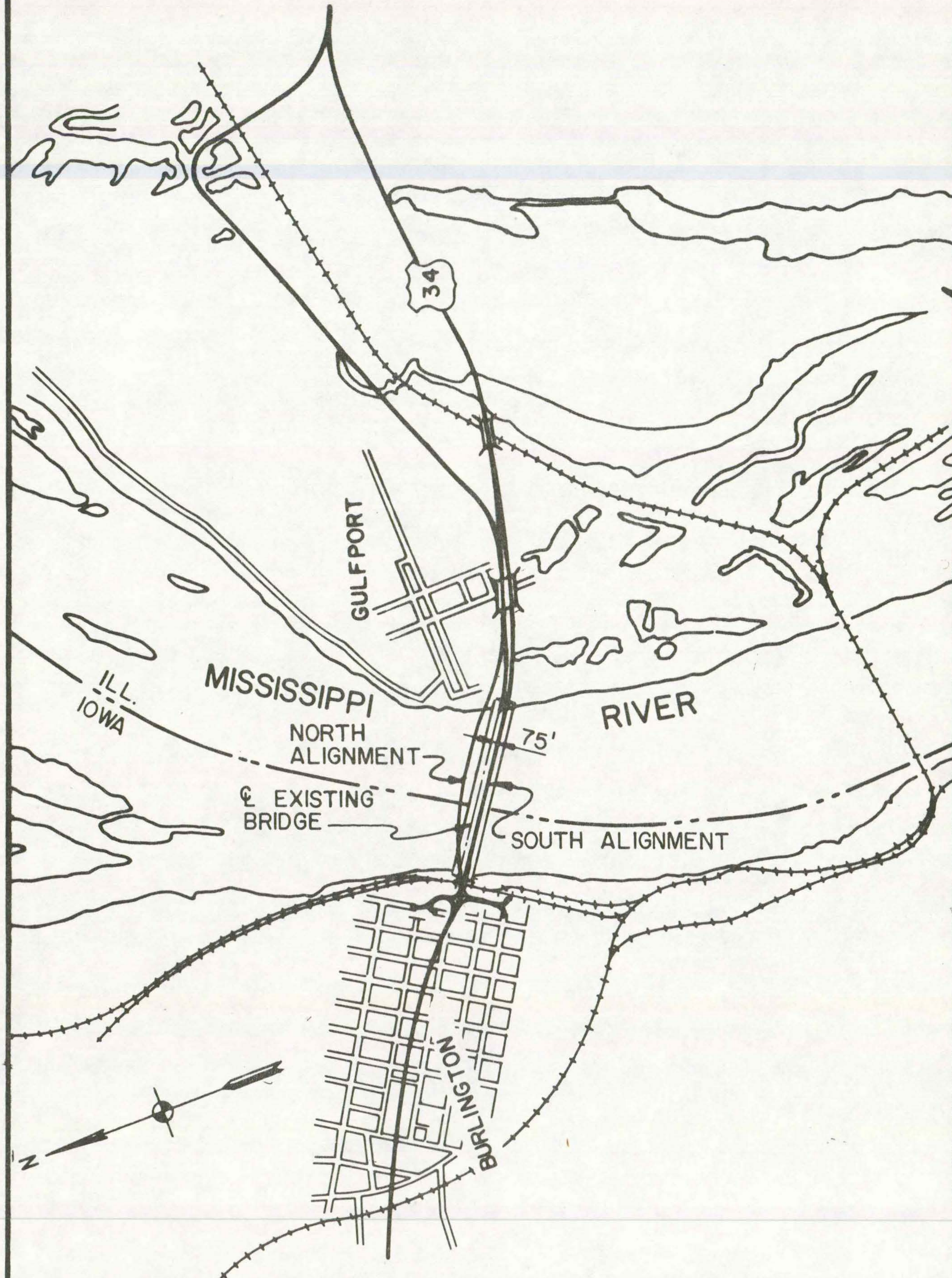
The ground on the Iowa side slopes upward away from the river to elevation 555± at Main Street, some 450 ft west of the river bank.

The ground on the Illinois side, all relatively low at elevation 520±, is protected by an earthen levee at the river bank which tops out at elevation 536±. Sand boils were evident in prior years behind the levee. Presently, seepage occurs when river levels are high.

C. EXISTING TRANSPORTATION FACILITIES

1. Iowa Side

At ground level in Iowa, tracks for the Burlington Northern railroad, Front Street and Main Street are approximately parallel to the Mississippi River. The railroad is about 100 ft, Front Street is about 150 ft, and Main Street is about 450 ft from the river bank at the proposed bridge alignments.



LOCATION MAP

FIGURE 2

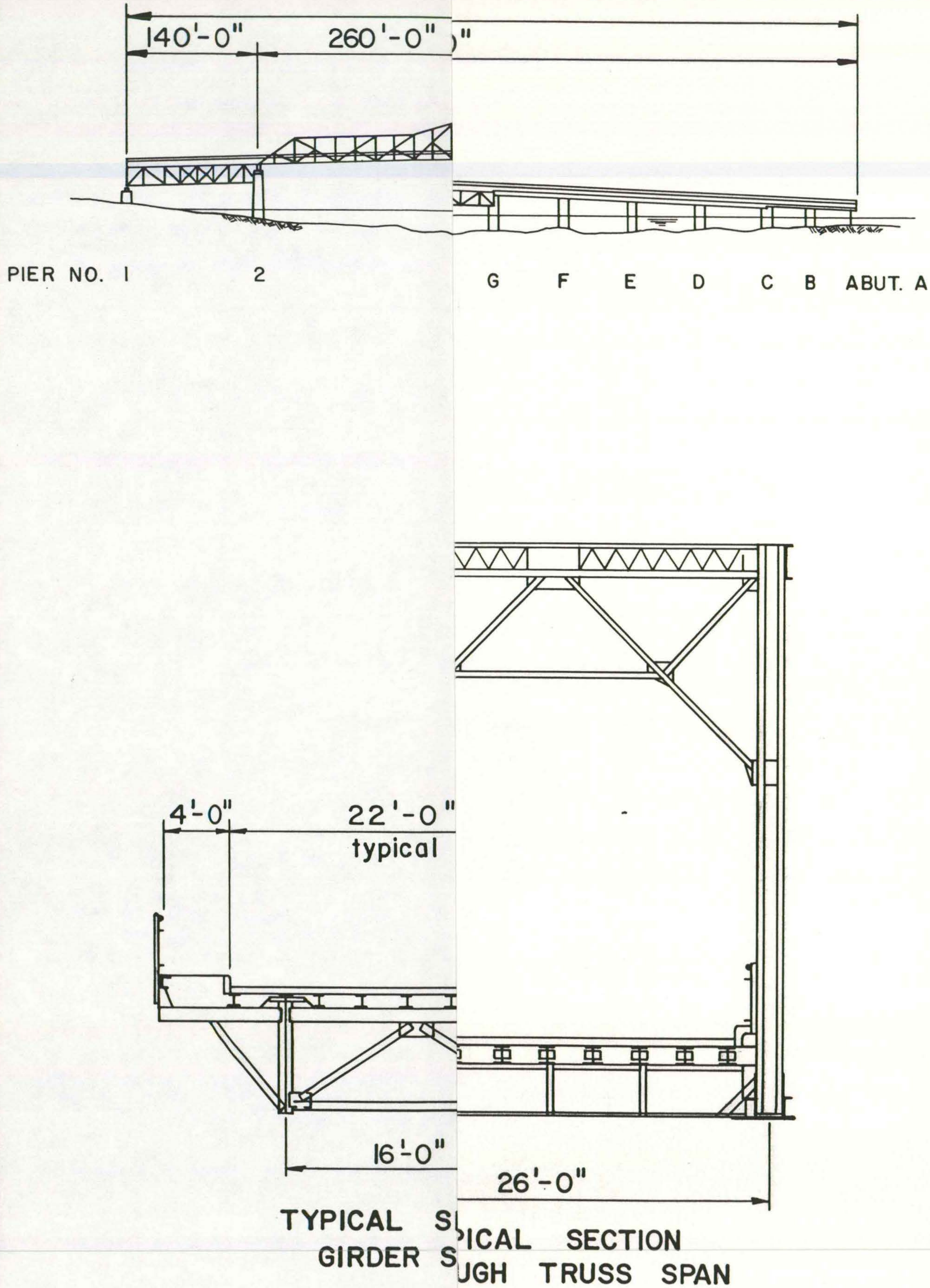
An elevated approach to the existing river bridge, with on and off ramps to the north and to the south, was constructed over these ground facilities in the early to mid 1970's. The elevated freeway, leading to the bridge, is composed of nine spans in the range of 51 ft to 75 ft. One span, Span 6, has been constructed with multiple welded girders with a 48-inch web and the remaining spans are multiple continuous composite 36-inch deep beam spans. The four ramp structures, one each on and off to the north and to the south are continuous composite curved welded girders with 48-inch deep webs. Spans for these ramp bridges range from about 46 ft to 82 ft. A field check of these structures on October 31, and November 1 and 2, 1984 revealed that the elevated bridges are structurally sound.

2. River Bridge

The MacArthur Bridge, opened to traffic in 1917 as a toll facility, was built by the Wisconsin Bridge and Iron Company of Milwaukee, Wisconsin. It was owned by the Citizens Bridge Company, Inc., of Burlington, Iowa until 1923 when the City of Burlington became the owner. The City has operated and maintained the structure ever since.

The structure consists of girder spans, deck truss spans and a three-span cantilever through truss as shown on Figure 3. The original structure had a 19-ft wide timber deck and a 4-ft wide sidewalk. The deck and sidewalk were supported on steel stringers and steel floorbeams.

In the early 1950's the structure underwent extensive repair and modifications. This consisted of increasing the roadway width to 22 ft by placing the walkway outside of the through truss spans and on an extension of the floorbeams of deck units; replacing the timber deck with open steel grating; strengthening floorbeams and girders by welding



MACARTHUR BRIDGE

FIGURE 3

structural tees and plates to the flanges; replacement of some stringers and increasing the capacity of other stringers by splicing for continuity; and corrective strengthening of truss components.

The piers were reconditioned in 1954 and in the late 1960's some piers were strengthened with steel truss towers and the pier caps of others were encased in steel plates tied through the cap with high strength rods. In addition, grillage beams have been replaced on some piers.

As the footings of the piers would otherwise be exposed to river bed scour, all piers are protected with a mound of riprap protection.

In a 1973 report by others the bridge was rated at "31 tons per truck and full trailer". It should be noted that this rating is for "Operating" stress levels of 75% of yield.

Following the 1973 report, specific repairs recommended therein were made and the bridge was repainted. Since that time the City of Burlington has been attentive to maintenance and protection of the structure. This has been accomplished by in-depth inspections approximately every two years and by underwater inspections in 1974 and 1979. In addition, the passage of heavy trucks is monitored by traffic control devices and personnel housed at each end of the bridge.

Repairs were made to the structure in 1983 by replacing a number of floorbeams, strengthening additional floorbeams and repairing welds in stringers. Following those repairs, it was recommended that a 300-ft spacing be maintained between trucks traveling at a maximum speed of 20 mph. Truck loads were to be limited to:

Single Vehicle (Two Axle)	25 Tons
Single Vehicle (Three Axle)	27 Tons
Tractor Trailer (382)	40 Tons
Truck and Full Trailer (3-3)	40 Tons

The City was requested to monitor the floor system to note any structural changes that would be visible from a walk-over inspection.

As an early effort in the study of a replacement structure a cursory examination of the MacArthur Bridge was made in October and November of 1984, to verify previous modifications and present conditions.

Although the City of Burlington has been conscientious in its maintenance procedures, the protective painting continues to break down with many large areas of flaking. Rust is again beginning to build in areas immediately beneath the deck on stringers and floorbeams.

Some details remain sensitive to the capacity of original components in their presently deteriorated condition. An example is the flange angles of the girder spans which must transmit stresses from cover plates through the connecting welds. Any further deterioration of those flange angles will render them inadequate for their function.

The steel reinforcement on the main piers, which was designed to relieve the concrete section from its load, is in fair condition but very vulnerable to navigation.

The "Operating" level determined in previous reports does not provide for fatigue in the modified structural components reinforced by welding. Although this field inspection did not reveal any deficiencies from fatigue, the overall structure is subject to deterioration from that phenomenon.

The nonredundant structures in this bridge (two components supporting a floor system) together with suspected limited fatigue life in its deteriorated condition, does not lend credence to any imagined effort to improve the structure for further use.

In addition, this structure with its 22-ft wide roadway is functionally obsolete for a two-lane bridge under present standards. Any possibility of widening the structure would be difficult and expensive.

In view of all items considered, it would not be practical nor cost effective to rehabilitate this bridge.

3. Illinois Approach and Grade Separation

The Illinois Approach, constructed in the early 1970s, consists of approximately 2,900 ft of highway running from the river bridge abutment to the Burlington Northern railroad bridge. This highway consists of two 12-ft lanes with 10-ft shoulders.

Included in the 2,900 ft is a grade separation structure over Chinn Avenue about 1,400 ft from the river bridge abutment. This structure is a standard Illinois 2-span welded plate girder bridge with girder-spans of 71'-5" and 76'-5". The out-to-out dimension of the concrete deck is 46'-0". A bridge inspection by personnel of the Illinois Department of Transportation in January 1984 indicated the need for minor repairs to the deck, some painting for exterior bearings and alleviation of the scour occurring at the edge of the slope protection. This condition was verified by Sverdrup & Parcel bridge inspectors in November 1984.

A two-quadrant partial cloverleaf interchange is located on the westerly side of Chinn Avenue. The ramps are 2 lanes at 12 ft each with 10-ft shoulders. A river front access road intersects the southerly ramp about 200 ft from the mainline.

Chinn Avenue, a divided highway at the interchange, has an 18-ft median, two 14-ft lanes and 10-ft shoulders. Pavement is 8-inch S.R. PCC, reinforced with fabric.

The condition of mainline, ramps and Chinn Avenue pavements was reviewed in November 1984. The pavements were generally found to be in good condition except that some transverse joints in the ramps and Chinn Avenue need to be repaired or replaced.

II DESIGN CRITERIA

A. STRUCTURAL

1. Design

The Mississippi River bridge will be designed in accordance with Division I of the 1983 AASHTO Standard Specifications for Highway Bridges Thirteenth Edition, with the exceptions and interpretations noted below.

2. Construction

a) Burlington Interchange and the Mississippi River bridge construction will be in accordance with the Standard Specifications for Highway and Bridge Construction of the Iowa Department of Transportation series of 1984, plus current Supplemental Specifications and Special Provisions.

b) For the Chinn Avenue overpass, the Standard Specifications for Road and Bridge Construction, adopted October 1, 1983, Illinois Department of Transportation, will be used in conjunction with Addendums and Special Provisions.

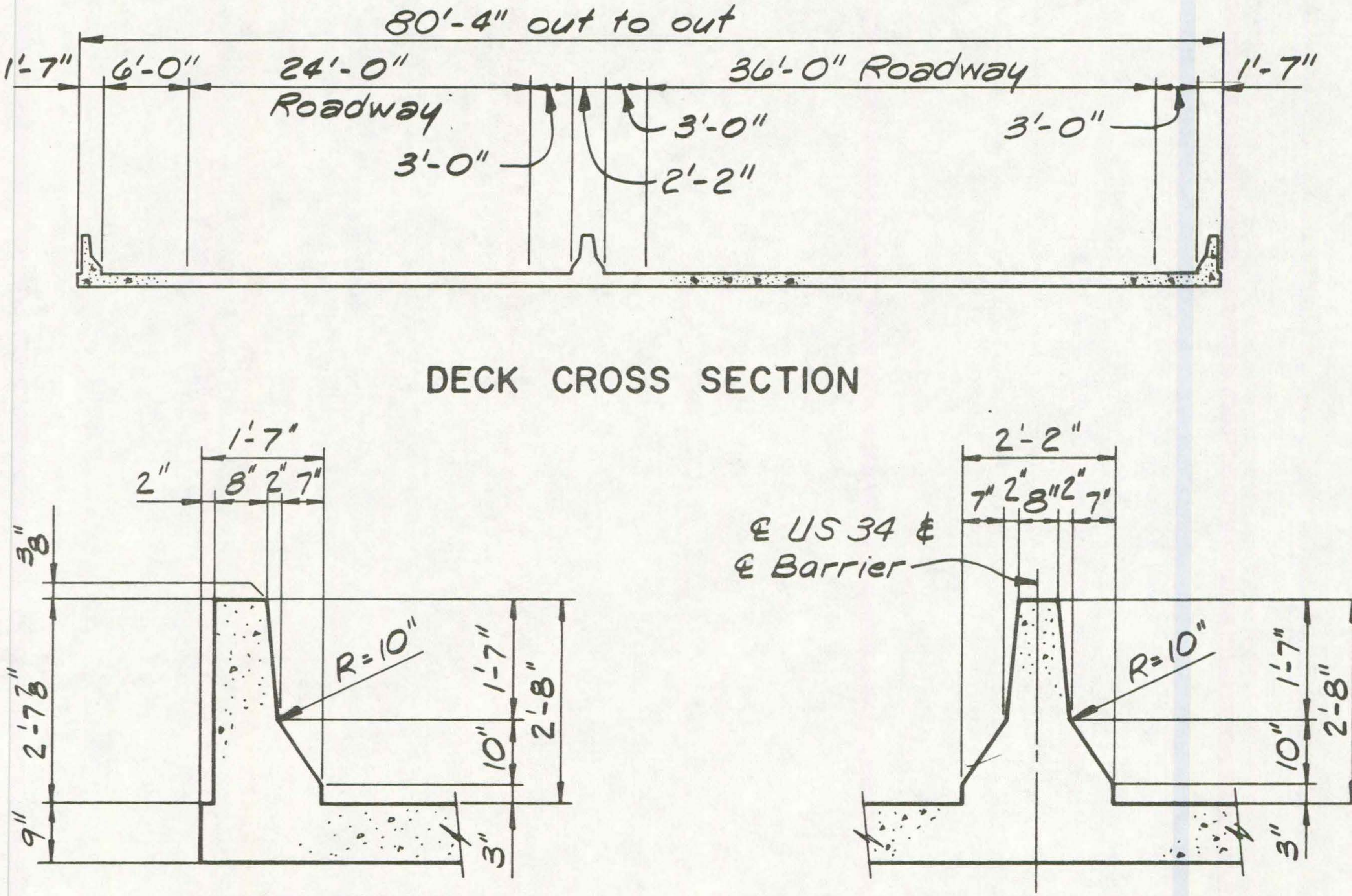
3. Deck Cross Sections

The normal river bridge cross section will provide for five 12-ft lanes, two westbound and three eastbound (see Figure 4).

4. Water Elevations

Corps of Engineers 1929 adjustment.

Normal Pool	Elev. 519.4
2 Percent Line	Elev. 526.6
High Water of Record (50-year flood)	Elev. 533.0



DECK CROSS SECTION

EDGE BARRIER

MEDIAN BARRIER

FIGURE 4

5. Dead Loads

The following unit loads will be used:

Item	Unit Loads (pcf)
Concrete	150
Steel	490
Earth	100
Compacted earth backfill	120
Submerged earth	125

Reinforced concrete deck specifications will include allowances for a concrete overlay and a future wearing surface of 20 psf (see discussion under Section V.B).

6. Live Load

HS20-44 loading and interstate highway bridge loading as per AASHTO Article 3.7.5 will be used, whichever controls the design.

7. Repetitive Loading

In accordance with AASHTO Article 10.3, Load Cycles for redundant and nonredundant structures.

8. Wind Loads

Wind loads will be as specified in AASHTO specifications and will be considered as unbroken moving loads. Identical loaded length for wind-on-live load and wind-on-structure will be used.

9. Thermal Forces

Thermal forces will be applied in accordance with AASHTO Article 3.16 consistent with a cold climate.

10. Force of Stream Current, Floating Ice and Sheet Ice

The force from the stream current and floating ice shall be applied in accordance with Article 3.18 of the AASHTO specifications, using 4 ft per second for stream flow and an 18-inch thickness of ice at elevation 526.6.

For the channel piers, sheet ice exerting a force of 20,000 lbs/ft applied parallel to the centerline of bridge at elevation 526.6 will be used. 5,000 lbs/ft will be used for approach piers.

11. Barge Impact

A barge impact force of 1,000,000 lbs will be applied normal to the centerline of bridge at elevation 526.6 for the channel piers. 250,000 lbs will be used for approach piers.

12. Buoyancy

Full hydrostatic pressure shall be considered acting over the entire base of the substructure units, with water surface elevations from 519.4 to 533.0.

13. Seismic Forces

Seismic loads will be applied in accordance with AASHTO Article 3.21 by the Equivalent Static Force Method, Zone 1, using a combined response coefficient of C = 0.06.

Unit stresses for dead load, earth pressure, buoyancy, stream flow and earthquake shall not exceed 133 percent of normal unit stress, except for lateral systems where the unit stress shall not exceed the normal allowable. Vertical dynamic forces will be disregarded.

14. Centrifugal Forces

Centrifugal force shall be applied in accordance with AASHTO Article 3.10 using a 60-mph design speed.

15. Earth Pressure

Equivalent fluid pressure shall be 40 psf/ft based on Rankine's formula using $\phi = 30°$ and unit weight of 120 pcf.

16. Structural Steel

The following structural steels will be used, with selection based on stress requirements and overall economy:

- o Structural Carbon Steel A36.
- o High Strength Low-Alloy Steel A572, Grade 50 and A588 with thickness limitations as found in Table 10.2A of the AASHTO Specifications.

Load factor design will be used for proportioning members and their attachments except for truss members, arch ribs, arch ties, cable hangers and cable-stayed girders where working stress design will apply. A capacity reduction factor consistent with AASHTO requirements will be used in load factor design.

Where applicable, the charpy V-notch impact requirements shall be for Temperature Zone 2.

17. Welding

All welding shall be in accordance with the current Structural Welding Code of the American Welding Society, as modified by the Supplemental Specifications.

18. Cables

For stay-cables, 0.6" diameter strand stays will be used with VSL or Strong Hold HiAm sockets. 2,000,000 load cycles will be used in design.

$f = 0.45\ f_{ult}$ for Case I and $0.5\ f_{ult}$ for all other cases.

Cables are to be enclosed in pressure grouted polyethylene pipe, continuous extruded seamless elastomeric covering or elastomeric cable wrap systems.

For tied arch hangers, structural strand, ASTM A586, f=0.40 foot will be used.

19. Reinforced Concrete

Deck Slab	$f'_c = 3{,}500$ psi	$n = 10$
Parapet	$f'_c = 3{,}500$ psi	$n = 10$
Substructure	$f'_c = 3{,}500$ psi	$n = 10$
Stayed-girder tower	$f'_c = 5{,}000$ psi	$n = 6$
Stayed-girder deck slab	$f'_c = 5{,}000$ psi	$n = 6$
Prestressed girders	$f'_c = 6{,}000$ psi	$n = 6$

20. Prestressed Concrete

$f'_c = 5{,}000$ psi $n = 6$

Allowable stress at time of initial prestress:

$f'_{ci} = 4{,}000$ psi

Compression $f_{ci} = .55\ (f'_{ci}) = 2{,}200$ psi

Tension $f_{ci} = 0$ psi

Allowable stress at service loads after losses:

Compression $f_c = .40\ f'_c = 2{,}000$ psi

Tension:

For Group Loadings I, II, and III $f_c = 0$ psi

For All Other Loadings $f_c = 3\ (f'_c)^{\frac{1}{2}}$ psi

21. Reinforcing Steel

ASTM A-615 Grade 60. All slab and parapet bars shall be epoxy-coated.

22. Prestressing Steel

Wire Strand ASTM A-416 Grade 270 uncoated seven-wire stress-relieved strand for prestressed concrete:

Ultimate strength $f'_s = 270$ ksi

Yield point $f_y = 0.85\ f'_s = 229$ ksi

Allowable stresses:

Temporary stress

before losses $f_{si} = 0.70\ f'_s = 189$ ksi

Jacking stress $f_{sj} = 0.75\ f'_s = 202$ ksi

(Maximum temporary stress prior to seating)

Effective stress

after losses $f_{se} = 0.68\ f'_s = 184$ ksi

(Stress of service load $f = 0.8\ f_y = 0.8 \times 0.85\ f'_s = 0.68\ f'_s$)

23. Substructure

The criteria for final design of substructure units will be based on subsurface information to be obtained. For preliminary design, the suggested criteria in the following paragraphs will be used.

24. Piling

Drilled caissons into rock will be used for the main piers on the south alignment adjacent to existing piers. A capacity of 50 tons per square foot end bearing plus 2.5 tons per square foot side friction will be used.

150-ton HP 14 x 89 piles will be used for all other river piers

70-ton HP 10 x 42 piles or 90-ton HP 12 x 53 piles will be used in the Burlington interchange area

30-ton concrete piles are to be used at the Chinn Avenue overpass

25. Stability

The stability of substructure units will be computed to include buoyancy, with water surface elevations from a normal pool of 519.4 to a high water of record of 533.0. The dead load will consist of the weight of concrete and the weight of soil and water on top of the footing.

For pile footings, no tension will be allowed in any pile for any final loading condition except Group VII (earthquake). In this instance, one-sixth of the pile capacity will be allowed in tension.

26. Seal Course

A water surface elevation of 523.0 (average of normal pool and 2% water elevation) will be used for designing seal courses.

A 10 psi uplift force on the piles, acting on an area equal to the perimeter of the pile, times the depth of the seal course, will be allowed up to a maximum of one-sixth of the pile capacity.

27. Roadway Drainage

Roadway drains will accommodate a 10-year-frequency rainfall of 5-minute duration with 100-percent runoff. However, drains at the low points will accommodate a 50-year-frequency rainfall of 5-minute duration with 100-percent runoff. Sufficient size and spacing of drains will be provided to restrict the flow of water to an area within 8 ft of the curbs.

28. Utilities

Navigational lighting for water craft and aircraft will be provided in accordance with current U.S. Coast Guard requirements and the Federal Aviation Authority.

Roadway lighting will be provided as determined by the Iowa Department of Transportation.

Signing and sign lighting, where required, will meet the requirements of the Iowa Department of Transportation.

B. GEOMETRICS

The design features presented in this study are based on the design policies of Iowa and Illinois Departments of Transportation, which in some instances are different, and the American Association of State Highway and Transportation Officials' A Policy on Geometric Design of Highways and Streets, 1984. Areas of the existing facility which are being modified or new construction which ties into the existing roadways may not satisfy current design policies. Deviations from these policies are noted in the section on project geometrics. The design criteria for the Burlington Bridge project are as follows:

1. Mainline

   a. Access Control

      Full

   b. Design Speeds

      1) Iowa and River Bridge

         55 mph Desirable; 50 mph Minimum

      2) Illinois

         70 mph

   c. Horizontal Alignment

Minimum tangent distance between curves is not to be less than the distance required for superelevation runout.

Spirals are not to be used.

1) Iowa and River Bridge

   Desirable Maximum Curvature = 5°-15'

   Maximum Curvature = 6°-45'

2) Illinois

Maximum Curvature = 2°30'

Minimum Length of Curve = 500 ft

d. Vertical Alignment

1) Iowa and River Bridge

Grade: Desirable Maximum = 4%; Maximum = 5%

K Values for Crest Vertical Curves: Desirable Minimum = 150; Minimum = 110; Maximum = 167 for curbed roadways

K Values for Sag Vertical Curves: Desirable Minimum = 100; Minimum = 54 (since the Iowa approach is lighted, this minimum K is based on comfort control for 50 mph); Maximum = 167 for curbed roadways

2) Illinois

Maximum Grade = 3%

Minimum Length of Vertical Curves = 400 ft

Minimum K Value for Crest Vertical Curves = 290

Minimum K Value for Sag Vertical Curves = 150

e. Stopping Sight Distances

Stopping sight distances are to be based on a height-of-eye of 3.5 ft to a height-of-object of 6 inches. If, when measuring horizontal sight distance, the line of sight crosses the median barrier or the bridge parapet, the effect of these items blocking the 6-inch height of object is not to be considered.

1) Iowa and River Bridge

450 ft Desirable Minimum; 400 ft Minimum

2) Illinois

Minimum = 625 ft

f. Superelevation

Maximum = 0.06 ft/ft

1) Illinois

Refer to Figure 2-310.01a of Illinois Location and Environment Manual.

Minimum Transition Length = 200 ft

g. Pavement

Two 12-ft lanes each direction.

h. Shoulder Widths

Left (Inside) = 3 ft

1) Iowa and River Bridge

Right (Outside) on Roadways = 10 ft for through lanes and 6 ft for auxiliary lanes

Right (Outside) on Bridges = 6 ft for through lanes and 3 ft for auxiliary lanes

2) Illinois

Right (Outside) = 10 ft

i. Slopes

1) Iowa

Minimum 4 ft border with 3:1 backslope or foreslope

2) Illinois - Fill Slopes

Heights less than 30 ft: 6:1 within 40-ft clear zone then 4:1.

Heights over 30 ft: 3:1

2. Ramps

a. Design Speeds

1) Iowa

20 mph Minimum

2) Illinois

25 mph Minimum

b. Horizontal Alignment

Spirals are not to be used.

1) Iowa

Minimum Radius = 100 ft

2) Illinois

Minimum Radius = 150 ft

c. Vertical Alignment

1) Iowa

Maximum Grade = 6%

2) Illinois

Maximum Grades = 6% Down and 4% Up

Refer to Illinois Location and Environment Manual Figures 2-315.04a and 2-315.05 for minimum lengths of crest and sag vertical curves.

d. Minimum Stopping Sight Distances

400 ft for 50 mph

275 ft for 40 mph

200 ft for 30 mph

150 ft for 25 mph

125 ft for 20 mph

e. Superelevation

1) Iowa

Maximum = 0.06 ft/ft

2) Illinois

Maximum = 0.08 ft/ft

f. Pavement

1) Iowa

Widths of new ramps are to be in accordance with Case IIC in Table X-3 of AASHTO's A Policy on Geometric Design of Highways and Streets, 1984

2) Illinois

Width = 16 ft

g. Shoulder Widths

1) Iowa

Left = 4 ft; Right = 6 ft

2) Illinois

Left = 6 ft Overall, 4 ft Stabilized

Right = 8 ft Overall, 6 ft Stabilized (Roadway)

Right = 6 ft (Bridge)

h. Fill Slopes

1) Iowa

Minimum 4 ft border with 3:1 slopes

2) Illinois

Heights less than 30 ft: 6:1 within 40-ft clear zone then 4:1

Heights over 30 ft: 3:1

3. Connector Road in Illinois

a. Design Speed

15 mph

b. Horizontal Alignment

Minimum Radius = 50 ft

c. Vertical Alignment

Maximum Grade = 8%

d. Stopping Sight Distance

Minimum = 80 ft

e. Superelevation

Remove Crown

f. Surface Width

18 ft

g. Shoulder Width

2 ft

h. Side Slopes

2:1

4. Minimum Horizontal Clearances

a. Mississippi River Navigation Channel

No less than provided by the existing bridge being replaced.

b. Burlington Northern Railroad

9 ft 0 inches from centerline of track

c. Roads and Streets

1) Iowa

6 ft from edge of traffic lane or parking lane or no less than existing clearance

2) Illinois

Chinn Avenue: No less than existing clearance

Connector Road: Total Opening = 40 ft

5. Minimum Vertical Clearances

a. Over Mississippi River Navigation Channel

60 ft above normal pool

b. Bridges Over Flood Prone Areas

2 ft above high water of record

c. Over Burlington Northern Railroad

23 ft (Subject to approval of BNRR)

d. Over Roads and Streets

14 ft 6 inches or no less than existing

e. Over Primary Highways

16 ft 6 inches

f. Clearance for thru Trusses and Overhead Signs

17 ft 3 inches

III PROJECT GEOMETRICS

A. SOUTH ALIGNMENT

1. Mainline

a. Horizontal

Existing Iowa stationing was established and projected forward along the proposed centerline across the Mississippi River to form a station equation, shown on Plate II, with the stationing used in Illinois. Illinois stationing is based on equating the existing stationing to the point of compound curve at Station 28+41.41.

Iowa Department of Transportation has established that the centerline of the proposed US Route 34 will be 75 ft downstream and parallel to the centerline of the existing MacArthur Bridge. In order to minimize reconstruction of the Iowa approach, the existing centerline is maintained up to Station 1003+57, which is the beginning of the last existing curve in Iowa. Refer to Plate I. The back tangent of this existing curve was projected forward to intersect with the proposed tangent of the river bridge at Station 1009+28. A 2°20' curve was selected such that the curve will not extend into the navigation span. This curve will end 30 ft from the centerline of the west navigation span pier when this pier is located immediately downstream of the existing navigation pier. Since a spiral would extend onto the navigation span, spirals will not be used. The superelevation transition will run out on the main span.

The existing median width on the Iowa approach roadway is 18 ft, and the proposed median width for the river bridge is 8 ft 2 inches. A median transition is proposed from Station 997+04 (opposite the beginning of the deceleration taper for Ramp B) to existing PT Station 1000+16. This results in an edge of pavement taper ratio of 63:1.

The horizontal alignment, shown on Plate III, for the Illinois approach was developed from Illinois DOT's Intersection Design Study (IDS), dated May 1983. The 0°15' curve, requiring no superelevation, provides for a smooth alignment through the ramp area of the Chinn Avenue interchange and allows relatively easy reconstruction of the bridge over Chinn Avenue. A 1°29'57.4" curve is used to connect the 0°15' curve to the existing 1°29' curve at the end of the project. The end-of-project tie-in point falls about 113 ft westerly of the west abutment of the existing bridge over the Burlington Northern Railroad and forms an angle point of about 0°21' with the existing pavement. A 1,000-ft long taper is used to transition from the four-lane mainline section to meet the existing 24-ft wide pavement.

b. Vertical

Since a median transition and the addition of a weaving lane and lane drop taper are the only necessary revisions along the mainline roadway of the Iowa approach, a three-inch asphalt concrete overlay on roadways and a concrete overlay on structures are proposed, and the existing descending 5.0% grade in the vicinity of the abutment will be maintained.

The existing 370-ft sag curve (K value of 41), shown on Plates V and VI, of the Iowa approach provides for a comfort control criteria of only 44 mph. Only a slight improvement (K=44 and comfort control speed of 45 mph) is possible because of the following constraints:

1) The proposed grade must approximate the existing elevations in order to utilize most of the existing approach structure and to maintain US 34 traffic during construction.

2) The proposed grade near Station 1006+80 needs to be as low as possible in order to tie-in Ramp D without exceeding the maximum ramp grade of 6%.

3) Near the Iowa bank, an ascending grade of 4% for a stayed-girder bridge or 5% for a through structure is necessary to provide the required clearance at the west navigation pier. The location of the west navigation pier is a critical control; any relocation of the pier towards the Iowa shore will adversely effect the geometrics.

The minimum vertical clearance over the Mississippi River navigation channel will be 60 ft above the normal pool elevation of 519.4 or 52 ft above the 2% water line elevation of 526.6. The 60 ft above normal pool controls; therefore, the minimum low structure elevation of the main span will be 579.4.

Because the structure depth for a through structure is about 10 to 12 ft, it is necessary to use grades of 4.0 and 5.0%. A minimum length of crest vertical curve for a 50-mph design speed is necessary to provide the required clearance over the navigation channel, refer to Plate VII. For the cable-stayed girder bridges the desirable maximum grades of 4.0%, shown on Plate VIII, are used. For the concrete alternate, the 1,270-ft length of crest vertical curve is adequate for a 55-mph design speed; however, for the steel alternate (with increased structure depth), a minimum length of crest vertical curve for a 50-mph design speed is necessary to provide the minimum clearance over the navigation channel. For each alternate, the profile grade control occurs at the west navigation pier.

The high water of record elevation of 533.0 (which equals the 50-year flood elevation) was obtained from the Corps of Engineers'

Standard Flood Profile of the Mississippi River. The minimum clearance of 2 ft above the high water of record to the bottom of the superstructure controls the profile grade at the Illinois abutment for the River Bridge with the stayed-girder alternates. The profile grade at the Illinois abutment for the through structure alternates is controlled by the clearance over the navigation span and by the minimum length of a sag vertical curve for a 70-mph design speed.

The profile grade, shown on Plates IX and X, of 1.4% from Station 17+50 to Station 23+70 was set by the controls established by widening the existing bridge over Chinn Avenue.

Reverse vertical curves are used from Station 23+70 to Station 26+00 for the purpose of ending the mathematical grade on a tangent with the 1.6% grade produced by the widening and three-inch asphalt concrete overlay which begins at Station 26+00. Although these reverse curves have less than minimum lengths, the result should not be noticeable to the driver because the grade changes are minor.

2. Ramps and Connectors

Iowa ramps are shown on Plate I, and Illinois ramps and connectors are shown on Plate III.

a. Iowa Ramp C

Ramp C will not be modified, but the existing acceleration taper does not provide sufficient distance for westbound entrance traffic to obtain adequate speed to safely merge with through traffic. A weaving lane with a six-ft paved shoulder will be extended westerly to the existing deceleration taper for the exit ramp to Central Avenue. A lane drop taper will be provided at this exit ramp.

b. Iowa Ramp A

Ramp A must be modified to meet the new mainline structure. To prevent rebuilding Ramp A from Main Street, the existing width of 23 ft between gutter lines and the existing radius of 130 ft were used for the new portion of the ramp. Since the distance between the existing PT Station 3003+49 to the proposed PC of the 130-ft radius curve is less than 50 ft, a superelevation transition rate of one percent per ten ft will be required. The standard deceleration taper of 15:1 would require widening of the 480-ft main navigation span; therefore, a taper of 12.9:1 is used and widening ends at the west navigation pier. Grades on Ramp A will not exceed two percent.

c. Iowa Ramp D

Ramp D must be rebuilt to meet the new mainline structure. To avoid complete reconstruction of the approach embankment from Columbia Street, the existing width of 23 ft between gutter lines and the existing radius of 100 ft were used for new Ramp D. The existing upgrade for the approach embankment is about 5.8% and the proposed maximum grade for Ramp D is 6.0%. It will be necessary to remove and replace about 134 ft of the eastern half of the approach pavement to develop superelevation. This superelevation transition produces a 5.2% grade along the Ramp D baseline on embankment. Because of the 550 ft of upgrade and the sharp radius of 100 ft, eastbound traffic from Ramp D will be moving very slowly when approaching the mainline structure. Based on a design speed of 20 mph for Ramp D, an acceleration distance of at least 800 ft is necessary for eastbound entrance traffic to safely merge with US 34 through traffic. This requires an auxiliary lane across the navigation span. Direction was given by Iowa DOT to continue this eastbound auxiliary lane across the bridge to intersect the exit ramp taper into Illinois.

d. Illinois Ramps

The four ramps for Chinn Avenue interchange were developed from Illinois DOT's IDS, dated May 1983. The Ramp A deceleration lane conforms with Illinois Bureau of Location and Environment manual criteria. Illinois standard entrance terminal design is used for Ramp B. Stop control will be used for Ramp C due to the proximity of the eastbound lane drop. The exit terminal for Ramp D conforms to Illinois design with exclusive (auxiliary) lane including a recovery area and taper beyond the gore.

e. Illinois Connectors

Alternates A and B were developed from Illinois DOT's IDS to provide connections to the Riverfront Access Road. Connector Alternate A closely follows an existing gravel road from Station 81+50 to Station 86+50 and crosses the slough at the location of an existing farm entrance. After crossing the slough, Connector A closely follows an existing dirt trail at the edge of an old cornfield. The alignment of Connector Alternate B is arranged to minimize the cost of the bridge over the Connector.

B. NORTH ALIGNMENT

1. Mainline

a. Horizontal

Existing Iowa stationing was established and projected forward along the north alignment across the Mississippi River to form a station equation, shown on Plate XXXIV, with the stationing used in Illinois. Illinois stationing is based on equating the proposed point of compound curve to the existing PC Station 25+30.35.

The north alignment is based on the approximate centerline as established by Iowa Department of Transportation for use during previous alignment studies and the public hearing. This alignment requires the acquisition of a one-story frame building on the Iowa bank of the Mississippi River. In order to minimize reconstruction of the Iowa approach, the existing centerline is used up to the proposed PC Station 1002+44.51 of a selected 4°00' curve to the left. Refer to Plate XXXIII. The proposed PI was set at the same point as the PI of the existing 3°30' curve. The forward tangent of the 4° curve was established by setting the proposed centerline 120 ft upstream from the centerline of the existing west navigation pier. The 120-ft distance from centerline of the existing pier is the minimum needed to construct a new pier using a cofferdam because of the riprap around the existing pier. As discussed later in this section, the proposed location of the 4° curve dictates major reconstruction of Ramps B and C. To minimize the reconstruction, an edge slope ratio of 1:110 (30 mph design) was used for the superelevation transition at the PC of this 4° curve. The cross slope at the PC was set at 0.03 ft/ft. Any easterly movement of this curve would increase the delta angle resulting in additional northerly movement of the curve over the river. The use of the maximum curve of 6°45' rather than the 4°00' would eliminate reconstruction of Ramps B and C, Main Street, the roadway approach west of the existing abutment and the first bridge span over Main Street. Although the 6°45' curve would result in considerable savings in construction costs, a 4°00' curve is recommended as a compromise between desirable geometrics and excessive construction costs. Even the 4° curve is not desirable because it occurs in a substandard interchange and would be the sharpest curve used on US 34 for miles in either direction.

The existing median width on the Iowa approach roadway is 18 ft, and the proposed median width for the river bridge is 8 ft 2 inches. A median transition is proposed from Station 997+04 (opposite the beginning of the deceleration taper for Ramp B) to existing PT Station 1000+16. This results in an edge of pavement taper ratio of 63:1.

The PC of the curve to the right over the river was located to provide a sufficient tangent distance east of the navigation channel to allow for the anchor span of a cable-stayed bridge as shown on Plates XLIX and L. Over the river, a 2°20' curve was selected as the flattest curve which allowed an acceptable tie-in to existing US 34 PC Station 25+30.35 just east of the bridge over Chinn Avenue in Illinois. Refer to Plates XXXIV and XXXV. The existing PC Station 25+30.35 of the 1°29' curve was converted to a proposed PCC of the same station. The 1°00' curve in the vicinity of Chinn Avenue was selected as the flattest curve to end at the proposed PCC and to provide a tangent alignment just east of the Illinois bank between the two curves. A 1,000-ft long taper is used to transition from the four-lane mainline section to meet the existing 24-ft wide pavement about 115 ft westerly of the west abutment of the existing bridge over the Burlington Northern Railroad.

Since spirals would require additional modifications to the Burlington interchange and approach and extend onto the anchor span of a cable-stayed bridge, spirals will not be used.

If a through-structure bridge is selected to be built, the degree of curve over the river could be reduced from the 2°20' (required because of the anchor span of a cable-stayed bridge) to 1°40'. The PC of a 1°40' curve would be 75 ft east of the east navigation pier. The superelevation would be reduced from 0.048 ft/ft to 0.038 ft/ft.

b. Vertical

Since a median transition and the addition of a weaving lane and lane drop are the only necessary revisions along the roadway of the Iowa approach until just before the existing abutment near Main Street, a three-inch asphalt concrete overlay on the roadways is proposed, and the existing descending 5.0% grade in the vicinity of the abutment will be maintained.

All of the vertical controls and the grades as shown on Plates XXXVI through XLI are the same or similar as previously discussed for the south alignment.

Although the mathematical grade ends at Station 27+00 on the Illinois approach, widening and a three-inch asphalt concrete overlay of the existing pavement will begin at Station 26+00. The vertical curve length of 100 ft at Station 26+50 is less than the minimum, but this will not be noticeable to a driver because the grade change is only 0.1%.

2. Ramps, Main Street and Connector A

Iowa ramps and Main Street are shown on Plate XXXIII, and Illinois ramps and Connector A are shown on Plate XXXV. Profile grades are shown on Plates XLII and XLIII.

a. Iowa Ramp A

Ramp A must be modified to meet the new mainline structure. To prevent rebuilding Ramp A from Main Street, the existing width of 23 ft between gutter lines and the existing radius of 130 ft were used for the new portion of the ramp. Since the tangent length between the existing curve and the proposed 130-ft radius curve is only about 60 ft, a superelevation transition rate of one percent per 12 ft is required.

The standard deceleration taper of 15:1 would require widening of the navigation span; therefore, a taper of 11.4:1 is used and widening ends at the west navigation pier.

b. Iowa Ramp B

The attainment of the superelevation for the proposed 4° curve on US 34 and the requirement to widen the mainline structure dictates the need for raising Ramp B a maximum of two feet in the vicinity of Station 4003+50 (over Main Street). A maximum descending grade of 6.82% is proposed to meet the existing grade just before reaching the south abutment. If a maximum grade of 6% were used, Ramps B and D would need to be to reconstructed down to Columbia Street before meeting the existing 5.8% grade. The existing width of 23 ft between gutter lines and the existing radius of 230 ft will be maintained in order to stay within the limits of construction shown.

c. Iowa Ramp C

The attainment of the superelevation for the proposed 4° curve on US 34 and the requirement to widen the mainline structure dictates the need for lowering Ramp C a maximum of nine inches in the vicinity of Station 5003+00 (over Main Street). A maximum grade of 6.5%, which approximates the existing grade, is required to meet the constraints established by the mainline grade, superelevation transitions, and by limiting construction. The end of Ramp C reconstruction was established at an existing field splice at Station 5004+48.2. To prevent rebuilding Ramp C from Main Street, the existing width of 23 ft between gutter lines and the existing radius of 150 ft were maintained. The existing acceleration taper does not provide sufficient distance for westbound entrance traffic to attain adequate speed to safely merge with through traffic. A weaving lane with a six-ft paved shoulder will be extended westerly to the existing deceleration taper for the exit ramp to Central Avenue. A lane drop taper will be provided at this exit ramp.

d. Iowa Ramp D

Ramp D must be rebuilt to meet the new mainline structure. To avoid complete reconstruction of the approach embankment from Columbia Street, the existing width of 23 ft between gutter lines and the existing radius of 100 ft were used for new Ramp D. The existing upgrade for the approach embankment is about 5.8%, and the proposed maximum grade for Ramp D is 5.5%. Because of the 550 ft of upgrade and the sharp radius of 100 ft, eastbound traffic from Ramp D will be moving very slowly when approaching the mainline structure. Based on a design speed of 20 mph for Ramp D, an acceleration distance of at least 800 ft is necessary for eastbound entrance traffic to safely merge with US 34 through traffic. This requires an auxiliary lane across the navigation span. This eastbound auxiliary lane continues across the bridge to intersect the exit ramp taper into Illinois.

e. Main Street

Since Ramp C must be lowered nine inches, so must Main Street in order to maintain the existing minimum vertical clearance. A 5.0% grade was used to limit the reconstruction length to 220 ft.

f. Illinois Ramps

The four ramps for Chinn Avenue interchange were developed from Illinois DOT's IDS, dated May 1983. The Ramp A deceleration lane conforms with Illinois Bureau of Location and Environment manual criteria. Illinois standard entrance terminal design is used for Ramp B. Stop control will be used for Ramp C due to the proximity of the eastbound lane drop. The exit terminal for Ramp D conforms to Illinois design with exclusive (auxiliary) lane including a recovery area and taper beyond the gore. The initial curve for Ramp D was changed from a simple 250-ft radius curve to a compound curve with radii of 764 ft and 382 ft to make it the same as used for the south alignment and to comply with Illinois DOT's latest criteria.

g. Illinois Connector A

Connector A is the same as for the south alignment.

IV BURLINGTON INTERCHANGE

A. ROADWAY

1. South Alignment

Between Stations 977+30 and 982+30 a lane drop taper is added by removing the existing westbound paved shoulder and constructing a 9-inch concrete pavement with a minimum 6-ft paved shoulder. This requires moving the exit ramp gore 120 ft westerly.

A 12-ft weaving lane with a 6-ft paved shoulder is added along the westbound lanes of US 34 from Station 988+30 to Station 1001+28 by widening the existing paved shoulder. A reinforced concrete barrier is needed along the outside of the new 6-ft shoulder for most of its length to minimize rock excavation and reconstruction of the existing cut slopes. Concrete approach barriers are used to transition from the curb and gutter section to the barrier section.

A minimum of three inches of asphalt concrete overlay (thicker over the existing paved shoulder areas to reduce the four percent cross slope) is used on the westbound lanes from Station 988+30 to the abutment at Station 1002+12 and on the eastbound lanes from Station 997+04 to the abutment at Station 1002+12. In addition to surfacing the 12-ft weaving lane and shoulder, this overlay is used as a means to transition the existing median width from 18 ft to 8 ft 2 in. Plate IV shows two typical sections of the proposed reconstruction of the mainline approach roadway to the bridge.

The location of new Ramp D on structure requires that a portion of the approach pavement be removed and replaced. About 134 ft of the eastern half of the approach pavement to Ramp D is to be reconstructed with a 9-inch concrete pavement and 6-inch integral curb on a revised cross slope to meet new Ramp D.

About 100 ft of the access street between Main Street and Front Street must be relocated to provide a minimum horizontal clearance of six ft from the face of the curb to the face of the new piers for Ramp D. Eight-inch concrete pavement with 6-inch integral curbs are used for this relocation.

2. North Alignment

The construction for the mainline, weaving lane and lane drop taper are the same as for the south alignment except that between US 34 Station 1001+50, and Ramp C Station 5001+00, to the abutment at Station 1002+12 the existing 9-inch concrete pavement and bridge approach sections are to be removed and replaced to begin the superelevation transition for the 4° curve.

About 220 ft of Main Street must be lowered to maintain the existing vertical clearance under Ramp C. A 1½-inch asphalt concrete surface course over an 8-inch concrete base with 6-inch intergral curbs are used for the lowered street.

B. EXISTING STRUCTURES

For the purposes of this study, the interchange on the Iowa Approach shall be the structures carrying the freeway between Abutment 1 at Station 1002+09.6 (backface of backwall) and Pier 10 at Station 1007+68, 2 entrance ramps (Ramps C and D) and 2 exit ramps (Ramps A and B). See Plate XIII.

1. Mainline Structures

The structural system carrying the freeway is a combination of a 5-span continuous unit (Spans 1 through 5), a single span unit (Span 6) and a 3-span continuous unit (Spans 7 through 9). The entire deck consists of a 7.5-inch slab of variable width supported on steel beams or girders. The depth of steel varies between 36 inches for rolled beams and a 48-inch web for welded plate girders. All piers are concrete frames consisting of a cap beam and round columns supported on individual footings which, in turn, are supported on steel piles driven to rock. For preliminary study purposes, rock level elevations have been taken from existing interchange structure plans

Span lengths are as follows:

Span 1	58.00 feet
Span 2	75.00 feet
Span 3	63.21 feet
Span 4	65.75 feet
Span 5	51.10 feet
Span 6	72.10 feet
Span 7	57.50 feet
Span 8	61.00 feet
Span 9	51.47 feet

2. Ramp A and Ramp C

Ramp A allows traffic to exit the freeway and go into Main Street. Ramp C allows traffic to enter the freeway from the same street.

The two ramp decks are supported on the same substructure between Abutment 9C (Station 5007+26) and Pier 6C (Station 5005+45) and are separated by a longitudinal open joint. From Pier 6C onward they diverge into separate structures.

The deck for both ramps is 25 ft wide between centerline open joint and the gutter line. Slab thickness is 8 inches.

Structurally, both ramps are a system of two 3-span continuous girders. Main supporting elements are two welded plate girders 19 ft apart with 48-inch webs.

3. <u>Ramp B and Ramp D</u>

Ramp B allows traffic to exit the freeway and go into Columbia Street. Ramp D allows traffic to enter the freeway from Columbia and Front streets. The deck for both ramps is an 8-inch slab, 23 ft curb-to-curb.

Structurally, Ramp B is a system of two 3-span continuous girders. Main supporting elements are two welded plate girders 19 ft apart with 48-inch webs.

Ramp D structural system is a 3-span continuous girder. Main supporting elements are also two welded plate girders 19 ft apart with 48-inch webs.

C. PROPOSED STRUCTURAL MODIFICATIONS

1. <u>General</u>

Two main objectives must be met in designing the new structures for this interchange.

- o Existing crossing must be kept open to traffic at all times during construction of the new bridge.
- o Existing structures should be incorporated to the maximum extent possible.

These two parameters and the proposed alignment determine the structural concept suitable for the new interchange regardless of the type of bridge structure selected for the river crossing.

2. South Alignment

For this alignment, the difference between actual top-of-deck elevations and proposed elevations will make it advantageous to use Span 3 as a cut-off point between structures to be removed and those that can be used. This line has been established near an existing field splice located 14 ft from Pier 3.

From this cut-off line onward the entire slab shall be removed. Additional reasons to select Span 3 as a cut-off point are:

- o Proposed alignment starts to deviate significantly from existing alignment in this span which makes reuse of the existing deck beyond Pier 3 somewhat impractical.
- o Span 3 deck is wide enough to allow detouring traffic during construction.

a. Structural Considerations

The new alignment will allow incorporation of most of the existing structure with the exception of the mainline east of Pier 3 and Ramps A and D. This limits selection of structural systems for the superstructure and substructure to structures similar to those presently in place. The major considerations are for maintenance of traffic during construction and the span arrangements of the mainline east of Pier 3 and Ramps A and D.

b. Superstructure - Mainline

It has been determined from the cross-sections of the existing mainline deck, taken by field surveys, that some areas of the deck can be overlaid, replaced on existing steel or completely reconstructed (see Plate XVI). The deck area can be overlaid when a wearing surface of 1-inch minimum thickness plus depth of scarification or a 2½-inch maximum thickness can achieve the revised cross slope. Wherever these limits would be exceeded, the deck would be replaced with variable depth haunches over the stringers.

On this basis, all beams in Spans 1 through 5 would be used in the new structure with extra beams added on each side for the required widening. New girders with 48-inch webs would be placed in Spans 6 through 8 as continuous composite components.

The deck would be retained in Spans 1 and 2 and overlaid with concrete. New decks would be constructed for Spans 3 through 8.

All details (bearings, cross-frames, diaphrams and bracing) would reflect those used in the existing installations.

c. Superstructure - Ramp A

The new deck will begin at Station 3003+28± which is 5 ft from the location of field splice A5-A10 line in Span 3A and will end at Pier 8A which lines up with Pier 8 on the mainline at Station 3006+20± as measured along Ramp A baseline. (See Plate XIV).

The deck will be an 8-inch slab supported on two curved plate girders 19 ft apart with 54-inch webs. Radius of curvature will be 130 ft.

The structural system will be a three-span continuous girder. Spans are about 90 ft long as measured along the ramp baseline. Details will reflect those in the existing structure.

d. Superstructure - Ramp D

The new deck will begin at Abutment 7B at existing Station 6003+97 along ramp baseline and will end at Pier 8EB at Station 1006+72± along centerline of the mainline. (See Plate XIV).

The deck will be an 8-inch slab supported on two curved plate girders 19 feet apart with 42-inch webs. The radius of curvature will be 100 ft.

The structural system will be a three-span continuous girder with spans about 58 ft long as measured along the baseline. Details will reflect those in the existing structure.

e. Substructure - Mainline

Since one of the basic premises in the design of this project is to retain existing structures as much as possible, modification of existing piers is therefore restricted to adding new columns as required by the extension of the corresponding cap beam.

New and revised piers will have the same geometrical configuration as the existing piers with the exception of Pier 8EB which will have a collision wall due to its proximity to the railroad tracks.

Concrete cap beam cantilevers will be removed as shown on Plates XVII and XVIII, leaving rebars in place so they can be spliced with new reinforcement. Cantilevers will have the same proportions as found in existing structures.

Frames for all piers will be analyzed for this new configuration.

For the purposes of this study, all pier cap beams have been located at the same elevation as the existing beam, though some changes in elevation are anticipated.

All columns will be round columns of the same diameter of the other columns of that pier. Column diameters vary between 30 inches and 36 inches.

Each new column will have an individual footing, either square or rectangular, supported on steel bearing piles. Only in those cases where the new column is too close to the existing structure will a combined footing be used by enlarging the existing footing as required.

It is anticipated that new steel piles will be similar to the existing piles (HP10 x 42).

The following modifications are anticipated:

- <u>Abutment</u> - Only the area between this structure and Abutment 1B will be modified to accommodate one new beam. The wingwall will be replaced. Modifications on the north side of the abutment structure are not anticipated.
- Existing expansion device will be replaced.
- Piers 2 - through 6 will be modified as shown on Plate XVII.
- Piers 7 and 8 westbound will be modified as shown on Plate XVIII.
- Pier 8 eastbound, Pier 9 (the first land pier in Iowa) and Piers for Ramps A and D will be constructed as shown on Plate XIX.

f. <u>Substructure - Ramp A</u>

Existing Piers 4A, 5A and 6A will be removed and new Piers 4A(N) and 5A(N) will be constructed. These new piers will also be single hammerhead type and will keep the geometrical proportions of existing piers. (See Plate XIX)

Pier footings will be supported on steel bearing piles. It is anticipated that new steel piles will be similar to the existing piles (HP 10 x 42).

g. Substructure - Ramp D

Existing Piers 2D, 3D and 4D will be removed and new Piers 2D(N) and 3D(N) will be constructed. These new piers will also be single hammerhead type and will keep the geometrical proportions of existing piers. (See Plate XIX).

Pier footings will be supported on steel bearing piles.

It is anticipated that new steel piles will be similar to the existing piles (HP 10 x 42).

Although the location of Abutment 7B will be maintained, some modification in the backwall is anticipated.

3. North Alignment

a. Structural Considerations

The north alignment, with a superelevation transition to a superelevation of 5.2% for the 4-degree curve, will require considerably more demolition and reconstruction than that for the south alignment. All of the mainline superstructure, Ramp B, Ramp D and portions of the superstructure of Ramps A and C will have to be modified. All work can be accomplished while maintaining US 34 traffic.

b. Superstructure - Mainline

All the mainline deck will have to be replaced and the structural steel will have to be removed and replaced. The length of Spans 1 through 5 will be maintained in order to make maximum use of substructures. Spans 6 through 8, leading to the first land pier, will be totally new. Rolled beams will be used for all span modifications and new spans.

All details (bearings, cross-frames, diaphragms and bracing will reflect those used in the existing installations for Spans 1 through 5.

c. <u>Superstructure - Ramp A</u>

This ramp will be reconstructed from the girder field splice in Span 3A (a distance of 21'-6" from Pier 7C) to Pier 9 (the first river pier). A total length of replacement of 332 ft along the Ramp A baseline will be required.

The deck will be an 8-inch slab supported on two curved plate girders 19 ft apart with 48-inch webs.

The structural system will be a 3-span leading to a 4-span continuous girder with spans measuring about 60 ft to 70 ft. Details will reflect those in the existing structure.

d. <u>Superstructure - Ramp B</u>

This ramp will be reconstructed from the west abutment on the mainline to Abutment 7B a distance of about 390 ft as measured along the baseline. The deck will be removed and the girders in Spans 1B, 2B and 3B will be raised. The girders in Spans 4B, 5B and 6B will be replaced.

The deck, girders, and details will be similar to those presently constructed.

e. <u>Superstructure - Ramp C</u>

The deck and girders for Ramp C will be reconstructed from a splice in Span 4C (21'-6" from Pier 5C) to the mainline structure a distance of about 115 ft. The deck, girders and details will be the same as those presently in place.

f. <u>Superstructure - Ramp D</u>

This ramp will be a totally new three-span continuous unit about 186 ft in length. It will start at Abutment 7B and terminate at mainline Pier 7.

The deck will be an 8-inch slab resting on two curved girders with 48-inch webs. All details will reflect those used on the existing structure.

g. Substructure - Mainline

As the superelevation of the mainline is considerably different than on the existing bridge and as the abutment and Piers 1 through 5 can be retained in their present position, the following modifications could be entertained for these units.

- Abutment - This unit is in a superelevation transition area and all bridge seat elevations will be lowered about 1½" on the north and raised about 6" on the south. This will be accomplished with new concrete pedestals on the existing bridge seat or through a modification of the bearings. That portion of the backwall, previously poured with the deck slab, will be removed and replaced with the new deck.
- Piers 2 through 5 - These units will have to be modified by extending the existing cap to receive additional girders to support the widened roadway. In addition, the superelevation transition and/or superelevation will be accommodated with concrete pedestals or a modification of the bearings on Piers 2 and 3, and cap replacement on Piers 4 and 5.
- Piers 6 through 8 - These units will be totally new.

h. Substructure - Ramps

Ramp A - For the new construction of a portion of this ramp, Pier 6C will have to be modified and three new piers, 4A, 5A and 6A, will have to be built.

Ramp B - The west abutment bridge seat and the backwall will have to be modified on Ramp B as was proposed for the mainline. Piers 2B and 5B will have modified pedestals and the cap on Piers 3B and 4B will be replaced.

Ramp C - For this substructure, the pedestals will be adjusted at Piers 3C and 4C.

Ramp D - For Ramp D, a new Pier 3D will be constructed.

For all foundations on the Burlington Interchange, steel piles (HP10 x 42) will be used as end bearing to rock.

V RIVER BRIDGE

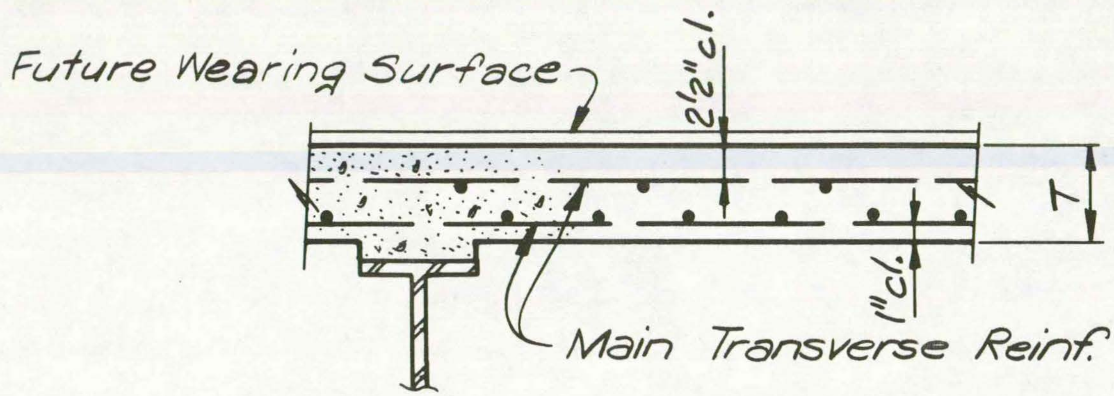
A. GENERAL

For either alignment, north or south, the river bridge for the MacArthur bridge replacement is measured from the first land pier in Iowa to the east abutment adjacent to and riverside of the levee in Illinois.

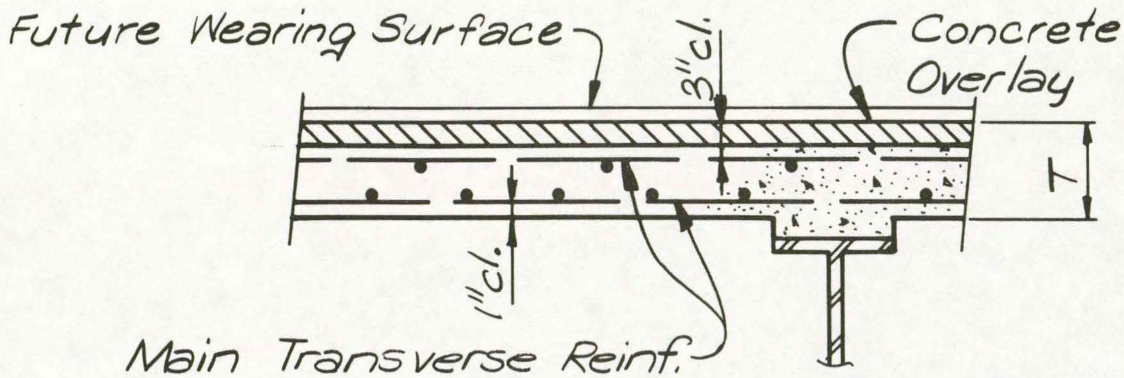
A number of structural concepts are applicable for consideration for both the navigation span and its east and west approach spans. A navigation span, providing for a minimum horizontal clearance as presently exists and as measured parallel to present structure could be a simple span through truss, a simple span tied arch, or cable-stayed girders in concrete or steel. Deck girders are not considered feasible because of their required structural depths which would adversely affect approach gradients that are already too steep. For the approach spans, multiple steel girders, multiple prestressed concrete girders and post-tensioned concrete box girders are appropriate concepts. These structures are discussed in this section of the report.

B. DECK SLAB

For a major river crossing it is desirable, from the standpoint of maintenance, to provide for wear in the prime deck slab or to install a durable wearing course. This can be accomplished by discounting a portion of the prime deck slab for structural capacity or by providing a concrete overlay for a wearing surface. In both instances the weight of a future wearing surface is to be included in the designs.



WITHOUT CONCRETE OVERLAY



WITH CONCRETE OVERLAY

ALTERNATIVE SLAB DESIGNS

FIGURE 5

In this study both systems are considered as follows (see Figure 5):

- o A concrete slab with 1" clear cover to the reinforcing bar at the bottom of the slab and a 2½" clear cover of bar at the top is used. One-half inch of the concrete at the top of slab is not considered effective in resisting loads.
- o A concrete slab with a 2" concrete overlay and a 3" clear cover to the bar as measured from the top of the concrete overlay is used. One-half of the thickness of the concrete overlay is considered effective in resisting loads. A 1" clear cover to the reinforcing bar at the bottom of slab is also used in this design.

The floor systems of the river bridge are designed for both installations and the construction costs are evaluated as shown in Section VIII. Although the design using the concrete overlay is more expensive initially, it is recommended for the reduction in future maintenance costs.

C. NAVIGATION SPANS

1. General

Concepts for navigation spans on two alignments, one south and one north of the existing bridge, and for two different span lengths on the south alignment are evaluated for the MacArthur Bridge replacement.

On the south alignment, which is 75 ft downstream and parallel to the existing bridge, a 480-ft minimum length channel span with the piers aligned with the existing piers is discussed. Because of the concerns associated with constructing the new piers in close proximity to the existing piers, which must remain in service during construction, a longer span placing the piers beyond any influence on the stability of the existing structure is also studied.

The north alignment, on an angle of 10°-12'-30" to the existing structure and 120 ft upstream of the centerline of the existing bridge, as measured on an extension of the existing west channel pier, will require a minimum length channel span of 515 ft. This will provide the channel clearance as measured parallel to the existing structure. This scheme is also investigated.

2. South Alignment - 480' Span

a. Simple Span Through Truss

A through truss concept has served the transportation needs of this country for a great number of years. In the recent past, the appearance of the structure, with its overhead mass of steel in sway frames, portals and lateral bracing, has reduced its acceptance. Although it is a very cost effective structure, other bridge types have been used for aesthetic reasons.

The first step in the design of a truss is the selection of a depth to span ratio, the setting of the panel lengths, and the adoption of a truss shape in order to optimize the structural components. For the 480-ft span, an inclined top chord is selected with a depth of 72 ft between the chords at mid-span or a depth to span ratio of 0.15. With this depth of structure, 40 ft panel lengths optimize the slope of diagonals from a minimum of 1 on 1 to a maximum of 1 on 1.8. A Warren truss (alternating compression and tension diagonals) with verticals is selected for the truss shape. Center-to-center of trusses of 82'-8" is set to provide 2" of clearance between the edge of slab and 2-ft wide truss members. See Plate XX for the General Plan and Elevation of the truss.

Following the selection of the truss, the floor system, the bracing and truss members are designed and sized in that order. The bracing design is recycled following the selection of the main truss members to insure adequate sizing.

For the floor system, two methods of framing for the stringers are considered. In the first method, the stringers are designed as continuous units resting on bearings on top of the floorbeams. The second method considers the stringers to be simple span beams framed between floorbeams. Both systems have their advantages and disadvantages as indicated on the following discussion.

- o Continuous stringers seated on the floorbeams and spaced at 9'-2", as shown in Figure 6, are W30 rolled beams of A36 material. The use of three four-span continuous units of 160 ft each requires two transverse joints in the deck slab. An arrangement of fixed and expansion seats provides stability to the floorbeam compression flange and prevents distortions from temperature variations in the structure. Welded stiffeners on the web of the floorbeams, placed under each stringer seat, act as bearing stiffeners and as required to stiffen the web.
- o Simple span stringers framed to the floorbeams and spaced at 9'-2", as shown in Figure 7, are W36 rolled beams of A36 material. These stringers are framed to the floorbeams with bolted connection angles or bolted web stiffeners and angles. Welded stiffeners are not recommended in this instance. Three transverse joints are recommended in this instance at a spacing of 120 ft. These joints relieve temperature distortions.

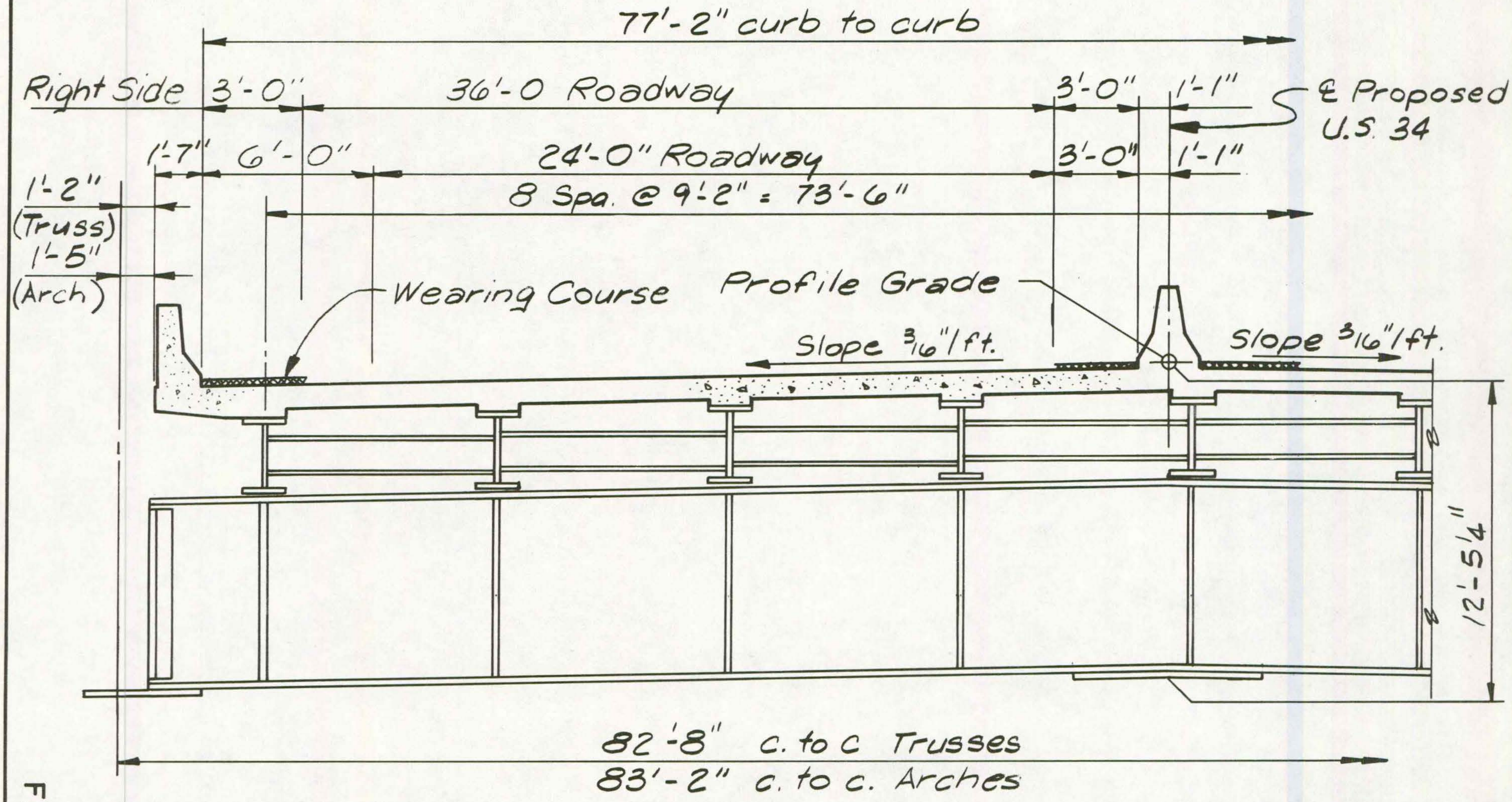
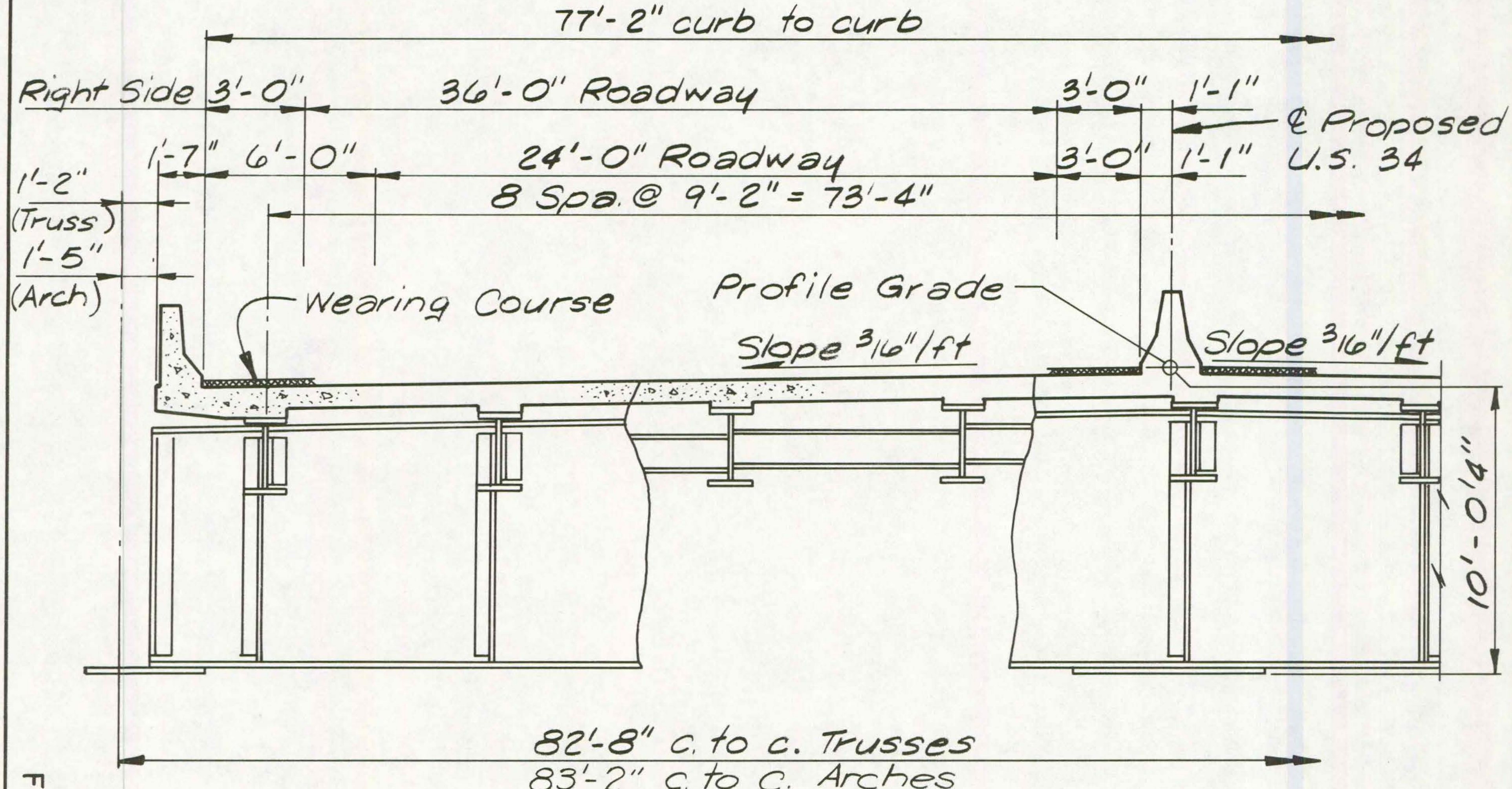


FIGURE 6

FLOOR SYSTEM
SIMPLE SPAN TRUSS
OR SIMPLE SPAN TIED ARCH
CONTINUOUS STRINGERS



FLOOR SYSTEM
SIMPLE SPAN TRUSS
OR SIMPLE SPAN TIED ARCH
SIMPLE SPAN STRINGERS

FIGURE 7

A comparison of these two framing schemes for stringers, with all other items being equal, indicates that with framed stringers a reduction in deck depth of 2'-5¼" is possible. However, the additional framing and erection would increase the cost of the floor system by approximately $200,000 and must be offset by a reduction in cost of approach piers.

In order to reduce the profile grade over the channel and decrease approach gradients, the stringers framed to the floorbeams are used in this study.

The floorbeams span 82 ft 8 inches between centerline of trusses and are designed using all high strength low alloy steel. The webs are tapered to follow the roadway cross slope. Thus stringer details and slab haunches will be uniform. The floorbeam web is stiffened with transverse stiffeners which are used at each stringer to double as bearing stiffeners and as required for shear near floorbeam ends.

After designing the floor system, a truss bracing concept is studied. A "K" configuration is used for both the upper and lower lateral systems. This system results in an efficient arrangement of bracing members due to the relationship between the structure's width and the truss panel lengths. Sway frames are laid out using double "X" bracing. At this stage, welded box bracing member sizes are assumed in order to develop loads for the main load carrying trusses.

Initially, all truss members are designed as welded boxes. Top and bottom chord members use 32-inch side plates and 23-inch top and bottom plates. Diagonals use 23-inch top and bottom plates with side plates varying from 18 inches to 28 inches depending upon the member load carrying capacity requirements. Top and bottom chord members are of high strength low alloy steel; and diagonals, hangers and posts are of A36 steel.

The main tension members of each truss are nonredundant and will be classified as fracture critical. To avoid the stringent welding control and inspection requirements, which may eliminate some fabricators from bidding on the project, and to provide additional component redundancy, all fracture critical members will be fabricated using high strength bolts. This procedure will require additional material over and above a welded box. This increase has not been included in the estimates of material for this concept.

After the truss members are designed, the bracing design is reevaluated. It is found that the box members initially assumed for the upper and lower laterals are not required and the more economical "I" sections are used instead. All bracing is A36 steel.

The truss span will require the installation of at least one falsework bent in the channel for erection. An alternative would be to assemble elsewhere and float the assembled steel structure into place. A float-in concept would require about a 48-hour closure of the channel. Falsework erection would necessitate channel restrictions for a few months.

b. Simple Span Tied Arch

The simple span tied arch has become a favorite for a variety of span lengths because of its pleasing proportions. However, the arch tie, a nonredundant tension member receiving about fifty percent of its load in tension from the thrust of the rib and the remaining load in tension from bending in the passage of live load has had problems in a number of instances. The FHWA issued a memorandum in the recent past indicating that of the 81 tied arches in service 6 had serious problems with cracking in the tie girder necessitating a bridge closure. Appropriate details and the highest of quality control in materials, workmanship, fabrication and erection are required to overcome these problems.

For this study, as shown on Plate XXI, a hanger and floorbeam spacing of 40 ft is selected for the 480-ft span. Thus the floor system is similar to that for the simple span through truss described above (see Figure 7).

A 100-ft rise of the arch rib is determined to be the most suitable, providing an adequate structure rigidity to keep combined live load and impact deflections within AASHTO tolerances.

The center-to-center of arches is set at 83'-2" to accommodate clearance between the deck and a 30" width for the rib and tie section. The rib is a box section made up of 42" web plates and 30" flange plates all of Grade 50 steel. The rise of 100 ft is achieved with a second degree parabola. The tie is a box section made up of 100" web plates and 30" flange plates all of Grade 50 steel. The centerline of tie is set parallel to the crest curve profile grade.

As for the through truss, the upper and lower lateral bracing systems is in a "K" configuration and in this instance is composed of sealed box sections of A36 steel.

Bridge strand is to be utilized for the hangers between rib and tie at each floorbeam.

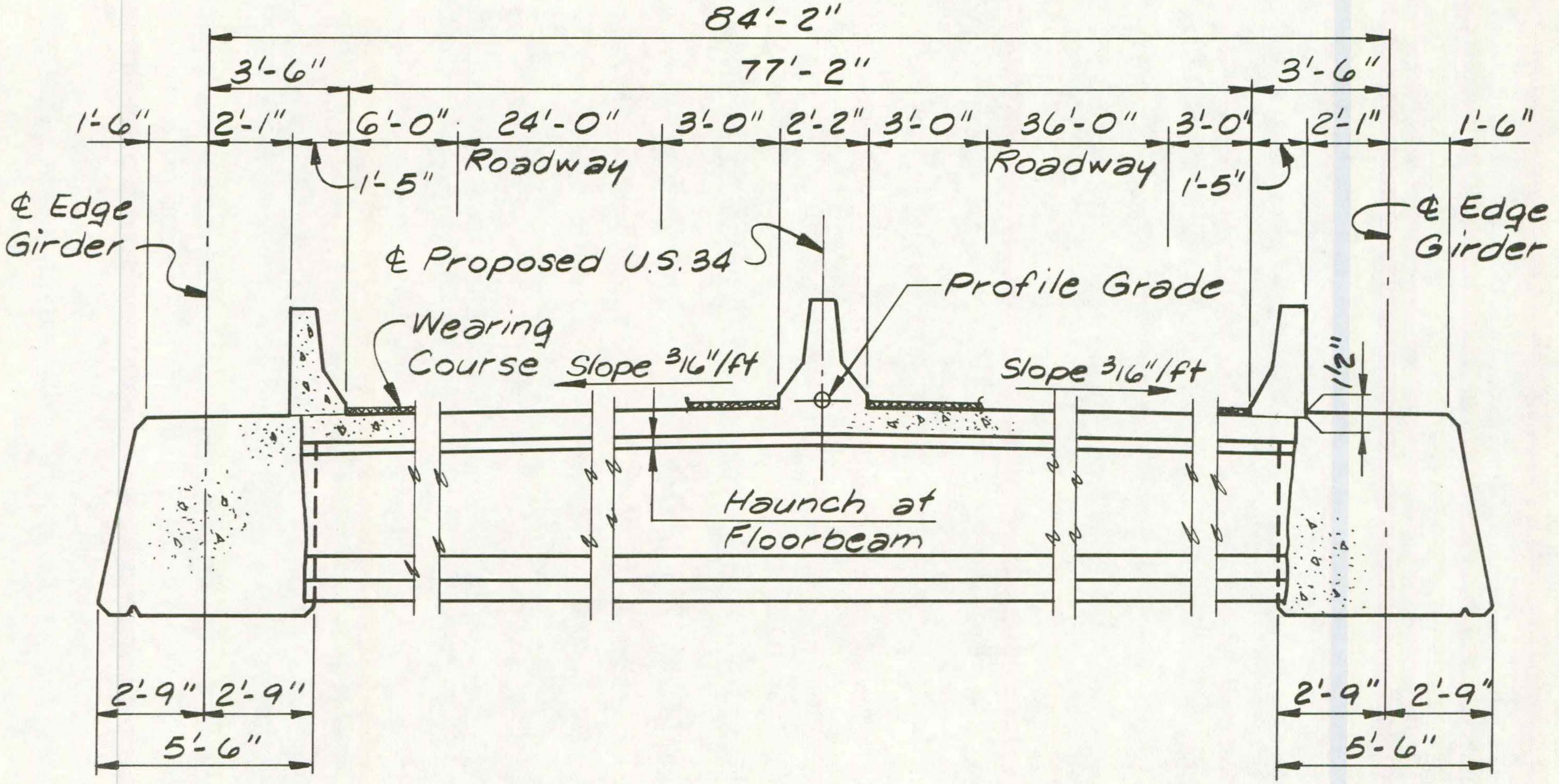
Arch erection is more easily achieved with two symmetrically placed falsework bents. However, this would restrict the channel to too great a degree and one falsework bent with tie backs may have to be utilized. An alternative would be a float-in of the erected steel.

c. Cable-Stayed Girders

Cable-stayed girder bridges have become increasingly popular in this country. As more bridges of this type are designed and constructed, a valuable base of expertise is developed. Widespread contractor familiarity with cable-stayed bridges may make them competitive for structures with shorter main spans than those previously constructed. Cable-stayed bridges have been well received aesthetically since they present a clean, uncluttered appearance. This clean appearance is largely due to the slender deck structure employed. Therefore, where structure depth must be minimized, a stayed girder bridge is an attractive option. For these reasons cable-stayed structures were studied.

The roadway to the west of the 480-ft main span is of variable width and is on a horizontal curve. This makes cable-stayed structures inappropriate for this area. Therefore, the cable-stayed concepts studied are limited to an asymmetrical arrangement with a single tower located on the east pier of the navigation channel. Both a concrete and a steel scheme are studied using this arrangement and the results are discussed below. See Plates XXII and XXIII for General Plan and Elevations of the concrete and steel alternates, respectively.

Cable-stayed concrete bridges have been constructed with the superstructures using a box girder or edge girders. The box girder, being torsionally stiff, can be designed with a single-plane arrangement of cable stays. However, with an unsymmetrical roadway (three lanes eastbound and two lanes westbound) this would not be recommended. The edge girder layout which will require two planes of stays to develop adequate stiffness is therefore chosen for this project.



CABLE-STAYED CONCRETE GIRDER BRIDGE
DECK CROSS SECTION

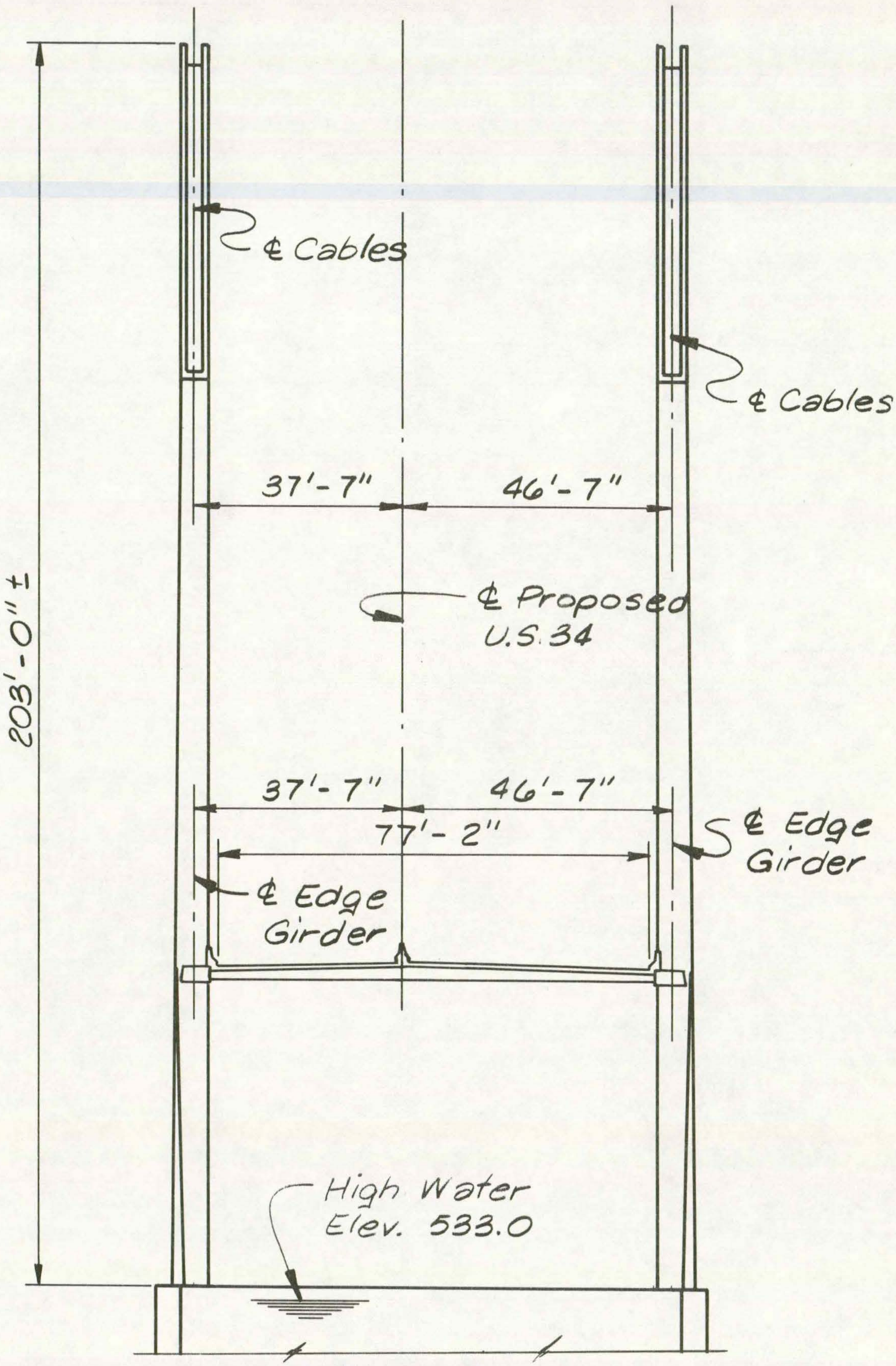
FIGURE 8

For the 480-ft channel span, a side span of 255 ft is used for a ratio of side span to main span of 0.53. The deck structure consists of a roadway slab supported by transverse floorbeams spanning between edge girders. Figure 8 illustrates the basic features of the deck.

The two edge girders are spaced transversely at about 84'-2" in line with the two vertical planes of cable stays. The cross section of these girders is approximately trapezoidal in shape, 5 ft deep at the centerline of cables and 5'-6" wide at the bottom and 4'-3" wide at the top. The clear distance between edge girders is 78'-8". Precast floorbeams are used and spaced nominally at 12'-4" on centers. These are prestressed initially for handling loads only. Once in place between the edge girders they are post-tensioned as required to carry all additional loads. The floorbeams are 3'-8" deep at the face of the edge girders and are of variable depth, following the roadway cross slope. A 10-inch cast-in-place slab is designed to act compositely with the floorbeams and the edge girders in resisting live load and superimposed dead loads.

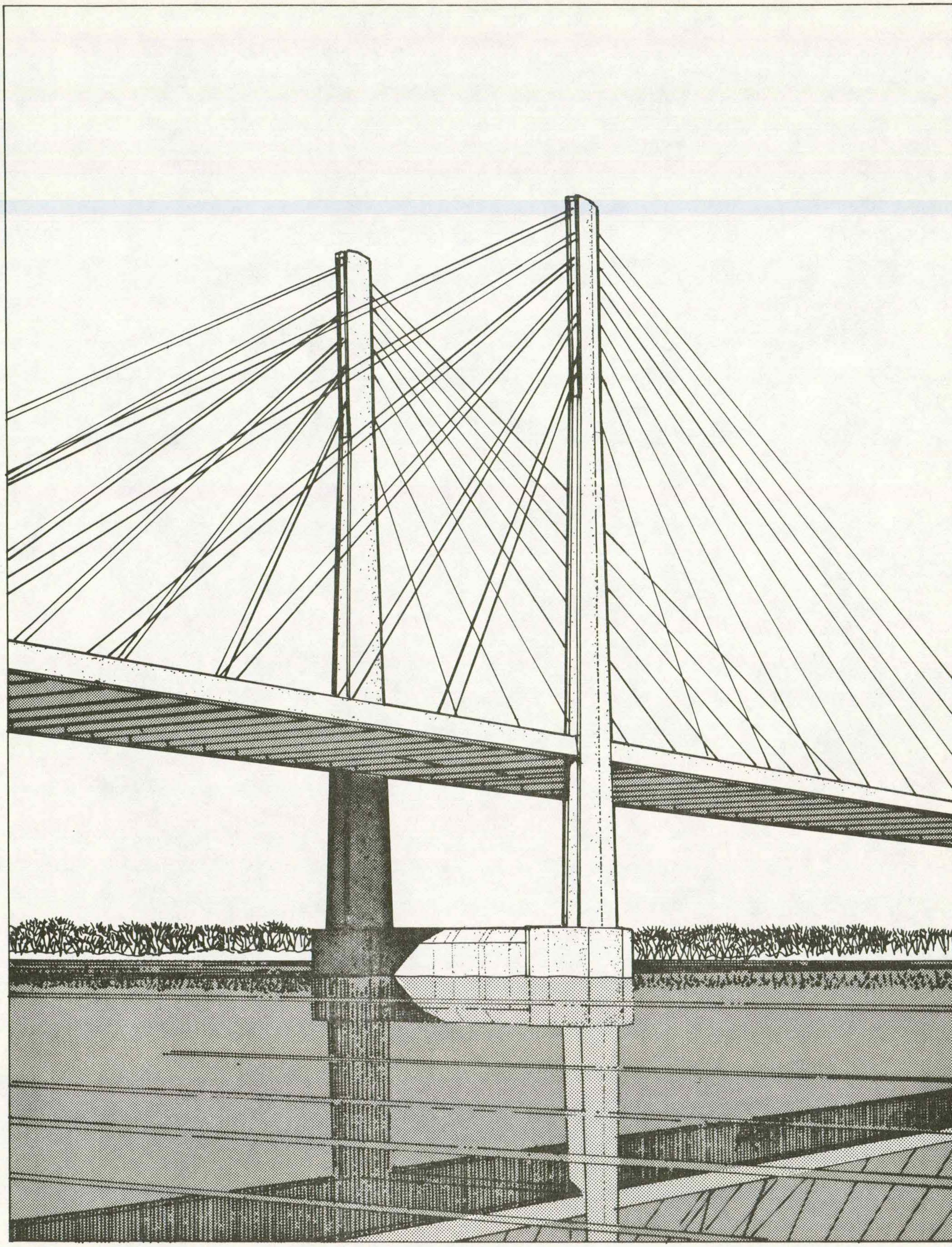
The tower is made up of two pylons integral with the edge girders, see Figures 9 and 10. The deck structure is designed at the pylons to act transversely as an integral strut. The pylons are box sections from the pedestal up to the cable anchorage zone, where a solid cross section is used. The working point of the uppermost stay is set at 144 ft above the deck level.

A fan arrangement is used for the cables. These are dead ended at the tower and tensioned from the deck level. Provision for future cable adjustment and replacement is incorporated into the structure. The cables in each plane are laid out in pairs. Nine pairs of cables are used for the main span and six pairs for the side span. The



CABLE-STAYED CONCRETE GIRDER TOWER

FIGURE 9



CABLE-STAYED CONCRETE GIRDER TOWER RENDERING

FIGURE 10

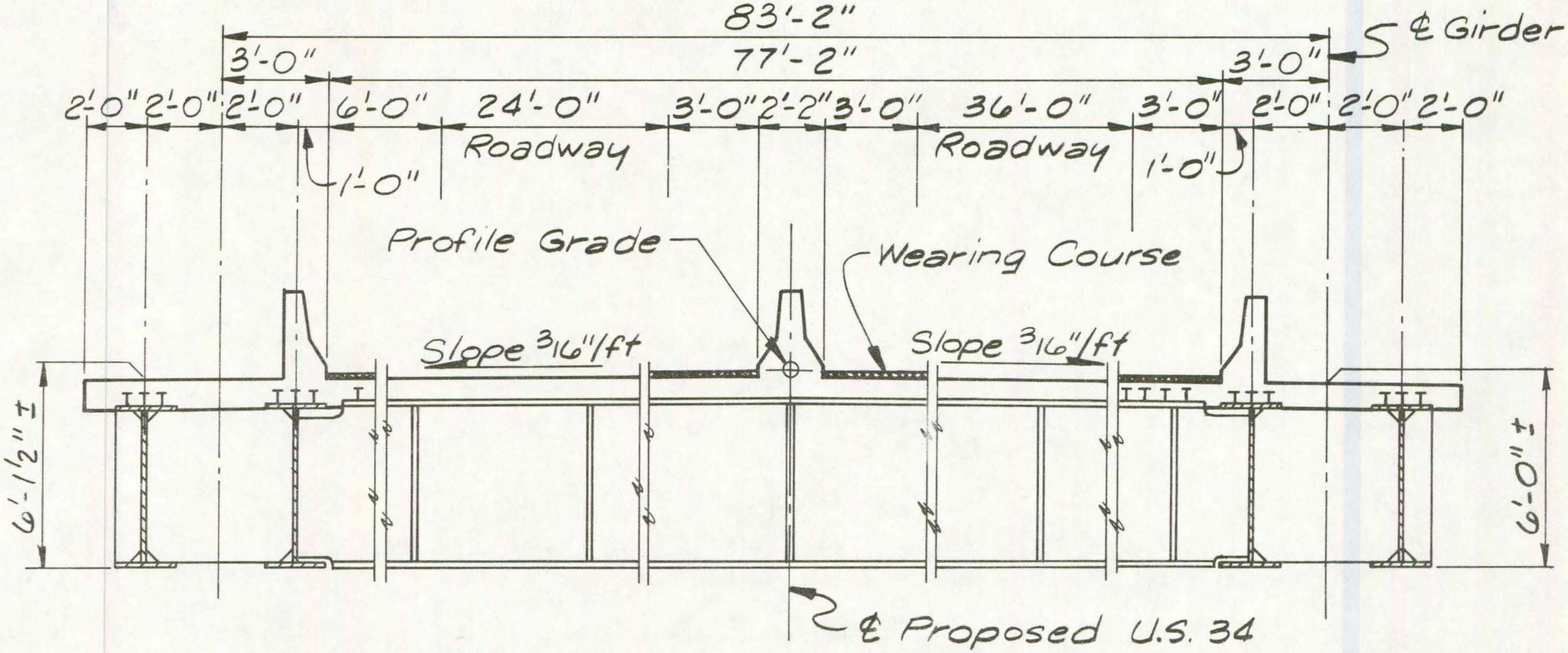
cable spacing is 37 ft horizontally at the edge girder, but is variable vertically at the pylon. The cable stays are made up with 0.6-inch diameter parallel strands encased in a polyethylene tube with anchorages compatible with the VSL system.

For the cable-stayed steel girders, various types of decks have been used. An orthotropic deck provides a light, efficient structure, but has become increasingly uneconomical due to high fabrication costs. A concrete deck was, therefore, selected for study. A system employing only transverse steel floorbeams and longitudinal edge girders is developed to simplify the deck structure. Figure 11 shows the basic features of the deck. A two-plane, stay cable system is required with this deck arrangement as discussed in the concrete girder previously.

The edge girders, which consist of a pair of girders spaced 4 ft apart, are spaced transversely at 83'-2" center-to-center. This spacing allows for the roadway and provides room to place the cable stays between the edge girders and the parapet barriers. The stays are connected to the edge girder through specially designed frames. Floorbeams are spaced at 15 ft center-to-center to minimize floorbeam depth and slab thickness. The deck slab is designed to perform three functions resulting in a 10" thickness being required:

- o Act as the main driving surface, spanning longitudinally between floorbeams.
- o Act compositely with the floorbeams.
- o Act compositely with the edge girders.

The slab is designed as a precast element made composite through the use of welded studs and cast-in-place plugs. This structure system requires no lateral bracing as the deck structure is sufficiently rigid to carry all horizontal loads to the piers.



CABLE-STAYED STEEL GIRDER BRIDGE
DECK CROSS SECTION

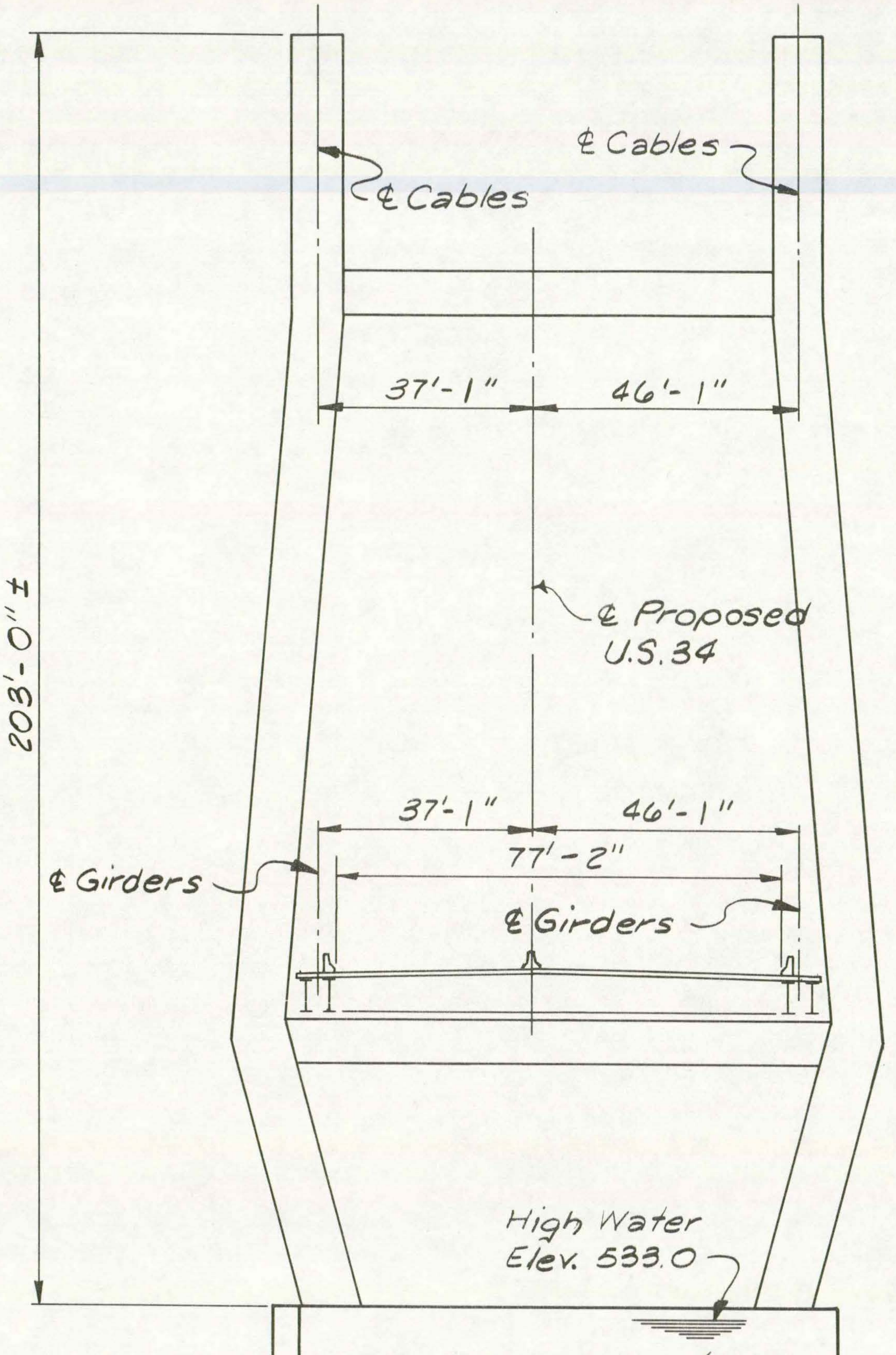
FIGURE 11

The floorbeams spanning between edge girders are designed as plate girders of high strength, low-alloy steel. The webs are 54" deep at the edge girder and of variable depth, tapering to match the roadway cross slope. The high strength, low-alloy steel edge girders are also simple "I" sections with 54-inch constant depth webs. These edge girders are designed to resist both bending moments and axial thrusts.

Basic span, tower, and cable arrangements were studied after having developed the deck structure. It was found that varying side span lengths between 250 ft and 400 ft did not have a marked effect on the main span structural response. Therefore, a length of 255 ft was chosen to minimize the length of cable-supported structure, making it comparable to the concrete scheme.

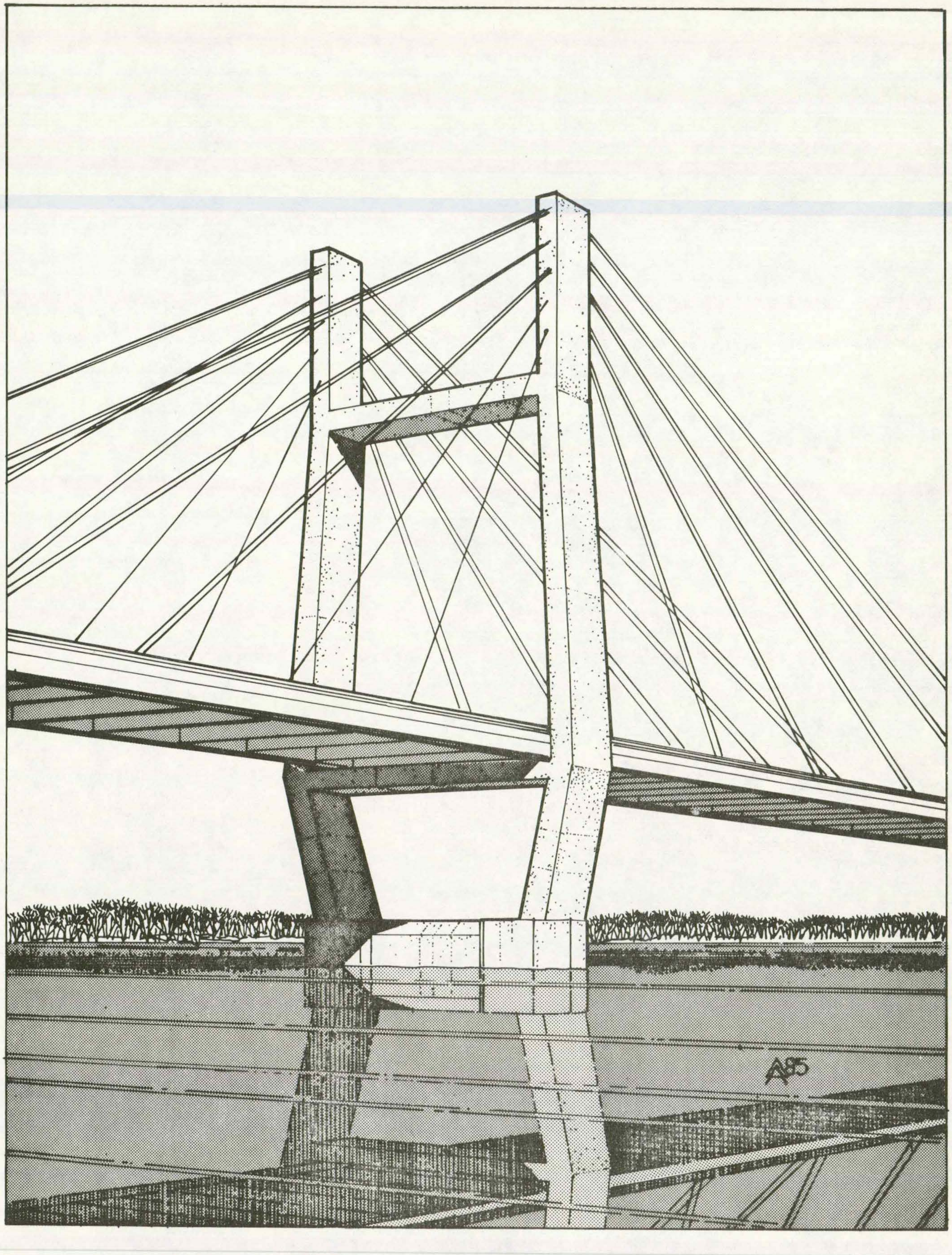
The towers for cable-stayed bridges have been constructed of both concrete and steel. However, because of the high cost of a stiffened plate steel tower, a concrete tower is selected for economy.

The tower consisted of two pylons and two struts. The pylons are arranged in a bow-legged configuration, see Figures 12 and 13. The upper portion of the pylons where the cables would be anchored are set vertically at 83'-2" apart. This allows the cable stays to lie in two vertical planes. Below that, the pylons are sloped outward to below the deck level to let the deck pass through the pylons, and then sloped inward to the top of the pier foundation. The inward slopes were necessary because of lateral interference with adjacent existing pier foundations. Two struts are introduced to brace the pylons, one at each point in the change of pylon slopes.



CABLE-STAYED STEEL GIRDER TOWER

FIGURE 12



CABLE-STAYED STEEL GIRDER TOWER RENDERING

FIGURE 13

The cables are arranged in two vertical planes in a fan pattern with the anchorages uniformly distributed over the upper part of the tower. The working point at the tower of the uppermost cable is set at 144 ft above deck level. The cable anchorages are spaced at approximately 75 ft along the main span superstructure and 45 ft along the side span. These anchorages will be located between floorbeams. The cables are dead ended at the tower with provision for future adjustment at the deck level. The structure is also designed to allow for the removal of a single cable for future replacement. The cables are made up with 0.6-inch diameter parallel stays encased in a grouted polyethylene tube. The system is designed to be compatible with the VSL anchorage system.

3. South Alignment - 660' Span

a. General

Because of the relatively high construction cost and concerns for the concept of pedestal pile piers proposed on this alignment for the 480-ft channel span, an evaluation of a longer span which would move the piers beyond any influence on the stability of the existing bridge during construction is to be considered.

This increased span is set at a length where cofferdam and seal course construction can be used to build the channel piers. If it is assumed that the rip-rap scour protection is about 60 ft out from the existing piers and a 60-ft wide seal course will be required for the new pier, then a new span of 660 ft will be required to meet the requirements. A first cut of this span is therefor recommended where the channel piers will be 90 ft east and west of the existing channel piers.

b. Through Structures

By moving the west channel pier 90 ft towards the Iowa Shore, these structures, the simple span through truss and the simple span tied arch, will have to provide for the horizontal clearance of lane tapers approaching the Burlington Interchange. This will set the center-to-center of trusses at 92'-8" and the center-to-center of arches at 93'-2" or a 10' increase over that used for the shorter span. An alternative would be to skew one truss or arch to the other but this would not be recommended because of the fabrication cost of a variety of lengths of floorbeams, sway frames, portals and lateral bracing.

In addition, the minimum vertical clearance for navigation would move 65 ft closer to the Iowa shore and increase the 5% gradient leading to the bridge.

With these considerations, the increased width of the structure and the undesirable approach gradient from the west, the through structures were dropped from any further consideration for this longer span.

c. Cable-Stayed Girders

The asymmetrical cable-stayed girders in concrete and in steel can be arranged to provide for the widening on the west end and with their relatively shallow deck depth the right descending pier can be moved closer to the Iowa shore without unduly increasing the approach gradients. Therefore, these structures, as shown on Plates XXX and XXXI, were investigated.

The main span of these structures is 660' long straddling the existing channel piers and the side span is 405' long in order to clear existing Pier J by 60 ft.

The deck arrangement, the structural depth of the deck, the cables and the towers are similar to those described previously for the 480' channel span.

4. North Alignment

a. General

As indicated above, a 515-ft channel span is required on the north alignment to provde the same horizontal clearance as that which exists. All concepts are studied for this span and are described in the following paragraphs.

b. Simple Span Through Truss

The simple span through truss for this 515' span will again be a Warren type with verticals (see Plate XLVII). Panel lengths of 36'-9" and 42'-11" were reviewed and it was determined that the longer panel length can be used while maintaining the same structure depth (10'-¼") as determined for the south alignment. The stringers would again be framed into the floorbeams. The height of truss, center-to-center of chords would vary from 45 ft at the first panel point to 78 ft at the center of the span. With this arrangement, truss members, portals, sway frames and lateral bracing would be similar to those described for the truss on the south alignment.

c. Simple Span Tied Arch

The simple span tied arch on the north alignment will have twelve panels of 42'-11" as for the truss. Therefore, the floor system will be identical to that of the truss with the stringers framed to the floorbeams.

The arch rib will take a parabolic shape raising to 105' center of rib to center of tie girder (see Plate XLVIII). The tie girder, rib, and lateral systems will be similar to those discussed for the 480-ft tied arch on the south alignment.

d. Cable-Stayed Girders

The north alignment, between the widening for lane taper leading to the Burlington Interchange and the point of curvature to the 2°-20' curve approaching the Illinois shore, has a tangent length of 787.24 ft in which the asymmetrical stayed girder schemes can be placed. Preliminary designs indicated a need to encroach on the 2°-20' curve with the anchor pier and a 515-ft channel span with a 282-ft side span was developed with a total length of 797 ft. The lateral displacemet of the anchor pier into the curve will not be enough to compromise the cable-stays.

All details for the steel and concrete girders for these cable-stayed schemes reflect those used for the 480-ft concept and are shown on Plates XLIX and L and on Figures 8 through 13.

D. APPROACH SPANS

Approach spans in the range of 100 ft to 250 ft are economically attractive in steel and in concrete for either the south or north alignments. For the lower range of span length, steel girders or prestressed concrete girders are feasible. As span lengths increase, steel girders remain economical along with post-tensioned concrete girders. All are discussed in the following paragraphs (see Figure 14).

1. Multiple Steel Girders

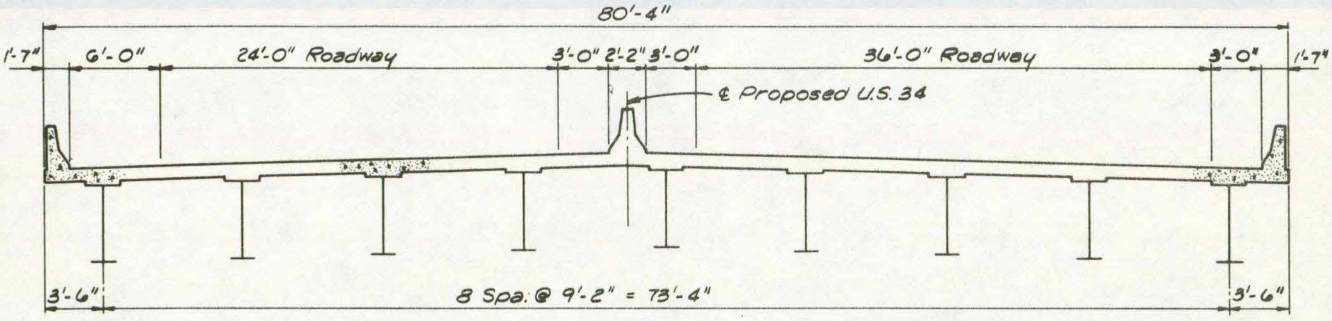
Multiple continuous composite plate girders have been used for many medium to short span structures. They are attractive both visually and economically. Various steels can be combined to achieve the greatest economy. Multiple steel girders can easily accommodate a variable width roadway or a horizontally curved alignment. It was only natural, therefore, to study this concept for the approach structure.

The first step in developing this concept is to layout a cross section providing for the 80'-4" wide deck and ignoring the roadway widening. Nine girders are spaced at 9'-2" on centers. The slab out-to-out dimensions is 80'-4" and the distance from edge of slab to center-line of exterior girder is 3'-6". Cross frames are used between the girders at all piers and at a maximum spacing of 25 ft between piers. It is anticipated that no lower lateral bracing system will be required. An 8" slab is used.

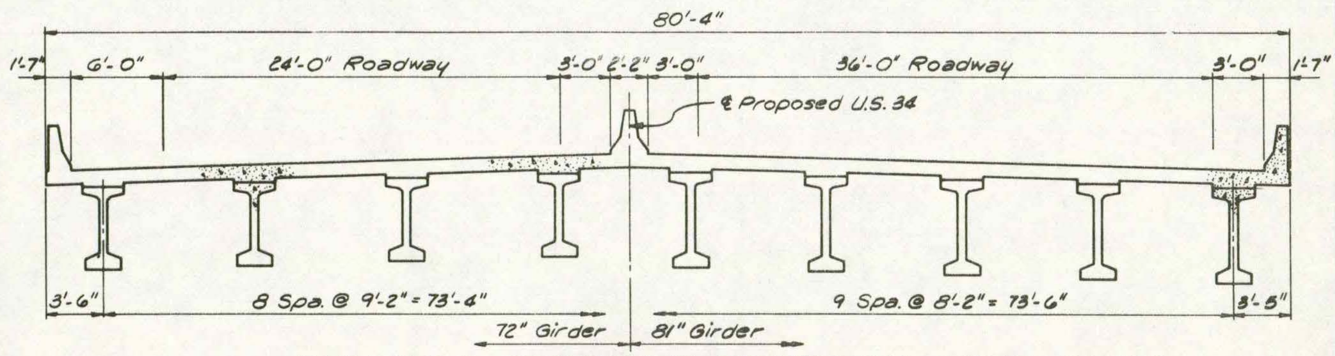
This cross section is investigated for four different span lengths. Seven span units incorporating a single intermediate hinge are analyzed and designed. All spans are of equal length. A homogeneous Grade 50 section is used at points of high negative moment. In positive moment areas, hybrid girders using a Grade 50 bottom flange and an A36 top flange and web are generally used. Webs are designed as transversely stiffened only, no longitudinal stiffeners are employed. The results for the study are summarized below.

Span	Web Depth	A36 (lbs/sq ft)	Grade 50 (lbs/sq ft)
175'	78"	18.1	16.2
200'	87"	22.1	18.1
225'	92"	21.7	24.5
250'	96"	25.9	32.1

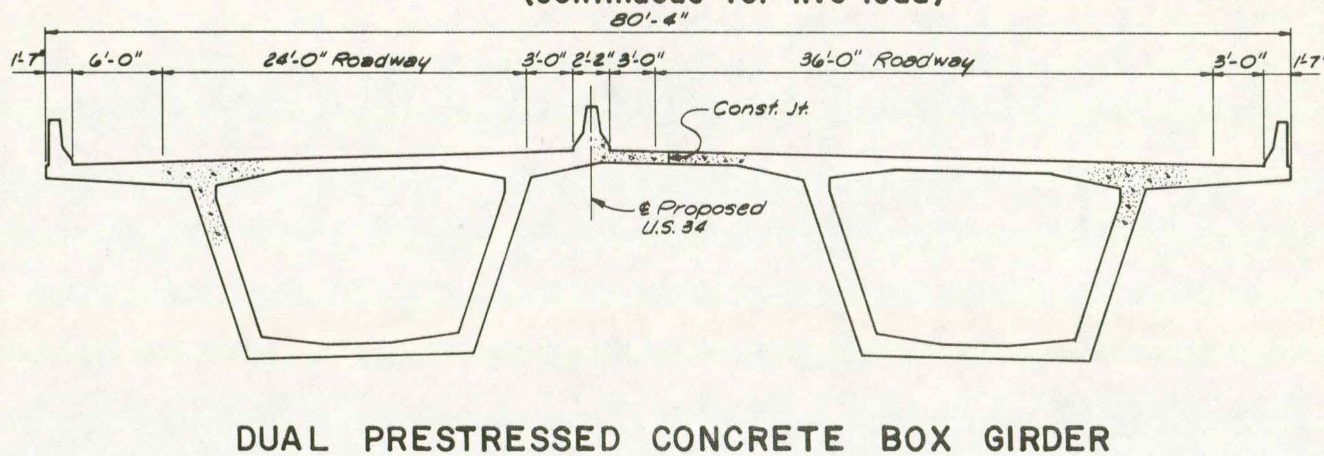
These superstructure quantities are combined with the sub-structure costs to arrive at optimum span arrangements to be used in conjunction with the navigation span concepts previously discussed.



MULTIPLE HYBRID STEEL GIRDER



MULTIPLE CONCRETE MINNESOTA TYPE GIRDER
(continuous for live load)



DUAL PRESTRESSED CONCRETE BOX GIRDER

APPROACH SPAN CONCEPTS

FIGURE 14

2. <u>Multiple Concrete Girders</u>

Prestressed concrete girders, made continuous for live load, provide an alternative to steel girders. The girders are precast and are prestressed to carry the girder diaphragm and deck slab dead loads as simple girders. If the full required prestressing force in the casting yard cannot be achieved, supplemental post-tensioning may be used at the construction site. Longitudinal reinforcement is provided in the slab over the piers to resist negative moments resulting from superimposed dead load and live load plus impact. Unit lengths of 700 ft or more are possible.

Preliminary investigations showed that 100-ft span Type IV AASHTO girders are only competitive with steel girders if trestle type pile bents were used. From an appearance standpoint and safety from barge impacts, this concept was rejected. To achieve economy it is necessary to reduce the number of substructure units and to study longer spans utilizing deeper girders. Four girder sections were chosen for study.

- Modified Type VI AASHTO Girders
- Modified Bulb-Tee Girders
- 72-inch Minnesota Type Girders
- 81-inch Minnesota Type Girders

The two Minnesota sections were included based on their efficiency and availability. Both Andrews Prestressing of Clear Lake, Iowa and Prestress Concrete Operation of Champion International, Iowa Falls, Iowa were contacted and they had indicated that they had or could obtain the necessary forms to produce the Minnesota sections.

The results of this study are summarized in Figure 15. A maximum girder spacing of 10'-6" was established for this comparison. As the span length was increased from 110 ft, this spacing was reduced as required. Therefore, each data point represents the maximum girder spacing that is compatible with the span length and girder type shown. The relative cost is the ratio of the total girder concrete volume for the particular scheme to that for the Modified Bulb-Tees at 110-ft spans. It was assumed that the deck slab would be the same for all girder types and span lengths.

Based on the above comparison, it was decided to develop the following concepts in conjunction with the navigation spans previously discussed.

- o 120-ft span 72-inch Minnesota Girders
- o 140-ft span 81-inch Minnesota Girders

The 72-inch girders are spaced at 9'-2" centers, the 81-inch at 8'-2", see Figure 14. Two types of deck as shown in Figure 5 are used. For spans in excess of 140 ft, as required on the Iowa Approach, the spacing is reduced.

3. Multiple Concrete Girders with Box Girders

To achieve spans greater than those possible using only precast concrete girders, a concept was developed combining these elements with box girders. This concept consisted of box girders over the piers cantilevering 30 ft on either side. Precast "drop-in" girders, 120 ft long were used between the cantilevered arms for a total span of 180 ft.

The box girders were designed integral with the piers. They would be cast-in-place using falsework supported from the piers. These boxes would be of multicell construction with the number of webs matching

PRETENSIONED CONCRETE GIRDERS

SPAN LENGTH VS. RELATIVE COST

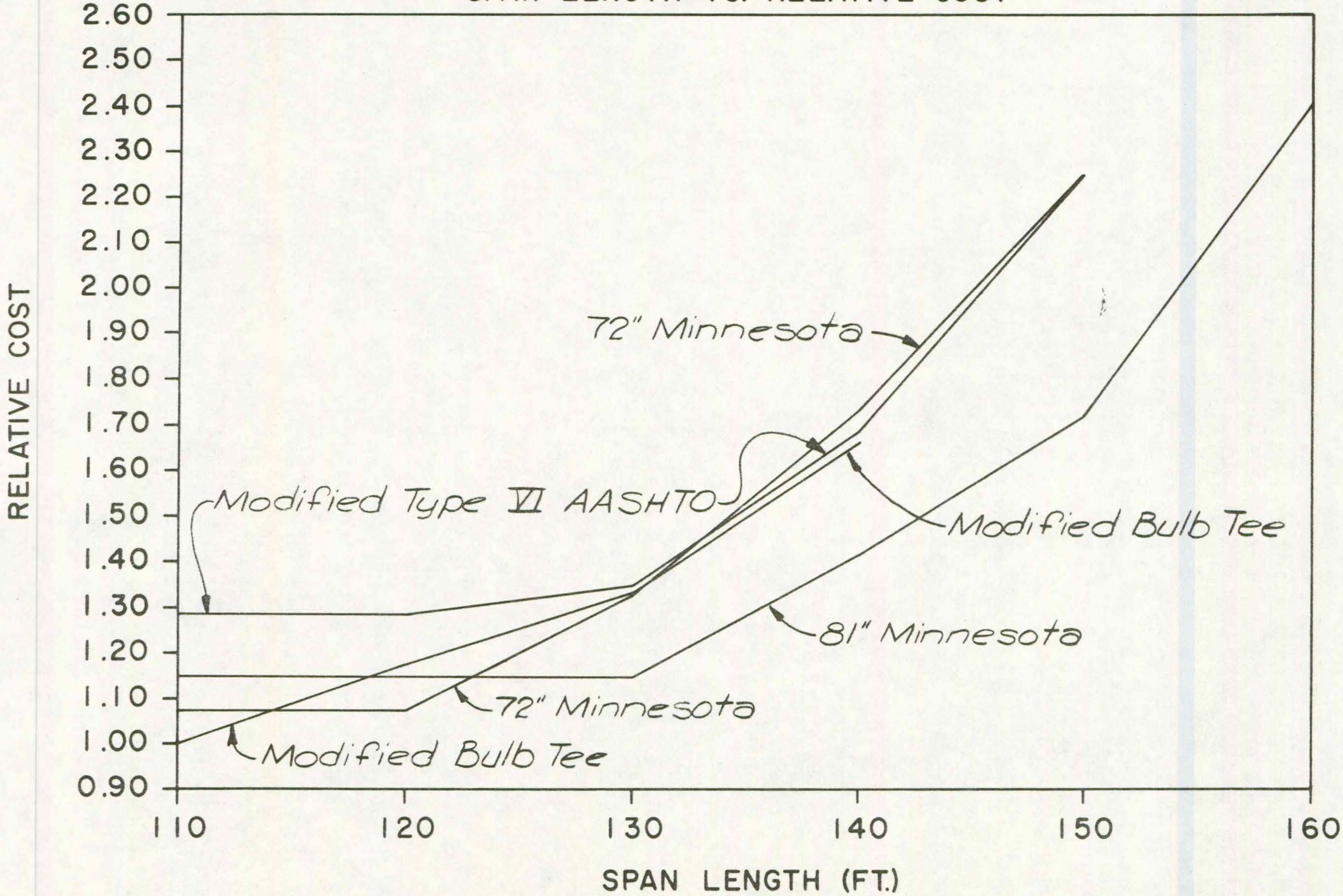


FIGURE 15

the "drop-in" girders. Transversely, mild reinforcing will be used. Longitudinally, the box section will be post-tensioned to resist its own dead weight, the dead load of the "drop-in" girders and their cast-in-place slab and diaphragms.

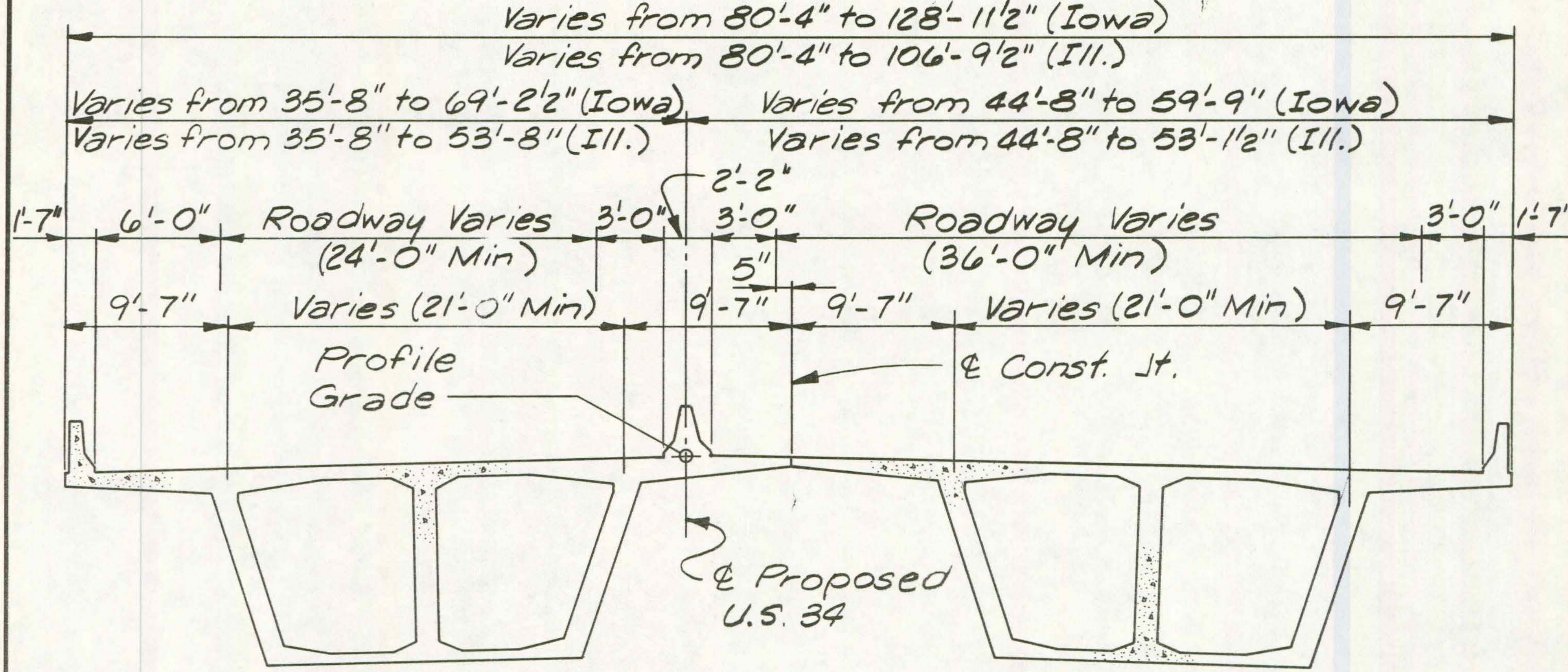
The "drop-in" girders were the 72-inch Minnesota girders discussed previously. These would be prestressed and/or post-tensioned to carry the girder dead load, and slab and diaphragm loads as simple girders. A cast-in-place diaphragm would be used to make the girders continuous with the box girder pier sections. Continuity post-tensioning in each web will resist the effects of superimposed dead loads, live load plus impact, and creep redistribution. The precast girder ends will be dapped at the seats provided on the box girder section. A total of ten precast girders were used in developing the cross-section.

This concept proved not to be economical and was dropped from further consideration.

4. Concrete Box Girders

For spans of about 200 ft and greater, concrete box girders were used in the study. This scheme was developed for two different span lengths. A 9'-6" deep box spanning approximately 185 ft was investigated as was an 11-ft deep box spanning about 225 ft.

As shown in Figure 14, two boxes combined with a transversly post-tensioned slab makes up the cross section. To accommodate increased roadway widths, the boxes are widened and a third central web is added (see Figure 16). The addition of the web will occur at a superstructure hinge.



POST-TENSIONED CONCRETE BOX GIRDER FOR WIDENED ROADWAY

Note: Dimensions shown are for the south alignment. Dimensions for the north alignment are similar

FIGURE 16

Construction of the boxes will be by the span-by-span method. The span in this case will be approximately from the 0.2 point of a one span to the 0.2 point of the next span. Since the roadway is divided by a longitudinal open joint, each box can be constructed independently of the other. Due to the variable width required, precast construction would not be economical for this project and the boxes were designed as cast-in-place. The use of an underslung movable truss to support falsework was assumed.

The deck slab of the boxes will be transversely post-tensioned. Longitudinal post-tensioning will be by grouted tendons contained within the girder webs. Tendon couplers will be used at the construction joints to achieve continuity of post-tensioning.

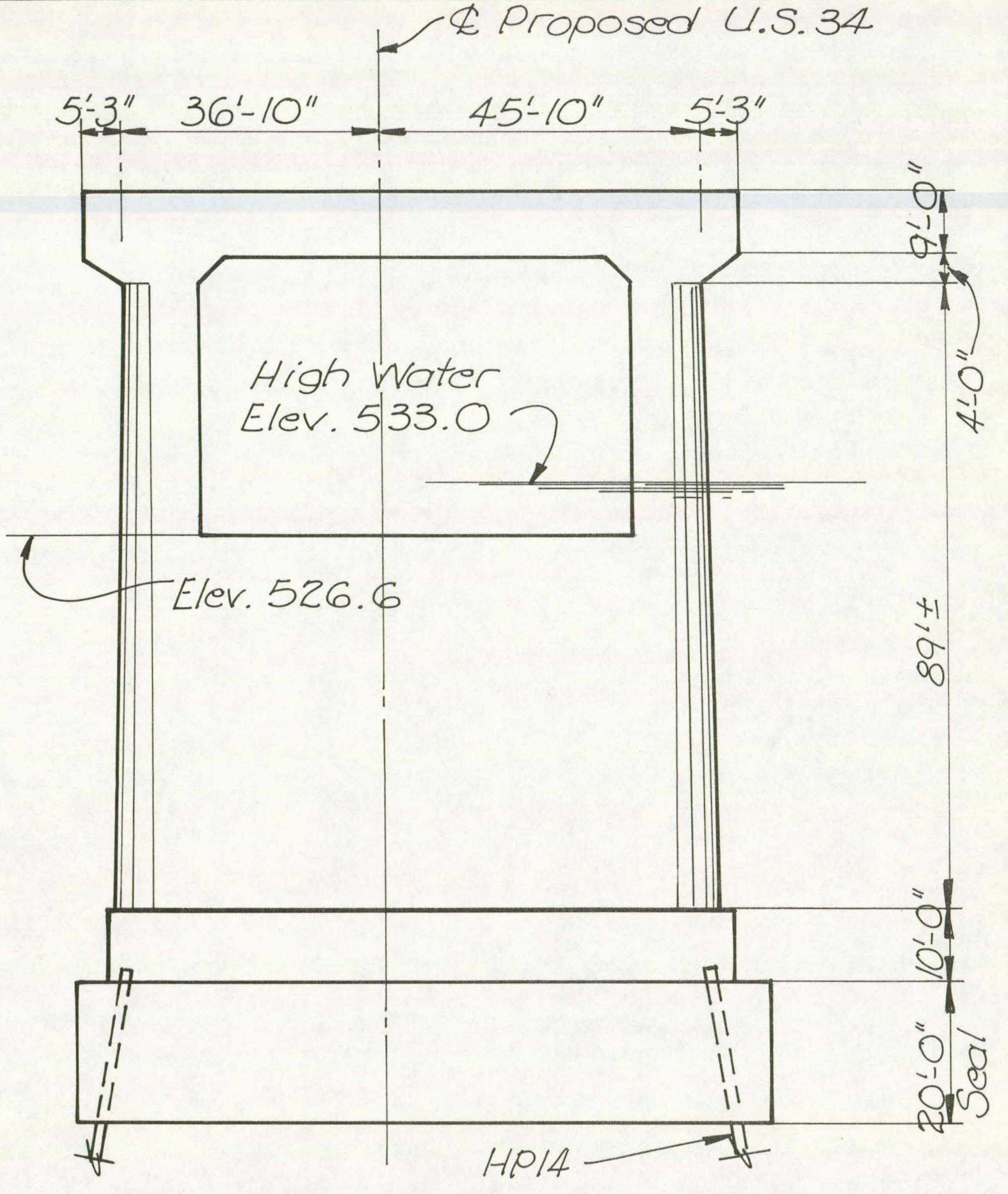
Pot bearings will be used at each of the substructure units. It would not be economical to design for an integral superstructure in combination with the relatively short substructure columns of this project.

Due to the widening, the box girder concept proved to be more expensive than steel or concrete multiple girders and was no longer considered an option.

E. SUBSTRUCTURE

1. General

The substructure for the river bridge, whether on the south alignment or north alignment, will consist of an Iowa shore land pier, water based approach and channel piers, and an abutment river side of the Henderson County levee. A discussion of these installations follows.



TRUSS SPAN PIER
NORTH ALIGNMENT

Batter shaft 1/4" on 12" all faces.

FIGURE 17

2. Iowa Land Pier

This pier, for the south alignment, is a multicolumn structure as shown on Plate XIX. The foundation of the pier will be HP10 x 42 piles driven to refusal in rock. The land pier for the north alignment will be similar in all respects to that shown.

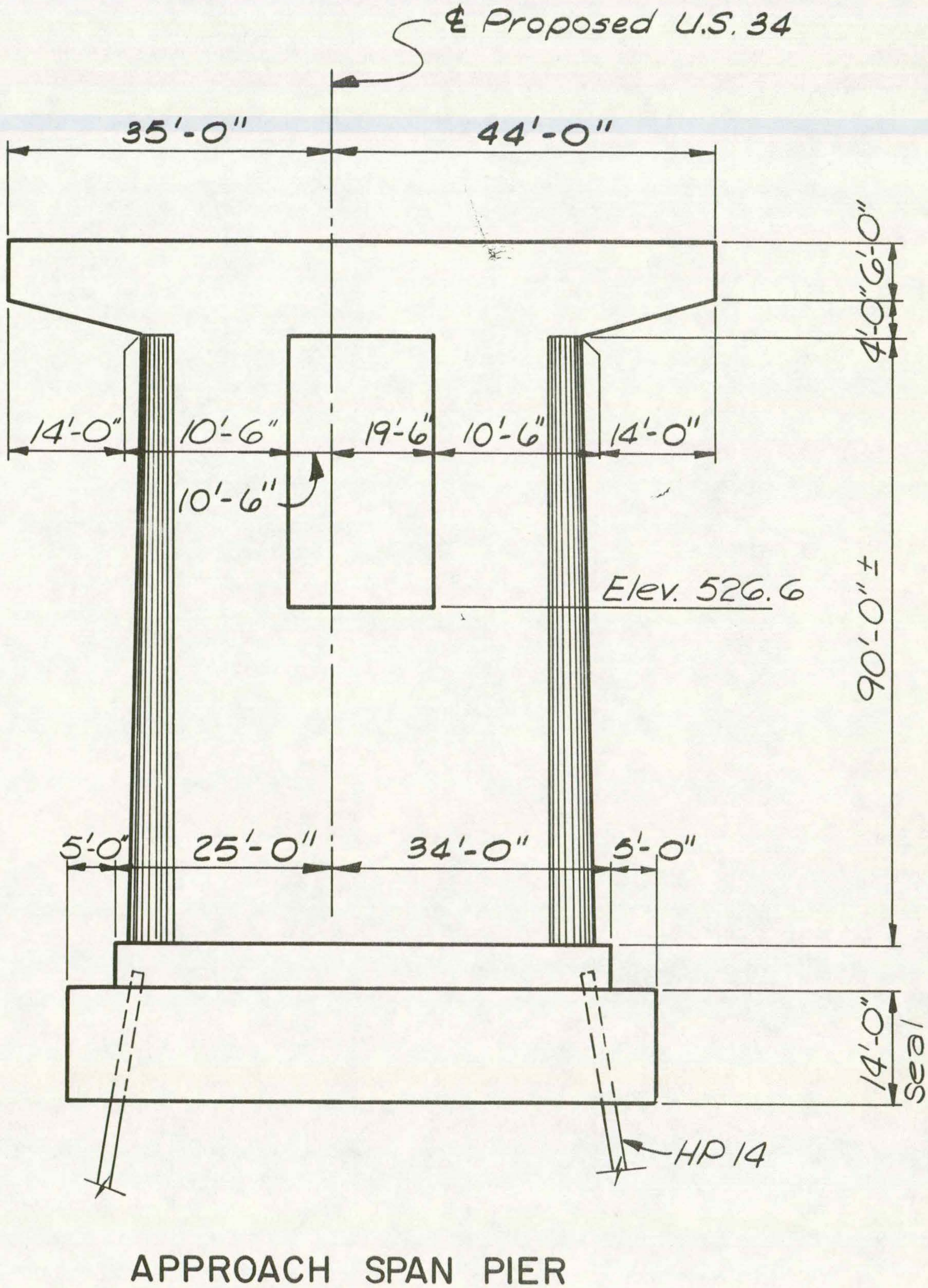
3. River Piers

In most instances, cofferdam and seal course construction techniques will be used to build the river piers as shown on Figures 17 and 18. The one exception is the channel piers for the south alignment, where concerns for the stability of the MacArthur Bridge during construction led to the development of a perched pier on drilled caissons as shown on Figures 19 and 20.

a. Cofferdam and Seal Construction

Cofferdam and seal course construction, as shown on Plate XXXII, is a common construction procedure for water piers. In this procedure, the Contractor is responsible for the design of the cofferdam and must protect the permanent construction from damage throughout that phase when the cofferdam is required for access.

This construction begins with the fabrication of bracing for the sheet piling. This includes wales, struts, knee braces, vertical posts and sway bracing. As the frame is assembled on site, soldier pile spuds are driven to support it at the proper level. The frame then forms the periphery around which the sheet piles are driven. The sheet piling is driven to a predetermined elevation below the proposed bottom of seal concrete to key it into the river bed. The river bed material inside the enclosure is then excavated to the bottom of seal concrete, permanent piles are driven, the excavation is cleaned of any loose material



APPROACH SPAN PIER

Batter shaft 1/4" on 12" all faces

FIGURE 18

and the seal concrete is placed through tremie pipes. After the seal concrete is cured, the cofferdam can be pumped down and pier construction can proceed in the dry.

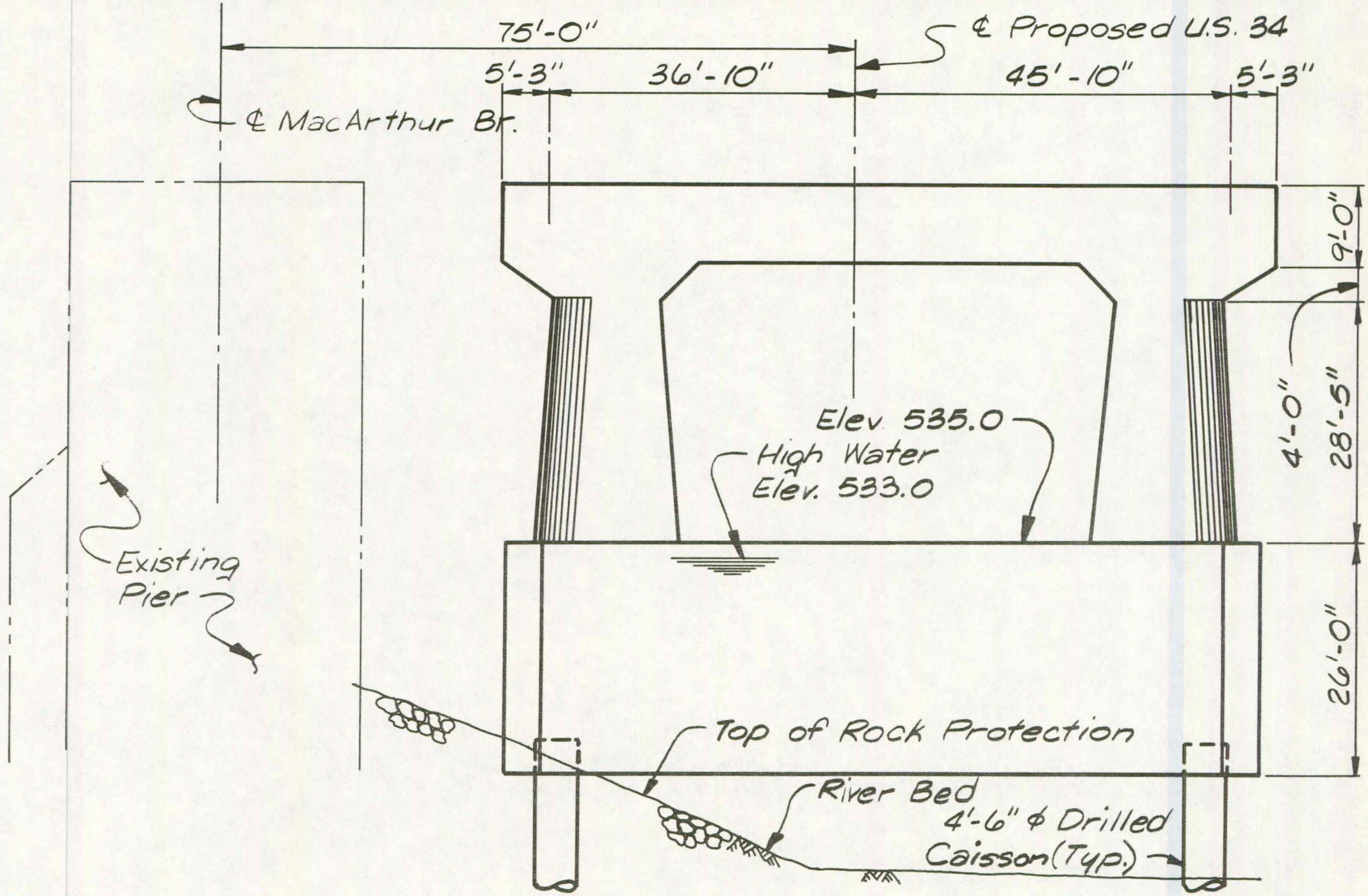
The cofferdam and seal concrete are designed to provide access to the pier construction for a range of water levels. In this instance, a high water elevation of 523 has been selected as an average of normal pool and the 2% duration levels. When water elevations exceed 523, provisions are to be provided for flooding the enclosure. In addition, the Contractor must protect the installtion from collapse due to river bed scour. This can be accomplished with upstream diversion walls, by rounding the upstream face of the cofferdam or by placing riprap around the periphery of the cofferdam at the mudline.

As previously stated, this procedure will be used for all river piers on both alignments with the exception of those piers on the south alignment which interfere with the riprap around the MacArthur Bridge piers.

b. Perched Piers with Drilled Caissons

The existing footing and pier shaft of the navigation span piers have had riprap scour protection mounded around them. The depth of riprap at the centerline of the pier is approximately 30 ft and slopes down and away from the pier on about a 2 on 1 slope. The toe of the slope, in some instances, is 60± ft from the centerline of the pier.

As the centerline of the proposed US 34 bridge on the south alignment is only 75 ft from the centerline of the existing bridge and as a seal course at the main pier would be approximately 100 ft wide (45 ft from the centerline towards the existing structure), all riprap would



TRUSS SPAN PIER
SOUTH ALIGNMENT

FIGURE 19

have to be removed to a point where it would not influence the design or construction of a sheet pile cofferdam.

Any substantial removal of the riprap, as required by cofferdam construction, would jeopardize the integrity of the existing bridge. This concept was dropped from further consideration.

In order to minimize interference with the existing structure and its riprap scour protection, a concept using a foundation supported above the river bottom with drilled caissons was evaluated (see Figures 19 and 20). This was found feasible.

The construction sequence for this concept is as shown on Plate XXIV and as follows:

1) Remove riprap symmetrically from around the existing pier to reduce its depth to about 20 ft and clear the proposed footing.

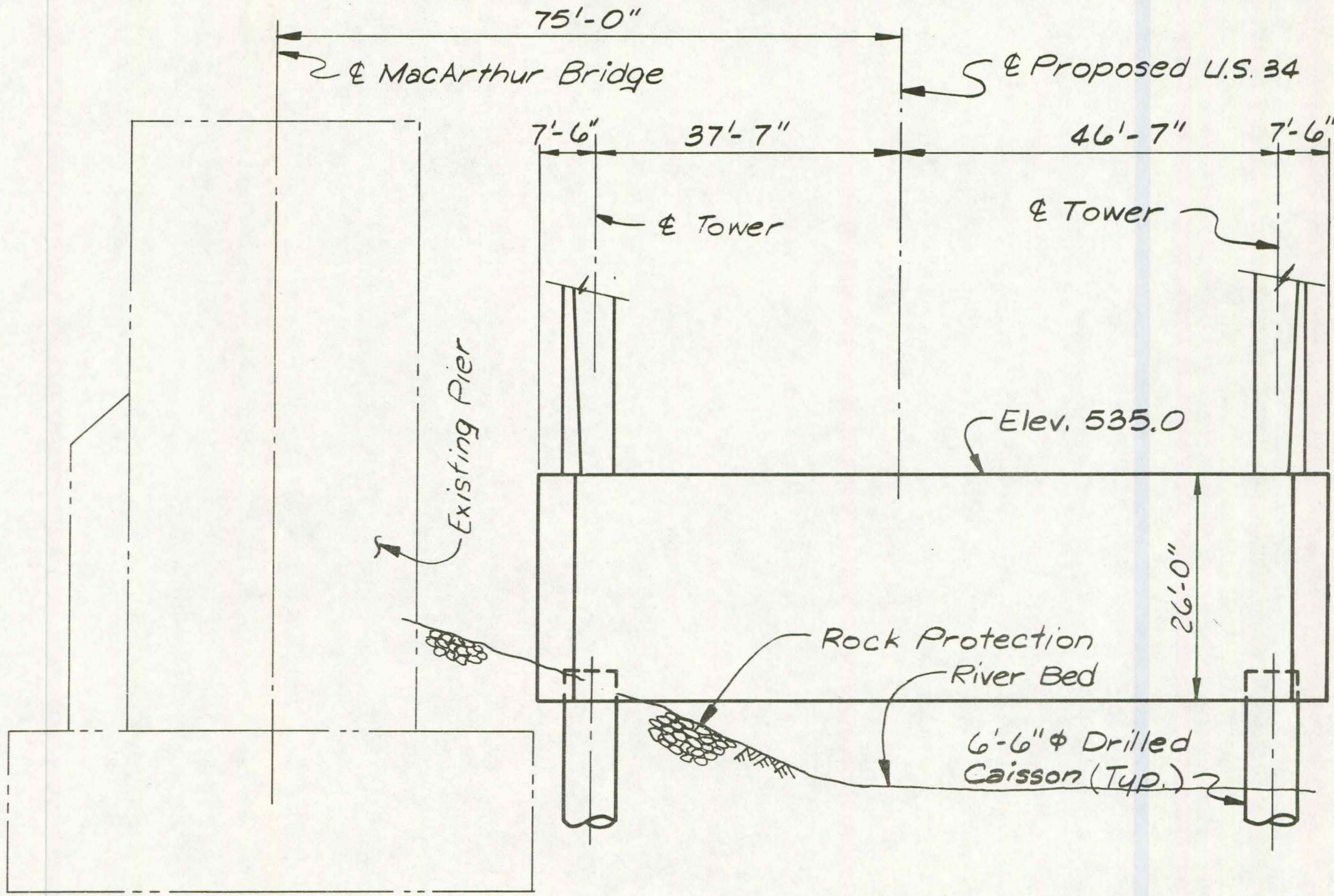
2) Grout the riprap a sufficient distance outside of the area to receive the caissons in order to stabilize the rock for drilling.

or

Place an oversized shell on the centerline of the caissons, each in turn, and with rock graples remove the riprap from the interior as the shell is lowered into the mud line.

3) Place a precast concrete box in position with spud piles and frames as shown on Plate XXIV. This box would have formed holes in the bottom to receive the shells for the caissons.

4) Drill, place reinforcement, and pour concrete for the caissons.



TOWER FOUNDATION

FIGURE 20

5) Dewater and pour the pier base in the dry.

If rock is less than 100 ft below the river bed, the caissons will be drilled to rock and socketed into it. If rock exceeds 100 ft, caisson capacity developed through end bearing and side friction will be considered.

The foundation on drilled caissons is used for both navigation span piers amd the anchor pier of the stayed-girder schemes.

In order to eliminate the need for the perched pier, a longer span can be used on this south alignment as discussed under Section V.C, Navigation Spans. Cofferdam and seal course construction can then be used. However, to move 90 ft west with the west channel pier places it in a river bed scour area with the mudline in the vicinity of Elevation 484. A preliminary design of a pier at this location indicates it would require 90-ft long sheet piling and/or a seal course in excess of 20-ft thick.

4. <u>East Abutment</u>

The east abutment on either the north or south alignment will be located on the river side of the Henderson County levee in much the same manner as the existing abutment. This is considered appropriate as any attempt to span the levee would increase the grades prohibitively.

As shown on Figure 21 for the south alignment, an embankment is placed around the abutment area intersecting the levee on the south side and the existing embankment adjacent to the US 34 roadway. River side slopes are 4 on 1 with filter cloth and riprap scour protection. On the top of this embankment, a medium height abutment with turn back wings is placed using HP10 x 42 piles as support.

The river side embankment and abutment will be completely constructed and the approach embankment will be staged with the eastbound roadway construction prior to rerouting traffic.

The abutment for the north alignment will reflect similar details.

5. Scour Protection

It is not anticipated that any scour protection will be used at any location other than at the east abutment.

The footings for all piers built by cofferdam and seal course construction are being set at that elevation which will protect the seal course and piling from becoming exposed. This has been judged from preliminary scour investigations using the Iowa Highway Research Bulletin No. 4 "Scour Around Bridge Piers and Abutments." By rounding the pier nose of a typical pier shaft and setting the footing about 10 ft below the mudline, the footing will intercept the slope of a scour hole. For those pier shafts set on an angle to the stream flow, as in the instance of the north alignment, the anticipated scour depth will increase slightly requiring an additional depth of embedment.

For perched piers, there is very little information available as to the behavior of the river bed from scour. One actual case, the Brazos River Bridge in Texas, where piles were exposed for a pier footing, the diving currents at low water created more scour than with high water. That is, the footing acted as a scour arrester for the diving currents at high water.

To approximate the amount of scour that would occur at the perched piers, the concepts of scour prediction from Bulletin No. 4 were applied assuming the drilled caisson acting as a pier shaft at the river bed. This led to the conclusion that a single 6-ft round caisson would create a valley around its periphery of about 9 to 14 ft deep, depending on water surface elevations. For design of these caissons, as a structural member with direct load, shear and bending it would be recommended that they be considered free-standing from the bottom of the suspended footing to a point of fixity well below the river bed and anticipated scour depths. That point of fixity would be determined from the soils data to be taken.

VI ILLINOIS APPROACH

A. ROADWAY

For the south alignment, the proposed centerline of US 34 varies from 75 ft south of the existing centerline at the Illinois bank of the Mississippi River to matching the existing centerline west of the bridge over the Burlington Northern Railroad. For the north alignment, the centerline varies from about 250 ft north of the existing centerline at the Illinois bank to matching the existing centerline at the existing PC east of Chinn Avenue. Since the facility is over low lands, borrow material is required to construct the embankments for the mainline, ramps and Connector A. Since the fill heights from the river to Chinn Avenue are less than 30 ft, mainline and ramp slopes need not be steeper than 6:1 within the 40-ft clear zone, except near bridge abutments. Slopes of 3:1 are used from Chinn Avenue to the end of the project because the fill height is over 30 ft. Existing drainage patterns are maintained and culverts are extended or replaced as required. The only additional right-of-way required is a small triangular piece south of Ramp D for the south alignment and that necessary for the construction of Connector A or B.

Ten-inch jointed concrete pavement over a 4-inch stabilized subbase is used for the mainline roadway from the river to Station 26+00 (the end of the full, divided four-lane section). From Station 26+00 to the end of the project, the existing concrete pavement is widened with a 9-inch concrete base course on a 4-inch stabilized subbase and all overlaid with three inches of asphalt concrete. Eight-inch jointed concrete pavement over a 4-inch stabilized subbase is used for the ramps and for the reconstructed portion of Roadway F. Various existing transverse joints and some transverse cracks in the concrete pavements for Chinn Avenue and Roadways E and F are to be replaced or repaired. A 3-inch asphalt concrete overlay is used over the pavement and shoulders.

B. CHINN AVENUE BRIDGE

1. General

The existing Chinn Avenue overpass consits of a concrete slab supported on six two-span continuous (71'-5": 76'-5") welded plate girders. Two abutments and one pier, at the centerline of Chinn Avenue, are standard Illinois structures for grade separation structures. The foundations are 50-ft long, 30-ton concrete friction piles.

Whether the new river bridge alignment will be to the south or north of the existing MacArthur Bridge, part of the Chinn Avenue overpass will be utilized during staged construction and a portion of the substructures will be utilized in the new structure. The entire superstructure, railings, parapets, deck slab and girders, will be removed and replaced in stages.

2. South Alignment

For a river bridge on the south alignment, the south portion of the revised Chinn Avenue Overpass can be constructed for two-way traffic without interrupting the movements on the existing bridge. As soon as the south one-half is complete, traffic will be rerouted and the north one-half completed.

As shown on Plates XXV and XXVI, reconstruction begins with the demolition of the south wingwalls of the two abutments and the extension of those abutment beam seats and backwall to the south. In addition, a new pier at the centerline of Chinn Avenue will be constructed which will reflect the shape of the existing pier. It is to be noted that there will be a minor conflict in the new pier cap beam with the existing which will require some minor demolition.

New welded plate girders will then be erected to receive a 7½"-thick deck slab. All details will reflect the standard details for bearings, girders, concrete slabs and railings currently in use by the Illinois Department of Transportation.

As soon as the south portion of the structure has been completed, traffic will be shifted to it and the existing superstructure will be removed. The existing pier and abutments will remain. However, new turn back wings will have to constructed on the new alignment on the north side. Beam seats on the existing abutments and pier will have to be adjusted to receive the new plate girders or the difference in elevation can be taken up in the bearings.

The north half of the superstructure can then be completed with the plate girders, deck slab and railings.

3. <u>North Alignment</u>

As in the instance of the modification as the Chinn Avenue Bridge for the south alignment, the structure can be modified under traffic for the north alignment.

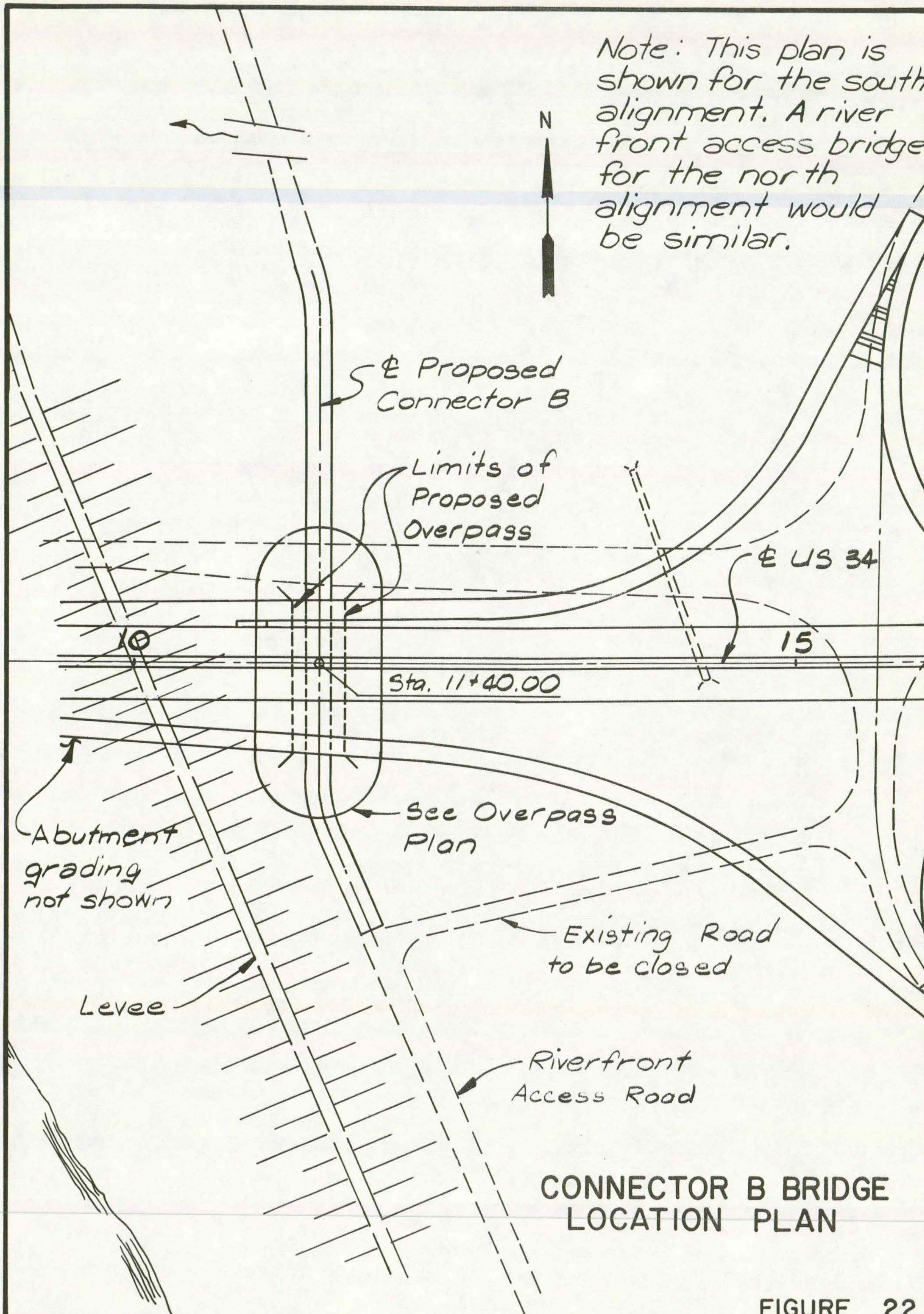
As shown on Plates LI and LII, the structure will be extended both to the north and south. Abutment wingwalls will be removed and the beam seat extended to that point where new turn back wings will be constructed. The pier at the centerline of Chinn Avenue will have to be extended and the existing cap will have to be replaced to achieve structural continuity as well as proper beam seat elevations.

With the completion of the substructure, the superstructure will be demolished in stages and reconstructed with new welded plate girders, deck and barriers. Again, all details will be in compliance with existing Illinois criteria and details.

C. RIVERFRONT ACCESS

Currently, traffic to and from the Riverfront Access Road uses a gravel road off of Roadway F. Proposed Ramps C and D will tie into Roadway F and be fully access controlled; therefore, the existing gravel road will be closed. Either Connector A or B as shown on Plate III will be constructed to provide access to the riverfront. Since the construction cost for Connector B is at least ten times more than Connector A, Connector B was only studied for the south alignment. Connector A is the same for either the south or north alignments. An 8-inch aggregate surface course, 18 ft wide will be used for the Connector Road. Connector A, although longer than Connector B (1,400 ft versus 550 ft), provides a more direct route to and from US 34 than Connector B. If Connector B were constructed, traffic from the Riverfront Access Road would have to use the streets in Gulfport to reach US 34. The additional right-of-way requirement for Connector A is about 1.5 acres as compared to only 0.04 acres for Connector B.

In providing riverfront access parallel to the levee via Connector B, an overpass as shown in Figure 22 will be required. A plan of the proposed overpass is shown in Figure 23. To provide the 14'-6" minimum vertical clearance, the proposed US 34 grade must be raised about two or three ft at Sta. 11+40.

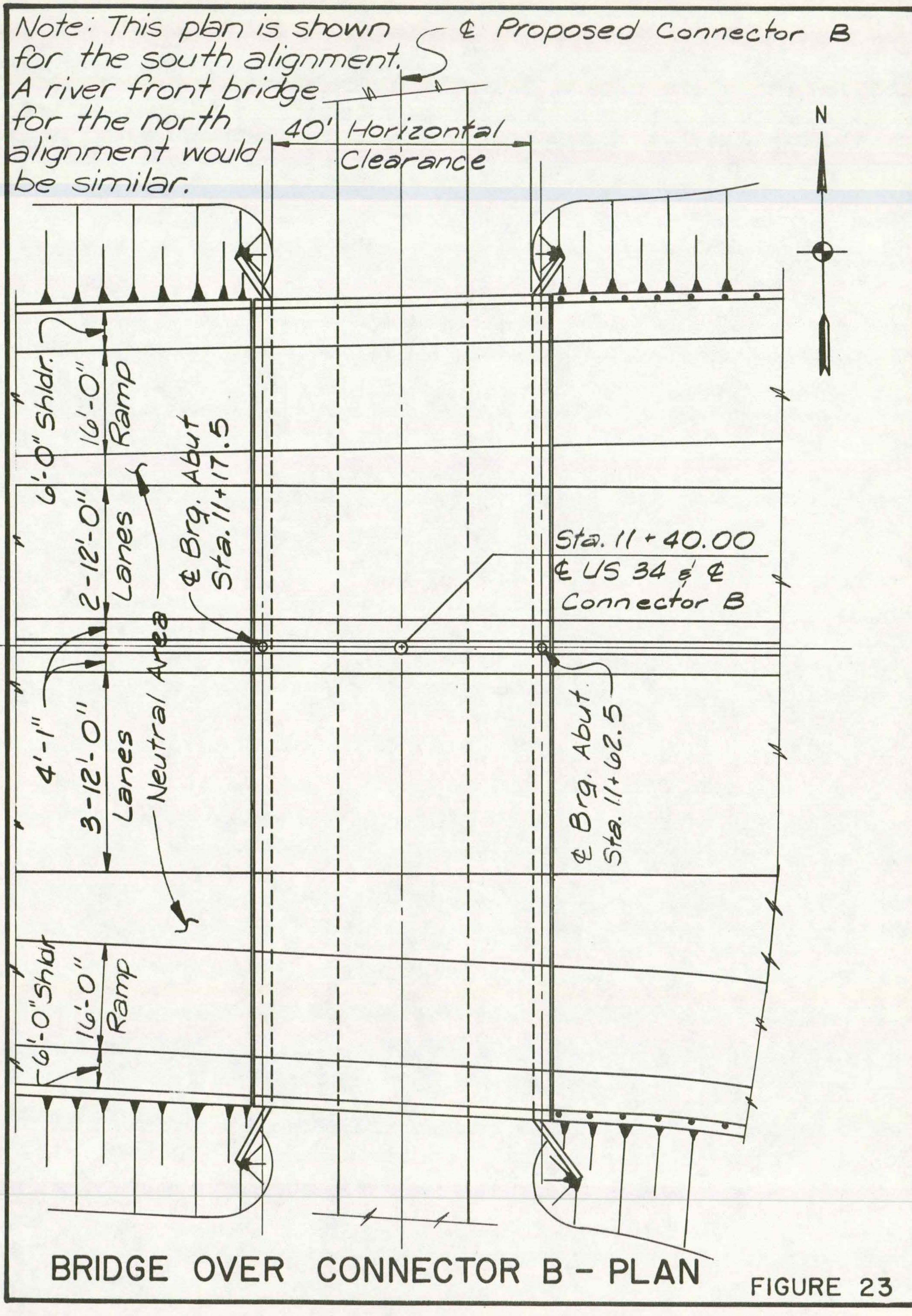


CONNECTOR B BRIDGE LOCATION PLAN

FIGURE 22

The deck structure will consist of a deck slab acting compositely with simply supported steel rolled beams. These beams will be spaced at about 9-ft 6-inch centers. The superstructure will bear on full height abutments founded on concrete friction piles. The construction cost of this structure would be about $320,000.

At the present time, the Village of Gulfport maintains Chinn Avenue, most of Roadway F and access to the riverfront. After the facility is complete, the Village should maintain Chinn Avenue and the Connector to the Riverfront Access Road. Because Ramps C and D and Roadway F will be access controlled, the maintenance of Roadway F should be transferred to the State.



BRIDGE OVER CONNECTOR B - PLAN

FIGURE 23

VII STAGE CONSTRUCTION

A. GENERAL

In order to construct a new river bridge, 75 ft downstream of the existing MacArthur bridge or a new river bridge north of the existing facility, and connect it with a modified Burlington interchange as well as a revised alignment in Illinois, it becomes necessary to study a means of maintaining an uninterrupted flow of traffic in both directions during construction.

The following paragraphs describe the means of accomplishing this objective.

B. BURLINGTON INTERCHANGE

1. South Alignment

As shown on Plate XXVII, the first order of business is to arrange for all traffic, one lane in each direction, to occupy the north side of the interchange and close Ramp D. This would be done by removing the median and toll booth and making any temporary repairs to the deck to take the traffic. A temporary barrier can then be erected to divert eastbound traffic to the westbound lanes and another barrier placed to maintain traffic on Ramp B.

All necessary modifications can then be accomplished on the south side, including deck widening, deck replacement, the reconstruction of Ramp D and new Spans 7 and 8. As soon as this is accomplished and the river bridge is ready for use, the traffic can be shifted to the south side.

At that time the north side can be modified after shifting the temporary barriers to protect mainline and Ramp C traffic (see Plate XXVIII). Deck widenings, deck replacement, new Spans 7 and 8 and Ramp A can be constructed. The 140-ft truss span and a portion of the anchor arm of the main bridge would have to be dismantled at this time.

2. North Alignment

As shown on Plate LIII, the first order of business is to move all the traffic, one lane in each direction, to occupy the south side of the interchange and close Ramps A and C. This would be accomplished by removing the median on the approach roadway west of the abutment and on structure approaching the toll booth in order that westbound traffic could move to the south side and back over to the westbound lanes before reaching Fourth Street. Temporary barriers will be erected to divert eastbound traffic to one lane on the south side of the structure and on the approximate centerline of the proposed bridge on the interchange to funnel all traffic away from the reconstruction.

All necessary modifications can then be made to the north side of the roadway, mainline structure and Ramps A and C. As soon as this is completed and the river bridge is ready for use, traffic can be shifted to the north side.

At that time, the south side of the approach roadway, mainline structure and Ramps B and D will be closed for reconstruction. The permanent median barrier on structure and a temporary barrier to divert eastbound traffic to the westbound lanes will be used to protect the traveling public from the reconstruction (see Plate LIV). The eastbound roadway, mainline structure and Ramps B and D can then be reconstructed. The existing river bridge approach at the toll booth will have to be demolished in order to complete this work.

C. RIVER BREIDGE

1. South Alignment

Almost all of the river bridge can be constructed independently of the Iowa or Illinois approach work. One exception would be the westbound lanes of River Spans 1 and 2, which cannot be constructed until traffic is shifted to the south lanes at the Burlington Interchange. At that time enough of the existing bridge can be dismantled in order to clear the area to finish the westbound lanes.

2. North Alignment

As in the instance of the south alignment, almost all of the river bridge can be built while traffic is diverted to the south side of the interchange and the MacArthur Bridge. A portion of the river bridge, Span 1, will have to wait for construction until traffic is shifted to the north side of the interchange and the old bridge is dismantled.

D. ILLINOIS APPROACH

1. South Alignment

The five general steps for the construction staging of the Illinois approach are shown on Plate XXIX. The sequence shown provides for all traffic movements during the construction period.

The first step is to close most of Chinn Avenue for the purpose of installing a new culvert south of the existing culvert, replacing pavement joints and overlaying the pavement with asphalt concrete. Second, the eastbound roadway (except for a short gap) including one-half of the bridge over Chinn Avenue is to be constructed. Third, construction for Ramps C and D and Roadway F is to be accomplished. Fourth, with all US 34 traffic using the new eastbound lanes, the westbound roadway and Ramps A and B are to be constructed. Fifth, all new roadways are to be opened to traffic and the remaining construction is to be completed under traffic.

2. North Alignment

The five general steps for the construction staging as shown on Plate LV provides for all traffic movements during the construction period.

The first step is to construct the widening for the last 1,000 ft of the project, the westbound roadway (except for a short gap), and portions of the eastbound roadway and Ramps A, B and D. This step includes the removal of the north portion of the bridge over Chinn Avenue and construction of the northern 33 ft of the new bridge. Two-way traffic will be maintained on US 34. Second, most of Chinn Avenue is to be closed for the purpose of replacing pavement joints and overlaying the pavement with asphalt concrete. Third, construction of Ramps A and B and Roadway E is to be completed. Fourth, with all US 34 traffic using the new westbound lanes, Ramp C and Roadway F and the remaining portions of the eastbound roadway and Ramp D are to be constructed. Fifth, all new roadways are to be opened to traffic and the remaining construction is to be completed.

VIII CONSTRUCTION COST

The construction cost estimates for the project have been based on preliminary designs and quantities. In addition, unit prices have been developed to represent current price levels with no allowance for escalation. These estimates include the cost of roadway, bridges, navigation and aviation obstruction lighting. A 15% contingency has been used to provide for variations in the preliminary demolition and construction quantities as well as the staging of the construction. These estimates are given in tabular form on the following pages.

TABLE I

ESTIMATED CONSTRUCTION COST SUMMARY
SOUTH ALIGNMENT

Main Spans	480-Ft. Simple-Span Through Truss		480-Ft. Simple-Span Tied Arch	
Approach Spans	Multiple Steel Plate Girders	Multiple Prestressed Concrete Girders	Multiple Steel Plate Girders	Multiple Prestressed Concrete Girders
River Bridge				
Structure Demolition	$ 1,000,000	$ 1,000,000	$ 1,000,000	$ 1,000,000
Main Span	9,237,760	9,237,760	9,779,670	9,779,670
Approaches	14,980,720	15,095,790	14,980,720	15,095,790
Embankment & Riprap	71,300	71,300	71,300	71,300
Lighting	240,530	240,530	240,530	240,530
Subtotal	25,530,310	25,645,380	26,072,220	26,187,290
Iowa Approach				
Roadway Items	259,000	259,000	259,000	259,000
Structural Demolition	397,320	397,320	397,320	397,320
Reconstruction	1,924,360	1,924,360	1,924,360	1,924,360
Subtotal	2,580,680	2,580,680	2,580,680	2,580,680
Illinois Approach				
Roadway Items	2,067,100	2,067,100	2,067,100	2,067,100
Structural Demolition	122,370	122,370	122,370	122,370
Reconstruction	491,410	491,410	491,410	491,410
Subtotal	2,680,880	2,680,880	2,680,880	2,680,880
Total	$30,791,870	$30,906,940	$31,333,780	$32,448,850
Contingency 15%	4,618,130	4,633,060	4,696,220	4,871,150
GRAND TOTAL	$35,410,000	$35,540,000	$36,030,000	$37,320,000

TABLE II

ESTIMATED CONSTRUCTION COST SUMMARY
SOUTH ALIGNMENT

Main Spans	480-Ft. Cable-Stayed Concrete Girder		480-Ft. Cable-Stayed Steel Girder	
Approach Spans	Multiple Steel Plate Girders	Multiple Prestressed Concrete Girders	Multiple Steel Plate Girders	Multiple Prestressed Concrete Girders
River Bridge				
Structure Demolition	$ 1,000,000	$ 1,000,000	$ 1,000,000	$ 1,000,000
Main Span	13,259,260	13,259,260	13,889,770	13,889,770
Approaches	13,407,530	12,571,740	13,407,530	12,571,740
Embankment & Riprap	71,300	71,300	71,300	71,300
Lighting	240,530	240,530	240,530	240,530
Subtotal	27,978,620	27,142,830	28,609,130	27,773,340
Iowa Approach				
Roadway Items	259,000	259,000	259,000	259,000
Structural Demolition	397,320	397,320	397,320	397,320
Reconstruction	1,924,360	1,924,360	1,924,360	1,924,360
Subtotal	2,580,680	2,580,680	2,580,680	2,580,680
Illinois Approach				
Roadway Items	2,085,100	2,085,100	2,085,100	2,085,100
Structural Demolition	122,370	122,370	122,370	122,370
Reconstruction	491,410	491,410	491,410	491,410
Subtotal	2,698,880	2,698,880	2,698,880	2,698,880
Total	$33,258,180	$32,422,390	$33,888,690	$33,052,900
Contingency 15%	4,991,820	4,867,610	5,081,310	4,957,100
GRAND TOTAL	$38,250,000	$37,290,000	$38,970,000	$38,010,000

TABLE III

ESTIMATED CONSTRUCTION COST SUMMARY
SOUTH ALIGNMENT

Main Spans	660-Ft. Cable-Stayed Concrete Girder		660-Ft. Cable-Stayed Steel Girder	
Approach Spans	Multiple Steel Plate Girders	Multiple Prestressed Concrete Girders	Multiple Steel Plate Girders	Multiple Prestressed Concrete Girders
River Bridge				
Structure Demolition	$ 1,000,000	$ 1,000,000	$ 1,000,000	$ 1,000,000
Main Span	14,977,330	14,977,330	15,778,670	15,778,670
Approaches	9,777,180	9,993,190	9,777,180	9,993,190
Embankment & Riprap	71,300	71,300	71,300	71,300
Lighting	240,530	240,530	240,530	240,530
Subtotal	26,066,340	26,282,350	26,867,680	27,083,690
Iowa Approach				
Roadway Items	259,000	259,000	259,000	259,000
Structural Demolition	397,320	397,320	397,320	397,320
Reconstruction	1,924,360	1,924,360	1,924,360	1,924,360
Subtotal	2,580,680	2,580,680	2,580,680	2,580,680
Illinois Approach				
Roadway Items	2,085,100	2,085,100	2,085,100	2,085,100
Structural Demolition	122,370	122,370	122,370	122,370
Reconstruction	491,410	491,410	491,410	491,410
Subtotal	2,698,880	2,698,880	2,698,880	2,698,880
Total	$31,345,900	$31,561,910	$32,147,240	$32,363,250
Contingency 15%	4,704,100	4,738,090	4,822,760	4,856,750
GRAND TOTAL	$36,050,000	$36,300,000	$36,970,000	$37,220,000

TABLE IV

ESTIMATED CONSTRUCTION COST SUMMARY
NORTH ALIGNMENT

Main Spans	515-Ft. Simple-Span Through Truss		515-Ft. Simple-Span Tied Arch	
Approach Spans	Multiple Steel Plate Girders	Multiple Prestressed Concrete Girders	Multiple Steel Plate Girders	Multiple Prestressed Concrete Girders
River Bridge				
Structure Demolition	$ 800,000	$ 800,000	$ 800,000	$ 800,000
Main Span	9,068,580	9,068,580	9,679,280	9,679,280
Approaches	12,989,220	12,912,490	12,989,220	12,912,490
Embankment & Riprap	56,800	56,800	56,800	56,800
Lighting	240,530	240,530	240,530	240,530
Subtotal	23,155,130	23,078,400	23,765,830	23,689,100
Iowa Approach				
Roadway Items	327,300	327,300	327,300	327,300
Structural Demolition	973,650	973,650	973,650	973,650
Reconstruction	2,769,640	2,769,640	2,769,640	2,769,640
Subtotal	4,070,590	4,070,590	4,070,590	4,070,590
Illinois Approach				
Roadway Items	2,229,000	2,229,000	2,229,000	2,229,000
Structural Demolition	127,890	127,890	127,890	127,890
Reconstruction	500,700	500,700	500,700	500,700
Subtotal	2,857,590	2,857,590	2,857,590	2,857,590
Total	$30,083,310	$30,006,580	$30,694,010	$30,617,280
Contingency 15%	4,506,690	4,503,420	4,605,990	4,592,720
GRAND TOTAL	$34,590,000	$34,510,000	$35,300,000	$35,210,000

TABLE V

ESTIMATED CONSTRUCTION COST SUMMARY
NORTH ALIGNMENT

Main Spans	515-Ft. Cable-Stayed Concrete Girder		515-Ft. Cable-Stayed Steel Girder	
Approach Spans	Multiple Steel Plate Girders	Multiple Prestressed Concrete Girders	Multiple Steel Plate Girders	Multiple Prestressed Concrete Girders
River Bridge				
Structure Demolition	$ 800,000	$ 800,000	$ 800,000	$ 800,000
Main Span	11,438,580	11,438,580	11,838,050	11,838,050
Approaches	10,607,680	10,787,710	10,607,680	10,787,710
Embankment & Riprap	56,800	56,800	56,800	56,800
Lighting	240,530	240,530	240,530	240,530
Subtotal	23,143,590	23,323,620	23,543,060	23,723,090
Iowa Approach				
Roadway Items	327,300	327,300	327,300	327,300
Structural Demolition	973,650	973,650	973,650	973,650
Reconstruction	2,769,640	2,769,640	2,769,640	2,769,640
Subtotal	4,070,590	4,070,590	4,070,590	4,070,590
Illinois Approach				
Roadway Items	2,243,300	2,243,300	2,243,300	2,243,300
Structural Demolition	127,890	127,890	127,890	127,890
Reconstruction	500,700	500,700	500,700	500,700
Subtotal	2,871,890	2,871,890	2,871,890	2,871,890
Total	$30,086,070	$30,266,100	$30,485,540	$30,665,570
Contingency 15%	4,513,930	4,543,900	4,574,460	4,604,430
GRAND TOTAL	$34,600,000	$34,810,000	$35,060,000	$35,270,000

TABLE VI

COST ESTIMATE
480-FT. NAVIGATION SPAN
SIMPLE-SPAN THROUGH TRUSS

Item	Unit	Quantity	Unit Price	Amount
SUPERSTRUCTURE				
Structural Steel				
Carbon	lb	1,673,320	$ 1.08	$ 1,807,190
Low Alloy	lb	1,731,210	1.15	1,990,890
Concrete - Deck	cy	744	300.00	223,200
Concrete - Barriers	cy	145	280.00	40,600
Reinf. Steel (Epoxy Coated)				
Deck	lb	262,080	0.50	131,040
Barriers	lb	27,070	0.50	13,540
Expansion Devices	lf	156	425.00	66,300
Concrete Overlay	sy	4,000	35.00	140,000
Bearings & Miscellaneous	ls	---	---	80,000
			Subtotal	$ 4,492,760
SUBSTRUCTURE				
Excavation (Rock Scour)	cy	1,613	$ 40.00	$ 64,520
Concrete				
Precast Foundation Shell	cy	1,180	275.00	324,500
Foundation	cy	3,191	130.00	414,830
Column and Cap	cy	1,169	250.00	292,250
Reinforcing Steel	lb	539,700	0.35	188,900
Drilled Caissons - 4.5'	lf	2,600	1,100.00	2,860,000
Tower Rig, Jacks, etc.	ls	---	---	600,000
			Subtotal	$ 4,745,000
		TOTAL ESTIMATED COST		$ 9,237,760

Cost of Superstructure Without Concrete Overlay: Superstructure Subtotal (minus Deck Concrete, Reinf. Steel and Concrete Overlay)				$ 3,998,520
Concrete Deck	cy	966	300.00	289,800
Reinf. Steel (Epoxy Coated)	lb	243,610	0.50	121,800
				$ 4,410,120

TABLE VII

COST ESTIMATE
480-FT. NAVIGATION SPAN
SIMPLE-SPAN TIED ARCH

Item	Unit	Quantity	Unit Price	Amount
SUPERSTRUCTURE				
Structural Steel				
Carbon	lb	1,284,200	$ 1.20	$ 1,541,040
Low Alloy	lb	1,990,380	1.35	2,687,010
Bridge Strands (Hangers)	lb	48,670	2.30	111,940
Concrete - Deck	cy	744	300.00	223,200
Concrete - Barriers	cy	145	280.00	40,600
Reinf. Steel (Epoxy Coated)				
Deck	lb	262,080	0.50	131,040
Barrier	lb	27,070	0.50	13,540
Expansion Devices	lf	156	425.00	66,300
Bearings & Miscellaneous	ls	---	---	80,000
Concrete Overlay	sy	4,000	35.00	140,000
			Subtotal	$ 5,034,670
SUBSTRUCTURE				
Excavation (Rock Scour)	cy	1,613	$ 40.00	$ 64,520
Concrete				
Precast Foundation Shell	cy	1,180	275.00	324,500
Foundation	cy	3,191	130.00	414,830
Column and Cap	cy	1,169	250.00	292,250
Reinforcing Steel	lb	539,700	0.35	188,900
Drilled Caissons - 4.5'	lf	2,600	1,100.00	2,860,000
Tower Rig, Jacks, etc.	ls	---	---	600,000
			Subtotal	$ 4,745,000
		TOTAL ESTIMATED COST		$ 9,779,670
Cost of Superstructure Without Concrete Overlay: Superstructure Subtotal (minus Deck Concrete, Reinf. Steel, and Concrete Overlay)				$ 4,540,430
Concrete Deck	cy	966	300.00	289,800
Reinf. Steel (Epoxy Coated)	lb	243,610	0.50	121,800
				$ 4,952,030

TABLE VIII

COST ESTIMATE
480-FT. NAVIGATION SPAN
CABLE-STAYED CONCRETE GIRDER

Item	Unit	Quantity	Unit Price	Amount
SUPERSTRUCTURE				
Concrete				
Deck	cy	1,455	$ 300.00	$ 436,500
Edge Girders	cy	1,558	350.00	545,300
Barriers	cy	222	280.00	62,160
Tower	cy	908	300.00	272,400
Prestressed Concrete Girders AASHTO Type III (Modified)	lf	4,462	110.00	490,820
Reinforcing Steel	lb	1,177,230	0.35	412,030
Reinf. Steel (Epoxy Coated)				
Deck	lb	510,200	0.50	255,100
Barriers	lb	41,440	0.50	20,720
Prestressing Steel-Strand	lb	158,150	1.75	276,760
-Bars	lb	97,610	1.50	146,420
Cables	lb	441,000	3.35	1,477,350
Expansion Devices	lf	156	425.00	66,300
Bearings & Miscellaneous	ls	---	---	150,000
Concrete Overlay	sy	6,125	35.00	214,380
			Subtotal	$ 4,826,240

Note: Superstructure costs include $100,000 for wind tunnel and cable anchorage tests.

Item	Unit	Quantity	Unit Price	Amount
SUBSTRUCTURE				
Excavation (Rock Scour)	cy	2,643	$ 40.00	$ 105,720
Concrete				
Precast Foundation Shell	cy	1,823	275.00	501,330
Foundation	cy	5,440	130.00	707,200
Column & Cap	cy	1,091	250.00	272,750
Reinforcing Steel	lb	822,900	0.35	288,020
Drilled Caissons - 4.5'	lf	2,400	1,100.00	2,640,000
Drilled Caissons - 6.5'	lf	1,980	1,600.00	3,168,000
Tower Rig, Jacks, etc.	ls	---	---	750,000
			Subtotal	$ 8,433,020
			TOTAL ESTIMATED COST	$13,259,260

Cost of Superstructure Without Concrete Overlay:
Superstructure Subtotal (minus Deck Concrete, Reinf. Steel, and Concrete Overlay) $ 3,920,260

Item	Unit	Quantity	Unit Price	Amount
Concrete Deck	cy	1,796	300.00	538,800
Reinf. Steel (Epoxy Coated)	lb	483,250	0.50	241,620
				$ 4,700,680

TABLE IX

COST ESTIMATE
480-FT. NAVIGATION SPAN
CABLE-STAYED STEEL GIRDER

Item	Unit	Quantity	Unit Price	Amount
SUPERSTRUCTURE				
Concrete				
Deck	cy	1,935	$ 300.00	$ 580,500
Barriers	cy	222	280.00	62,160
Tower	cy	1,172	300.00	351,600
Reinforcing Steel	lb	527,310	0.35	184,560
Rein. Steel (Epoxy Coated)				
Deck	lb	230,710	0.50	115,360
Barriers	lb	41,440	0.50	20,720
Structural Steel				
Low Alloy	lb	2,566,120	0.96	2,463,480
Cable Conn. - Low Alloy	lb	75,000	1.25	93,750
Carbon	lb	50,000	2.00	100,000
Prestressing Steel - Strand	lb	103,230	1.75	180,650
Cables	lb	263,110	3.35	881,420
Expansion Devices	lf	156	425.00	66,300
Bearings & Miscellaneous	ls	---	---	150,000
Concrete Overlay	sy	6,125	35.00	214,380
			Subtotal	$ 5,464,880

Note: Superstructure costs include $100,000 for wind tunnel and cable anchorage tests.

Item	Unit	Quantity	Unit Price	Amount
SUBSTRUCTURE				
Excavation (Rock Scour)	cy	2,643	$ 40.00	$ 105,720
Concrete				
Precast Foundation Shell	cy	1,823	275.00	501,330
Foundation	cy	5,440	130.00	707,200
Column & Cap	cy	1,062	250.00	265,500
Reinforcing Steel	lb	820,400	0.35	287,140
Drilled Caissons - 4.5'	lf	2,400	1,100.00	2,640,000
Drilled Caissons - 6.5'	lf	1,980	1,600.00	3,168,000
Tower Rig, Jacks, etc.	ls	---	---	750,000
			Subtotal	$ 8,424,890
			TOTAL ESTIMATED COST	$13,889,770

Cost of Superstructure Without Concrete Overlay:

Superstructure Subtotal (minus Deck Concrete and Concrete Overlay)				$ 4,670,000
Concrete Deck	cy	2,275	300.00	682,500
				$ 5,352,500

TABLE X

COST ESTIMATE
660-FT. NAVIGATION SPAN
CABLE-STAYED CONCRETE GIRDER

Item	Unit	Quantity	Unit Price	Amount
SUPERSTRUCTURE				
Concrete				
Deck	cy	2,120	$ 300.00	$ 636,000
Edge Girders	cy	2,251	350.00	787,850
Barriers	cy	321	280.00	89,880
Tower	cy	1,476	300.00	442,880
Prestressed Concrete Girders AASHTO Type III (Modified)	lf	6,609	110.00	726,990
Reinforcing Steel	lb	1,847,330	0.35	646,570
Reinf. Steel (Epoxy Coated)				
Deck	lb	740,300	0.50	370,150
Barriers	lb	60,200	0.50	30,100
Prestressing Steel-Strand	lb	193,800	1.75	339,150
-Bars	lb	100,000	1.50	150,000
Cables	lb	856,000	3.35	2,867,600
Expansion Devices	lf	166	425.00	70,550
Bearings & Miscellaneous	ls	---	---	200,000
Concrete Overlay	sy	8,925	35.00	312,380
			Subtotal	$ 7,670,100

Note: Superstructure costs include $100,000 for wind tunnel and cable anchorage tests.

Item	Unit	Quantity	Unit Price	Amount
SUBSTRUCTURE				
Cofferdam	ea	3	$400,000.00	$ 1,200,000
Excavation	cy	15,970	20.00	319,400
Concrete				
Tremie Seal	cy	7,130	130.00	926,900
Footing	cy	2,735	130.00	355,550
Shaft, Column & Cap	cy	8,435	275.00	2,319,630
Reinforcing Steel	lb	725,000	0.35	253,750
Steel Piles (HP 14 x 89)	lf	46,000	42.00	1,932,000
			Subtotal	$ 7,307,230
			TOTAL ESTIMATED COST	$14,977,330

Cost of Superstructure Without Concrete Overlay:				
Superstructure Subtotal (minus Deck Concrete, Reinf. Steel and Concrete Overlay)				$ 6,351,570
Concrete Deck	cy	2,616	300.00	784,800
Reinf. Steel (Epoxy Coated)	lb	704,000	0.50	352,000
				$ 7,488,370

TABLE XI

COST ESTIMATE
660-FT. NAVIGATION SPAN
CABLE-STAYED STEEL GIRDER

Item	Unit	Quantity	Unit Price	Amount
SUPERSTRUCTURE				
Concrete				
Deck	cy	2,834	$ 300.00	$ 850,200
Barriers	cy	321	280.00	89,880
Tower	cy	1,723	300.00	516,900
Reinforcing Steel	lb	947,600	0.35	331,660
Reinf. Steel (Epoxy Coated)				
Deck	lb	337,600	0.50	168,800
Barriers	lb	60,000	0.50	30,000
Structural Steel				
Low Alloy	lb	3,782,000	0.96	3,630,720
Cable Conn. - Low Alloy	lb	105,000	1.25	131,250
Carbon	lb	75,000	2.00	150,000
Prestressing Steel - Strand	lb	150,000	1.75	262,500
Cables	lb	643,000	3.35	2,154,050
Expansion Devices	lf	166	425.00	70,550
Bearings & Miscellaneous	ls	---	---	200,000
Concrete Overlay	sy	8,925	35.00	312,380
			Subtotal	$ 8,898,890

Note: Superstructure costs include $100,000 for wind tunnel and cable anchorage tests.

Item	Unit	Quantity	Unit Price	Amount
SUBSTRUCTURE				
Cofferdam	ea	3	$390,000.00	$ 1,170,000
Excavation	cy	15,430	20.00	308,600
Concrete				
Tremie Seal	cy	7,080	130.00	920,400
Footing	cy	2,425	130.00	315,250
Shaft, Column & Cap	cy	7,595	275.00	2,088,630
Reinforcing Steel	lb	714,000	0.35	249,900
Steel Pile (HP 14 x 89)	lf	43,500	42.00	1,827,000
			Subtotal	$ 6,879,780
		TOTAL ESTIMATED COST		$15,778,670
Cost of Superstructure Without Concrete Overlay: Superstructure Subtotal (minus Deck Concrete and Concrete Overlay)				$ 7,736,310
Concrete Deck	cy	3,314	300.00	994,200
				$ 8,730,510

TABLE XII

COST ESTIMATE
515-FT. NAVIGATION SPAN
SIMPLE-SPAN THROUGH TRUSS

Item	Unit	Quantity	Unit Price	Amount
SUPERSTRUCTURE				
Structural Steel				
Carbon	lb	1,965,010	$ 1.08	$ 2,122,210
Low Alloy	lb	1,956,660	1.15	2,250,160
Concrete - Deck	cy	798	300.00	239,400
Concrete - Barriers	cy	155	280.00	43,400
Rein. Steel (Epoxy Coated)				
Deck	lb	281,210	0.50	140,610
Barriers	lb	28,990	0.50	14,490
Expansion Devices	lf	156	425.00	66,300
Concrete Overlay	sy	4,292	35.00	150,220
Bearings & Miscellaneous	ls	---	---	85,000
			Subtotal	$ 5,111,790
SUBSTRUCTURE				
Cofferdam	ea	2	$350,000.00	$ 700,000
Excavation	cy	10,430	25.00	260,750
Concrete				
Tremie Seal	cy	5,230	130.00	679,900
Footing	cy	1,520	130.00	197,600
Shaft, Column and Cap	cy	3,880	275.00	1,067,000
Reinforcing Steel	lb	450,800	0.35	157,780
Steel Piles (HP 14 x 89)	lf	21,280	42.00	893,760
			Subtotal	$ 3,956,790
		TOTAL ESTIMATED COST		$ 9,068,580

Cost of Superstructure Without Concrete Overlay:
Superstructure Subtotal (minus Deck Concrete, Reinf. Steel, and Concrete Overlay) $ 4,581,560

Concrete Deck	cy	1,036	300.00	310,800
Rein. Steel (Epoxy Coated)	lb	261,390	0.50	130,700
				$ 5,023,060

TABLE XIII

COST ESTIMATE
515-FT. NAVIGATION SPAN
SIMPLE-SPAN TIED ARCH

Item	Unit	Quantity	Price	Amount
SUPERSTRUCTURE				
Structural Steel				
Carbon	lb	1,523,920	$ 1.20	$ 1,828,700
Low Alloy	lb	2,245,710	1.35	3,031,710
Bridge Strands (Hangers)	lb	53,330	2.30	122,660
Concrete - Deck	cy	798	300.00	239,400
Concrete - Barriers	cy	155	280.00	43,400
Reinf. Steel (Epoxy Coated)				
Deck	lb	281,210	0.50	140,610
Barrier	lb	28,990	0.50	14,490
Expansion Devices	lf	156	425.00	66,300
Bearings & Miscellaneous	ls	---	---	85,000
Concrete Overlay	sy	4,292	35.00	150,220
			Subtotal	$ 5,722,490
SUBSTRUCTURE				
Cofferdam	ea	2	$350,000.00	$ 700,000
Excavation	cy	10,430	25.00	260,750
Concrete				
Tremie Seal	cy	5,230	130.00	679,900
Footing	cy	1,520	130.00	197,600
Shaft, Column and Cap	cy	3,880	275.00	1,067,000
Reinforcing Steel	lb	450,800	0.35	157,780
Steel Piles (HP 14 x 89)	lf	21,280	42.00	893,760
			Subtotal	$ 3,956,790
		TOTAL ESTIMATED COST		$ 9,679,280

Cost of Superstructure Without Concrete Overlay: Superstructure Subtotal (minus Deck Concrete, Reinf. Steel, and Concrete Overlay)				$ 5,192,260
Concrete Deck	cy	1,036	300.00	310,800
Reinf. Steel (Epoxy Coated)	lb	261,390	0.50	130,700
				$ 5,633,760

TABLE XIV

COST ESTIMATE
515-FT. NAVIGATION SPAN
CABLE-STAYED CONCRETE GIRDER

Item	Unit	Quantity	Unit Price	Amount
SUPERSTRUCTURE				
Concrete				
Deck	cy	1,579	$ 300.00	$ 473,700
Edge Girders	cy	1,683	350.00	589,050
Barriers	cy	241	280.00	67,480
Tower	cy	1,059	300.00	317,700
Prestressed Concrete Girders AASHTO Type III (Modified)	lf	4,730	110.00	520,300
Reinforcing Steel	lb	1,319,290	0.35	461,750
Reinf. Steel (Epoxy Coated)				
Deck	lb	551,080	0.50	275,540
Barriers	lb	45,000	0.50	22,500
Prestressing Steel-Strand	lb	169,660	1.75	296,910
-Bars	lb	97,610	1.50	146,420
Cables	lb	501,160	3.35	1,678,890
Expansion Devices	lf	156	425.00	66,300
Bearings & Miscellaneous	ls	---	---	160,000
Concrete Overlay	sy	6,642	35.00	232,470
			Subtotal	$ 5,309,010

Note: Superstructure costs include $100,000 for wind tunnel and cable anchorage tests.

Item	Unit	Quantity	Unit Price	Amount
SUBSTRUCTURE				
Cofferdam	ea	3	$350,000.00	$ 1,050,000
Excavation (Rock Scour)	cy	13,510	25.00	337,750
Concrete				
Tremie Seal	cy	5,600	130.00	728,000
Footing	cy	2,371	130.00	308,230
Shaft, Column & Cap	cy	7,219	275.00	1,985,230
Reinforcing Steel	lb	619,300	0.35	216,760
Steel Piles	lf	35,800	42.00	1,503,600
			Subtotal	$ 6,129,570
			TOTAL ESTIMATED COST	$11,438,580

Item	Unit	Quantity	Unit Price	Amount
Cost of Superstructure Without Concrete Overlay: Superstructure Subtotal (minus Deck Concrete, Reinf. Steel, and Concrete Overlay)				$ 4,327,300
Concrete Deck	cy	1,948	300.00	584,400
Reinf. Steel (Epoxy Coated)	lb	524,010	0.50	262,000
				$ 5,173,700

TABLE XV

COST ESTIMATE
515-FT. NAVIGATION SPAN
CABLE-STAYED STEEL GIRDER

Item	Unit	Quantity	Unit Price	Amount
SUPERSTRUCTURE				
Concrete				
Deck	cy	2,098	$ 300.00	$ 629,400
Barriers	cy	241	280.00	67,480
Tower	cy	1,432	300.00	429,600
Reinforcing Steel	lb	660,520	0.35	231,180
Reinf. Steel (Epoxy Coated)				
Deck	lb	250,170	0.50	125,090
Barriers	lb	45,000	0.50	22,500
Structural Steel				
Low Alloy	lb	2,644,000	0.96	2,538,240
Cable Conn. - Low Alloy	lb	90,000	1.25	112,500
Carbon	lb	62,000	2.00	124,000
Prestressing Steel - Strand	lb	111,940	1.75	195,900
Cables	lb	383,440	3.35	1,284,520
Expansion Devices	lf	156	425.00	66,300
Bearings & Miscellaneous	ls	---	---	160,000
Concrete Overlay	sy	6,642	35.00	232,470
			Subtotal	$ 6,219,180

Note: Superstructure costs include $100,000 for wind tunnel and cable anchorage tests.

Item	Unit	Quantity	Unit Price	Amount
SUBSTRUCTURE				
Cofferdam	ea	3	$350,000.00	$ 1,050,000
Excavation	cy	12,688	25.00	317,200
Concrete				
Tremie Seal	cy	5,719	130.00	743,470
Footing	cy	1,836	130.00	238,680
Shaft, Column & Cap	cy	6,084	275.00	1,673,100
Reinforcing Steel	lb	577,200	0.35	202,020
Steel Piles (HP 14 x 89)	lf	33,200	42.00	1,394,400
			Subtotal	$ 5,618,870
		TOTAL ESTIMATED COST		$11,838,050

Cost of Superstructure Without Concrete Overlay:				
Superstructure Subtotal (minus Deck Concrete and Concrete Overlay)				$ 5,357,310
Concrete Deck	cy	2,467	300.00	740,100
				$ 6,097,410

TABLE XVI

COST ESTIMATE
APPROACH SPANS-SOUTH ALIGNMENT
MULTIPLE PLATE GIRDERS
(for 480-Ft. Navigation Span Through Truss or Tied Arch)

Item	Unit	Quantity	Unit Price	Amount
SUPERSTRUCTURE				
Structural Steel				
Carbon	lb	3,400,700	$ 0.80	$ 2,720,560
Low Alloy	lb	2,847,100	0.85	2,420,040
Concrete - Deck	cy	3,075	300.00	922,500
Concrete - Barriers	cy	539	280.00	150,920
Reinf. Steel (Epoxy Coated)				
Deck	lb	1,101,200	0.50	550,600
Barriers	lb	100,800	0.50	50,400
Expansion Devices	lf	342	375.00	128,250
Bearings & Misc.	ls	---	---	75,000
Concrete Overlay	sy	16,975	35.00	594,130
			Subtotal	$ 7,612,400
SUBSTRUCTURE				
Excavation	cy	15,730	$ 25.00	$ 393,250
Cofferdams	ea	8	195,000.00	1,560,000
Steel H-Piles				
HP 14 X 89	lf	45,300	42.00	1,902,600
HP 10 X 42	lf	1,600	25.00	40,000
Concrete				
Seal	cy	7,396	130.00	961,480
Footings & Columns	cy	8,985	250.00	2,246,250
Reinforcing Steel	lb	756,400	0.35	264,740
			Subtotal	$ 7,368,320
		TOTAL ESTIMATED COST		$14,980,720

Cost of Superstructure Without Concrete Overlay: Substructure Subtotal (minus Deck Concrete, Reinf. Steel, and Concrete Overlay)				$ 5,545,170
Concrete Deck	cy	4,075	300.00	1,222,500
Reinf. Steel (Epoxy Coated)	lb	1,023,600	0.50	511,800
				$ 7,279,470

TABLE XVII

COST ESTIMATE
APPROACH SPANS-SOUTH ALIGNMENT
MULTIPLE PRESTRESSED CONCRETE GIRDERS
(for 480-Ft. Navigation Span Through Truss or Tied Arch)

Item	Unit	Quantity	Unit Price	Amount
SUPERSTRUCTURE				
Concrete - Deck	cy	4,485	$ 300.00	$ 1,345,500
Concrete - Barriers	cy	539	280.00	150,920
Reinf. Steel (Epoxy Coated)				
Deck	lb	1,147,100	0.50	573,550
Barriers	lb	100,800	0.50	50,400
Prestressed Concrete Beams	lf	22,117	130.00	2,875,210
Expansion Devices	lf	431	375.00	161,630
Bearings & Misc.	ls	---	---	25,000
Concrete Overlay	sy	16,975	35.00	594,130
			Subtotal	$ 5,776,340
SUBSTRUCTURE				
Excavation	cy	21,110	$ 25.00	$ 527,750
Cofferdams	ea	11	170,000.00	1,870,000
Steel H-Piles				
HP 14 X 89	lf	60,400	42.00	2,536,800
HP 10 X 42	lf	1,600	25.00	40,000
Concrete				
Seal	cy	9,941	130.00	1,292,330
Footings & Columns	cy	10,921	250.00	2,730,250
Reinforcing Steel	lb	920,900	0.35	322,320
			Subtotal	$ 9,319,450
		TOTAL ESTIMATED COST		$15,095,790

Cost of Superstructure Without Concrete Overlay: Superstructure Subtotal (minus Deck Concrete, Reinf. Steel, and Concrete Overlay)				$ 3,263,160
Concrete Deck	cy	4,585	300.00	1,375,500
Reinf. Steel (Epoxy Coated)	lb	1,197,700	0.50	598,850
				$ 5,237,510

TABLE XVIII

COST ESTIMATE
APPROACH SPANS-SOUTH ALIGNMENT
MULTIPLE PLATE GIRDERS
(for 480-Ft. Navigation Span Cable-Stayed Steel or Concrete Girder)

Item	Unit	Quantity	Unit Price	Amount
SUPERSTRUCTURE				
Structural Steel				
Carbon	lb	2,948,000	$ 0.80	$ 2,358,400
Low Alloy	lb	2,476,300	0.85	2,104,860
Concrete - Deck	cy	2,68[illegible]	300.00	805,800
Concrete - Barri[illegible]	cy	46[illegible]	280.00	129,360
Reinf. Steel (Ep[illegible]oated)				
Deck	lb	961,900	0.50	480,950
Barriers	lb	86,400	0.50	43,200
Expansion Devices	lf	431	375.00	161,630
Bearings & Misc.	ls	---	---	75,000
Concrete Overlay	sy	14,846	35.00	519,610
			Subtotal	$ 6,678,810
SUBSTRUCTURE				
Excavation	cy	14,090	$ 25.00	$ 352,250
Cofferdams	ea	8	195,000.00	1,560,000
Steel H-Piles				
HP 14 X 89	lf	40,000	42.00	1,680,000
HP 10 X 42	lf	1,600	25.00	40,000
Concrete				
Seal	cy	6,697	130.00	870,610
Footings & Columns	cy	7,966	250.00	1,991,500
Reinforcing Steel	lb	669,600	0.35	234,360
			Subtotal	$ 6,728,720
		TOTAL ESTIMATED COST		$13,407,530

Cost of Superstructure Without Concrete Overlay:				
Superstructure Subtotal (minus Deck Concrete, Reinf. Steel, and Concrete Overlay)				$ 4,872,450
Concrete Deck	cy	3,560	300.00	1,068,000
Reinf. Steel (Epoxy Coated)	lb	894,300	0.50	447,150
				$ 6,387,600

TABLE XIX

COST ESTIMATE
APPROACH SPANS-SOUTH ALIGNMENT
MULTIPLE PRESTRESSED CONCRETE GIRDERS
(for 480-Ft. Navigation Span Cable-Stayed Steel or Concrete Girder)

Item	Unit	Quantity	Unit Price	Amount
SUPERSTRUCTURE				
Concrete - Deck	cy	3,103	$ 300.00	$ 930,900
Concrete - Barriers	cy	462	280.00	129,360
Reinf. Steel (Epoxy Coated)				
Deck	lb	1,001,800	0.50	500,900
Barriers	lb	86,400	0.50	43,200
Prestressed Concrete Beams	lf	19,022	130.00	2,472,860
Expansion Devices	lf	431	375.00	161,630
Bearings & Misc.	ls	---	---	25,000
Concrete Overlay	sy	14,846	35.00	519,610
			Subtotal	$ 4,783,460
SUBSTRUCTURE				
Excavation	cy	17,830	$ 25.00	$ 445,750
Cofferdams	ea	9	170,000.00	1,530,000
Steel H-Piles				
HP 14 X 89	lf	50,300	42.00	2,112,600
HP 10 X 42	lf	1,600	25.00	40,000
Concrete				
Seal	cy	8,546	130.00	1,110,980
Footings & Columns	cy	9,121	250.00	2,280,250
Reinforcing Steel	lb	767,700	0.35	268,700
			Subtotal	$ 7,788,280
		TOTAL ESTIMATED COST		$12,571,740

Cost of Superstructure Without Concrete Overlay: Superstructure Subtotal (minus Deck Concrete, Reinf. Steel, and Concrete Overlay)				$ 2,832,050
Concrete Deck	cy	4,004	300.00	1,201,200
Reinf. Steel (Epoxy Coated)	lb	1,045,800	0.50	522,900
				$ 4,556,150

TABLE XX

COST ESTIMATE
APPROACH SPANS-SOUTH ALIGNMENT
MULTIPLE PLATE GIRDERS
(for 660-Ft. Navigation Span Cable-Stayed Steel or Concrete Girder)

Item	Unit	Quantity	Unit Price	Amount
SUPERSTRUCTURE				
Structural Steel				
Carbon	lb	2,174,000	$ 0.80	$ 1,739,200
Low Alloy	lb	1,909,000	0.85	1,622,650
Concrete - Deck	cy	2,183	300.00	654,900
Concrete - Barriers	cy	362	280.00	101,360
Reinf. Steel (Epoxy Coated)				
Deck	lb	781,700	0.50	390,850
Barriers	lb	67,800	0.50	33,900
Expansion Devices	lf	332	375.00	124,500
Bearings & Misc.	ls	---	---	75,000
Concrete Overlay	sy	12,090	35.00	423,150
			Subtotal	$ 5,165,510
SUBSTRUCTURE				
Excavation	cy	9,860	$ 25.00	$ 246,500
Cofferdams	ea	5	195,000.00	975,000
Steel H-Piles				
HP 14 X 89	lf	28,800	42.00	1,209,600
HP 10 X 42	lf	1,600	25.00	40,000
Concrete				
Seal	cy	4,437	130.00	576,810
Footings & Columns	cy	5,599	250.00	1,399,750
Reinforcing Steel	lb	468,600	0.35	164,010
			Subtotal	$ 4,611,670
		TOTAL ESTIMATED COST		$ 9,777,180

Cost of Superstructure Without Concrete Overlay: Superstructure Subtotal (minus Deck Concrete, Reinf. Steel, and Concrete Overlay)				$ 3,696,610
Concrete Deck	cy	2,893	300.00	867,900
Reinf. Steel (Epoxy Coated)	lb	726,700	0.50	363,350
				$ 4,927,860

TABLE XXI

COST ESTIMATE
APPROACH SPANS-SOUTH ALIGNMENT
MULTIPLE PRESTRESSED CONCRETE GIRDERS
(for 660-Ft. Navigation Span Cable-Stayed Steel or Concrete Girder)

Item	Unit	Quantity	Unit Price	Amount
SUPERSTRUCTURE				
Concrete - Deck	cy	2,577	$ 300.00	$ 773,100
Concrete - Barriers	cy	362	280.00	101,360
Reinf. Steel (Epoxy Coated)				
Deck	lb	817,200	0.50	408,600
Barriers	lb	67,800	0.50	33,900
Prestressed Concrete Beams	lf	13,390	130.00	1,740,700
Expansion Devices	lf	421	375.00	157,880
Bearings & Misc.	ls	---	---	25,000
Concrete Overlay	sy	12,090	35.00	423,150
			Subtotal	$ 3,663,690
SUBSTRUCTURE				
Excavation	cy	14,580	$ 25.00	$ 364,500
Cofferdams	ea	7	170,000.00	1,190,000
Steel H-Piles				
HP 14 X 89	lf	41,700	42.00	1,751,400
HP 10 X 42	lf	1,600	25.00	40,000
Concrete				
Seal	cy	6,814	130.00	885,820
Footings & Columns	cy	7,508	250.00	1,877,000
Reinforcing Steel	lb	630,800	0.35	220,780
			Subtotal	$ 6,329,500
		TOTAL ESTIMATED COST		$ 9,993,190

Cost of Superstructure Without Concrete Overlay: Superstructure Subtotal (minus Deck Concrete, Reinf. Steel, and Concrete Overlay)				$ 2,058,840
Concrete Deck	cy	3,287	300.00	986,100
Reinf. Steel (Epoxy Coated)	lb	762,200	0.50	381,100
				$ 3,426,040

TABLE XXII

COST ESTIMATE
APPROACH SPANS-NORTH ALIGNMENT
MULTIPLE PLATE GIRDERS
(for 515-Ft. Navigation Span Through Truss or Tied Arch)

Item	Unit	Quantity	Unit Price	Amount
SUPERSTRUCTURE				
Structural Steel				
Carbon	lb	3,258,000	$ 0.80	$ 2,606,400
Low Alloy	lb	3,258,000	0.85	2,769,300
Concrete - Deck	cy	2,824	300.00	847,200
Concrete - Barriers	cy	507	280.00	141,960
Reinf. Steel (Epoxy Coated)				
Deck	lb	1,011,300	0.50	505,650
Barriers	lb	94,800	0.50	47,400
Expansion Devices	lf	319	375.00	119,630
Bearings & Misc.	ls	---	---	75,000
Concrete Overlay	sy	15,564	35.00	544,740
			Subtotal	$ 7,657,280
SUBSTRUCTURE				
Excavation	cy	11,280	$ 25.00	$ 282,000
Cofferdams	ea	6	195,000.00	1,170,000
Steel H-Piles				
HP 14 X 89	lf	32,500	42.00	1,365,000
HP 10 X 42	lf	1,600	25.00	40,000
Concrete				
Seal	cy	5,202	130.00	676,260
Footings & Columns	cy	6,439	250.00	1,609,750
Reinforcing Steel	lb	539,800	0.35	188,930
			Subtotal	$ 5,331,940
		TOTAL ESTIMATED COST		$12,989,220

Cost of Superstructure Without Concrete Overlay: Superstructure Subtotal (minus Deck Concrete, Reinf. Steel, and Concrete Overlay)				$ 5,759,690
Concrete Deck	cy	3,742	300.00	1,122,600
Reinf. Steel (Epoxy Coated)	lb	940,000	0.50	470,000
				$ 7,352,290

TABLE XXIII

COST ESTIMATE
APPROACH SPANS-NORTH ALIGNMENT
MULTIPLE PRESTRESSED CONCRETE GIRDERS
(for 515-Ft. Navigation Span Through Truss or Tied Arch)

Item	Unit	Quantity	Unit Price	Amount
SUPERSTRUCTURE				
Concrete - Deck	cy	3,344	$ 300.00	$ 1,003,200
Concrete - Barriers	cy	507	280.00	141,960
Reinf. Steel (Epoxy Coated)				
Deck	lb	1,058,100	0.50	529,050
Barriers	lb	94,800	0.50	47,400
Prestressed Concrete Beams	lf	17,430	130.00	2,265,900
Expansion Devices	lf	399	375.00	149,630
Bearings & Misc.	ls	---	---	25,000
Concrete Overlay	sy	15,564	35.00	544,740
			Subtotal	$ 4,706,880
SUBSTRUCTURE				
Excavation	cy	18,490	$ 25.00	$ 462,250
Cofferdams	ea	10	170,000.00	1,700,000
Steel H-Piles				
HP 14 X 89	lf	53,600	42.00	2,251,200
HP 10 X 42	lf	1,600	25.00	40,000
Concrete				
Seal	cy	8,483	130.00	1,102,790
Footings & Columns	cy	9,480	250.00	2,370,000
Reinforcing Steel	lb	798,200	0.35	279,370
			Subtotal	$ 8,205,610
		TOTAL ESTIMATED COST		$12,912,490

Cost of Superstructure Without Concrete Overlay: Superstructure Subtotal (minus Deck Concrete, Reinf. Steel, and Concrete Overlay)				$ 2,629,890
Concrete Deck	cy	4,262	300.00	1,278,600
Reinf. Steel (Epoxy Coated)	lb	986,800	0.50	493,400
				$ 4,401,890

TABLE XXIV

COST ESTIMATE
APPROACH SPANS-NORTH ALIGNMENT
MULTIPLE PLATE GIRDERS
(for 515-Ft. Navigation Span Cable-Stayed Steel or Concrete Girder)

Item	Unit	Quantity	Unit Price	Amount
SUPERSTRUCTURE				
Structural Steel				
Carbon	lb	2,879,700	$ 0.80	$ 2,303,760
Low Alloy	lb	2,358,500	0.85	2,004,730
Concrete - Deck	cy	2,394	300.00	718,200
Concrete - Barriers	cy	422	280.00	118,160
Reinf. Steel (Epoxy Coated)				
Deck	lb	857,300	0.50	428,650
Barriers	lb	79,000	0.50	39,500
Expansion Devices	lf	319	375.00	119,630
Bearings & Misc.	ls	---	---	75,000
Concrete Overlay	sy	13,208	35.00	462,280
			Subtotal	$ 6,269,910
SUBSTRUCTURE				
Excavation	cy	9,310	$ 25.00	$ 232,750
Cofferdams	ea	5	195,000.00	975,000
Steel H-Piles				
HP 14 X 89	lf	25,400	42.00	1,066,800
HP 10 X 42	lf	1,600	25.00	40,000
Concrete				
Seal	cy	4,204	130.00	546,520
Footings & Columns	cy	5,288	250.00	1,322,000
Reinforcing Steel	lb	442,000	0.35	154,700
			Subtotal	$ 4,337,770
		TOTAL ESTIMATED COST		$10,607,680

Cost of Superstructure Without Concrete Overlay: Superstructure Subtotal (minus Deck Concrete, Reinf. Steel, and Concrete Overlay)				$ 4,660,780
Concrete Deck	cy	3,172	300.00	951,600
Reinf. Steel (Epoxy Coated)	lb	796,800	0.50	398,400
				$ 6,010,780

TABLE XXV

COST ESTIMATE
APPROACH SPANS-NORTH ALIGNMENT
MULTIPLE PRESTRESSED CONCRETE GIRDERS
(for 515-Ft. Navigation Span Cable-Stayed Steel or Concrete Girder)

Item	Unit	Quantity	Unit Price	Amount
SUPERSTRUCTURE				
Concrete - Deck	cy	2,846	$ 300.00	$ 853,800
Concrete - Barriers	cy	422	280.00	118,160
Reinf. Steel (Epoxy Coated)				
Deck	lb	898,000	0.50	449,000
Barriers	lb	79,000	0.50	39,500
Prestressed Concrete Beams	lf	16,230	130.00	2,109,900
Expansion Devices	lf	319	375.00	119,630
Bearings & Misc.	ls	---	---	25,000
Concrete Overlay	sy	13,208	35.00	462,280
			Subtotal	$ 4,177,270
SUBSTRUCTURE				
Excavation	cy	14,980	$ 25.00	$ 374,500
Cofferdams	ea	8	170,000.00	1,360,000
Steel H-Piles				
HP 14 X 89	lf	43,300	42.00	1,818,600
HP 10 X 42	lf	1,600	25.00	40,000
Concrete				
Seal	cy	6,852	130.00	890,760
Footings & Columns	cy	7,611	250.00	1,902,750
Reinforcing Steel	lb	639,500	0.35	223,830
			Subtotal	$ 6,610,440
		TOTAL ESTIMATED COST		$10,787,710

Cost of Superstructure Without Concrete Overlay: Superstructure Subtotal (minus Deck Concrete, Reinf. Steel, and Concrete Overlay)				$ 2,412,190
Concrete Deck	cy	3,624	300.00	1,087,200
Reinf. Steel (Epoxy Coated)	lb	837,500	0.50	418,750
				$ 3,918,140

TABLE XXVI

COST ESTIMATE
BURLINGTON INTERCHANGE
IOWA APPROACH - SOUTH ALIGNMENT

Item	Unit	Quantity	Price	Amount
STRUCTURAL DEMOLITION				
Concrete Deck	cy	960	$ 150.00	$ 144,000
Structural Steel	lb	434,700	0.50	217,350
Substructure	cy	327	110.00	35,970
			Subtotal	$ 397,320
BRIDGE RECONSTRUCTION				
Concrete				
Deck	cy	1,273	$ 300.00	$ 381,900
Substructure	cy	678	275.00	186,450
Reinforcing Steel	lb	145,750	0.35	51,010
Reinf. Steel (Epoxy Coated)	lb	519,300	0.50	259,650
Structural Steel	lb	916,200	1.00	916,200
Steel H-Piles	lf	3,705	30.00	111,150
Excavation	cy	720	25.00	18,000
			Subtotal	$ 1,924,360
			TOTAL ESTIMATED COST	$ 2,321,680

TABLE XXVII

COST ESTIMATE
BURLINGTON INTERCHANGE
IOWA APPROACH - NORTH ALIGNMENT

Item	Unit	Quantity	Price	Amount
STRUCTURAL DEMOLITION				
Concrete Deck	cy	1,590	$ 150.00	$ 238,500
Structural Steel	lb	1,376,800	0.50	688,400
Substructure	cy	425	110.00	46,750
			Subtotal	$ 973,650
BRIDGE RECONSTRUCTION				
Concrete				
Deck	cy	2,180	$ 300.00	$ 654,000
Substructure	cy	781	275.00	214,780
Reinforcing Steel	lb	156,200	0.35	54,670
Reinf. Steel (Epoxy Coated)	lb	813,210	0.50	406,610
Structural Carbon Steel (Rolled Bms.)	lb	577,050	0.80	461,640
Structural Carbon Steel (Pl. Gdrs.)	lb	579,400	0.90	521,460
Structural Low Alloy Steel (Rolled Bms.)	lb	341,510	0.85	290,280
Steel H-Piles	lf	5,040	30.00	151,200
Excavation	cy	600	25.00	15,000
			Subtotal	$ 2,769,640
			TOTAL ESTIMATED COST	$ 3,743,290

TABLE XXVIII

COST ESTIMATE
CHINN AVENUE BRIDGE
ILLINOIS APPROACH-SOUTH ALIGNMENT

Item	Unit	Quantity	Unit Price	Amount
STRUCTURAL DEMOLITION				
Concrete Deck	cy	200	$ 150.00	$ 30,000
Structural Steel	lb	184,330	0.50	92,170
Substructure	cy	1	200.00	200
			Subtotal	$ 122,370
BRIDGE RECONSTRUCTION				
Concrete				
Deck	cy	355	$ 400.00	$ 142,000
Substructure	cy	183	250.00	45,750
Reinf. Steel	lb	17,700	0.35	6,200
Reinf. Steel (Epoxy Coated)	lb	95,850	0.50	47,930
Struct. Steel (Pl. Gdrs.)	lb	214,500	0.90	193,050
Concrete Piles (12"ø)	lf	1,890	24.00	45,360
Excavation	cy	159	20.00	3,180
Slope Wall (4")	sy	529	15.00	7,940
			Subtotal	$ 491,410
			TOTAL ESTIMATED COST	$ 613,780

TABLE XXIX

COST ESTIMATE
CHINN AVENUE BRIDGE
ILLINOIS APPROACH-NORTH ALIGNMENT

Item	Unit	Quantity	Unit Price	Amount
STRUCTURAL DEMOLITION				
Concrete Deck	cy	200	$ 150.00	$ 30,000
Structural Steel	lb	184,330	0.50	92,170
Substructure	cy	52	110.00	5,720
			Subtotal	$ 127,890
BRIDGE RECONSTRUCTION				
Concrete				
Deck	cy	364	$ 400.00	$ 145,600
Substructure	cy	175	250.00	43,750
Reinf. Steel	lb	19,520	0.35	6,830
Reinf. Steel (Epoxy Coated)	lb	98,280	0.50	49,140
Struct. Steel (Pl. Gdrs.)	lb	222,760	0.90	200,480
Concrete Piles (12"ϕ)	lf	1,824	24.00	43,780
Excavation	cy	159	20.00	3,180
Slope Wall (4")	sy	529	15.00	7,940
			Subtotal	$ 500,700
			TOTAL ESTIMATED COST	$ 628,590

IX CONSTRUCTION SCHEDULE

The construction schedule for the proposed improvement on US 34 over the Mississippi River from the west abutment of the Burlington Interchange in Iowa, to the Burlington Northern railroad overpass in Illinois, is based on the coordination of three phases of work. These are the reconstruction of the Burlington Interchange, the river bridge construction and the Illinois approach roadway and grade separation structure reconstruction. Maintenance of traffic during construction has a definite impact on the schedule.

The construction time required to build the river bridge will dictate the starting point of construction. This structure can be let in two contracts (channel span and approaches) depending on the bridge type or types selected for final design. For the schedule, the truss and multiple girders have been selected for the study, but alternate designs of cable-stayed girders would be essentially the same.

The beginning time for reconstruction in Iowa and Illinois will be that which will make the continued flow of traffic possible from one side to the other when the river bridge is essentially complete.

Figure 24 shows that a total time for construction will be about 3.5 years for the south alignment or 3.65 years for the north alignment.

There are several Iowa spans which conflict with the existing bridge, and these will be completed when the existing bridge is no longer needed. Construction of a portion of the first river spans, Spans 1 and/or 2, near the Iowa shore, are in conflict with the existing bridge and must be deferred until after the service of the existing bridge is no longer needed.

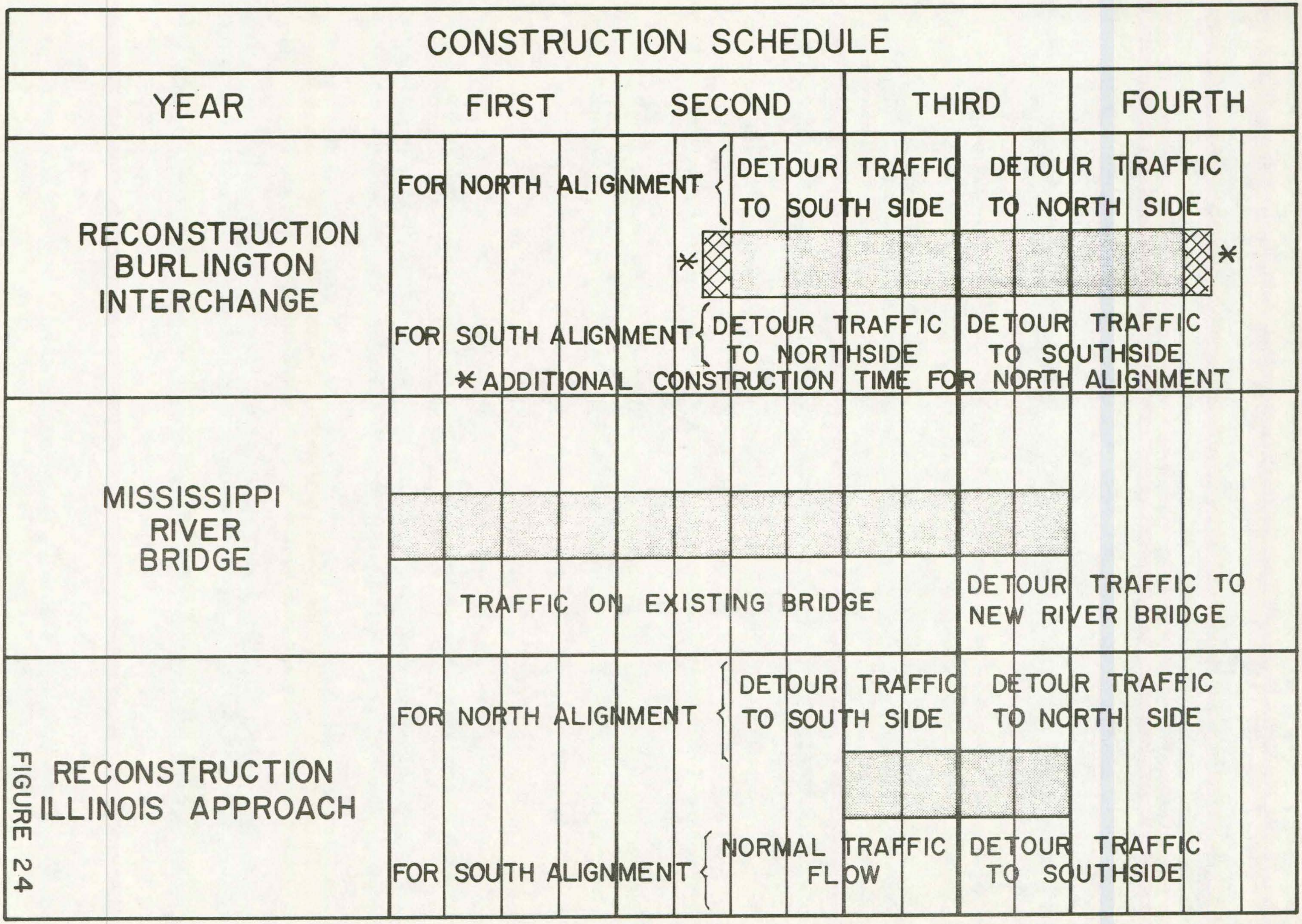

CONSTRUCTION SCHEDULE
YEAR
FIRST
SECOND
THIRD
FOURTH
RECONSTRUCTION BURLINGTON INTERCHANGE
FOR NORTH ALIGNMENT
DETOUR TRAFFIC TO SOUTH SIDE
DETOUR TRAFFIC TO NORTH SIDE
FOR SOUTH ALIGNMENT
DETOUR TRAFFIC TO NORTHSIDE
DETOUR TRAFFIC TO SOUTHSIDE
* ADDITIONAL CONSTRUCTION TIME FOR NORTH ALIGNMENT
MISSISSIPPI RIVER BRIDGE
TRAFFIC ON EXISTING BRIDGE
DETOUR TRAFFIC TO NEW RIVER BRIDGE
RECONSTRUCTION ILLINOIS APPROACH
FOR NORTH ALIGNMENT
DETOUR TRAFFIC TO SOUTH SIDE
DETOUR TRAFFIC TO NORTH SIDE
FOR SOUTH ALIGNMENT
NORMAL TRAFFIC FLOW
DETOUR TRAFFIC TO SOUTHSIDE

FIGURE 24

X CONCLUSIONS

In developing recommendations for the Mississippi River bridge or bridges to be selected for final design, whether on the south alignment or north alignment, available options rest primarily with the river crossing concepts with minor cost impact for the reconstruction in Illinois. The Burlington Interchange reconstruction has little or no bearing on the selection of a bridge type for any one alignment, but may have an impact for one alignment over the other.

To adequately evaluate the pros and cons of each of the concepts, construction cost, maintenance cost, constructability, gradient and motorist safety, and aesthetics must be considered for each structure. These are discussed in the following paragraphs.

A. COMPARISON OF SCHEMES

1. Construction Cost

As noted in Section VIII, the simple span truss with multiple girders is the most cost effective structure that can be constructed for the river bridge on either alignment. The construction cost, including demolition and contingencies for the three segments of work is listed below.

Segment	South Aligment	North Alignment
Iowa Approach	$ 2,968,000	$ 4,681,000
River Bridge	29,359,000	26,543,000
Illinois Approach	3,083,000	3,286,000
Total	$35,410,000	$34,510,000

If a simple span tied arch and multiple girders are used for the river bridges an increase in cost of 1.8% for the south and 2.0% for the north alignment would result.

If either a cable-stayed concrete or steel girder is utilized for either alignment an increase in cost of from less than 1.0% to 10.0% can be expected.

Should a 660-ft cable-stayed concrete girder be selected for the south alignment with multiple prestressed concrete girder approaches the project cost would increase by 1.8% over that for the 480-ft through structure.

The multiple prestressed girders and multiple steel girders, for the river bridge approach spans, are extremely competitive as noted by the less than 1.0% cost differential.

2. Maintenance Cost

Other than painting of structural steel, all concepts are comparable. Therefore, a comparison of steel tonnage would lead to the relative merits of each concept for the river bridge.

Scheme	Tons of Steel	
	South Alignment	North Alignment
Truss	1,702	1,961
Arch	1,637	1,885
Cable-Stayed Steel Girder	1,346-1,981	1,398
Cable-Stayed Concrete Girder	-	-
Multiple Steel Girders	2,712-3,142	2,619-3,258
Multiple Prestressed Concrete Girders	-	-

On this basis, the cable-stayed concrete girder with prestressed concrete girders would be the least expensive to maintain on either alignment.

3. Constructability

Of the navigation span schemes studied, the simple span through truss and the simple span arch will require falsework in the channel for a period of time. The cable-stayed structures contain their own elements for erection and only minor interference with river traffic will occur.

4. Architectural Considerations

For appearance, it is felt that any of the cable-stayed structures, with the clean shallow lines of the deck and without any or minor overhead obstructions, would accomplish the objective of adding to the visual appeal of the area.

The single span tied arches do have cleaner lines than the truss and would be selected over that structure for appearance.

5. Grade and Design Speed

The through structures require a higher profile grade than the cable-stayed concrete girder leading to an increase in fuel costs for the traveling public and a reduced design speed.

B. CONCLUSIONS

If construction cost is of prime importance, a simple span truss with multiple girders for the north alignment river bridge, together with the Iowa and Illinois approaches, would cost $34,510,000.

If other considerations, such as maintainability, ease of construction, grades and design speeds, and aesthetics are foremost in consideration, a cable-stayed concrete girder with multiple girders for the river bridge would cost $34,600,000, including the Iowa and Illinois approaches.

For the riverfront access in Illinois, Connector A is recommended over Connector B as it provides a more direct route and costs about one-tenth as much.

U.S. ROUTE 34
DES MOINES COUNTY, IA - HENDERSON COUNTY, IL
TENTATIVE SCHEDULE
FOR
ENGINEERING SERVICES

DATE	ACTIVITY
April 12, 1985	Review conference for Design Study Report Receive authorization to solicit boring proposals
April 15, 1985	Solicit proposals for boring contract
April 19, 1985	Submit proposal for final design (River Bridge)
April 26, 1985	Print final Design Study Report
May 3, 1985	Receive boring proposals and forward recommendations to Client
May 15, 1985	Attend Public Hearings
May 28, 1985	Stake boring locations and complete field survey pickups Commence boring operations, concentrate on channel pier borings first
May 31, 1985	Complete negotiations for final design services (River Bridge)
June 3, 1985	Submit proposals for all remaining design services (Approaches
June 11 or 25	Commission Approval to start final design of River Bridge
June 21, 1985	Complete field work for boring program
July 1, 1985	Notice to Proceed with final design (River Bridge)
July 8, 1985	Complete negotiations for all remaining design (Approaches) services
August 16, 1985	Submit soils report
July —	Commission Approval to start final design of Approaches
August 5, 1985	Notice to Proceed with all remaining design (Approaches) services
October 31, 1985	Submit contract package for channel piers
	Let Channel piers
December 31, 1985	Submit contract package for truss (F&E)
January 6, 1986	Submit proposals for construction phase services
February 7, 1986	Complete negotiations for construction phase services
March — Comm. Approval for Const. Services	Let truss or truss and channel piers
March 14, 1986	Notice to proceed with construction phase services
July 1, 1986	Submit contract package for approach spans
December 31, 1986	Submit contract packages for Iowa and Illinois road and structures