STATE OF OREGON
STATE HIGHWAY DEPARTMENT
SALEM, OREGON

ALTERNATE DESIGNS FOR THE
FREMONT BRIDGE
OVER THE WILLAMETTE RIVER
PORTLAND, OREGON

INTERSTATE ROUTE 405
FEDERAL-AID PROJECT I-405-8 (1) 301

SUPPLEMENTARY REPORT ON
THREE ADDITIONAL DESIGNS

PARSONS, BRINCKERHOFF, QUADE & DOUGLAS

JULY 1965

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PARSONS, BRINCKERHOFF, QUADE & DOUGLAS

ENGINEERS

FOUNDED BY WILLIAM BARCLAY PARSONS IN 1885

NEW YORK • SAN FRANCISCO

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July 1, 1965

Mr. Forrest Cooper
State Highway Engineer
Oregon State Highway Department
Salem, Oregon 97310

Dear Mr. Cooper:

In accordance with our contract dated February 8, 1965, we submit herein this Supplementary Report of our studies of three Alternate Designs for the Fremont Bridge over the Willamette River at Portland, Oregon. As the title indicates, this report is a supplement to our Report of August 1964 which covered our investigations and evaluations of seven Alternate Designs as provided in our contract with the Department dated October 10, 1963.

Because it is a supplement to our earlier report, it is intended that the two reports shall be used together; therefore, basic data as to location, design criteria, and physical data (except for the additional borings needed for Design S-2) are not repeated herein. However, for convenience of reference and comparative purposes, a summary of the estimates of cost of all Alternate Designs which we have studied is included in this Supplementary Report.

Renderings in color of Designs 1 through 6 were prepared and delivered to you as a part of our Report of August, 1964. Such renderings for the three alternates comprising this Supplementary Report are not required by our present contract.

As in the case of the original studies, further studies of this important bridge have been a most interesting engineering assignment. This further opportunity to assist you and your Department on this project is highly valued by our firm.

Very truly yours,

PARSONS, BRINCKERHOFF, QUADE & DOUGLAS

M. N. Quade
Registered Professional Engineer
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CONTENTS

TABLES

ILLUSTRATIONS

INTRODUCTION

In August 1964, Parsons, Brinckerhoff, Quade & Douglas submitted to the Oregon State Highway Department a report on certain Alternate Designs for the Fremont Bridge over the Willamette River in the City of Portland. The Department, in collaboration with the U. S. Bureau of Public Roads, proposes to construct this bridge as required to connect the Stadium Freeway (U.S. Interstate Highway Oregon I-405) west of the river with the Fremont Street interchange of Interstate I-5 east of the river.

The August 1964 Report considered and evaluated the following seven basic designs:

Design 1. Through Cantilever Truss - 650-ft. Main Span
Design 2. Continuous Tied Arch Truss - 650-ft. Main Span
Design 3. Self-Anchored Suspension Bridge - 650-ft. Main Span
Design 4. Single Span Tied Arch - 650-ft. Main Span
Design 5. Orthotropic Deck Plate Girders - 650-ft. Main Span
Design 6. Continuous Tied Arch Truss - 1, 135-ft. Main Span
Design 7. Orthotropic Deck Plate Girders - 1, 135-ft. Main Span.

The 1964 studies disclosed certain advantages in designs having a longer main span that eliminate the two piers in the river required for the 650-ft. main span that was originally contemplated and included among the design criteria given to the Consultants. The river piers were found to be very costly. Moreover, they would create hazards to navigation during and after construction. They also would require large and expensive fenders which would have to be maintained at a significant annual cost in the future. All designs considered herein have the main piers located on land.

Subsequent to the submission of the August 1964 Report and the six renderings in color that accompanied it, the City of Portland proposed for aesthetic reasons a design similar to that of the recently completed bridge over the Fraser River at Port Mann, B.C. and designated as a "stiffened tied arch."

In view of the above and other considerations, the Consultants were directed to study and evaluate three additional designs as provided in a contract between the Department and the Consultants dated February 8, 1965. These designs are as follows:

Design S-1. A Stiffened Tied Arch - 1,350-ft. Main Span

Design S-2. An Externally Anchored Suspension Bridge - 1,180-ft. Main Span

Design S-3. A Through Cantilever Truss - 1,150-ft. Main Span.

The location, the design criteria and physical data pertaining to the site are, of course, the same for these three additional designs and the description of them need not be repeated in this Supplementary Report. The study limit at the northeast end remains the same. At the southwest end the study limits are slightly different than those used in the August 1964 Report. The Department has requested that for Designs S-1 and S-3 the originally established limit at Station 368+50 be held in order to locate the end pier of the main bridge on the west side of the intersection of N.W. 14th Avenue and Thurman Street, thus eliminating the alteration to this intersection that was proposed in the August 1964 Report.

In the case of Design S-2, it is necessary to locate the cable anchorage beyond Station 368+50 in the block bounded by N.W. 15th Avenue, Thurman Street, N.W. 14th Avenue and Savier Street. This will require an adjustment in the design and cost of the end 170 feet of the approach structure at that end of the main bridge, as measured on the extended main bridge tangent.

DESCRIPTION OF ALTERNATE DESIGNS

General

All of the designs included in these supplemental studies have main spans which cross the entire width of the Willamette River. Because of the configuration of the arch rib, Design S-1 requires a main span of 1,350 feet to meet the required navigation clearances. Designs S-2 and S-3 accomplish this with spans of 1,180 feet and 1,150 feet respectively. These main span lengths are to be compared to the 1,135-foot main span in Designs 6 and 7 in the 1964 report. That span length is adequate to clear the river and the wharves along the river banks after completion of the bridge. Horizontal clearances and usage by ships of the wharves during construction are discussed on page 12 of the 1964 report.

In Design S-2 the main span is about 45 feet longer than the minimum required to clear the river and wharves and its length of 1,180 feet is fixed by the location of the southwest anchorage in relation to the intersection of N.W. 14th Avenue and Thurman Street, and the desirability of making the suspended side spans (as measured from the center line of main tower to the center line of the cable bent at the front of the anchorage) exactly one-half of the length of the main span.

Similarly, Design S-3 is 15 feet longer than the minimum required because of the 80-foot shift away from the river of the southwest anchor pier and the desirability of making the anchor spans exactly one-half of the main span in order to maintain equal panel lengths throughout the entire cantilever bridge. In neither case, do the small differences in lengths of the main span appreciably affect the costs of the bridges.

Roadway widths, overhead clearances and other geometrical features of the cross sections are the same as in the 1964 designs except that the distance center to center of trusses has been increased to 68'-0" for Design S-1. Live loads are the same.

In accordance with the Department's instructions, the longitudinal roadway profiles conform to those initially established. As in the cases of Designs 1, 2, 3, 4 and 6, there is excess vertical clearance between roadways and over the navigation channel in Designs S-2 and S-3 (see page 11 of 1964 report). This does not apply to Design S-1.

Also in accordance with the Department's instructions, both the upper and lower decks in Designs S-2 and S-3 and the lower deck in Design S-1 consist of conventional reinforced concrete slabs. It has been previously pointed out that the use of steel open grating on the lower deck between main piers would result in lower costs and, in the opinion of the Consultants, it would be quite satisfactory.

The upper deck in Design S-1 is, of necessity, of orthotropic construction. In fact, it is the stiffening element of the superstructure under unbalanced live load because the trusses contain no diagonal members. The subject of surfacing for steel plate decks is discussed on pages 18 and 19 of the 1964 report and the same comments apply to the upper deck in Design S-1. In addition, reference is made to the comments on page 38 of the 1964 report wherein the Consultants state that in their opinion the requirements for the use of a corrosion-resistant steel in the deck plates, metallizing of the deck plates with hot-applied zinc, and a three-ply membrane waterproofing placed directly on the steel plate are cumulatively excessive and that perhaps any one and certainly no more than two of the three requirements would be found adequate.

The kinds of steel and rope suspenders and the allowable unit stresses for each are for Designs S-1, S-2 and S-3 the same as those described on page 12 of the 1964 report. The suspension bridge cables in Design S-2 are, however, to be constructed of parallel wires.

Design S-1

The layout for Design S-1 is shown on Fig. 1. Beginning at Station 368+50 it consists of a 475-foot side span, a 1,350-foot main span, a 475-foot side span, and one Vierendeel truss span 335'-0"long.

The main bridge is 2,300 feet long. It is statically indeterminate. To determine the stresses in the main members algebraically under various conditions of loading would require the solution of eight simultaneous equations repeated at least 55 times — possibly a total of two man-years of manual work. The Consultants originally intended to employ a large computer owned by the American Bridge Division and were told that a program was available which could be adapted to the solution of this Design. However, it was learned later that that computer was not available until a much later date and that the nearest available program could not be readily adapted to such a complex problem as Design S-1.

The Consultants then broke the analysis down into several stages for solution on their own computer—a Bendix 15-D. This computer has a much smaller memory capacity and the various increments in the analysis had to be rehandled in the computer in stages that did not exceed its capacity.

Before the analysis was sufficiently complete for the purposes of this preliminary study, the Consultants learned of an available IBM 7094 computer with a program that could be adapted to the S-1 design. About eight man-weeks of time by our staff using available sub-routines has been required to adapt the program. It is interesting to note that after the programming was completed, the running time for the machine to solve two basically different designs (one with fixed shoes for the arch ribs at both main piers and a second with fixed shoes at one main pier and expansion shoes at the other main pier) was ten minutes. Other studies of special problems were made on the Bendix 15-D using data obtained from the IBM 7094. The design analyses to date are, of course, preliminary but they are adequate for the purposes of this report. A final design for contract plans might require one hour of computer time on the IBM 7094.

After comparing the two analyses referred to above, it was concluded that the arch ribs should be fixed at both main piers. Expansion and contraction of the center span due to changes in temperature will result in changes in arch thrust and the corresponding rise and fall of the arch. The important considerations which lead to this conclusion are greater rigidity, a significant saving in structural steel, and a desirable symmetry of design since maximum longitudinal forces arising from wind and earthquake loads are divided equally between the two main piers. This structure is quite flexible when compared to a truss composed of triangular elements. It becomes much more flexible when expansion shoes are placed at one of the main piers.

Moreover, if the shoes at both main piers are fixed, the expansion movements at the free ends of the 2,300-foot length of bridge are approximately eight inches at each end. If expansion shoes were to be used at one main pier, the movement at the end pier on the same side of the river would be of the order of magnitude of four feet. It should be noted that because of the unusual action of this type of structure, transverse loads would also result in longitudinal movements of expansion shoes placed at a main pier.

For the purposes of these studies it was assumed that earthquake forces would be as large or larger than those due to longitudinal wind loads. Because the two are never expected to occur simultaneously, no analysis was made for longitudinal wind. Transverse wind was assumed to be 75 psf on 1.5 times the projected area and was applied in combination with dead load (i.e. on the unloaded bridge). The Consultants believe that this will not prove to be overly conservative. The two deep, wide box girders in the orthotropic upper deck, together with the solid bridge decks above and below them are likely to produce a large wind drag effect.

In making a final design for S-1 it would be necessary to perform extensive wind tunnel tests on a scale model in order to determine the transverse and longitudinal wind loads to be applied to the structure. The design should be investigated for wind load on the bridge in combination with live load for both longitudinal and transverse forces. Also for final design, dynamic analyses for live loads, wind loads, and earthquake loads should be made; the need for them is indicated by the flexibility of the structure. The second deck and the complete absence of trussed framing between roadway decks is a significant factor in the flexibility of the bridge.

It is entirely possible that wind tunnel tests of a model might demonstrate that the use of open steel grating on the lower deck would reduce the magnitude of the wind forces and the dynamic response of the bridge.

In these preliminary calculations the shape of the arch ribs is parabolic. Since the dead loads are not uniform, some adjustment in the shape of the ribs should be made in the final design—especially in the side spans. The difference is not significant for the purposes of this report.

The arch ribs in all spans are provided with a full depth lateral bracing system except in the locations where the bracing must be interrupted to provide necessary roadway clearances at both the upper and lower decks. At these locations the transverse forces must be resisted by bending in the arch ribs. In the upper deck, the continuous orthotropic deck provides rigid lateral bracing in the plane of that deck. At intervals of about 150 feet the transverse truss beneath the upper deck shown in the section at the pier on Fig. 2 will brace the deep box girders and provide torsional rigidity. The lateral system in the plane of the lower deck is continuous throughout the bridge.

All vertical members between decks act as members of a rigid frame for transverse loads. A heavy frame will transfer transverse loads from the upper deck to the shoes at the end pier and, at the main piers, to vertical trussed portal frames between the lower deck and the arch ribs at the panel points over the main piers. Heavy rigid frames are required at the panel points where the above-deck lateral bracing between arch ribs ends in order to transfer lateral loads on the ribs to the orthotropic deck which, in turn, will transfer them to the portal frames at the main piers.

The vertical hangers suspending the two decks from the central portion of the arch ribs will consist of wire rope bridge strands attached to the upper deck except for the end hangers which are rigid members that will participate in the portal action. As previously stated, verticals between decks are rigid members which through participation in frame action provide transverse and torsional stiffness as well as suspending the lower deck at each panel point.

It is possible that studies made during a final design might indicate the need for trussed sway frames in the planes of the longer columns which support the roadway decks on the arch ribs at the panel points where the ribs are below the roadways. At present such sway frames do not appear to be necessary.

As required by the Department, the deck plate for the upper deck is corrosion-resistant steel. All other steel in the orthotropic upper deck consists of A-441 grade steel except that T-1 steel is used in the lower flanges and lower parts of the webs of the deep box girders and except that the transverse floor beams and trusses in the upper deck are A-36 steel. The arch ribs are made of T-1 steel throughout. The main truss verticals are made of A-36, A-441 and T-1 steel depending upon their location and the stress requirements.

Influence lines were computed for reactions and for the stresses in all main truss members. Those for the reactions at the piers, the horizontal thrust at three points in the arch rib, the stress in the orthotropic tie girders and the moment in the orthotropic girders at the main pier panel point are shown on Fig. 3. An examination of these influence lines shows the unusual hybrid action of this type of structure. It can best be simply described as a combination of three structural systems: a relatively slender tied arch, a continuous beam (the orthotropic deck), and a structural frame without diagonals.

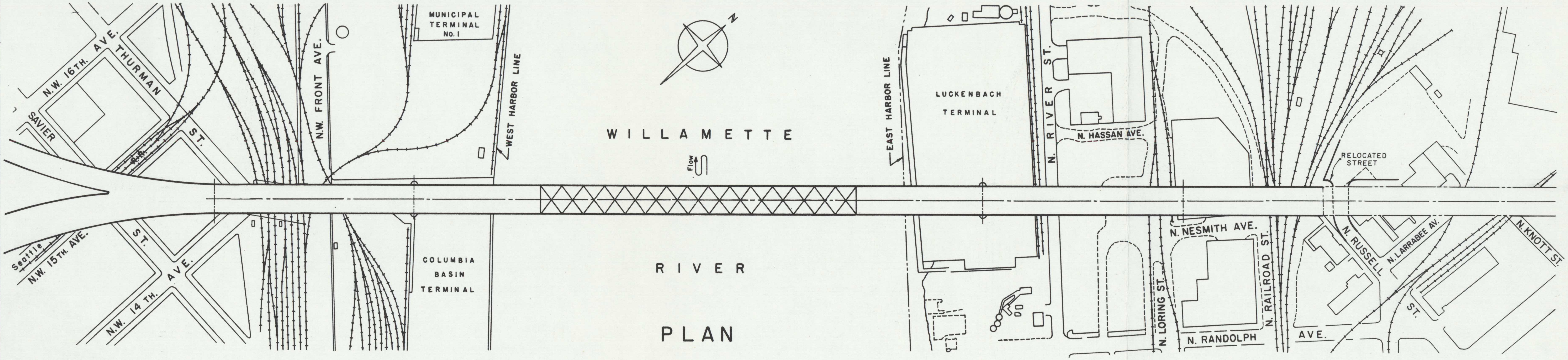
It should be noted that the lower deck will not participate in the tie action resisting the arch thrust at the ends of the ribs and between the intersection points where the rib rises above the upper roadway. The longitudinal members in the lower deck are discontinuous at appropriate points and expansion and contraction joints are provided at those points. Provision is made, however, for transfer of transverse wind shear in the lower deck at those points. It is impractical and undesirable to design the lower deck so that it would participate in the tie stress.

Problems of erection and the jacking forces required for closure of the arch ribs are described in later sections of this report.

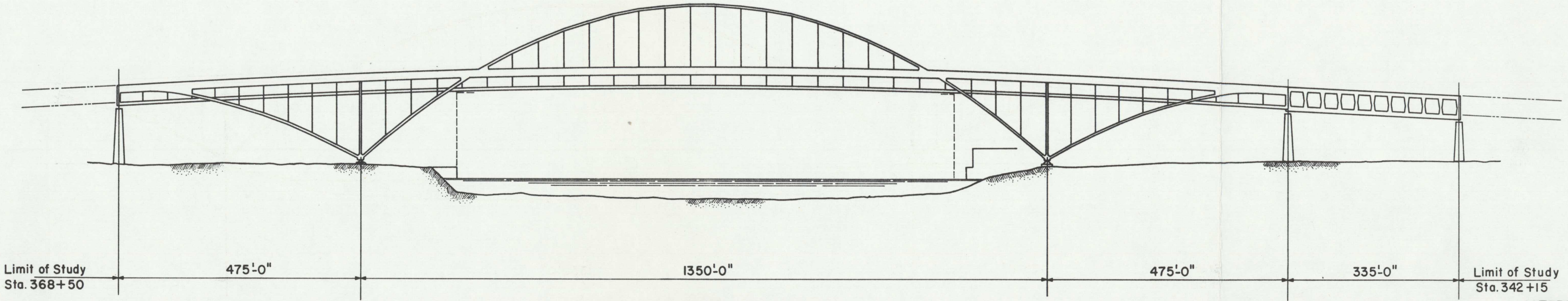
The west main pier is founded at elevation - 20 on steel H-piles. Since the reaction of the superstructure has reversing horizontal components several outer rows of piles on the east and west sides of the base will be battered as required to resist the horizontal components. Under dead load only, the total vertical load from the superstructure shoes is 13,000 tons and the total horizontal load is 660 tons applied in a direction toward the river (i.e. the horizontal arch thrust from the side span is greater than that from the center span). Live load reactions which are additive and produce the maximum horizontal force in the same direction are 640 tons vertical and 1,180 tons horizontal. Thus the greatest ratio of horizontal thrust to vertical thrust for dead plus live load is 1,840:13,640 which is a ratio of 1 to 8.3. Since this pier is about 100 feet farther away from the river than the west main pier in Design 6, it will not be necessary to leave any of the sheet pile cofferdams, within which it would be constructed, in place or provide riprap protection as was recommended for the west main pier of Design 6 (see page 9 of the 1964 report).

The east main pier is founded on a spread footing at elevation -30. It, too, would be constructed within a cofferdam. It is considered advisable to leave the sheet piling in place around the entire perimeter of this pier base for added security against movement under the horizontal components of the pier loads. The pier is too far from the river to require protection by riprap against scour.

The other three piers are founded at elevation +15 and are supported by steel H-piles.



PLAN



ELEVATION

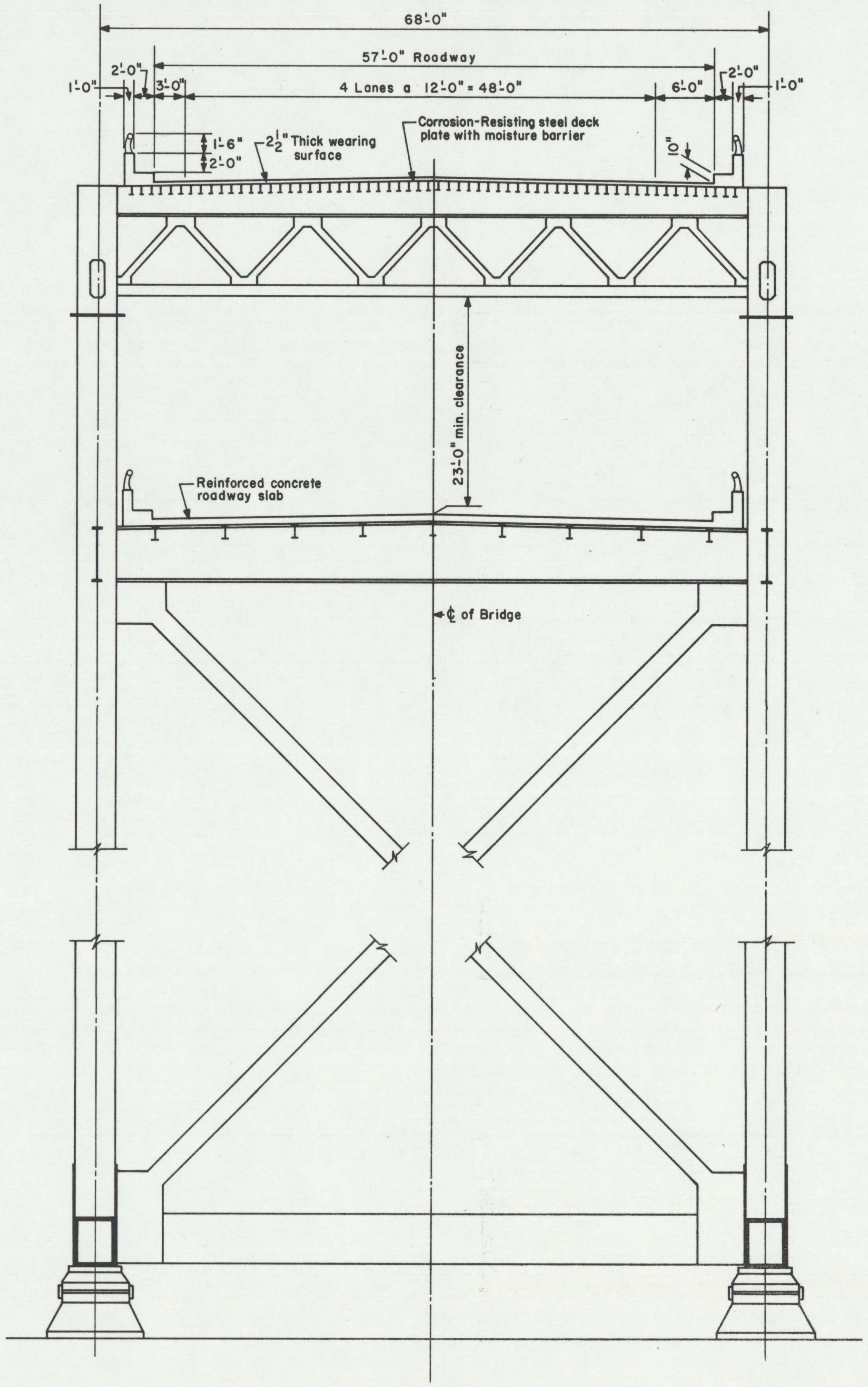
NOTE:
For "Bridge Profile and Navigation Clearances" see Figure 1 in Report of August 1964.

100 50 0 100 200 300 400
FEET

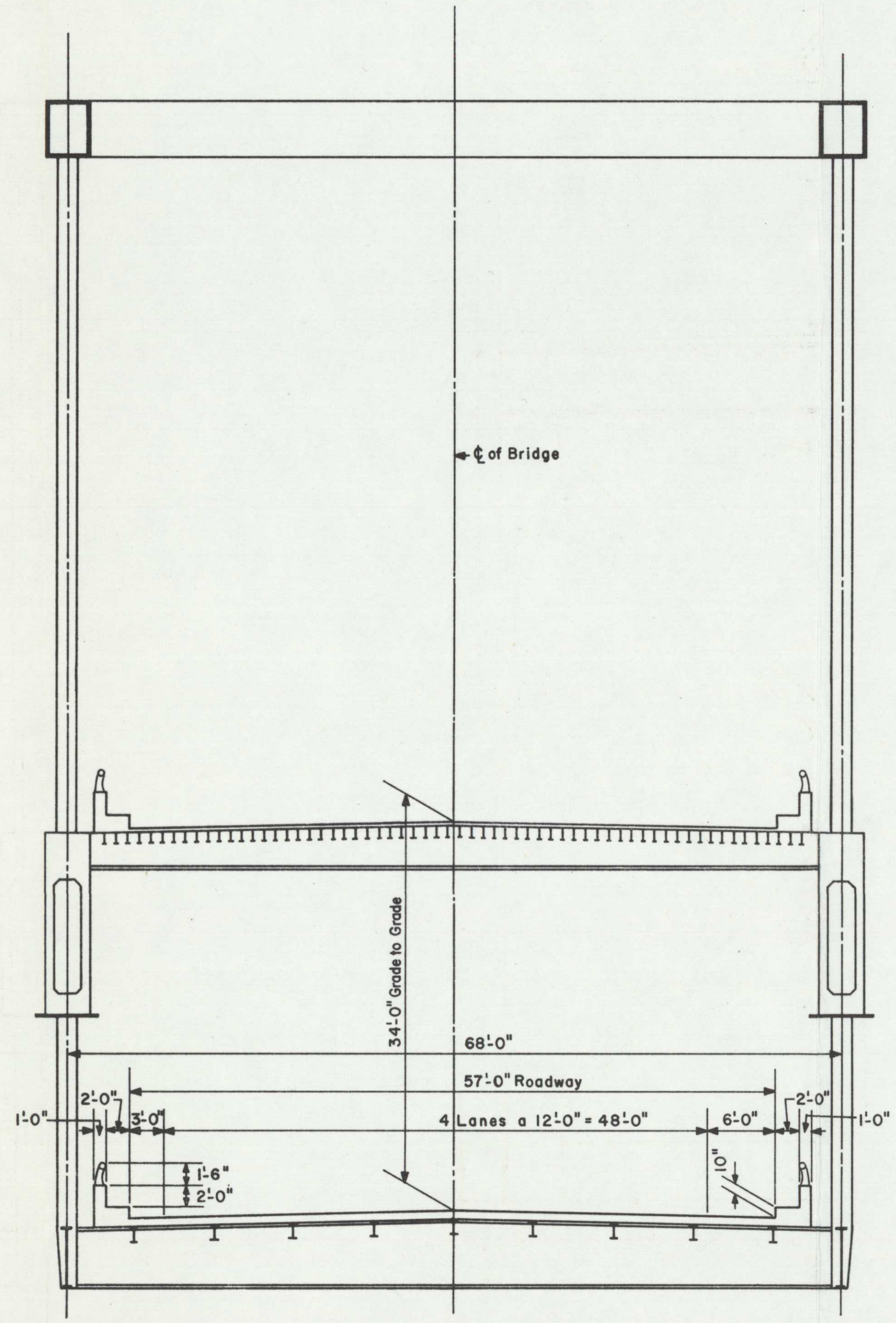
ALTERNATE DESIGNS
FOR THE FREMONT BRIDGE

DESIGN S1

STIFFENED TIED ARCH
WITH ORTHOTROPIC UPPER DECK
1350 FOOT MAIN SPAN

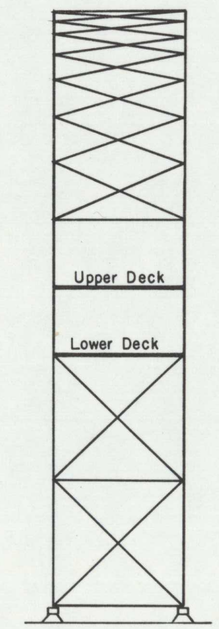


SECTION AT
MAIN PIER



SECTION THROUGH
MAIN SPAN

10 0 10 20
FEET



ELEVATION AT
MAIN PIER

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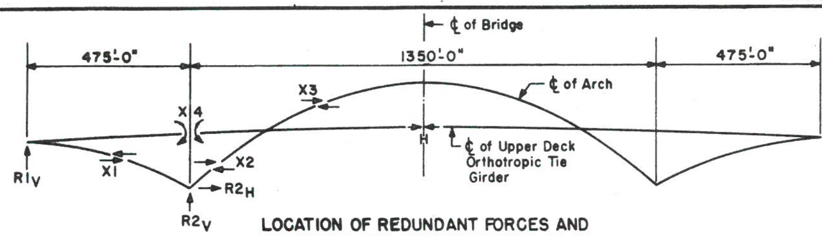
FEET

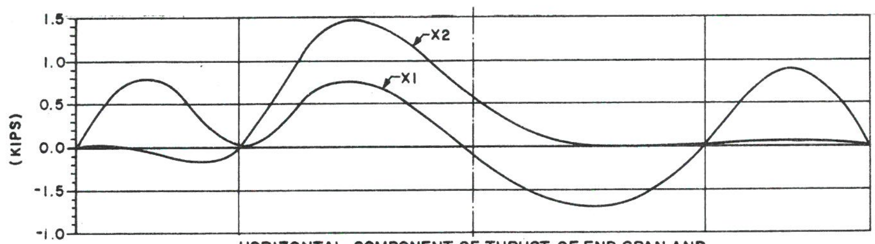
ALTERNATE DESIGNS
FOR THE FREMONT BRIDGE

TYPICAL CROSS SECTIONS

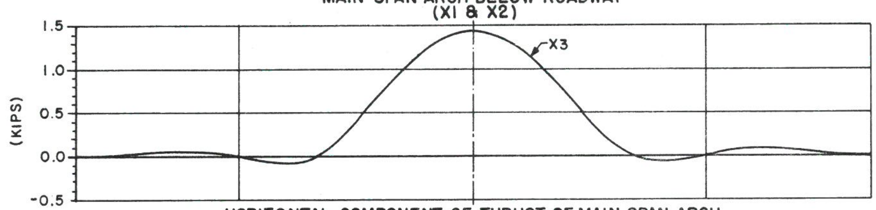
DESIGN SI



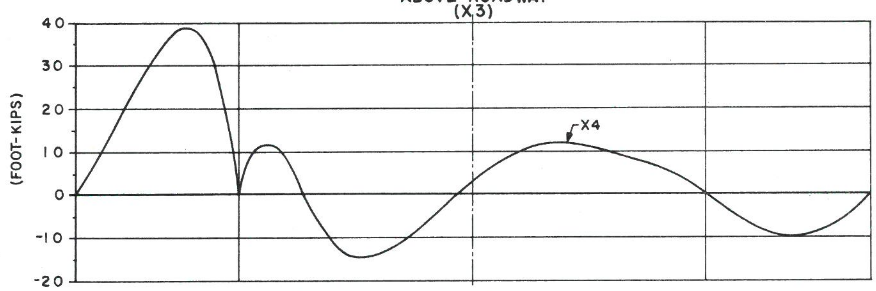
LOCATION OF REDUNDANT FORCES AND REACTIONS FOR STIFFENED TIED ARCH WITH ORTHOTROPIC UPPER DECK



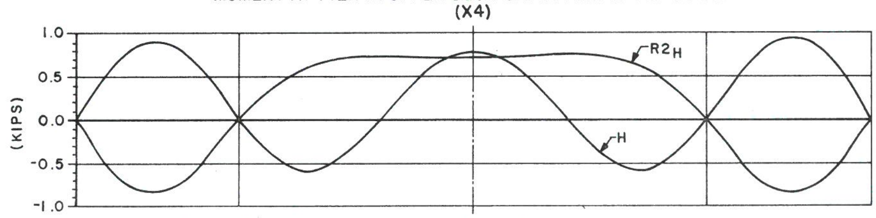
HORIZONTAL COMPONENT OF THRUST OF END SPAN AND MAIN SPAN ARCH BELOW ROADWAY (X1 & X2)



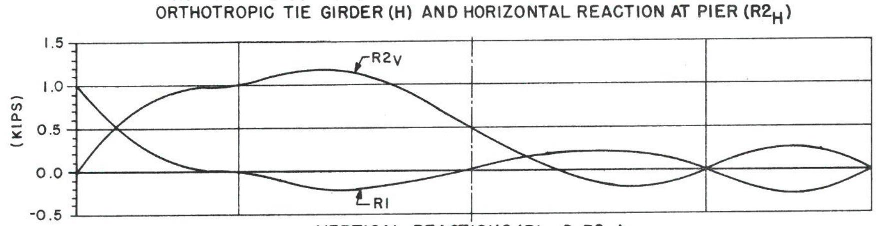
HORIZONTAL COMPONENT OF THRUST OF MAIN SPAN ARCH ABOVE ROADWAY (X3)



MOMENT AT PIER IN UPPER DECK ORTHOTROPIC TIE GIRDER (X4)



HORIZONTAL COMPONENT OF STRESS IN CENTER PART OF UPPER DECK ORTHOTROPIC TIE GIRDER (H) AND HORIZONTAL REACTION AT PIER ($R2_H$)



VERTICAL REACTIONS ($R1_V$ & $R2_V$)

NOTE:
Influence lines for unit vertical load of 1^k

ALTERNATE DESIGNS FOR THE FREMONT BRIDGE

INFLUENCE LINES

DESIGN S1

STIFFENED TIED ARCH WITH ORTHOTROPIC UPPER DECK

Design S-2

The plan and elevation of Design S-2 are shown on Fig. 4 and a typical cross section is shown on Fig. 6. The design consists of a conventional externally anchored suspension bridge having a main span of 1,180 feet and suspended side spans of 590 feet. A girder span 170 feet long is required to complete the bridge to Station 342 + 15— the easterly limit of the study.

As previously mentioned, the westerly study limit has been extended in order to locate the west anchorage beyond the intersection of N.W. 14th Avenue and Thurman Street.

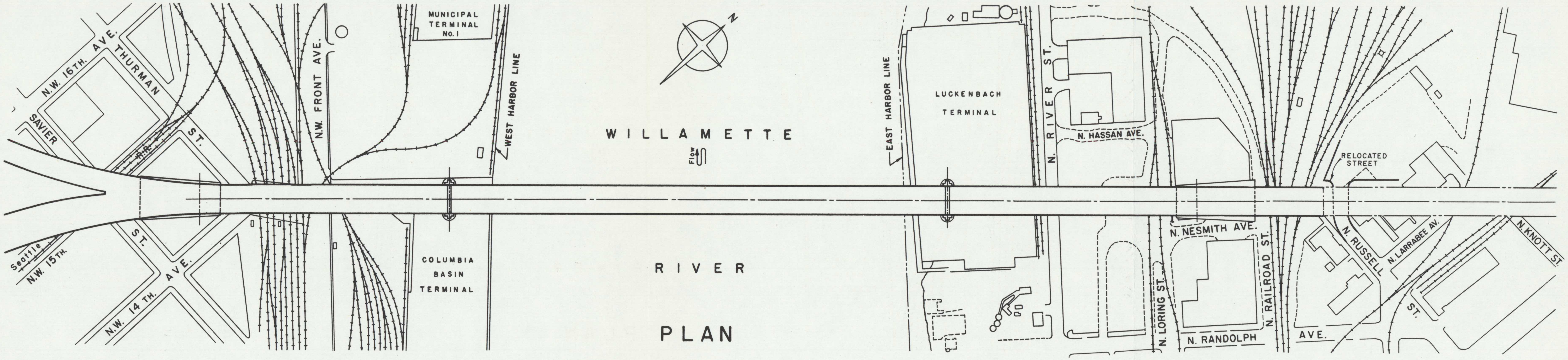
The cables are made of parallel wires and the suspenders are wire rope strands. Two kinds of steel, A-36 and A-441, are used in the stiffening trusses. The floor and lateral systems are made of A-36 steel.

The west main pier is founded at elevation -20 on steel H-piles. Since this pier is located nearly as close to the river as that in Design 6, the sheet piling for the cofferdam used for construction should be cut off about one foot above the top of the base and left in place along the river face and along the outer half of the two ends, leaving the lower portions of the sheets in place. Riprap should be placed between the harbor line and the base of the pier after pile driving for the restoration of the wharf has been completed.

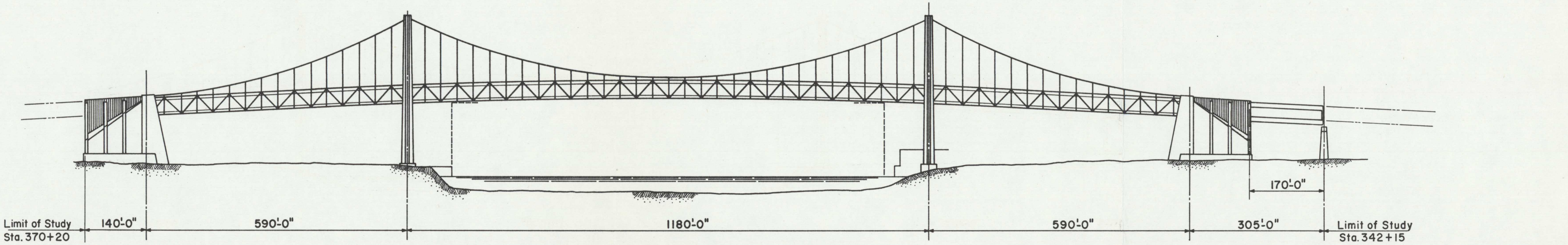
The east main pier will be founded on a spread footing at elevation -30. As in the case of the west main pier described above, provision against possible scour should be made by leaving a portion of the steel sheet piling in place and placing riprap in front of it.

Additional borings were made at the sites of the cable anchorages. The logs of these borings are shown on Fig. 5. Both anchorages can be founded on spread footings, the west anchorage being founded at elevation -10 and the east anchorage at elevation -20. Because the elevation of the cables at the saddles on the cable bents at the front of the anchorages is quite high above ground, the overturning moment of the cable forces is large and a massive block of concrete is required at each anchorage. However, the upper part of each anchorage is hollow. Both roadways are supported within the anchorages by steel spans. The west anchorage has been designed to accommodate the divergence in plan of the upper and lower roadways.

The approach pier at Station 342 + 15 is founded at elevation +15 on steel H-piles.



PLAN



ELEVATION

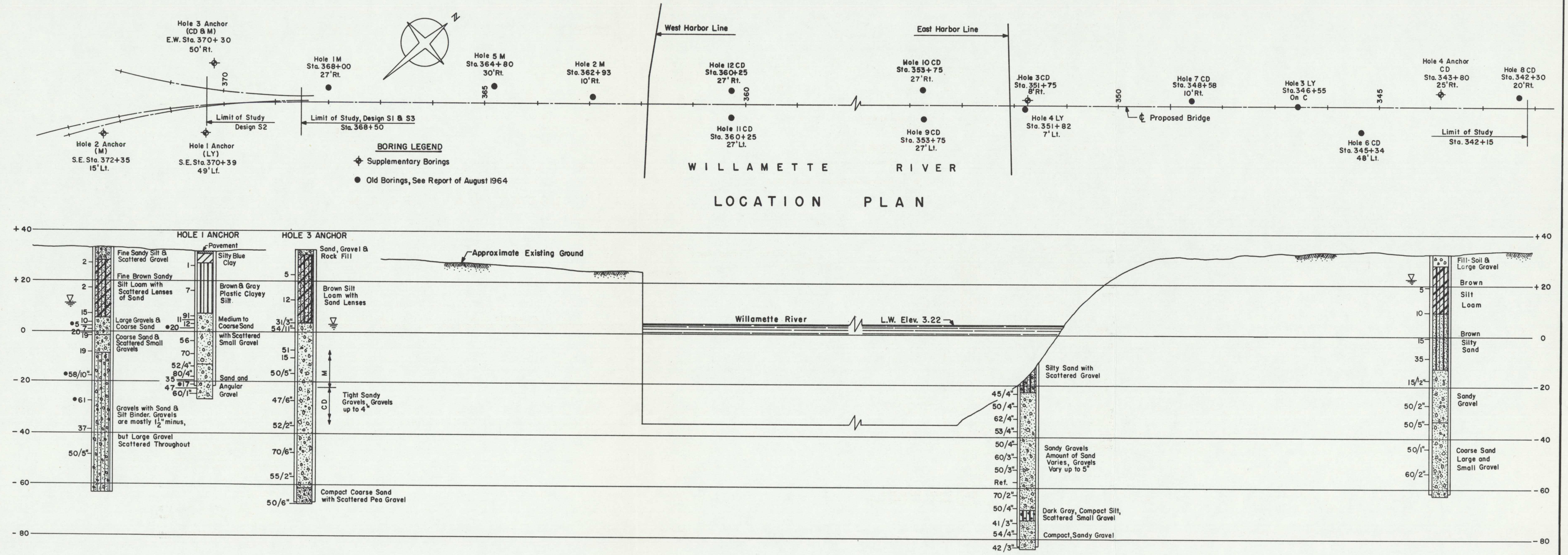
NOTE:
For "Bridge Profile and Navigation Clearances" see Fig. 1 in Report of August, 1964.

100 50 0 100 200 300 400
FEET

ALTERNATE DESIGNS
FOR THE FREMONT BRIDGE

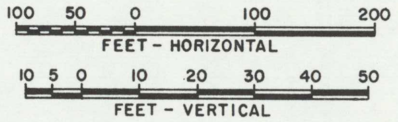
DESIGN S2

EXTERNALLY ANCHORED
SUSPENSION BRIDGE
1180 FOOT MAIN SPAN

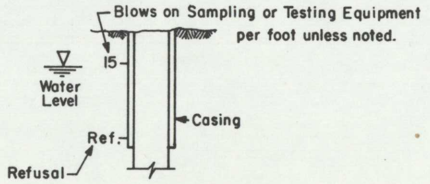


NOTES

1. All Elevations are from City of Portland Datum
2. Drilling Equipment:
   - CD - Churn Drill
   - LY - Longyear Rotary
   - M - Mobile Rotary
3. Sampling and Testing Equipment:
   - Unless noted, 2" O.D. Standard Split Spoon Sampler with 140 lb. hammer falling 30"
   - ● Oregon Miniature Pile with 166 lb. weight falling 72"
4. Gravel sizes shown are maximum in each classification.

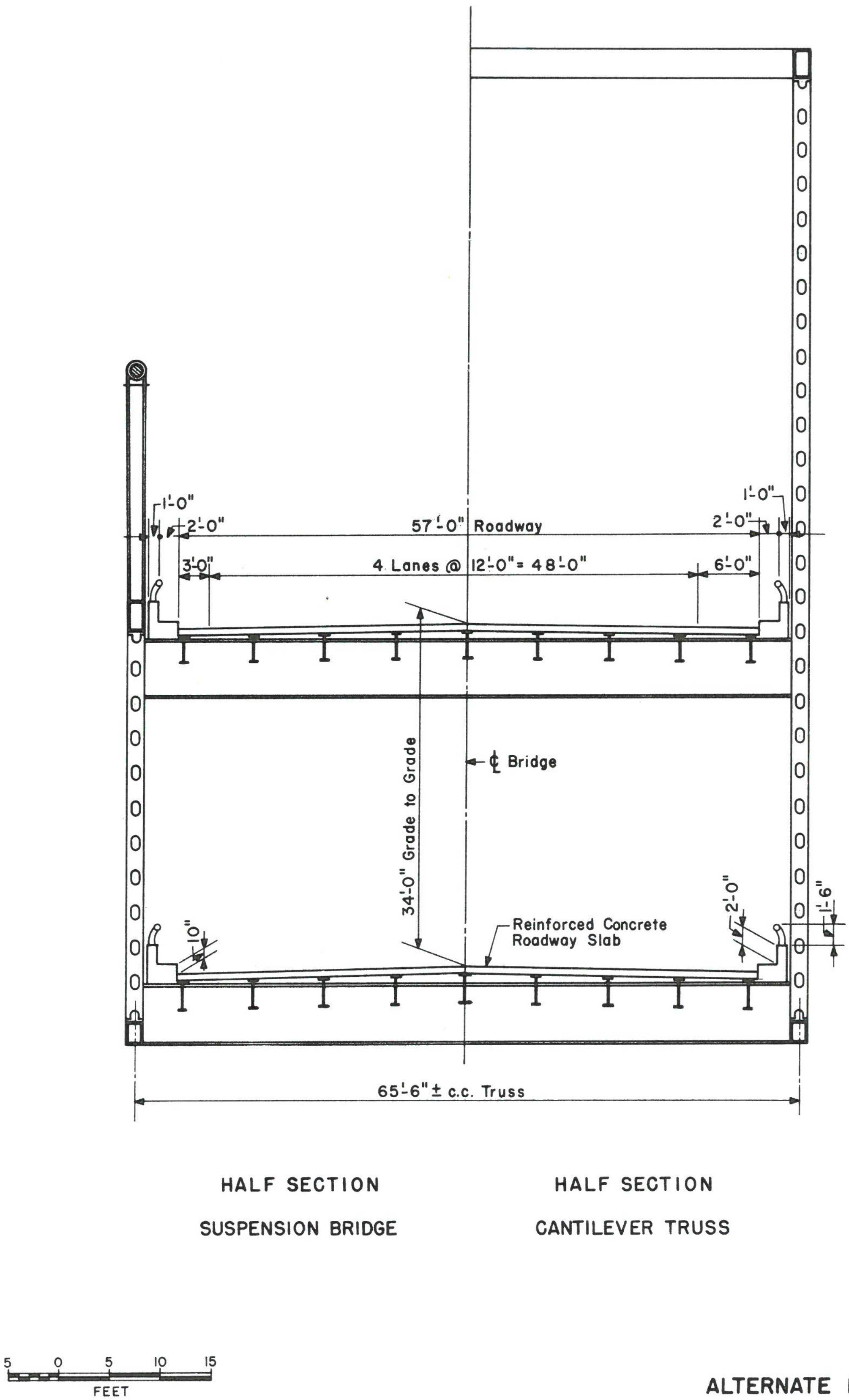


Legend:



ALTERNATE DESIGNS
FOR THE FREMONT BRIDGE

LOGS OF SUPPLEMENTARY TEST BORINGS



ALTERNATE DESIGNS
FOR THE FREMONT BRIDGE

TYPICAL CROSS SECTIONS

DESIGNS S2 AND S3

Design S-3

Design S-3 consists of a through cantilever truss having anchor spans 575 feet long on each side of an 1,150-foot main span. At the east end there is a 335-foot simple truss span. The plan and elevation are shown on Fig. 7 and a typical cross section is shown on Fig. 6.

The design of the through cantilever bridge is conventional; however, the double deck is a somewhat unusual feature, particularly because of the fact that the transverse bracing system at the top chord can not be carried through to the ends of the anchor spans, and because of the fact that all portals and sway frames must be designed as rigid frames to resist lateral forces throughout all or a portion of their height as required by roadway clearances for the two decks.

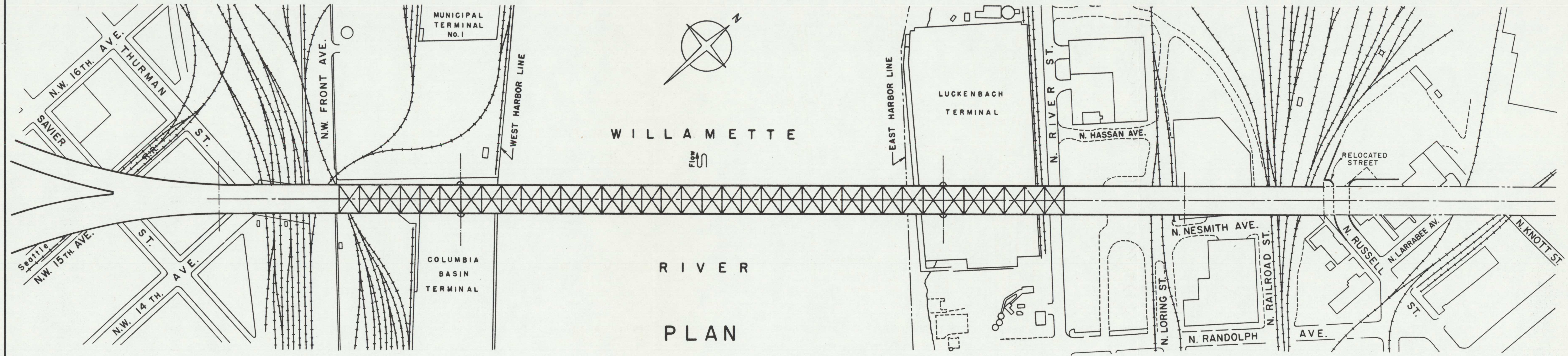
The floor system is composed of A-36 steel. Because of the panel length (48 feet), the stringers are designed for composite action with the conventional reinforced concrete deck slabs in each roadway. The long truss panel was selected for reasons of economy, appearance, and the relative reduction in secondary stresses in the deep chords in the trusses.

A-36, A-441 and T-1 steels are used in the main truss members—the choice being determined by economy.

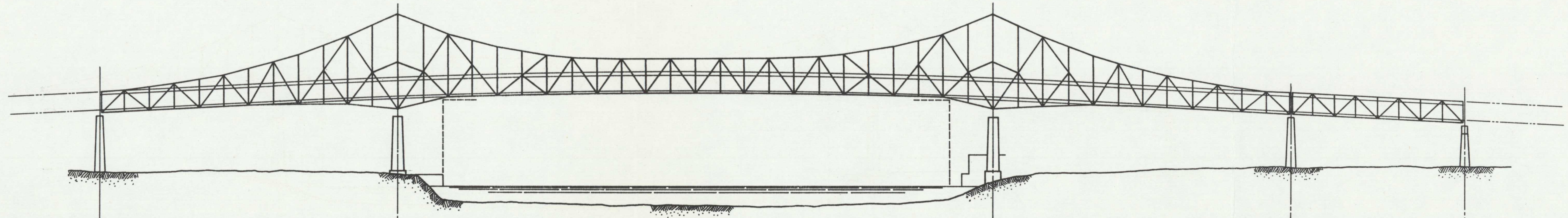
As in Design S-2, the west main pier is founded at elevation -20 and is supported by steel H-piles; the east main pier is founded at elevation -30 on a spread footing. The same protection against scour that is provided for the main piers in Design S-2 and that has been previously described is required here. The anchor piers and the end pier at Station 342 + 15 are founded at elevation +15 on steel H-piles.

Inasmuch as the length of the main span in relation to the horizontal limits of navigation clearance will permit lowering the bottom chords of the trusses about 20 feet at the main piers, considerable economy has been achieved by the reduction in pier heights. The top chord panel points have been lowered accordingly without a reduction in truss depth at the main piers that would be uneconomic. This profile, in combination with the longer panel length, has, in the opinion of the Consultants, resulted in a significant improvement in the appearance of the through truss cantilever as compared to Design 1 which is based on a 650-foot main span, anchor span lengths which, of necessity, are too long for good appearance, and four approach simple truss spans which are not particularly attractive. (see Figs. 5 and 5a in 1964 report).

To facilitate comparison, an alternate truss profile which is a more common and conventional profile for a through cantilever truss is shown on Fig. 8. In this profile, the bottom chords are parallel to the profile of the roadways. Since the upper chord panel points at the main piers are raised vertically by the same amount as the lower chord panel points in order to maintain the truss depth at the main piers, the profile of the top chords differs from that shown on Fig. 7. The cost due to the additional height of the main piers is estimated to be $250,000. This cost is not included in the estimate of cost of Design S-3 in Table 3.



PLAN



ELEVATION

NOTE:
For "Bridge Profile and Navigation Clearances" see Fig. 1 in Report of August, 1964.

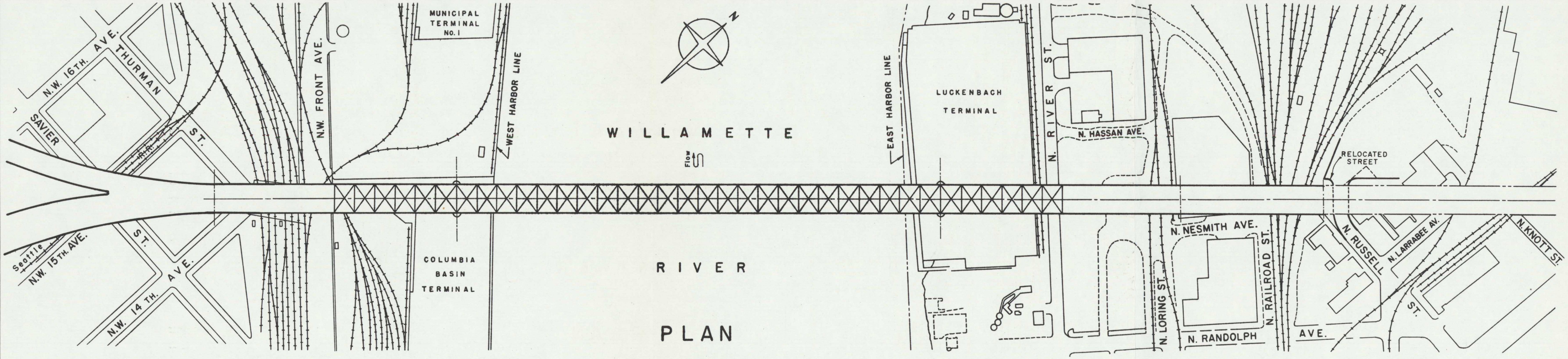
100 50 0 100 200 300 400
FEET

ALTERNATE DESIGNS
FOR THE FREMONT BRIDGE

DESIGN S3

THROUGH CANTILEVER TRUSS

1150 FOOT MAIN SPAN



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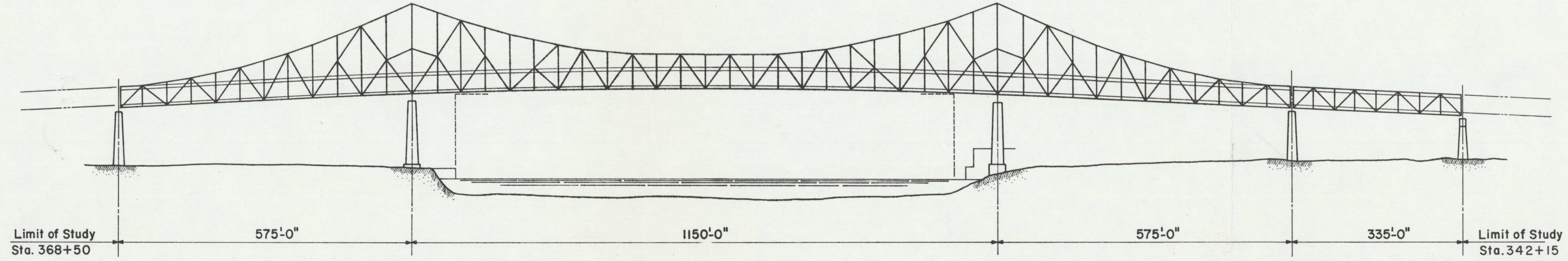

ELEVATION

NOTE:
For "Bridge Profile and Navigation Clearances" see
Fig. I in Report of August, 1964.

100 50 0 100 200 300 400
FEET

ALTERNATE DESIGNS
FOR THE FREMONT BRIDGE

DESIGN S3

THROUGH CANTILEVER TRUSS
ALTERNATE TRUSS PROFILE

1150 FOOT MAIN SPAN

ADDITIONAL STUDIES

The externally anchored suspension bridge (S-2) and the through cantilever truss bridge (S-3) are quite conventional and, for the purposes of this report, they required no special additional studies. In the case of S-2, architectural study was given to the cable anchorages to develop a form and outline that is pleasing while meeting, at the same time, the structural requirements. The anchorages rise higher above ground than the usual suspension bridge anchorage.

Consideration was given to the use of a Wichert continuous truss for Design S-3 but this proposal was abandoned when it was learned that it offered no significant advantages.

Although the Consultants were instructed by the Department to evaluate the quantities of materials and cost of Design S-1 without completely resolving the many complex design and erection problems that are involved, several additional studies not normally required for preliminary estimates of cost were, of necessity, carried out for that design. Certain critical details were developed to the stage necessary to be sure that the design and the erection are feasible for this novel and unprecedented bridge and to obtain information needed for cost estimates.

Several erection problems were studied—principally those relating to jacking the arch ribs for closure and the methods and jacking forces required to reduce the dead load moments in the orthotropic tie girders to a minimum. Since changes in length of the orthotropic tie girders (elongation) and in the length of the arch ribs in various parts of the spans (shortening) are large, a final design of this structure will require detailed study of the main truss verticals below deck and of the types and details of the end connections that are required for both temporary and permanent purposes.

Temporary or even permanent hinges may be required, since the possibility of residual bending stresses in the verticals resulting from the adoption of a practicable sequence of construction operations must not be overlooked.

Rather complete details of the shoe at the main piers for the arch ribs and the main vertical above the pier were developed in order to study the problem of jacking the side span arch rib at this panel point. This solution was found to be practicable. However, additional studies for a final design might show that other jacking locations and methods of closure and stress transfer to the tie girders would be advantageous.

Jacking forces for closure and transfer of stress to the orthotropic tie girder were calculated on the basis of all structural steel being in place except the floor steel for the lower deck in the central portion of the main span, but with no surfacing on the upper roadway and no concrete for the entire lower deck roadway in place. The full effect of this added dead load after closure has not been studied.

In accordance with this preliminary plan, simultaneous jacking would be accomplished during closure at a total of six locations in the arch ribs as follows: each side span rib at each main pier and each rib at the center of the main span. The jacking force per side span arch rib at each main pier has been calculated to be 8,500 kips and the movement to be 16 inches. At the top of the main span the jacking force is 4,600 kips per rib and the movement is 14 inches. This total movement of 46 inches is not a measure of the elongation of the tie girders since much of the work done in jacking serves to bend the girders as required to reduce to a minimum the dead load bending after closure and after all dead load is in place. The tie girders are elongated 12 inches in their 2,300-foot length under full dead load. These forces and movements are indicative of the many design problems yet to be solved with respect to the closure when the steel ceases to be supported by the temporary erection towers and cables and begins to support itself.

Many influence lines for stresses in the principal members and for vertical and horizontal movements of various panel points have been computed and plotted for Design S-1. Many others would be needed for final design and for erection. In general, the influence lines for deflection that have been prepared indicate a high degree of flexibility of the structure. It is possible that at times movements and vibrations from unsymmetrical live load or wind or both might be sensed by the public and therefore become a cause for adverse criticism. Movements due to changes in temperature are too slow to cause public reaction. Those from a severe earthquake would certainly be sensed by the public, but this would be true of any bridge. However, the movements of the S-1 structure would undoubtedly be larger and be accompanied by larger amplitudes of vibration.

At one stage of the work the Consultants considered the advantages of two variations to the S-1 design. In the first alternate design, diagonals were added to the truss panels below the lower deck to convert the side spans into statically determinate anchor spans and each end portion of the central span into a statically determinate cantilever arm. The suspended span would become a tied arch with orthotropic construction at both the upper and lower decks to resist moments due to unsymmetrical live load. Only the lower deck would act as the arch tie. Concrete decks could be used in both levels on the cantilever and anchor arms. Such a design would be less flexible and would be easier and less costly to erect. The truss depth at the main pier would be too shallow for the most economical use of steel.

The second alternate substituted a continuous Warren truss between the upper and lower roadways as the stiffening element in all spans as well as the tie for the arches. This substitution would eliminate the need for an orthotropic deck and would permit the use of concrete deck slabs for both roadways. At the request of the Department no further consideration was given to either alternate.

CONSTRUCTION

There are no unusual problems involved in the construction of the substructure for any of the three designs. The substructure for Design S-1 could be constructed in the shortest time because of the relatively short height of the main piers. However, this gain in construction time on the substructure would be more than offset by the additional time to erect the superstructure with its very considerable amounts of temporary towers and cable supports, and the meticulous and time consuming control of the geometry of the arch ribs and orthotropic upper deck during the entire erection and especially during jacking for closure. The Consultants are of the opinion that the through cantilever could be built in the least overall construction time, exceeded by the suspension bridge and the stiffened tied arch in that sequence.

No falsework in the river would be required for the erection of the superstructure of any of the designs. In the cantilever bridge the anchor spans over land would be erected on falsework bents. The number required would depend on whether the contractor elected to start the erection at each main pier using a balanced erection procedure. After the anchor spans are in place the entire main span could be erected as cantilevers from both main piers and closed at the center in a normal manner.

In the suspension bridge the cables would be erected first (either with the wires spun in place or as premeasured, pre-wrapped parallel wire strands) followed by erection of the stiffening trusses and roadway decks which are suspended from the cables. No falsework would be required in either the side or center spans.

The erection of the stiffened tied arch bridge would be accomplished by balanced erection starting at the main piers using one or more falsework bents under each side span as required. Temporary towers at the main piers perhaps reaching to a height of 400 feet above ground together with supporting cables would be necessary. Accurate control of the geometric positions and alignment of the ribs and orthotropic deck would be necessary during erection as well as during the jacking at six points for closure (as previously described). Precise control of shop procedures during the fabrication of the orthotropic deck to prevent warping would also be necessary. The size and weight of the floor and girder units would depend upon shipping facilities and the capacity of the erection equipment that would be employed. The number of floor and girder units would greatly influence the extent of the work done in the field during erection and the time required to accomplish it. Precise positioning and control of the welding of the units in the field would also be required.

With regard to the estimated time required to prepare final designs and contract plans and specifications for the three designs included in this report, there would be no difference in time between Design S-1 and S-3 despite the more complex problems of design analysis, development of unusual structural details, and of erection. It is assumed that separate substructure and superstructure contracts would be awarded in each case. Including the initial time required for preparation and for obtaining final approvals, the plans and specifications for the substructure contract would require about six months, unless utility relocations which might be included in the contract should be more complex and time-consuming than are presently anticipated. An additional five months would be required to complete the plans and specifications for the superstructure contract. This difference in time between contracts would be compatible with the construction schedule because the substructure contract would need about that much lead time over the award of the superstructure contract.

For Design S-2, eight months would be required to prepare the substructure contract plans and specifications because of the complexity of the cable anchorages. As in the case of Designs S-1 and S-3, the superstructure plans and specifications would require an additional five months.

ESTIMATES OF CONSTRUCTION COSTS - 1964 PRICE LEVELS

Tables 1, 2 and 3 record the estimated total costs of construction for the three designs investigated in this supplementary study based on 1964 price levels. Table 5 is a summary of the estimates of cost for all designs that have been studied and includes those that appear in the 1964 report.

As in the 1964 report, the estimates given in the tables which follow cover only the bridge structure including roadway, navigation and airways lighting between the study limits that have been designated. Excluded are the following costs:

1. Signs on the structure
2. Right-of-way
3. Demolition
4. Restoration of buildings or waterfront structures
5. Permanent or temporary street, railroad, and utility relocations
6. Maintenance of railroad or highway traffic
7. Engineering, legal, or administrative services
8. An allowance for contingencies

Tables 1 through 3 show detailed substructure and superstructure estimates for Designs S-1, S-2 and S-3 in which the estimated quantities of the individual items are listed together with the estimated unit prices. In the interest of simplicity in the presentation of these tables, some items such as cofferdams and lighting have been shown as lump sum costs rather than as listings of the individual items which make up the work.

The unit prices for 1964 levels used in preparing the estimates are the result of considerable study, and represent the Consultants' best information based on bids received on similar projects. Additional information on the steel unit prices was obtained from discussions with two of the principal steel companies, who were furnished the steel quantities by the Consultants. These companies made their recommendations after study of the details of all designs, taking into consideration erection methods, types of structures, and site location. The concrete and foundation excavation unit prices are average figures.

It should be noted that the cost of lighting includes only the cost of conduit, wiring, and the light units, many of which are mounted on brackets attached to the steel framing rather than on standards. Costs of transformers, feeders, controls, and other similar items necessary for the entire bridge are assumed to be included in the costs of the approaches.

An allowance for contingencies has not been included since it seems more appropriate to add such an allowance to the overall cost of the project, including those items of cost which have been omitted from these estimates. Although the estimates are considered adequate, some allowance for unforeseeable factors should be made. An allowance of five percent of the construction costs given in this report is considered adequate.

In the 1964 report most of the designs that were studied were based on a 650-foot main span with both main piers in the river. The high cost of these piers and the disadvantages arising from interference with navigation both during and after construction indicated to the Consultants that a design that would span the main river should be investigated. Accordingly, Design 6 was included in the report. These supplementary studies of three long-span designs have caused the Consultants and the two principal steel companies who provided information for both reports to re-examine the costs of Design 6. This new study has shown that the unit prices used in the 1964 report were too low as of 1964 price levels. For this reason, a revised estimate of Design 6 is presented in Table 4.

Table 1

Fremont Bridge, Portland, Oregon

ESTIMATED CONSTRUCTION COSTS - 1964 Price Levels
DESIGN S-1 - STIFFENED ~~TIED~~ ARCH WITH ORTHOTROPIC UPPER DECK
(Main Span Unit 475 - 1,350 - 475 feet)

Substructure				
Foundation Excavation	39,650 c.y.	@	$ 6.00	$ 238,000
Tremie Concrete	11,000 c.y.	@	30.00	330,000
Footing Concrete	15,200 c.y.	@	40.00	608,000
Concrete Above Footings	8,650 c.y.	@	60.00	519,000
Reinforcing Steel	1,250,000 lb.	@	0.12	150,000
Steel H-Piles	62,200 l.f.	@	12.00	746,000
Cofferdams	l.s.	@		510,000
Structural Carbon Steel	160,000 lb.	@	0.40	64,000
			Total	$ 3,165,000
Superstructure				
Carbon Steel (Girders, Bracing, Floor Systems, and Bearings)	14,950,000 lb.	@	$ 0.36	$ 5,382,000
Low Alloy Steel (Girders and Arch Ribs)	7,460,000 lb.	@	0.38	2,835,000
Corrosion-Resistant Steel Deck Plate	3,520,000 lb.	@	0.41	1,443,000
High Strength Steel	11,620,000 lb.	@	0.40	4,648,000
Ropes and Sockets	120,000 lb.	@	0.80	96,000
2-1/2 in. Asphalt Wearing Course	2,030 tons	@	12.00	25,000
Deck Concrete	4,180 c.y.	@	90.00	376,000
Reinforcing Steel	840,000 lb.	@	0.12	101,000
Three-Ply Membrane Waterproofing	14,600 s.y.	@	3.50	51,000
Bridge Railing	10,540 l.f.	@	4.50	48,000
Roadway and Navigation Lighting	l.s.			50,000
			Total	$15,055,000
			Total Cost	$18,220,000

Table 2

Fremont Bridge, Portland, Oregon

ESTIMATED CONSTRUCTION COSTS - 1964 Price Levels
DESIGN S-2 - EXTERNALLY ANCHORED SUSPENSION BRIDGE
(Main Span Unit 590 - 1,180 - 590 feet)

Substructure				
Foundation Excavation	87,700 c.y.	@$ 6.00		$ 526,000
Tremie Concrete	27,300 c.y.	@ 30.00		819,000
Footing Concrete	43,800 c.y.	@ 40.00		1,752,000
Concrete Above Footings	54,200 c.y.	@ 52.00		2,819,000
Reinforcing Steel	3,120,000 lb.	@ 0.12		375,000
Structural Carbon Steel	1,500,000 lb.	@ 0.30		450,000
Structural Low Alloy Steel	1,820,000 lb.	@ 0.35		637,000
Steel H-Piles	51,200 l.f.	@ 12.00		615,000
Cofferdams	l.s.			980,000
			Total	$ 8,973,000
Superstructure				
Carbon Steel (Trusses, Bracing, Floor System, Towers)	15,000,000 lb.	@ $0.305		$ 4,575,000
Low Alloy Steel (Trusses, Towers and Girders)	8,330,000 lb.	@ 0.325		2,707,000
Cables	4,410,000 lb.	@ 0.65		2,867,000
Cable Wrapping	100,000 lb.	@ 1.37		137,000
Castings	710,000 lb.	@ 0.75		533,000
Deck Concrete	7,900 c.y.	@ 90.00		711,000
Reinforcing Steel	1,580,000 lb.	@ 0.12		190,000
Bridge Railing	11,200 l.f.	@ 4.50		50,000
Roadway and Navigation Lighting	l.s.			50,000
Elevators in Towers	l.s.			50,000
			Total	$11,870,000
			Total Cost	$20,843,000

Table 3

Fremont Bridge, Portland, Oregon

ESTIMATED CONSTRUCTION COSTS - 1964 Price Levels
DESIGN S-3 THROUGH CANTILEVER TRUSS
(Main Span Unit 575 - 1150 - 575 feet)

Substructure					
Foundation Excavation	33,400	c.y.	@	$ 6.00	$ 201,000
Tremie Concrete	11,050	c.y.	@	30.00	332,000
Footing Concrete	10,400	c.y.	@	40.00	416,000
Concrete Above Footings	20,750	c.y.	@	60.00	1,245,000
Reinforcing Steel	2,350,000	lb.	@	0.12	282,000
Structural Carbon Steel	140,000	lb.	@	0.40	56,000
Steel H-Piles	66,400	l.f.	@	12.00	797,000
Cofferdams		l.s.			490,000
				Total	$ 3,819,000
Superstructure					
Carbon Steel (Trusses, Bracing and Floor System)	20,170,000	lb.	@	0.345	$ 6,959,000
Low Alloy Steel (Trusses)	2,050,000	lb.	@	0.355	727,000
High Strength Steel (Trusses)	9,320,000	lb.	@	0.385	3,588,000
Shear Connectors		l.s.			80,000
Deck Concrete	7,430	c.y.	@	90.00	669,000
Reinforcing Steel	1,490,000	lb.	@	0.12	179,000
Bridge Railing	10,660	l.f.	@	4.50	48,000
Roadway and Navigation Lighting		l.s.			50,000
				Total	$12,300,000
				Total Cost	$16,119,000

Table 4

Fremont Bridge, Portland, Oregon

REVISED ESTIMATED CONSTRUCTION COSTS - 1964 Price Levels
DESIGN 6 - CONTINUOUS TIED ARCH TRUSS
(Main Span Unit 530 - 1135 - 530 feet)

Substructure				
Same as in 1964 Report				$ 3,023,000
Superstructure				
Carbon Steel (Trusses, Bracing and Floor System)	23,071,000 lb.	@	$ 0.30	6,921,000
Low Alloy Steel (Trusses)	4,868,000 lb.	@	0.34	1,947,000
High Strength Steel (Trusses)	7,860,000 lb.	@	0.38	2,987,000
Deck Concrete	7,230 c.y.	@	90.00	651,000
Reinforcing Steel	1,446,000 lb.	@	0.12	173,000
Bridge Railing	10,220 l.f.	@	4.50	46,000
Roadway and Navigation Lighting	l.s.			50,000
			Total	$12,775,000
			Total Cost	$15,798,000

Table 5
Fremont Bridge, Portland, Oregon

SUMMARY OF ESTIMATED CONSTRUCTION COSTS - 1964 Price Levels

Design No.	Type	Main Span Lengths (feet)	Sub-structure	Super-structure	Total
1	Through Cantilever Truss	455 - 650 - 455	$6,039,000	$ 7,403,000	$13,442,000
2	Continuous Tied Arch Truss	455 - 650 - 455	6,459,000	8,165,000	14,624,000
4	Single Span Tied Arch	650	6,710,000	8,337,000	15,047,000
3	Self-Anchored Suspension Bridge	455 - 650 - 455	6,459,000	9,055,000	15,514,000
6	Continuous Tied Arch Truss	500 - 1,135 - 500	3,023,000	12,775,000	15,798,000
S-3	Through Cantilever Truss	575 - 1,150 - 575	3,819,000	12,300,000	16,119,000
5	Orthotropic Deck Plate Girders	490 - 650 - 490	6,029,000	10,960,000	16,989,000
7	Orthotropic Deck Plate Girders	500 - 1,135 - 500	No Detailed Estimate		17,500,000 to 18,000,000
S-1	Stiffened Tied Arch with Orthotropic Upper Deck	475 - 1,350 - 475	3,165,000	15,055,000	18,220,000
S-2	Externally Anchored Suspension Bridge	590 - 1,180 - 590	8,973,000	11,870,000	20,843,000

ESTIMATES OF CONSTRUCTION COSTS - 1965 PRICE LEVELS

Although there is no evidence of an appreciable increase in the costs of bridge substructures, there is evidence of an increase in the costs of steel superstructures during the past year. This increase is estimated to be approximately seven and one-half percent. It is due largely to an increase in the cost of shop labor and a significantly larger increase in the cost of labor for field erection. The latter increases are due in part to larger fringe benefits and, because of a scarcity of erection labor, the need to provide a certain amount of overtime at premium wages. Moreover, the uncertainties of a possible strike in the steel industry later this year and the possibility of further increases in wage rates must be considered in the estimates.

Table 6, which follows, shows the adjusted costs of the various designs—including those in the 1964 report—for this increase in steel prices. It should be noted that Designs S-1, S-2, and S-3 are based on overall lengths of main bridge that are longer than those in the 1964 report. For comparative purposes, a deduction amounting to approximately $250,000 in the cost should be applied to the estimates for those designs.

Table 6

Fremont Bridge, Portland, Oregon

SUMMARY OF ESTIMATED CONSTRUCTION COSTS - 1965 Price Levels

Design No.	Type	Main Span Lengths (in feet)	Sub-structure	Super-structure	Total
1	Through Cantilever Truss	455 - 650 - 455	$6,039,000	$ 7,891,000	$13,930,000
2	Continuous Tied Arch Truss	455 - 650 - 455	6,459,000	8,711,000	15,170,000
4	Single Span Tied Arch	650	6,710,000	8,890,000	15,600,000 — 15,887,000
3	Self-Anchored Suspension Bridge	455 - 650 - 455	6,459,000	9,601,000	16,060,000
6	Continuous Tied Arch Truss	500 - 1,135 - 500	3,023,000	13,667,000	16,690,000
S-3	Through Cantilever Truss	575 - 1,150 - 575	3,819,000	13,141,000	16,960,000
5	Orthotropic Deck Plate Girders	490 - 650 - 490	6,029,000	11,721,000	17,750,000
7	Orthotropic Deck Plate Girders	500 - 1,135 - 500	No Detailed Estimate		18,500,000 to 19,000,000
S-1	Stiffened Tied Arch with Orthotropic Upper Deck	475 - 1,350 - 475	3,165,000	16,135,000	19,300,000
S-2	Externally Anchored Suspension Bridge	590 - 1,180 - 590	8,973,000	12,677,000	21,650,000

AESTHETIC CONSIDERATIONS

As stated in the 1964 report, the double-deck construction of the Fremont Bridge creates a special aesthetic problem in bridge design because the two decks appear as heavy parallel curving lines of steel and concrete and the light and shadows accentuate the deck lines. It was further stated in that report that the selection of a particular design because of its appearance is so much a matter of individual opinion and taste that evaluation from this point of view alone becomes difficult.

There seems to be no doubt that for this bridge the designs having the long main spans (6, 7, S-1, S-2 and S-3) are superior in appearance as well as having other advantages by reason of spanning the entire river. In all probability the double deck construction is an important factor in the superior appearance of the longer span designs. An important factor in the appearance of those designs having a 650-foot main span is the length of the side spans which, because of railroad tracks and other topographical features along the banks of the river, are too long in proportion to the length of the main span to permit the most pleasing appearance. This is especially noticeable in Designs 1, 3, and 5. It is somewhat less noticeable in Design 2 but the disadvantage has been partly overcome in Design 4 by the use of an additional side span at some sacrifice in total cost.

Design 1, although the least costly, is probably the least attractive of the designs having a 650-foot main span. With due regard for the unique and striking appearance of Design 5 and for the opinion of the architect who created it, the Consultants are of the opinion that Design 2 is the most attractive of the 650-foot main span designs. It is second in order of cost.

Of the five designs having main spans varying from 1,135 feet to 1,350 feet, S-1 is considered to be the most pleasing in appearance but it has certain disadvantages which are discussed elsewhere in this report. S-2 is perhaps second in order of appearance but is the most costly of all by quite a large margin. Design 7 was presented in the 1964 report by the Consulting Architect. It is superior in appearance to Design 5, its shorter span counterpart, and, like it, is unusual and striking in appearance.

Design 6, in the opinion of the Consultants, is an attractive design. With its high rise central arch span it would, however, dominate the surrounding terrain and it has the appearance of being a massive though attractive structure.

The Consultants are well pleased with the appearance of the long-span through cantilever truss, S-3. The increase in panel length has resulted in fewer members and a more open appearance than was possible in Design 1. Its outline is well adapted to the double-deck construction and it suggests a rugged strength and rigidity which is not suggested in Designs S-1 and 7. Overall it has a more slender outline than Design 6. The breaks in the line of the lower chord at the main piers, made possible by the longer main span without encroaching on the navigation clearance, have also improved the appearance, in the opinion of the Consultants, who favor the truss outline shown on Fig. 7.

CONCLUSIONS AND RECOMMENDATIONS

All of the designs submitted herein and in the 1964 report are practicable from an engineering viewpoint in that they can be designed and constructed. From the Consultant's aesthetic point of view, the designs having the greatest merit are Designs S-1, S-2, S-3, 6 and 7. From the viewpoint of cost, the short span through cantilever truss (Design 1) is the cheapest and provides the basis of cost comparison.

Although it is pleasing in appearance, the Consultants reject Design S-2, the externally anchored suspension bridge, on the basis of its high cost, which is 55 per cent greater than that of Design 1 and is about $2 million greater than its nearest competitor in cost, Design S-1, even after applying an adjustment in cost for its greater overall length.

Although it can be considered as having a superior appearance, the Consultants also reject Design S-1, the stiffened tied arch with orthotropic upper deck. In addition to the higher cost, amounting to about $2.3 million above that of a trussed design having approximately the same main span length, this design can be expected to be relatively flexible under the heavy live loads from eight lanes of traffic on two decks and under wind and earthquake forces. In addition, it has like Designs 5 and 7, an orthotropic deck. A completely satisfactory solution for the wearing surface on an orthotropic deck has not yet been found. (See pages 18 and 19 of 1964 report.)

In the opinion of the Consultants, the choice for the recommended design lies between Designs 6 and S-3. There is little difference in cost. In fact, it is so small as to be within the limits of accuracy in estimating at this preliminary stage of the engineering. The superstructure of S-3 costs a little less than that of 6 but the substructure of 6 costs less than the substructure of S-3 because the side span trusses in Design 6 are deeper and the piers are shorter and therefore have less dead load weight and small overturning moments under lateral and longitudinal loads. The Consultants prefer Design S-3, the through cantilever truss design-particularly the truss profile shown in Fig. 7—and recommend that it be adopted.

Re-examination of the cost of the structural steel in Design 6 that was presented in the 1964 report has resulted in a substantial increase in the cost as of 1964 price levels. (See Table 7 in 1964 report and Table 4 in this supplementary report.) This increase has altered the relative economics of the short and long main span designs because the long span designs contain much more structural steel. The estimated increase of approximately seven and one-half per cent in price level of structural steel between 1964 and 1965 further alters the relative economics of the short and long span designs. The Consultants have had to rely to a greater than usual degree upon data obtained from the steel companies in estimating the costs of these uncommon designs. The decision to re-examine the costs of Design 6 originated with the steel companies and the Consultants have no reason to question the need for it.

A comparison between the costs of the cheapest design (Design 1) and the recommended design (Design S-3) is in order. Both are through cantilever truss designs. Their estimated costs are \$13,930,000 and \$16,960,000 respectively—in round numbers a difference of \$3 million. However, two adjustments in comparative cost should be made. Design S-3 is longer by 80 feet; therefore, the cost of 80 feet of double deck approach spans must be added to Design 1. This is estimated to be \$250,000. In addition the Consultants regard the fender protection presently provided and as shown on the Permit Application for the main piers of all designs having 650-foot main spans as being inadequate. They continue to be of the opinion that much heavier fenders will be found necessary upon final design and that the increase in cost of such fenders would be at least $350,000 for Designs 1 through 5. In addition, maintenance costs for the fenders can be expected to be quite high in future years. With these two adjustments the difference in cost is reduced to $2.4 million. A further reduction in the relative cost could be achieved by the use of open steel grating for the lower deck between main piers because the large reduction in dead load would have a greater effect on the longer main span. The Consultants have previously and do now recommend open steel grating in that location for all designs except Designs 5 and 7 where both the upper and lower decks are of orthotropic construction. If used in the recommended location the savings in cost in all designs would be large.

It should be noted that if the $600,000 adjustments in cost are applied to Design 2, the continuous tied arch truss with a 650-foot main span, the increase in cost of Design S-3 over Design 2, so adjusted is $1.2 million.

Design S-3, the through cantilever truss with a 1,150-foot main span, is recommended for the following reasons:

1. Elimination of the main piers in the river will remove all hazards to navigation both during and after construction, and will permit free and uninterrupted use of the wharves along the river banks. Also eliminated is the need for expensive and unsightly fenders and the future costs of maintaining them. Without piers in the river the time of construction would be reduced.

2. A bridge that is much more pleasing in appearance can be obtained at a cost of $2.4 million more than the cost of the cheapest design. This differential is twice that of the best appearing design having a 650-foot main span (Design 2) but is only one-half of the difference in cost between the cheapest design and the design having the best appearance (Design S-1). Design S-3 therefore appears to be a very satisfactory compromise between cost and appearance.

3. Design S-3 is a type that has a long history of successful usage and application to bridge crossings. It is a rugged, rigid type well adapted to the heavy dead and live loads and is entirely determinate in all respects. It is the easiest to erect and requires very little additional steel for erection purposes.

The same comments and reasons also apply to Design 6, the continuous tied arch truss and the difference in cost between Designs S-3 and 6 is so small as to be negligible at this stage of the work. However, in view of these further studies the Consultants prefer Design S-3 and recommend that it be adopted.