THE DESIGN OF AN ALL-WELDED 120' SPAN, TWO LANE, DECK HIGHWAY BRIDGE

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By

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INTRODUCTION

In the years preceding World War II welding was considered a structural outcast. The several committees and associations responsible for the standard specifications and procedures viewed welding as adequate for minor structural fabrication, where stresses were for all practical considerations predominately static. The dynamic load properties of welded connections were untested, and the construction field felt a vague understandable fear of its use in multi-million dollar buildings and bridges.

Today, the field testing of welding is over. Military and industrial application of welding to our gigantic war machine needs gave convincing evidence to engineers that a welded joint, properly understood, could do its job cheaply and successfully. Now all that remains is to overcome the inertia of the specifications and codes.

In the latest printing of the American Association of State Highway Officials' Standard Specifications for Highway Bridges (1944), welding is classified as permissible on incidental parts of the structure only. Warn the specifications, "Welding is not recommended in main members or their connections where the failure of the weld would endanger the stability of the structure." What a blow to the proponents of welding! While these A.A.S.H.O. standards serve as a guide for most highway bridge construction, some states in the years since 1940 have recognized the value of welding as an economic fabrication medium and have assumed responsibility for the construction of several welded highway bridges. California and New York, with a great number of miles of highways and

numerous bridges to accommodate them, have been leaders in the promotion of the welded structure.

At this time the most popular application of bridge welding technique is to the deck girder and the rigid frame types. These are all of moderate spans in the neighborhood of sixty to eighty feet, seldom exceeding 100 feet. Both of these bridge types present a neat unobstructed roadway for the motorist utilizing them. In elevation, also, they tend to give a pleasing architectural effect if such an effect is considered in design. For this reason their use is extensive at grade eliminations and for suburban stream crossings. When considered architecturally, the fundamental cleanliness of shape and joints of a welded structure often eliminate the necessity for a complex exterior veneer of stone or concrete to dress up the bridge.

An additional economic advantage is offered by the use of a composite steel beam and concrete slab. In this design method the concrete floor slab of a deck-type bridge acts as a part of the compression flange of each of the welded girders. All that is required for such composite action is an adequate means of resisting the shear between the steel beam and the concrete. Shear keys, fastened to the beam flange and imbedded in the concrete, are the answer. So far, riveted shear keys have not proved satisfactory; all their difficulties have been overcome by use of a welded shear key.

Considering these facts, it is apparent that a bright future exists for the welded bridge. It can successfully compete with riveted bridges using old-style design and materials that were developed for the peculiar

needs of riveted work. And, hamstrung by many of the old specifications, it can still prove itself a cheaper, better-looking, longer-lasting bridge.

Perhaps the welded bridge could be much better yet, if it were designed from scratch as a welded structure, not just a bridge for which welds are substituted for rivets. This is the current thought that drifts through the industry today. A good many technical articles and several competitions have stressed the desirability of breaking away from the old patterns of riveted construction. The optimum welded bridge may have vastly different structural characteristics than that which is considered correct today. Only design investigation of the remotest possibilities in all their myriad combinations will produce the answer.

In an effort to advance one step toward that goal this thesis has been undertaken.

The design of a welded bridge, discarding the familiar specifications, procedure, and materials, is a tedious undertaking. As a steadying influence, the authors elected to design the structure according to the competitive restrictions of the James F. Lincoln Arc Welding Foundation's Welded Bridges of the Future award program for 1949. It was felt that if the finished design had sufficient merit it could be submitted as a contest entry with the performance of the additional work required for detailed drawings and cost estimates.

For this design the bridge was classified as a two-lane deck highway bridge supported on piers 120 feet apart. The design of the piers or abutments was not considered. The requirements of the steel satisfied A.S.T.M.-A7-46 specifications. For proportioning members, determining

allowable unit stresses, and sizing welds used in the fabrication the 1947 edition of the Standard Specifications for Welded Highway and Railroad Bridges of the American Welding Society were followed. Contest specifications designated the loadings to be applied to the designed structure. Those other features of the design normally considered by specifications when designing a riveted bridge were evaluated with reference to a welded structure. The authors attempted to judge logically the necessity for the use of these old specifications. Any that should apply or seemed to apply were given consideration in the solution of the problem.

A deliberate attempt was made to incorporate into the bridge structure new or seldom used shapes. This has resulted in a structural system which, to the knowledge of the authors, is as yet untried. It is fundamentally a backbone-and-rib system, utilizing a single box-shaped girder as the primary vertebrae resisting shear, bending, and torsion. Transmitting loads to the girder are the eight pairs of ribs or floor beams. These beams have a wide-flange type cross-section and are cantilevered from the girder to give support to the floor system and its stringers. To economically utilize the metal in these floor beams their vertical longitudinal section has been designed as a wedge section, a section suggested only recently by A. Amirikian for use in mill buildings, shops, warehouses, and similar structures formerly requiring rigid frames and trusses to span large floor areas.

Architecturally, the thinnest possible elevation that was presented to the eye seemed to be desirable. The use of the wedge beams tends to accentuate this thinness from almost any position that the observer might

select. In order to break the otherwise long (120 feet) straight line of the bottom flange of the girder, the flange is shaped to a parabolic curve rising three feet at mid-span. This is also an economical solution for fixity of the ends of the girder, a condition incorporated in the girder design. As a result of these features, the bridge presents a graceful, willowy silhouette that is ideally suited for grade-crossing eliminations and stream crossings in suburban areas.

The valve of the torsional shear that would be applied was much smaller than first guesses had estimated. No previous literature was located to give any hint as to the magnitude of this shear and early in the design there was some doubt in the authors' minds as to the ability of a reasonable girder section to resist the maximum torque condition.

A modern two-lane 24-foot highway is assumed to be served by the bridge. As recommended by safety considerations, the bridge roadway is widened to 26 feet between curbs and 29 feet clear between guard railings. The curbs are flared at the approaches to prevent vehicles from striking the guard rail end posts. No sidewalks were designed, although they may be installed without increasing the size of any of the present main structural members. Occasional pedestrians may cross by using the 18 inch curb top.

By trying small scale elevations of the bridge and using different panel lengths for each, a seven-panel design was chosen as most pleasing architecturally. The standard panel is 17 feet. To overcome the shortening illusion of the solid vertical abutment wall upon the end panels, the end bearing connection was placed beyond the last floor beam.

A light-weight floor system that utilizes welding to fasten it to

the stringers is the U. S. Steel Company's Armored I-Beam-Lok. It is a steel grid floor filled with low-grade concrete. It is tack welded to each stringer to give a structure strong enough to resist the lateral forces applied by wind and lateral loads. The floor stringers run longitudinally at an assumed spacing of 5 feet 3 inches. By using longitudinal stringers any permanent deflection of the floor between stringers will not destroy the smooth riding quality built into the floor. An attempt was made to fasten the floor and the stringer together to act as composite beams. Investigation proved that this was impractical. Instead, each stringer is designed as continuous over the seven panels.

The roadway loadings as specified by the contest specifications are identical to the H20-44 loadings given by the A.A.S.H.O. in their 1944 specifications.

Field erection of this bridge is felt to be an additional advantage. If conditions permit, the girder will be completely shop assembled and transported to the site in one piece. There it will be installed and used as the base for all further erection. With the girder up the floor beams are erected in pairs by use of a wide flange erection piece that straddles the top of the girder between the beams and holds them in place for welding. Before welding the stringers rest on the top flanges of the beams without the use of special fastenings. As soon as the stringers are set, the steel grid may be laid and immediately used as a working platform for the remainder of the erection and finishing.

Looking over the completed design, the authors feel that they have accomplished at least in some measure the following: First, the

sections of the cantilever beams, the girder, and the floor system are particularly adapted to welding practices. Second, complete use is made of the material. Each member was proportioned throughout its length, with due regard to costs, for maximum allowable stress. Third, the total dead load of the structure is kept to a minimum and compares favorably with the designed value of live load that it will support. Fourth, the bridge is made of simple components that fit together in a simple manner.

> "If a work such as a bridge be well composed constructively, whatever may be the constituent material or materials employed, and whatever may be the kind of construction, it can hardly fail to be an agreeable object for it will certainly possess the essentials to beauty in architectural composition, simplicity, and harmony. The introduction of anything not necessary to the construction, the omission of what is requisite, or the substitution of a bad expedient for a good one, will assuredly tell injuriously upon the eye, how incompetent soever the observer may be to determine the cause of the defect, or even in what the defect may consist."

<u>Bridge Architecture</u>

DESIGN PROCEDURE

By designing the bridge "from the top down," that is, the floor system first and the girder last, it was possible to eliminate most of the guess work involved in choosing the dead loads that were transmitted to each structural member. Only the dead weight of the member being investigated needed to be assumed for its preliminary design. Therefore, the design proceeded in this order:

Curb and Guard Rails

Floor System

Floor Stringers

Cantilever Beams

Main Girder

Bridge Seat

Ordinarily, the design of the welded connections was not considered at the time the member was investigated. These weld sizes and details of their application were made a part of the detail drawings and design of them was done at that time.

In the following discussion reference will be made to all the assumptions made in the design of each structural component. However, only that assumption that finally ruled the design method will be included in the design computations that are tabulated in the next section of this paper.

When reading through the explanation of the design procedure, it may be helpful to refer to the drawings in the appendix. Adequate details have been prepared for all portions of the structure.

CURBING AND GUARD RAILS

It was felt that adequate attention is too often lacking in the design of curbs and railings. Well chosen proportions for the railing may add much to the fundamental gracefulness of the slim horizontal bridge lines. Conversely, a hastely accepted railing design may upset the whole balance of the bridge.

Both curbs and railings were designed in accordance with the American Institute of Steel Construction folder, "Bridge Railings, Their Design and Construction." The distance between curbs is two feet greater than the approaching highway pavement width. A curb nine inches high is used, which will deflect a car and yet not catch fenders or running boards. To provide a substantial curb a horizontal force of 500 pounds per lineal foot was applied at the top of the curb.

The inside railing faces are set back eighteen inches from the curb line. A smooth railing surface is presented to traffic. On the lower rail a horizontal design force of 500 pounds per lineal foot was applied. The two top rails had a horizontal force of 150 pounds per lineal foot applied. A vertical force of 100 pounds per lineal foot was applied to each rail. The spacing of railings was chosen to give a height that did not harmfully obstruct the motorist's view. Yet it is adequate for safety and compliments the gracefulness of the bridge's elevation.

To balance the upper and lower portions of the bridge as viewed in elevation it was necessary to space the vertical railing posts on 8 foot 6 inch centers, which is equivalent to one-half a panel length. These posts are anchored to the exterior floor stringers at each panel point and mid-panel point.

A molding of light gage metal extends from the curb level to just below the cantilever beam's bottom flange on the outside of the railing and serves to finish the outside edge of the roadway.

FLOOR SYSTEM

A steel grid, concrete-filled, flooring was chosen for its advantages of lightness, strength, long life, economy, and ease of installation. The saving in weight is not a small item. The 5 inch depth chosen, with its 1 inch bituminous wearing surface, weighs only 60 pounds per square foot. If a normal concrete slab were laid, the depth including wearing surface would be at least 10 inches and would weigh 125 pounds per square foot. The saving in floor dead load is, therefore, in excess of 50 per cent, a considerable item when the floor area amounts to more than 3100 square feet as it does in this bridge. This reduced dead load results in a decrease in size of all other structural members, stringers, beams, and girder. Hence, a lighter, more graceful bridge is possible. Although an economic study is difficult for the authors to make, it is felt that the added cost of the steel floor system is more than balanced by a saving in concrete forms and weights of material supplied for the other structural components.

Ease of installation is an important advantage of the steel grid floor. It reaches the bridge site already cut to the proper lengths and widths and with openings cut as necessary for drains. It rests directly on the upper flanges of the stringers and is welded to them to provide a rigid network of steel at the floor level to resist lateral and longitudinal loads.

During erection the steel grid is laid and welded to the stringers. By using United States Steel Armored I-Beam-Lok, light gage metal form strips fit between the main beams of the flooring on its under side and

act as forms for the concrete and as a protection for the underside of the floor. These strips are shop assembled and welded to the floor system. The concrete may be poured from transit-mix trucks running onto the bridge over the steel grid.

The stresses resulting from wheel load concentrations on the floor system and the influence of the distribution of these loads on the moments studied has been based on the theory proposed by Prof. H. M. Westergaard, of the University of Illinois, and modified by the Bureau of Public Roads. These modifications were developed to simplify the design computations for the two types of moment conditions; first, for bridge floor slabs with main reinforcement parallel to the direction of traffic; and second, main reinforcement transverse to the direction of traffic. This second condition was chosen for the design.

In this bridge the floor was made continuous over several supports and is firmly welded to the stringer flanges to produce for practical considerations a fully restrained condition. To allow for some flexibility all stresses were computed for an end restraint of 75 per cent.

LONGITUDINAL FLOOR STRINGERS

To support the floor a system of longitudinal stringers was used. This allows the sag of the floor to be parallel to the traffic travel and does not hinder the smooth riding characteristics of the bridge. The normal span of the stringers between the cantilever beams is 17 feet. Any typical cross-section contains four stringers, two on each side of the centerline. They are spaced 7.25 feet and 12.50 feet, respectively, from the centerline. The main girder also acts as support for the floor, handling the loads over the center section of the traffic area.

Three alternative designs were considered. First, the feasibility of composite action of the stringer and a portion of the concrete in the floor immediately above was considered. By assuming the designed floor depth in combination with various beam sizes, stresses were computed by transformed section analysis. Since the concrete stresses had to be superimposed upon those stresses already existing because of slab action, the actual allowable stress in the concrete was low. This meant that the floor would have to be made thicker to increase the concrete area and the concrete stress available to resist this composite beam action. The gain in weight of the thickened slab could not be regained in a similar saving in weights of the four stringer sections. Therefore, this alternative was abandoned.

The second design was based on simple beam action for each stringer. This gave a reasonable section for the stringers and was kept in reserve in case the stringer-to-beam connection became unwieldy. After the design of the beams, it became apparent that this solution was not necessary.

The final alternative was to consider the stringers as continuous over the entire seven panel lengths of the bridge. Any splices are made where study showed that moment values were low. Normally this is considered to be about one-fifth of the panel length from any support. Using such a long continuous beam, it was uncertain whether the lane loadings or the truck loadings as specified would produce the greater moments. Investigation showed that moments over the supports were greater for the lane loadings. The mid-span moments, however, were greater for the truck loadings, and these moments ruled the design of the stringer.

A slight increase in weight per foot was allowed in order to reduce the stringer depth to a practical minimum. This increase amounted to 6 pounds per foot in the three center panels and 4 pounds per foot in the four exterior panels over the weights of the most economical sections. This is an over all weight increase of about 2200 pounds in the bridge. Interior and exterior stringers were made the same since the curb position allowed the same loads to come onto the exterior stringer as were used for interior stringer design. Ordinary wide-flange sections were chosen. The loads applied made such a section ideal in resisting the generated moments. The floor fits snugly on the top flange; the bottom flange rests upon the cantilever beam and is welded to it. No difficult framing was needed.

CANTILEVER FLOOR BEAMS

The use of the wedge beam section for the cantilever floor beams was chosen as the simplest of several shapes that offered themselves for this structural member. At first, a tapered box section was considered to be more attractive to the observer. However, this type of section had no structural advantages, and it had the important disadvantage of being an expensive fabrication and erection job.

Another consideration was the built-up section consisting of a web plate and two flanges, with the lower flange bent to some curved profile, most probably parabolic. This lower curve would add gracefulness to the underside of the bridge and would possibly conform to the requirements of the applied moments. This design was discarded because of the added cost of fabrication with only small saving in the weights of steel over the triangular wedge shape.

As previously mentioned in the early pages of this report, the wedge beam has the advantages of attractiveness, lightness, and economical use of metal to resist the shears and moments. A standard rolled wide flange section was chosen as the parent material. It was split along its web in a straight diagonal line for the full length of one cantilever beam. This diagonal cut was so proportioned that the webs are rejoined along the cut after one half of the beam is reversed end for end. With the proper design it was possible to make the section modulus curve of the newly welded wedge parallel to the required section modulus of the cantilever beam. And this fabrication requires but one flame cutting and one shop weld for each wedge beam.

It was feared that the wedge might not have the required stiffness

to resist excessive deflection tendencies. To investigate the deflection it was necessary to make a combined graphical and analytical solution of the beam by the slope deflection method. The deflection was not critical, indicating that the extra stiffness near the support more than compensated for the beam's flexibility near its free end.

MAIN GIRDER

For resistance to torsional shear the most efficient cross-section that could be applied to this bridge was the symmetrical box with rounded corners. However, the material in this cross-section also was called upon to resist transverse shear in the web portions and direct stresses in the flange portions. This led to the adoption of a box section with rounded corners that had very heavy flanges to develop the necessary moment of inertia to resist the bending moments, with only light web pieces to resist shear.

Preliminary studies indicated that the torsional shear would be small compared to the transverse shear and bending stresses. Consequently, the girder was designed for these latter forces only, and an allowance was made in proportioning the members to keep the stresses slightly below their limiting values. When these stresses were later combined by principal stresses with the torsional shear, the total stresses were still below their limits.

Rounded corners were used for two purposes. First, these corners, made with adequate radii, eliminate any concentration of the torsional stress as it flows around the cross-section. These concentrations when caused by square corners are not properly investigated, and leading authorities disagree on the increase to be assumed. Most allow a 150 to 200 per cent increase. Second, rounded corners eliminate the use of fillet welds at points of stress concentration, a condition prohibited by welding specifications.

The girder cross-section was designed for moment at the end supports and at the center of the span. At these points the steel was proportioned

so that one size of plate is used for both flanges and another size for both webs throughout the entire length of the girder. Any necessary increase in moment of inertia was handled by the parabolic curve of the bottom flange, which increased the depth of the girder from 5 feet at the mid-span to 8 feet at the supports. These design dimensions for the parabola were influenced by the appearance in elevation as well as by the requirements for moment of inertia. Fortunately, the two conditions did not contradict one another.

For negative moment over the supports 100 per cent fixity was assumed, for this gave the worst condition. For maximum positive moment at mid-span the rigidity was reduced to 75 per cent to allow for any small deformations of the bridge seat. In the first solution of the girder it was necessary to consider it as being of uniform cross-section throughout and to be simply supported. Under these assumptions four loading conditions were tested, all of which gave practically identical results. Then the value of moment chosen was modified according to the effect of end restraint on moments existing at the supports and at mid-span. These modified moments were designated as the trial design moments, from which the first proportioning of the girder was made.

With the girder dimensions chosen the moment of inertia of sections along the span was computed. The cube root of all these moments of inertia were plotted against distance along the girder. The resultant curve approached a parabola so that it was considered safe to use the Handbook of Frame Constants as published by the Portland Cement Association for calculation of the fixed end moments.

Torque conditions along the girder were examined for all conditions of unbalanced loading. From this inspection it was shown that the

torsional shear could vary from zero to one definite maximum, and that this same maximum could exist at any point along the girder cross-section, the maximum shear shifting as the maximum unbalanced loads shifted.

BRIDGE SEAT

Four sometimes incompatible functions had to be performed by the bridge seat in order that it resist the forces brought to it by the main girder. First, it was to carry the vertical thrusts to the pier or abutment. Second, it had to resist the end moments, that is, it must be rigid in the plane of the longitudinal centerline of the bridge. Third, it had to resist the transverse torque by being rigid in a transverse plane. Fourth, it was to allow for the expansion of the girder over the specified temperature ranges expected. It is believed that the joint as designed satisfies all these conditions and is still not overly complex.

DESIGN COMPUTATIONS

CURB AND RAILINGS

CURB

Assume a curb 9 inches high. Support the inside of the curb on the floor over the exterior stringer. Support the outside edge of the curb on a channel section running between railing posts.

Loads.

Horizontal -- 500 lbs. per foot of curb

Vertical -- Dead Load -- 169 lbs. per foot of curb

Live Load -- One front wheel of design truck or 4000 lbs. placed critically

Shear between curb and flooring.

H = 500 lbs. per foot

Steel area required to resist shear $= \frac{500}{13000} = 0.037$ sq. ins.

Channel section.

Assume $\frac{1}{2}$ of wheel load and $\frac{1}{2}$ of dead load are supported by channel and the remainder supported on exterior stringer.

Assume channel simply supported at each railing post.

Moments.

Dead Load. $M = \frac{wl^2}{8} = \frac{169 \times 8.5^2}{2 \times 8} = 762$ ft. lbs.

Live Load. $M = \frac{Pl}{4} = \frac{4000 \times 8.5}{2 \times 4} = 4250$ ft. lbs.

Impact. $\frac{50}{8.5 + 125} = 0.375$

use 0.30 $\quad 4250 \times .30 = 1275$ ft. lbs.

Total Moment. 6287 ft. lbs.

Required Section Modulus $= \frac{6287 \times 12}{18000} = 4.19$ cu. ins.

Use a 6 x 2 x 8.2 lb. channel; S = 4.3 cu. ins.

LOWER RAILING

Span. 8.5 ft.

Live Loads.

Horizontal -- 500 lbs. per foot

Vertical -- 100 lbs. per foot

Shear.

Horizontal -- $500 \times \frac{8.5}{2} = 2125$ lbs

Vertical -- $100 \times \frac{8.5}{2} = 425$ lbs

Moment.

Horizontal -- $500 \times \frac{8.5^2}{8} = 4510$ ft lbs

Vertical -- $100 \times \frac{8.5^2}{8} = 902$ ft lbs

Use f = 18000 psi

Required Section Modulus.

$S_h = \frac{4510 \times 12}{18000} = 3$ cu ins

$S_v = \frac{902 \times 12}{18000} = 0.602$ cu ins

Try a cross-section 6" x 2" x 3/16"

$$I_v = 2Ax^2 + 2\frac{b\,h^3}{12} = 2 \times 2 \times 3/16 \times 2.906^2 + 2 \times 3/16 \times \frac{6^3}{12} = 13.07 \text{ in}^4$$

$S_h = \frac{I_v}{c} = \frac{13.07}{3} = 4.35$ cu ins

$I_h = 2 \times 6 \times 3/16 \times 0.906^2 + 2 \times 3/16 \times \frac{2^3}{12} = 2.07 \text{ in}^4$

$S_v = \frac{2.07}{1} = 2.07$ cu ins

UPPER RAILING

Span. 8.5 ft.

Live Loads.

Horizontal -- 150 lbs. per foot

Vertical -- 100 lbs. per foot

Shear.

Horizontal -- $150 \times \frac{8.5}{2} = 638$ lbs

Vertical -- $100 \times \frac{8.5}{2} = 425$ lbs

Moment.

Horizontal -- $150 \times \frac{8.5^2}{8} = 1352$ ft lbs

Vertical -- $100 \times \frac{8.5^2}{8} = 902$ ft lbs

Use f = 18000 psi

Required Section Modulus.

$$S_h = \frac{1352 \times 12}{18000} = 0.9 \text{ cu ins}$$

$$S_v = \frac{902 \times 12}{18000} = 0.602 \text{ cu ins}$$

Try a cross-section 6" x 2" x 1/8"

$$I_v = 2 \times 2 \times 1/8 \times 2.938^2 + 2 \times 1/8 \times \frac{6^3}{12} = 8.8 \text{ in}^4$$

$$S_h = \frac{8.8}{3} = 2.93 \text{ cu ins}$$

$$I_h = 2 \times 6 \times 1/8 \times 0.938^2 + 2 \times 1/8 \times \frac{2^3}{12} = 1.47 \text{ in}^4$$

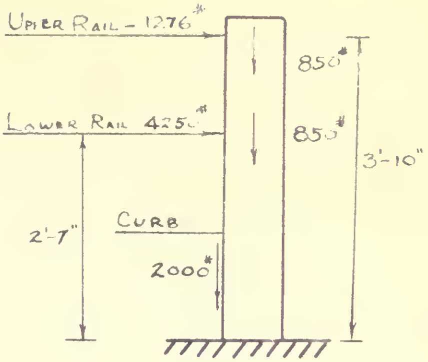
$$S_v = \frac{1.47}{1} = 1.47 \text{ cu ins}$$

RAILING POSTS

Span. (Cantilever) 3' - 10"

Live Loads.

See Figure at right.



Shear.

Horizontal -- 5526 lbs

Vertical -- 3700 lbs

Moment.

4250 x 2.584 = 11000 ft lbs

1276 x 3.83 = 4880 ft lbs

1700 x 0.167 = 284 ft lbs

Total = 16164 ft lbs

Required Section Modulus.

$$S = \frac{16164 \times 12}{18000} = 10.77 \text{ cu ins}$$

Try a cross-section $4\frac{1}{2}" \times 4\frac{1}{2}" \times \frac{1}{2}"$

Area = $4 \times 4\frac{1}{2} \times \frac{1}{2}$ = 9 sq ins Safe for shear.

$$I = 2 \times \frac{1}{2} \times 4\frac{1}{2} \times 2^2 + 2 \times \frac{1}{2} \times \frac{4^3}{12} = 25.6 \text{ in}^4$$

$$S = \frac{25.6}{2.25} = 11.35 \text{ cu ins}$$

FLOOR SYSTEM

WEARING SURFACE

Use a 1 inch wearing surface of concrete or of asphalt.

FLOOR SLAB

Use United States Steel I Beam Lok Armored slabs.

Lay flooring transverse to traffic flow.

Compute stresses by modified Westergaard theory.

Span.

Stringer spacing = 5.25 ft

Assume stringer flanges 8 inches wide.

Clear span = 5.25 ft − 8 ins = 4 ft 7 ins

Design span = $4' - 7'' + \frac{8''}{2}$ = 4 ft 11 ins

Assume a monolithic slab. Interior spans assumed to have a 75 per cent end restraint condition. Design a portion of the slab 1 foot wide.

Loads.

Dead Loads

Wearing Surface = $1 \times 1 \times \frac{1}{12} \times 150$ = 12.5 lbs per foot

Assume 5 inch I Beam Lok Armored = 47.0 lbs per foot

Total = 59.5 lbs per foot

Live Loads

Truck loading rules

1 front axle of 8000 lbs (4000 per wheel)

1 rear axle of 32000 lbs (16000 per wheel)

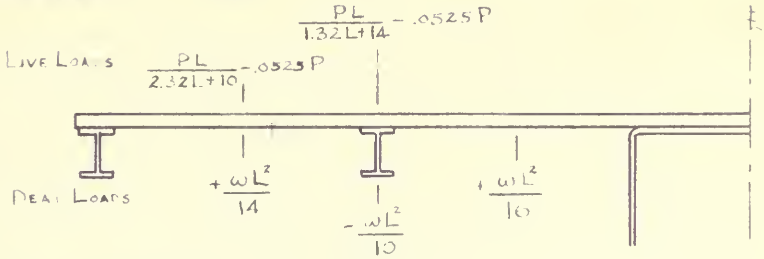
Impact Allowance

$I = \frac{50}{L + 125}$, where L is the design span in feet

$I = \frac{[illegible]}{[illegible] + 125} = [illegible].335$ Use a maximum I of [illegible]

Moments.

See sketch to right.



Positive moment

Dead load = $\frac{wL^2}{14} = \frac{[illegible] \times 4.917^2}{14}$ = [illegible] ft lbs

Live load = $\frac{PL}{2.32L + 10} - 0.0525P$

$\frac{16000 \times 4.917}{2.32 \times 4.917 + 10} - 0.0525 \times 16000$ = [illegible] ft lbs

Impact = 0.[illegible] x [illegible] = [illegible] ft lbs

Total = [illegible] ft lbs

Stresses resulting from positive moment

Section Modulus of compression concrete = [illegible].8 cu ins

Section Modulus of tensile steel = [illegible].02 cu ins

The above values are average values for I Beam Lok.

Concrete stress = $\frac{M}{S_c} = \frac{37[illegible]5 \times 12}{[illegible].8}$ = [illegible] psi

Steel stress = $\frac{M}{S_s} = \frac{[illegible] \times 12}{[illegible].02}$ = 15[illegible] psi

Low grade concrete is used to fill I Beam Lok. For design a value of n = 15 is used and the stresses allowed are: for concrete, [illegible] psi, and for steel, [illegible] psi. Concrete is assumed not to have any value in tension.

Negative Moment

Dead load = $\frac{wL^2}{10} = \frac{[illegible].5 \times 4.917^2}{10}$ = 144 ft lbs

Live load = $\frac{PL}{1.32L + 14} - 0.0525 P$

$\frac{16000 \times 4.917}{1.32 \times 4.917 + 14} - 0.0525 \times 16000$ = 3000 ft lbs

Impact = 0.30 x 3000 = [illegible] ft lbs

Total = 4044 ft lbs

Stresses resulting from negative moment

Section Modulus of compression concrete = 56.3 cu ins

Section Modulus of tensile steel = 2.54 cu ins

Concrete stress = $\frac{M}{S_c} = \frac{4044 \times 12}{56.3}$ = [illegible] psi

Steel stress = $\frac{M}{S_s} = \frac{4044 \times 12}{2.54}$ = 19000 psi

The steel stress is a little over the allowable value chosen but will be adequate within the design limits of the problem.

[illegible]

<u>[illegible]</u>

Span.

Floor-beam spacing = 17 feet

Design span = 17 feet

Loads.

Dead loads

Floor = 5[illegible] x 5.[illegible] = 31[illegible] lbs per foot

Assume stringer weight = 50 lbs per foot

Total = 36[illegible] lbs per foot

Live Loads

Truck loading rules over lane loading.

Place one 16,000 lb wheel at mid-span for maximum moment.

Impact Allowance

$I = \frac{50}{17 + 125} = 0.35$ [illegible] Use a maximum I of 0.30

Moments.

Dead load $= \frac{wL^2}{8} = \frac{36[illegible] \times 17^2}{8} = 13100$ ft lbs

Live load $= \frac{PL}{4} = \frac{16000 \times 17}{4} = 68000$ ft lbs

Impact = 0.30 x 68000 = 20400 ft lbs

Total = 101500 ft lbs

<u>[illegible]</u>

Span.

Design span = 17 feet

Loads.

All loads are the same as for first solution

Truck loading [illegible] lane loading. Truck in [illegible] panels [illegible]

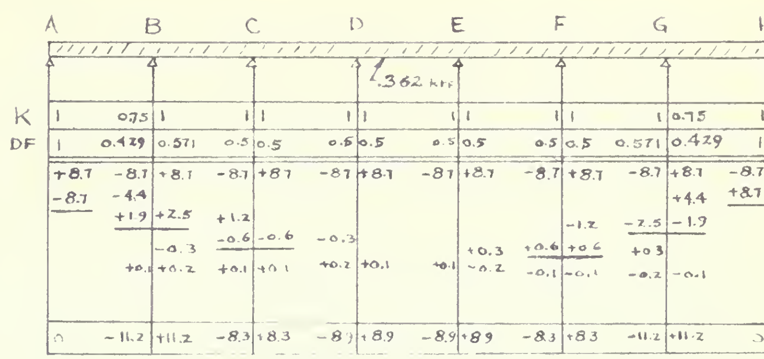
[illegible] spans were [illegible] distribution details.

Moments.

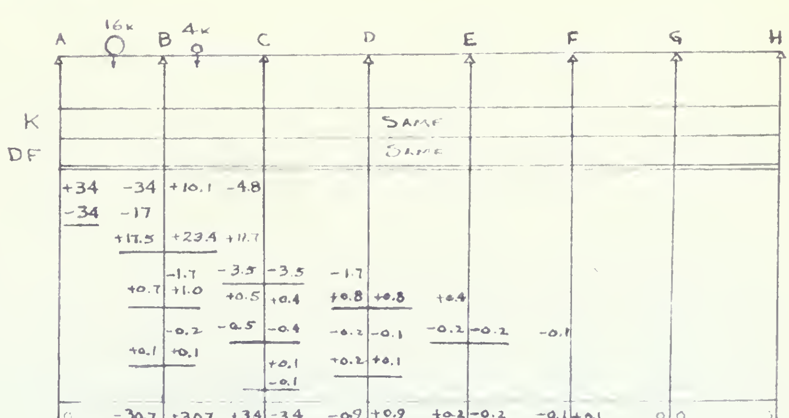
Moment distribution was used to solve the continuous beam. Lane loadings and truck loadings were solved for all positions of loads.

Solution is given for the maximum condition only. Maximum moments in other panels or at other joints are tabulated.

Dead loads



Live loads



Positive moments

Span AB

Dead load

[illegible] x 0.5 + .[illegible] x [illegible]

= 3670 ft lbs

Live load

0.19 x [illegible] = [illegible]2600 ft lbs

Impact

0.[illegible] x [illegible] = [illegible] ft lbs

Total = 77[illegible] ft lbs

Span BC

Total = 61[illegible] ft lbs

Span CD

Total = [illegible] ft lbs

Span DE

Total = [illegible] ft lbs

Negative moments, totals only

Support A = 0

Support B = [illegible]111 ft lbs

Support C = 4[illegible]50 ft lbs

Support D = [illegible] ft lbs

Shear.

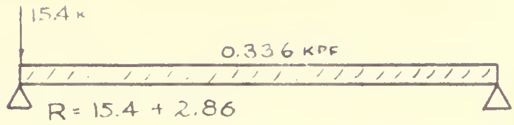
Assume [illegible] le [illegible].

Lane loading [illegible] over truck loading.

Assume that [illegible].5 feet of 1[illegible] foot lane loading and its concentrated line load are applied to stringer.

Dead load - 362 x [illegible] = [illegible] lbs

Live load



= 18,[illegible] lbs

Impact = [illegible] x [illegible] = [illegible]47[illegible] lbs

Total = [illegible] lbs

An examination of the moments developed by the loads under the two design conditions shows that a 24 per cent reduction in moment is accomplished by use of the continuous stringer.

Required Section Modulus.

Use a design moment of 77,050 ft lbs

$S = \frac{77050 \times 12}{18000} = 51.5$ cu ins

In order to keep depth at a minimum use a

12" Wide Flange 40 lb section; S = 51.9 cu ins

Most economical section is

16" Wide Flange 36 lb section

Web Shear Check.

Area web = 12 x 5/16 = 3.75 sq ins

Shear stress = $\frac{V}{A}$ = $\frac{2[illegible]}{3.75}$ = [illegible]140 psi

Allowable shear stress = 11000 psi

[illegible]

Span.

Assume web girder width = [illegible] feet

Design span = 15 - [illegible] = 1[illegible].5 feet

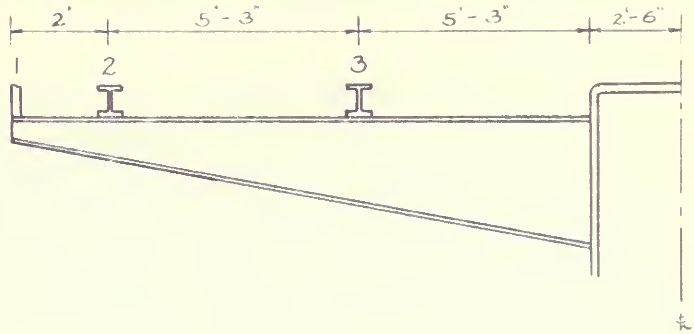
Loads.

For the following solution these abbreviations will be used:

Load 1 means any load applied at free end of beam.

Load 2 means any load applied by exterior stringer.

Load 3 means any load applied by interior stringer.



Dead Loads

Floor

Load 2 = 57.5 x 17 x 5.125 = [illegible]130 lbs

Load 3 = [illegible] x 17 x 5.25 = [illegible]550 lbs

Railings

Load 1 = weight of 8.5 feet

of railing = 175 lbs

Load 2 = same as above = 175 lbs

Curb

Load 1 = weight of concrete

curb and supports = [illegible] lbs

Load 2 = 500 lbs

Stringers

Load 2 = 40 x 17 = 680 lbs

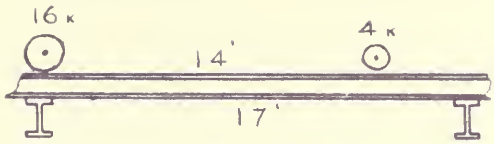
Load 4 = 40 x 17 = 680 lbs

No estimate made here of beam dead weight.

Live Loads

Truck loading ruled.

See sketch.



Stringer reaction = 16 + $\frac{4 \times 3}{17}$ = 16.7 kips

Impact Allowance

I = $\frac{50}{12.5 + 125}$ = 0.364 Use a maximum I of 0.30

Moments.

Moments are computed for each foot along the beam. Using a flexure stress of 18000 psi, the required section modulus at each foot is also found.

A complete computation is given for the moment solution at the beam support.

Moment at beam support

Dead load

Load 1 = 12.5(175 + 500) = 8440 ft lbs

Load 2 = 10.5(175 + 500 + 3180 + 680) = 47500 ft lbs

Load 3 = 5.75(5350 + 680) = 31700 ft lbs

Total = 87640 ft lbs

Live load

Truck

Load [illegible] = 1[illegible].5 x 10710 = 1[illegible] ft lbs

Load [illegible] = [illegible].2[illegible] x 10710 = [illegible]379[illegible] ft lbs

[illegible] line and cars

Load [illegible] = [illegible] x 12.5 + 1[illegible] = [illegible] ft lbs

Load [illegible] = [illegible] x [illegible].5 = [illegible]1[illegible] ft lb

Total = [illegible]66[illegible] ft lbs

Impact = [illegible] x [illegible] = 1[illegible] ft lbs

Total = [illegible]40[illegible] ft lbs

Section modulus [illegible] beam support

$$S = \frac{[illegible]40[illegible] \times 12}{18[illegible]} = [illegible]1 \text{ cu in}$$

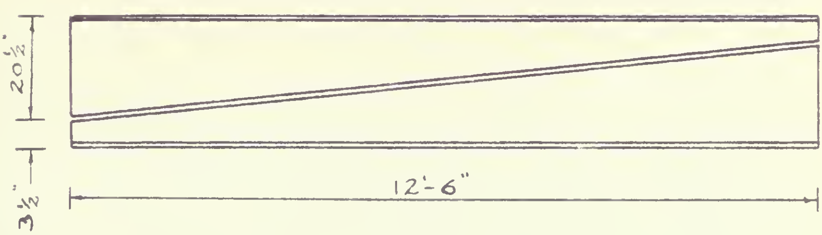
Distance from bridge center line	Dead load	Live load	Impact	Total	Section Modulus
3.5 feet	765[illegible]	307[illegible]	[illegible]500	476200	[illegible]18
4.5	[illegible]100	[illegible]5200	[illegible]0000	41[illegible]	276
[illegible].5	[illegible]400[illegible]	[illegible]1[illegible]	[illegible]3800	[illegible]51900	235
[illegible]	42[illegible]	1[illegible]	[illegible]100	[illegible]0100	194
7.5	31500	1[illegible]700	[illegible]200	227400	151
8.5	[illegible]300	1[illegible]	[illegible]300	1[illegible]00	124
9.5	1[illegible]000	1[illegible]170[illegible]	[illegible]500	151300	102
10.5	14[illegible]00	[illegible]700	[illegible]500	115300	77
11.5	9200	[illegible]7[illegible]	1[illegible]100	[illegible]300	[illegible]
12.5	4[illegible]00	[illegible]4000	10400	[illegible]9000	[illegible]3
14	[illegible]	1[illegible]00[illegible]	5900	[illegible]200	18
1[illegible]		1[illegible]	4[illegible]	[illegible]700	14

Wedge Beam Section.

The problem is to proportion a wedge beam whose section modulus at any transverse cross-section is equal to or slightly greater than that required.

Splitting a wide flange section longitudinally, reversing one half, and welding the webs together again results in an economical wedge beam.

Split a 24 WF [illegible] beam as shown:



Weld the split 24 WF [illegible] as shown:

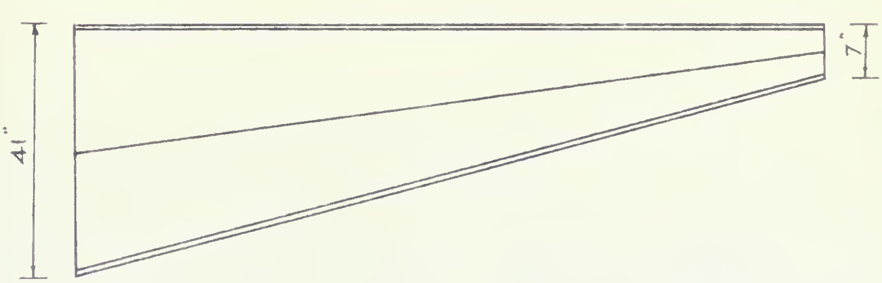


Table [illegible] Comparison of Actual and Required Section Moduli.

Distance from bridge centerline	24 WF 84 Moment of Inertia	24 WF 84 Section Modulus	Required Section Modulus
2.5 feet	[illegible] ins^4	[illegible]92 ins^3	[illegible]61 ins^3
4.5	[illegible]740	[illegible]24	276
6.5	[illegible]885	259	194
8.5	[illegible]486	202	124
10.5	1406	146	7[illegible]
12.5	671	[illegible]	[illegible]
15	[illegible]	4[illegible]	14

Check the increase in section modulus to handle the wedge beam dead load. Check at support only.

Moment = W x d = total wt x dist. to CG

= 84 x 12.5 x 4.77 = 5020 ft lbs

Section modulus = $\frac{5020 \times 12}{18000}$ = 3.35 cu ins

Shear.

Unsupported end

Dead load = 500 + 175 = 675 lbs

Live load = 1700 + 2000 = 3700 lbs

Impact = 0.30 x 3700 = 1110 lbs

Total = 5485 lbs

Web depth = 7 ins; web thickness = 0.47 ins

Shear stress = $\frac{5485}{7 \times .47}$ = 1670 psi

Under the exterior stringer

Dead load = [illegible] + [illegible] + 570 + [illegible] = [illegible] lbs

Live load = 10710 + [illegible] = [illegible] lbs

Impact = [illegible] x [illegible] = [illegible] lbs

Total = [illegible] lbs

Web depth = 12.5 ins; web thickness = 0.47 ins

$$\text{Shear stress} = \frac{[illegible]}{12.5 \times .47} = 5040 \text{ psi}$$

Under interior stringer

Dead load = 5270 + [illegible] + 360 + [illegible] = [illegible] lbs

Live load = [illegible] + 10710 = [illegible] lbs

Impact = [illegible] x [illegible] = [illegible] lbs

Total = [illegible] lbs

Web depth = 27 ins; web thickness = 0.47 ins

$$\text{Shear stress} = \frac{[illegible]}{27 \times .47} = 4550 \text{ psi}$$

[illegible] end

Dead load = [illegible] + [illegible] = [illegible] lbs

Live load = [illegible] lbs

Impact = [illegible] lbs

Total = [illegible] lbs

Web depth = 41 ins; web thickness = 0.47 ins

$$\text{Shear stress} = \frac{[illegible]}{41 \times .47} = [illegible] \text{ psi}$$

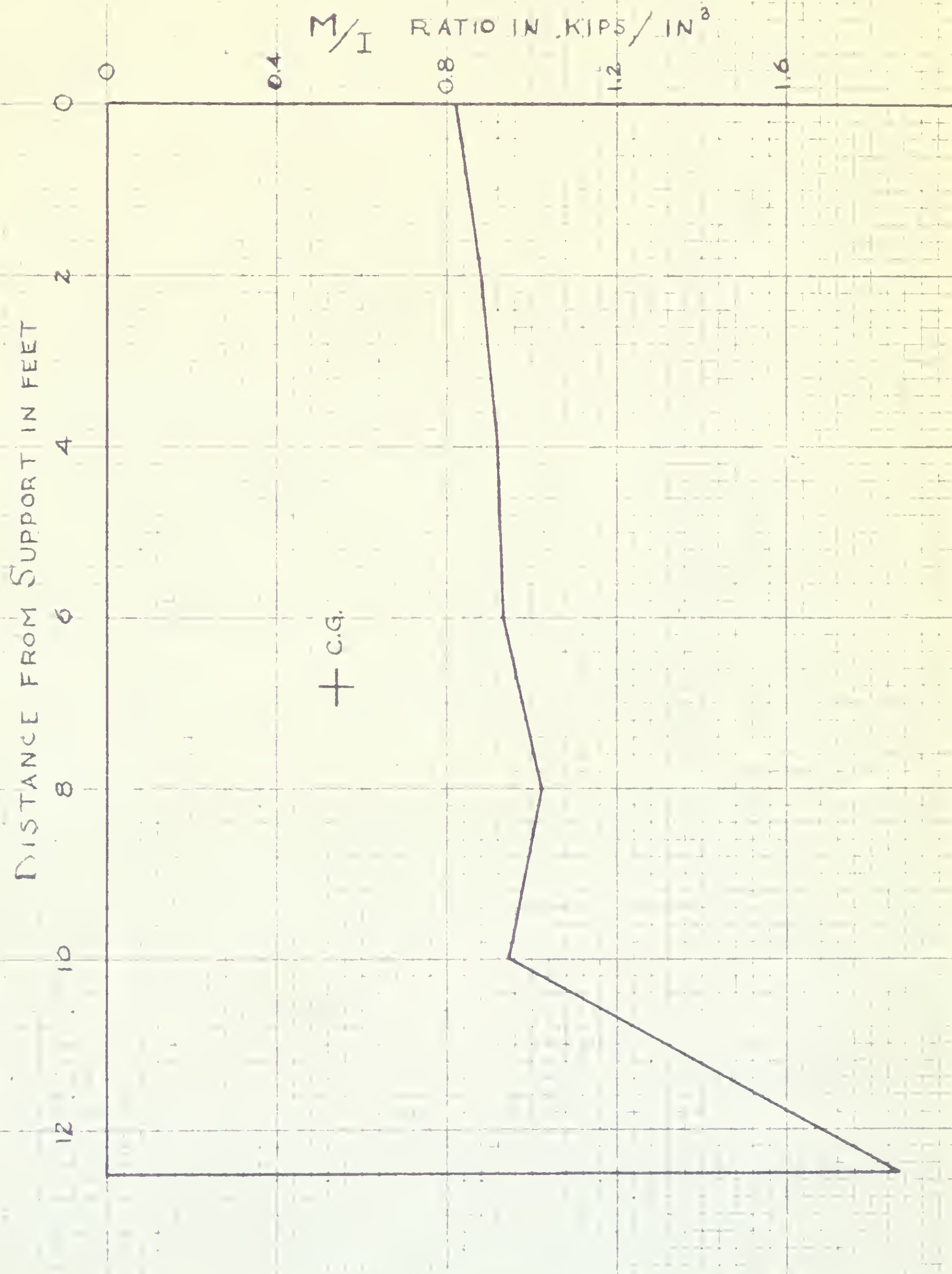
Deflection.

Assume [illegible] fixity at girder support and compute the deflection of the free end from the tangent to the fixed end using [illegible] deflection method.

[illegible] diagram to find the area and center of gravity.

Plot of M/I Diagram for Typical Floor Beam

June 1949



[illegible]

Stiffeners.

[illegible] 60

[illegible]

t figures under [illegible]

[illegible]

Allowable bearing stress on stiffeners [illegible]

[illegible]

[illegible]

[illegible] = [illegible] spaced [illegible] ft [illegible] spaced.

[illegible]

Span.

Design Load: [illegible]

Loads considered for figure of girder.

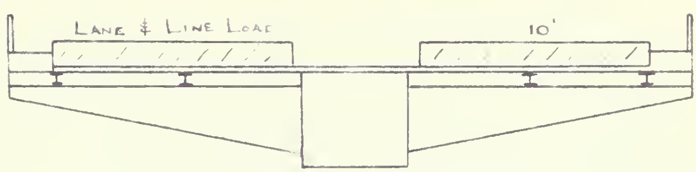
Dead Load:

Slab = [illegible] x [illegible] = [illegible] lbs per ft

[illegible] = 100 lbs per ft

Live Load:

Use lane loading shown.



[illegible] does not change the final moment more than 40 ft kips in 10000 ft kips.

[illegible] = [illegible] x [illegible] = [illegible] lbs per ft

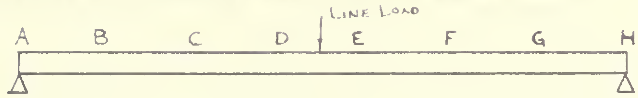
[illegible]

Impact allowance

[illegible] = [illegible]

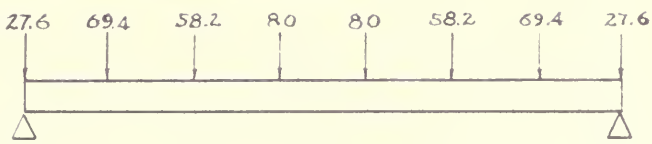
[illegible] moment [illegible] loading.

[illegible] = [illegible] ft kips



LINE LOAD
A
B
C
D
E
F
G
H

[illegible]



[illegible]

Moment at mid-span = [illegible]

[illegible]

[illegible]

[illegible] from 1.0 and 1.5 per cent based on [illegible] the girder supports. Therefore, [illegible] negative moment at mid-span will be [illegible] per cent of the [illegible] moment for simple support.

Assume a trial [illegible] = 0.5[illegible] x [illegible] = [illegible] ft-kips [illegible] moment [illegible] full support and mid-span. [illegible] value is [illegible] of its [illegible] positive moment at mid-span. The value of this positive moment will be [illegible] per cent of the positive moment for simple support.

Assume a trial design positive moment = [illegible]

= 7[illegible] ft-kips

Steel box girder.

Assume girder width = 5 feet

Assume girder depth at supports = 8 feet

Assume girder depth at mid-span = 5 feet

[illegible] flanges to meet the [illegible].

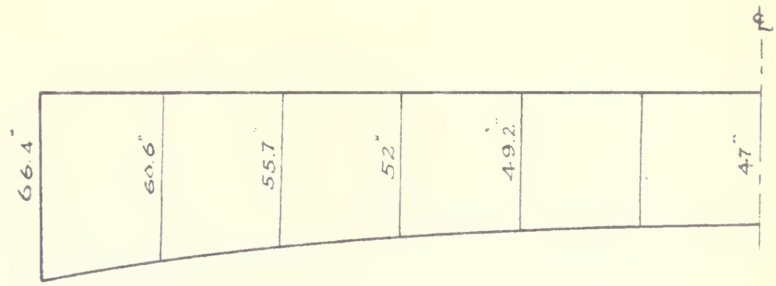
The final design is shown [illegible], using [illegible], 7/8" flanges

[illegible] cross-section.

Distance from support	Girder depth	I of webs	I of flanges	Total I (in.^{4})
0 ft	[illegible]	[illegible]	237[illegible]	292500 in.^{4}
[illegible]	[illegible]	[illegible]	184[illegible]	[illegible]2400
[illegible]	[illegible]	[illegible]	147500	174400
[illegible]	[illegible]	[illegible]	121[illegible]	141500
[illegible]	[illegible]	[illegible]	[illegible]	120400
[illegible]	[illegible]	[illegible]	[illegible]	10[illegible]

[illegible]

develop in the real girder it is necessary to change the girder to an equivalent girder with the same width at every section and the same moment of inertia at every section. Therefore, the cube of the depth of the equivalent girder is equal to the I of the real girder. By plotting the depth of the equivalent girder it can be seen that the curve of its lower chord is approximately parabolic.



With this data it is possible to solve for the moment on the equivalent girder using the frame constant published by the Portland Cement Association. The moment on the equivalent girder will equal the moment on the real girder.

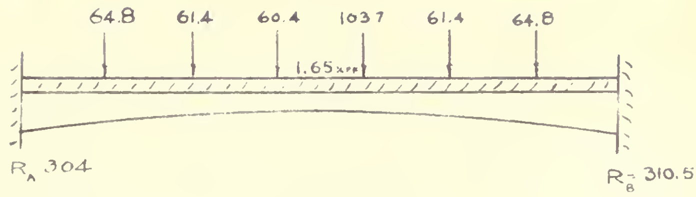
Check girder proportions.

Span.

Design span = 120 feet

Loads.

Use the same loads as with elastic supports. However,


64.8
61.4
60.4
103.7
61.4
64.8
1.65 k/ft
R_A 304
R_B = 310.5

[illegible]

[illegible]

[illegible] = [illegible] in^4

Bending stresses

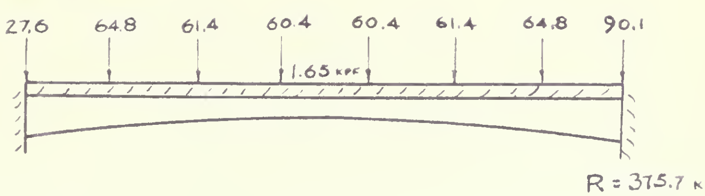
[illegible]

[illegible]

Web shear in girder.

[illegible]

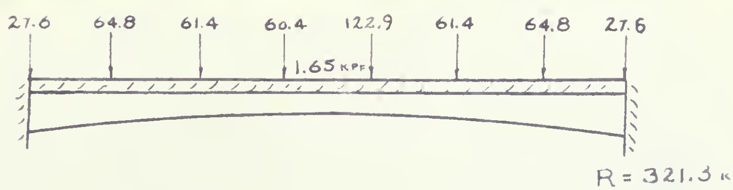
At support



Web area = [illegible] in^2

Shear stress = [illegible] = 5[illegible] psi

At panel point nearest [illegible]



Web area = [illegible] in^2

Shear stress = [illegible] = [illegible] psi

Torsion.

Maximum torsional stress will occur with an unbalanced positioning of the line and live loads on the bridge. The bridge rolling will also be considered in view of this critical torsional moment.

Effects of unbalanced loads on the bridge are pictured on the following pages. A description follows.

1. Load one-half of bridge with a uniform load. The torque is a maximum at the support and zero at the centerline.
2. Load one side of the bridge with a concentrated line load. The torque at the support varies as the distance of the load from the support.
3. Load the bridge with two uniform loads as shown. The torque is zero at the supports and is a maximum at the floor beam nearest the centerline.
4. For the maximum torque at the support load as shown.
5. For the maximum torque at the center load as shown. It is necessary to apply two line loads, each equal to the line load, in order to obtain maximum torques equal in cases 4 and 5.

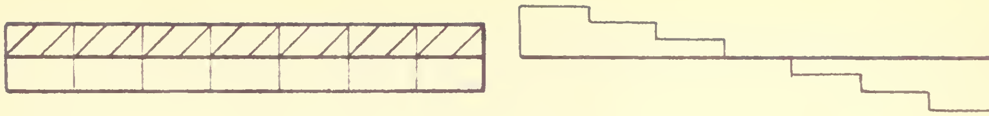
The following torque equations have been used:

$$\tau_t = \frac{T}{2A_o t} \quad \text{and torsional rotation} = \frac{T}{4A_o^2 G} \oint \frac{ds}{t}$$

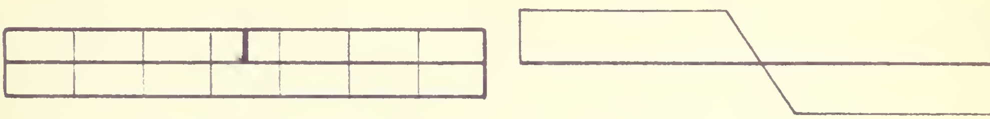
where τ_t = torsional shear stress

T = applied torque

1



2



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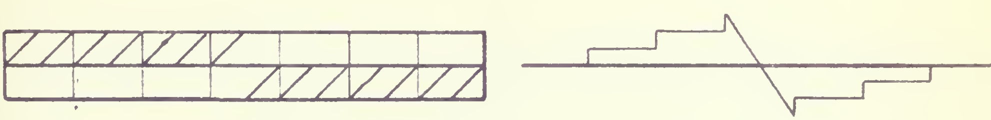

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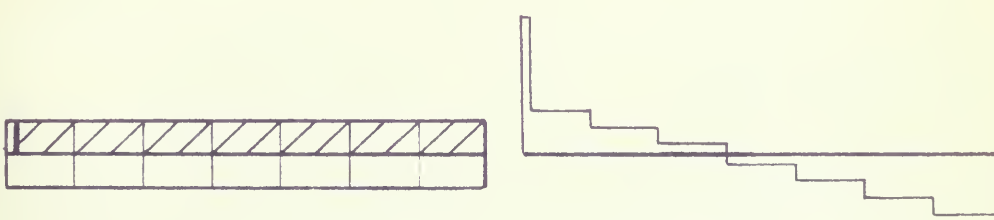
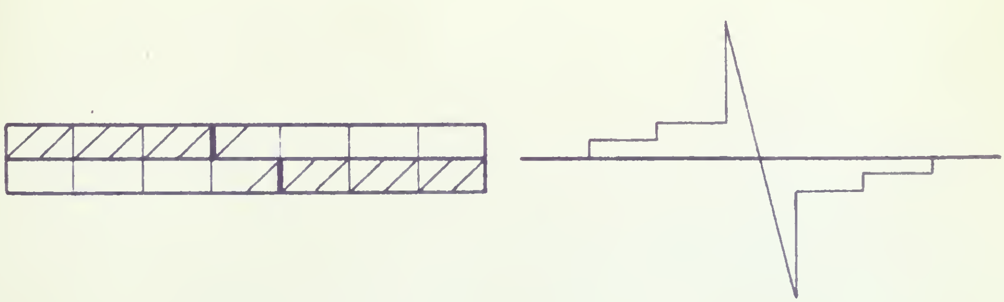


5



A_m = area enclosed by the median line of the cross-section in in^2

t = thickness of the cross-section at the point in the cross-section being considered in in.

G = shear modulus = [illegible] psi

s = length of the part of shear stress in a cross-section

Applied Torque.

At support using type 4 loading

Concentrated load = 2500 x [illegible] = [illegible] ft lbs

Uniform load = $\frac{140 \times [illegible] \times 8}{2}$ = [illegible] ft lbs

Rolling load = [illegible] x 17 x 14.[illegible] + 200 x 17 x 4.5

+ [illegible] x 17 x 5.0 + [illegible] x 17 x 5.3

= [illegible] ft lbs

Total = [illegible] ft lbs

Impact = [illegible].254 x [illegible] = [illegible] ft lbs

Total torque = [illegible] ft lbs

Torsional Shear at the Support

$S_s = \frac{[illegible] \times 12}{2 \times 144 \times 40 \times .375}$ = 2225 psi in webs

$S_s = \frac{0.375}{0.960} \times 2225$ = 870 psi in flanges

Torsional Deflection.

Assume as the worst condition that the entire girder is only 5 feet deep.

Rotation per inch length =

$\frac{[illegible] \times 12}{144^2 \times 4 \times [illegible] \times [illegible]} \left(\frac{10 \times 12}{.375} + \frac{10 \times 12}{.[illegible]} \right)$

Rotation per inch length = .00000709 radian per inch

Total roatati n at mid-span = .00000709 x 60 x 12

$$\times \frac{180}{3.14} = .293 \text{ degrees}$$

Mid-span deflection = 15 x 12 x tan 0.293 = 0.553 inches

Total Stresses.

Web stress = vertical shear + torsional shear

At support = 5220 + 2225 = 7445 psi

At mid-span = 2470 + 2225 = 4695 psi

Flange stress = Bending stress + torsional shear

Combine by principal stresses

$$\text{At support} = \frac{f}{2} + \left(\frac{f^2}{4} + s^2\right)^{\frac{1}{2}}$$

$$= \frac{15300}{2} + \left(\frac{15300^2}{4} + 2225^2\right)^{\frac{1}{2}} = 15620 \text{ psi}$$

$$\text{At mid-span} = \frac{16500}{2} + \left(\frac{16500^2}{4} + 2225^2\right)^{\frac{1}{2}} = 16830 \text{ psi}$$

Girder Deflection Under Design Load.

Assume 100 per cent end restraint at supports and compute deflection by slope deflection method.

Plot M/I diagram to find its area and center of gravity.

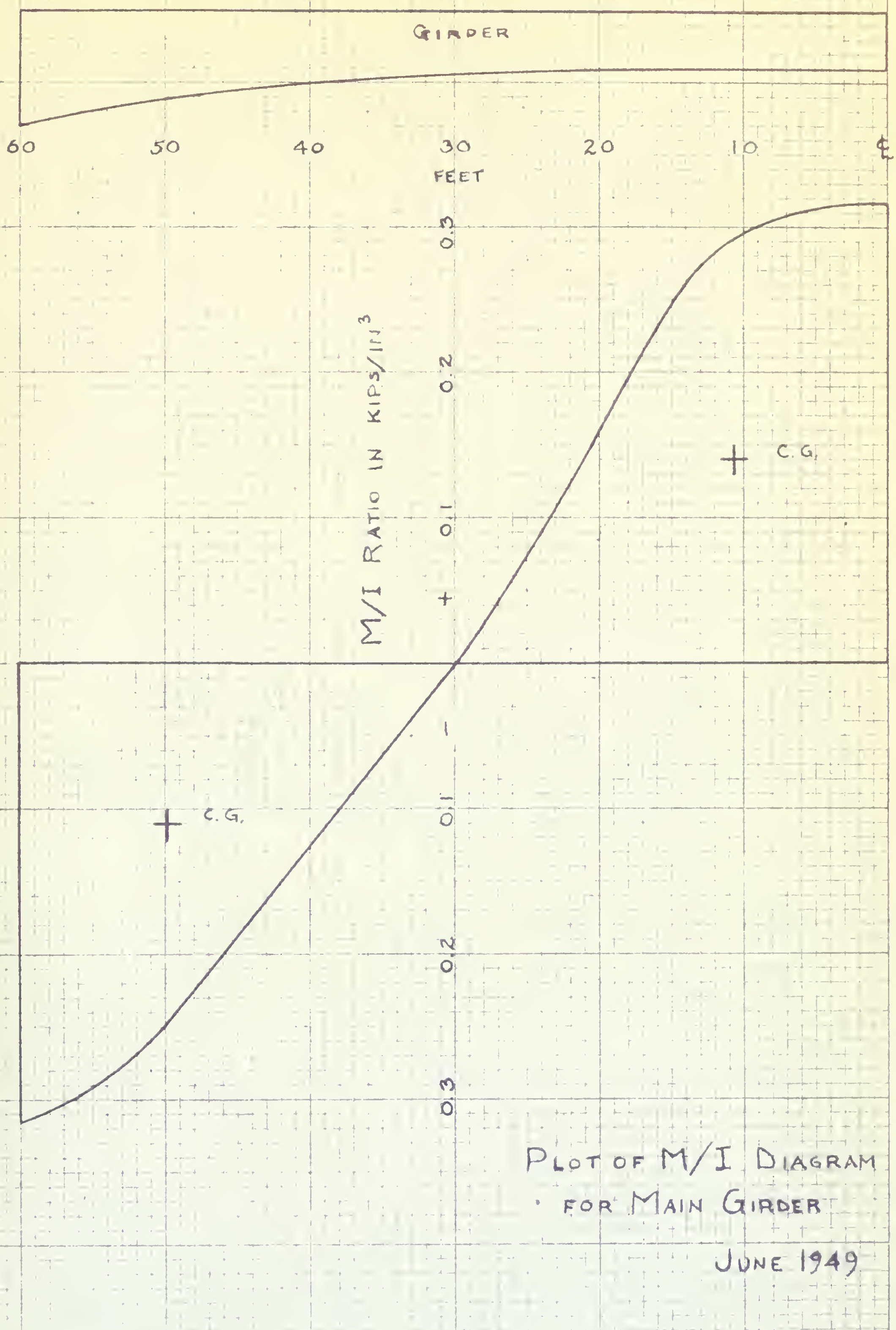
Negative area = $5.478 \frac{\text{kip ft}}{\text{in}^3}$, C.G. = 49.97' from mid-span

Positive area = $6.17 \frac{\text{kip ft}}{\text{in}^3}$, C.G. = 10.06' from mid-span

Moment of M/I diagram = 30500 kip/in

Deflection at mid-span = $\frac{30500}{30000}$ = 1.015 ins

Usual allowable deflection = $\frac{\text{span}}{800} = \frac{120 \times 12}{800}$ = 1.8 ins

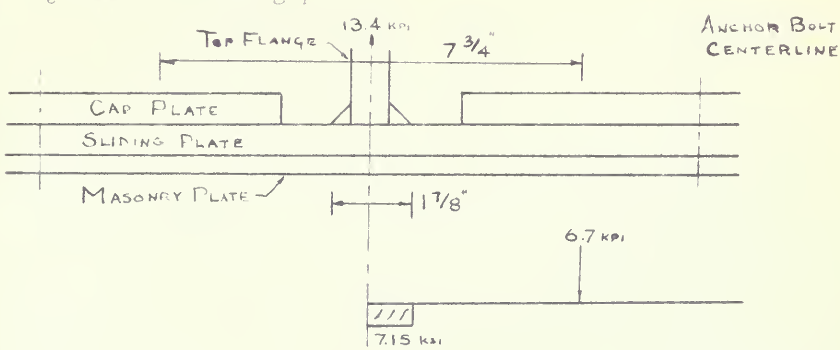

GIRDER
60
50
40
30
20
10
℄
FEET
M/I RATIO IN KIPS/IN³
0.3
0.2
0.1
+
−
0.1
0.2
0.3
C.G.
C.G.
PLOT OF M/I DIAGRAM
FOR MAIN GIRDER
JUNE 1949

BRIDGE SEAT

In order to resist the bending moment and the applied torque and at the same time allow some horizontal longitudinal movement, it was decided to use a three plate assembly for the end connection which would allow some sliding. The middle plate is the sliding plate and is an enlarged continuation of the bottom flange of the girder. The girder top flange is bent in an arc of 8 foot radius and welded to the sliding plate. The flange fibre stress is thus smoothly transferred to a vertical force. Three web plates are used in the end connection to transfer the shear.

SLIDING PLATE

Design thickness to resist the vertical force of the top flange where it joins the sliding plate.



Moment under flange plate $= 6.7 \times 3.825 - \frac{7.15 \times .938^2}{2}$

$= 22.46$ in kips

Required Section Modulus $= \frac{22.46}{18} = 1.25 \text{ in}^3$

Required thickness $= \left(\frac{6S}{b}\right)^{1/2} = \left(\frac{6 \times 1.25}{1}\right)^{1/2} = 2.75$ in Use 3 inch

MASONRY PLATE

Use a 3/4 inch plate for this plate. The area of bedding is not critical.

CAP PLATES

Use 3 inch plates to give them the same relative stiffnesses as the sliding plate in order to keep a plane surface between them.

ANCHOR BOLTS

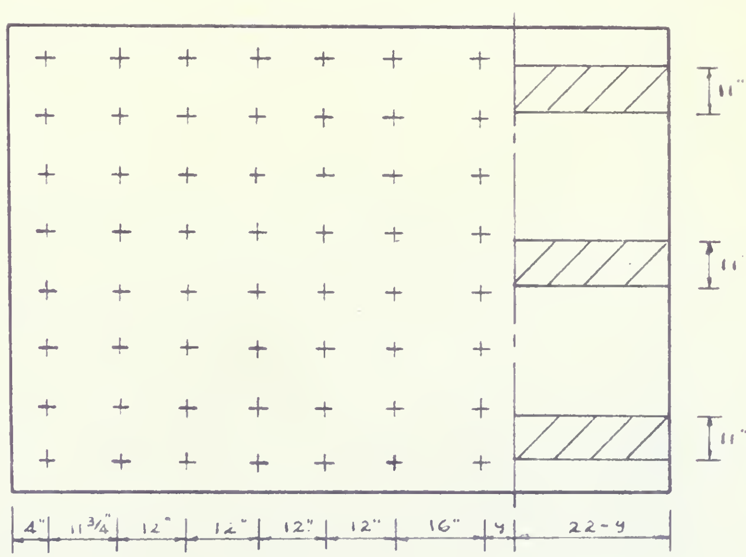
Required section modulus to resist bending moment.

Consider concrete bearing area in compression and the bolts in tension. Use bolt tension = 13.5 ksi.

$$S = \frac{7740 \times 1000 \times 12}{13500} = 6880 \text{ ins}^3$$

Neutral Axis of compression concrete and tension steel.

Space $2\frac{1}{4}$ inch bolts as shown. Bolt area = 3.976 in^2



Solve for y

$$3\left[\frac{11(22-y)^2}{2}\right] = 6 \times 3.976(275.75 + 7y) + 2 \times 3.976(139.75 + 2y)$$

$y = 0.35$ in Assumption for Neutral Axis correct

Moment of inertia of compression concrete and tension steel.

$$I = 3\left[\frac{11(21.65)^3}{3}\right] + 23.8(16.35^2 + 28.35^2 + 40.35^2 + 52.35^2$$

$$64.35^2 + 76.10^2) + 7.95(64.35^2 + 76.10^2)$$

$$= 556000 \text{ in}^4$$

Section Modulus.

$$S = \frac{556000}{76.10} = 7310 \text{ in}^3$$ Adequate.

Required Section Modulus to resist torsion.

Consider concrete bearing area in compression and the bolts in tension.

$$S = \frac{803500 \times 12}{13500} = 778 \text{ in}^3$$

Find neutral axis and moment of inertia of concrete and steel by the same method as just described above.

$I = 315000 \text{ in}^4$ $c = 66.04$ in

Section Modulus.

$$S = \frac{315000}{66.04} = 4770 \text{ in}^3$$ Adequate.

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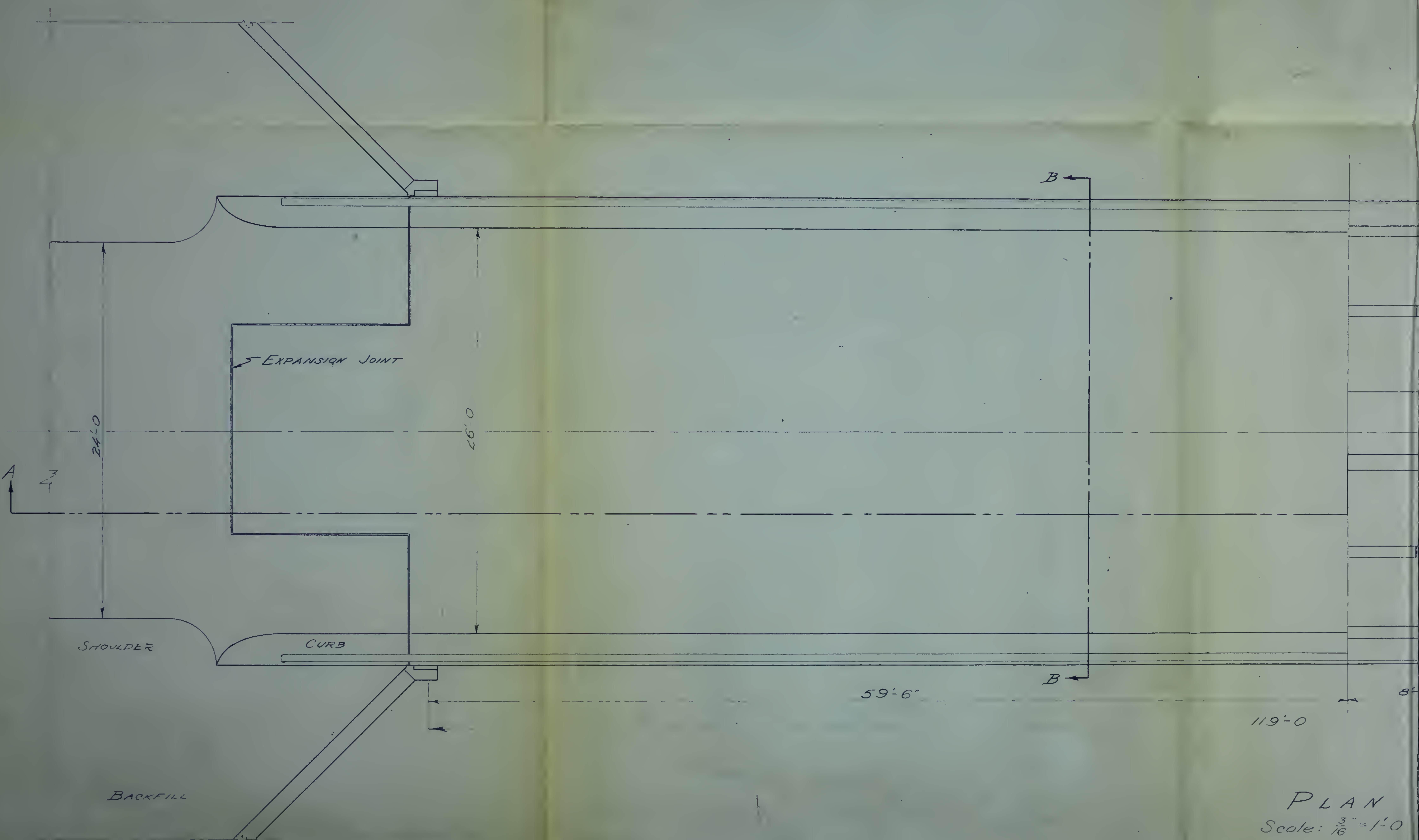
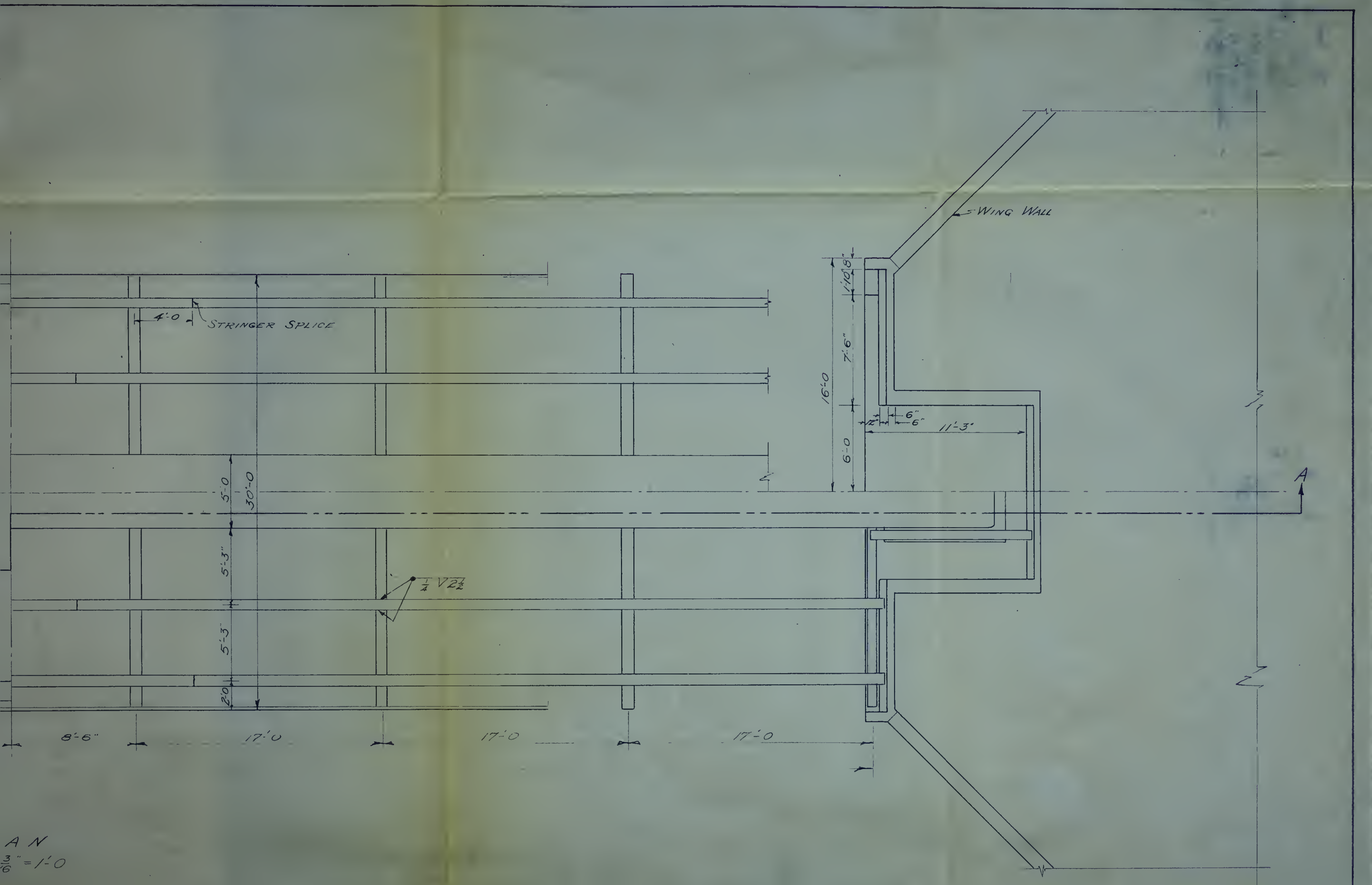
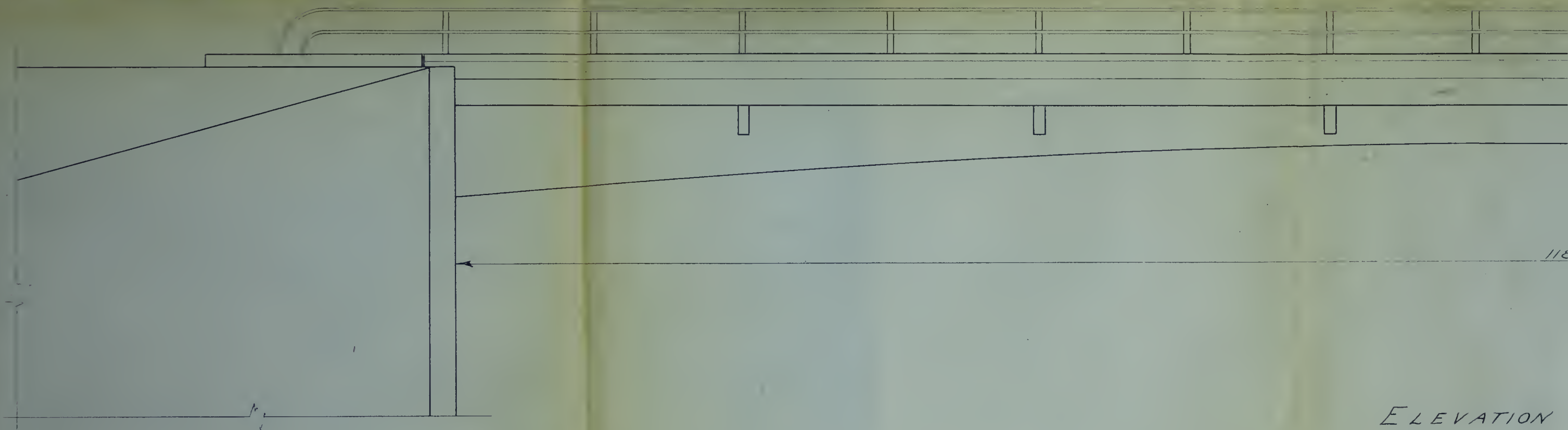


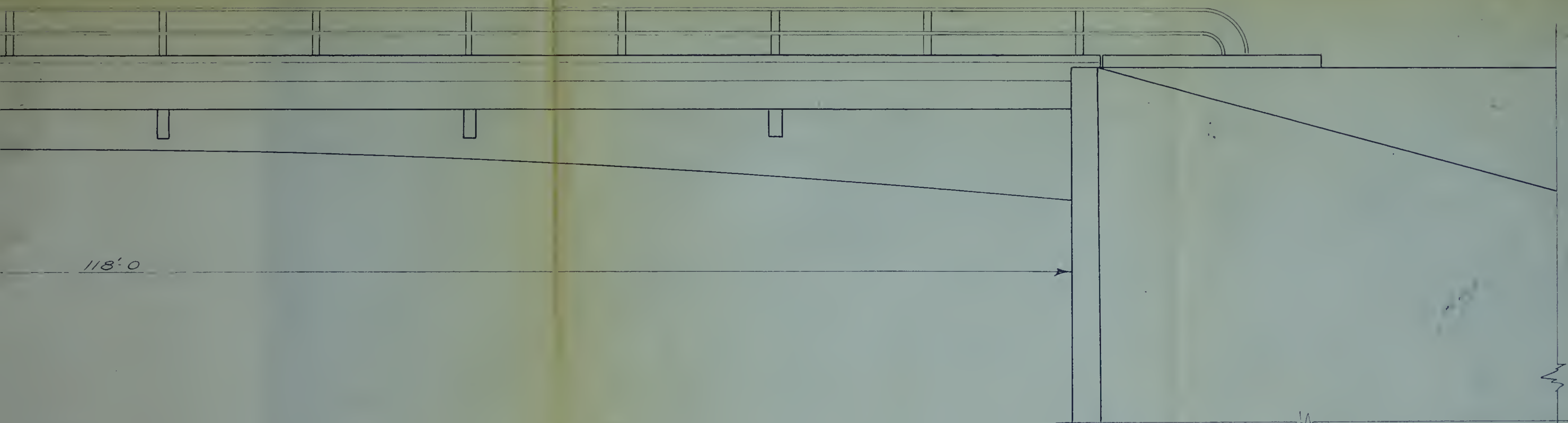
B
B
EXPANSION JOINT
24'-0
26'-0
A
SHOULDER
CURB
59'-6"
119'-0
BACKFILL
PLAN
Scale: 3/16" = 1'-0



WING WALL
STRINGER SPLICE
4'-0
16'-0
1'-10
8"
7'-6"
6'-0
6"
6"
2"
11'-3"
30'-0
5'-0
5'-3"
5'-3"
2'-0
A
$\frac{1}{4}$ V2$\frac{1}{2}$
8'-6"
17'-0
17'-0
17'-0
A N
$\frac{3}{16}$" = 1'-0



ELEVATION
Scale: $\frac{3}{16}$" = 1'-0



ATION
= 1'-0

TWO-LANE, 120' SPAN, DECK TYPE

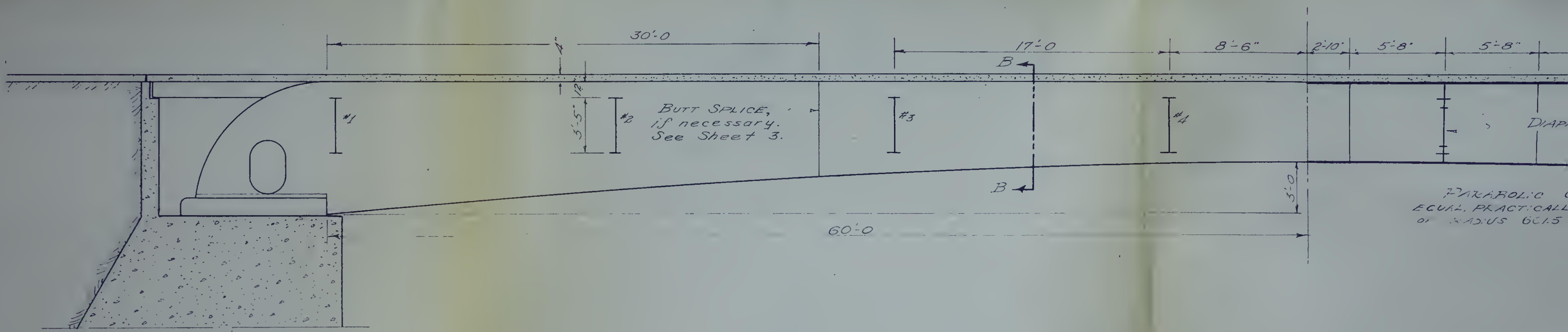
HIGHWAY BRIDGE

PLAN & ELEVATION

DESIGNED BY:

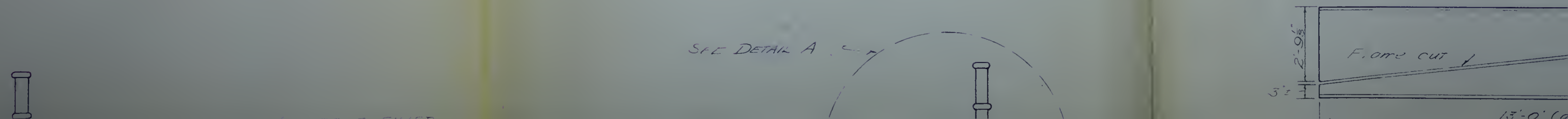
JUNE 1949

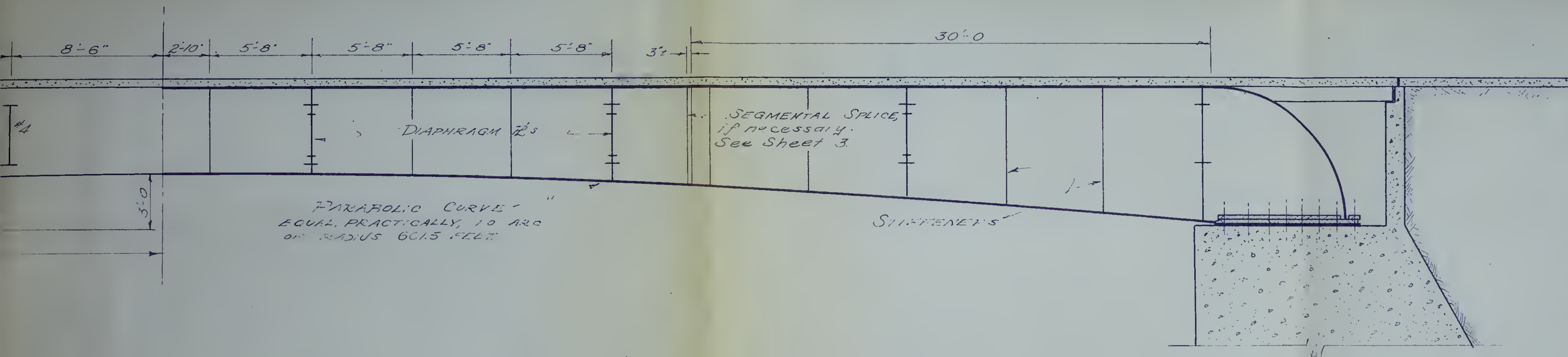
SHEET 1 of 4



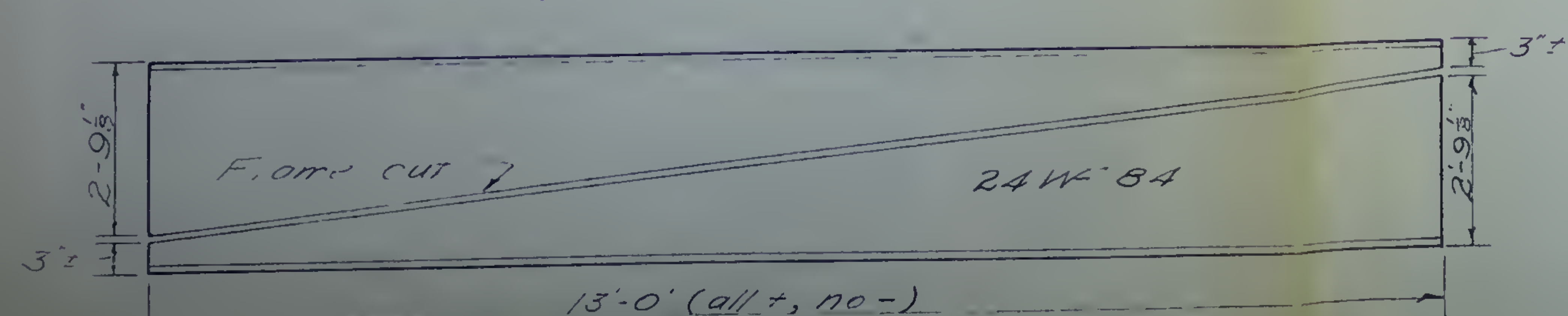
LONGITUDINAL SECTION

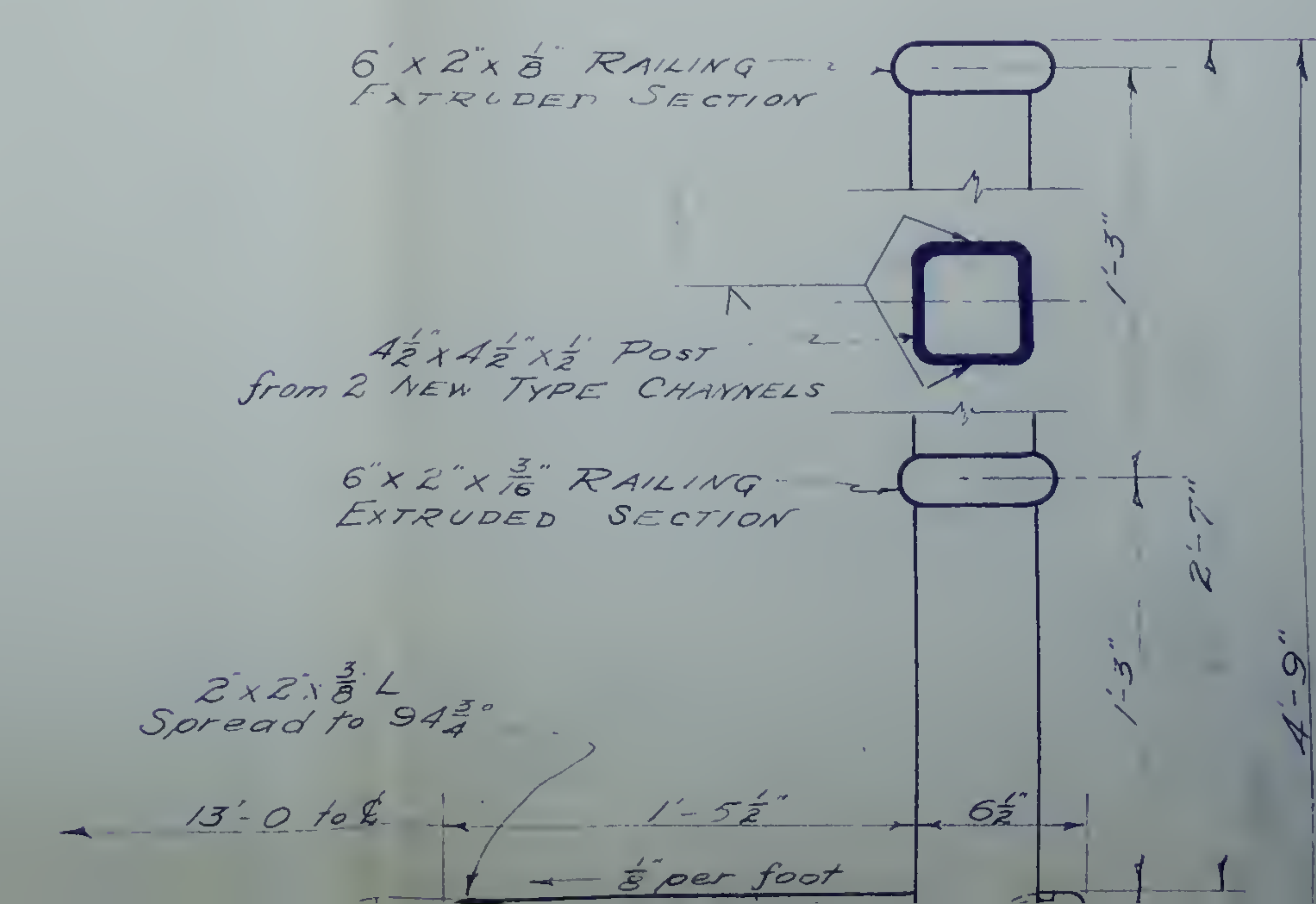
Scale: $\frac{3}{16}$ = 1'-0

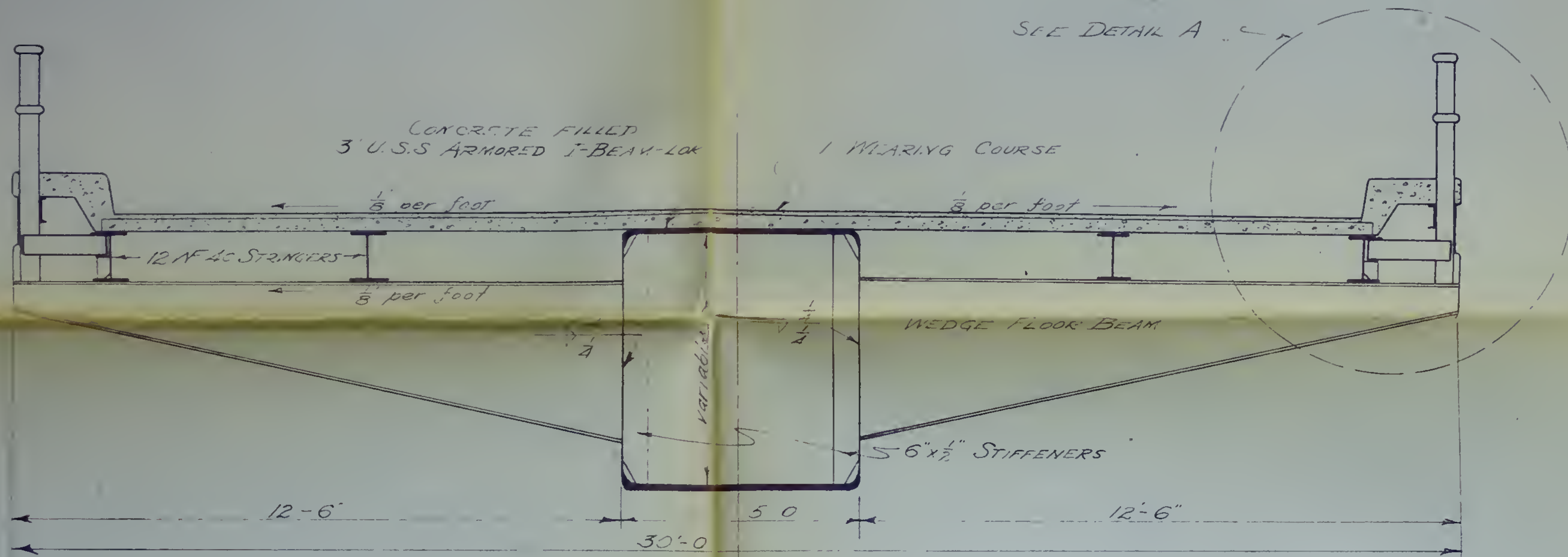




NGITUDINAL SECTION at A-A
Scale: $\frac{3}{16}$" = 1'-0

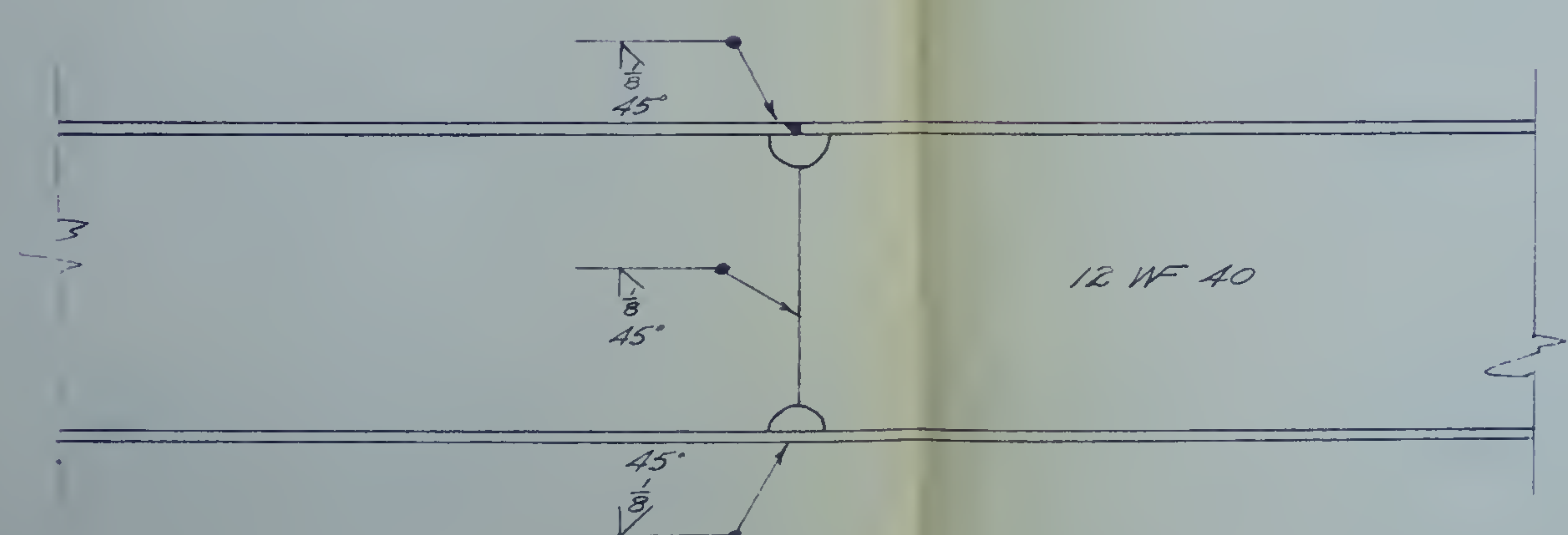






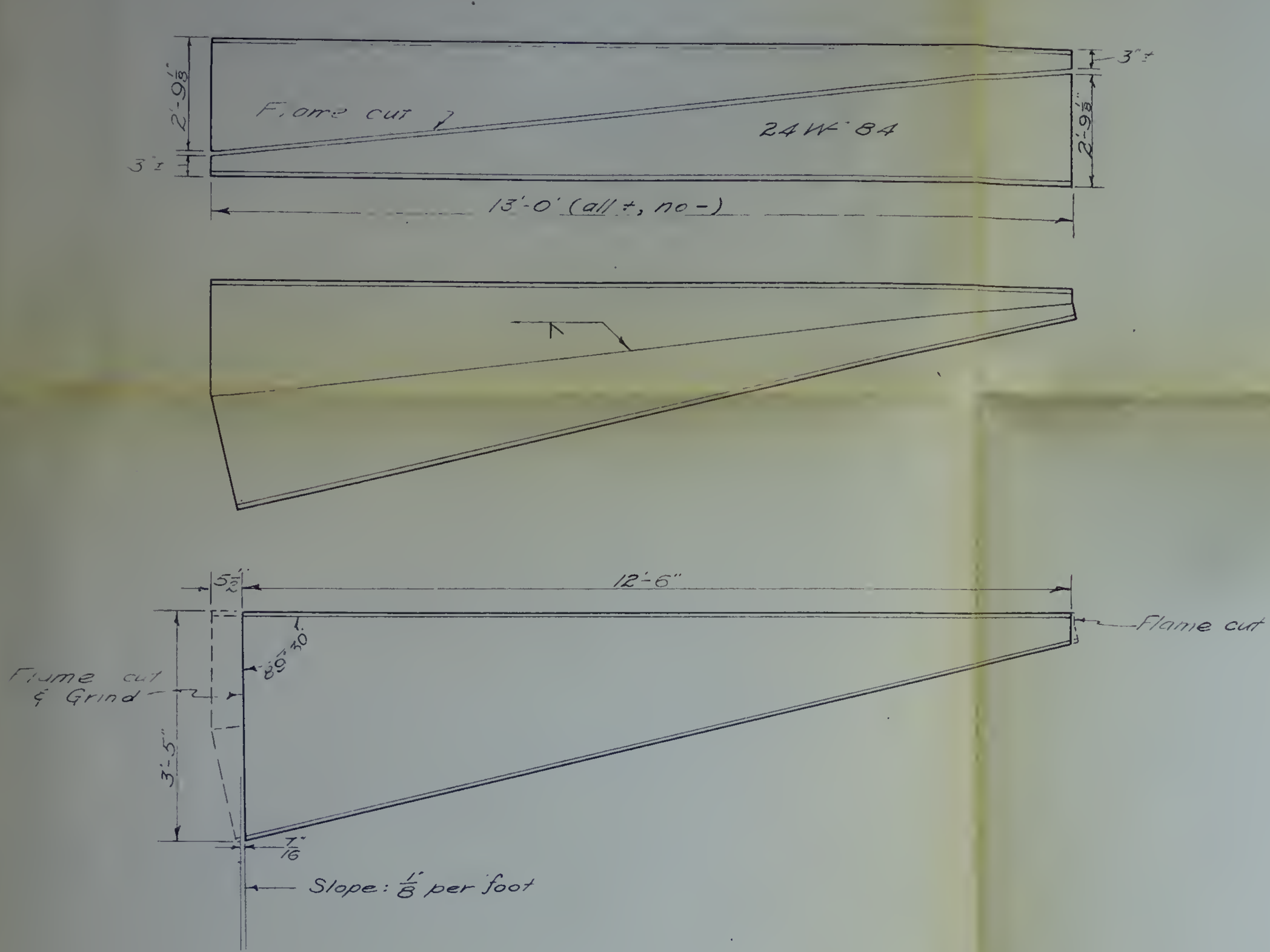
TYPICAL TRANSVERSE SECTION at B-B

Scale: 3/8" = 1'-0

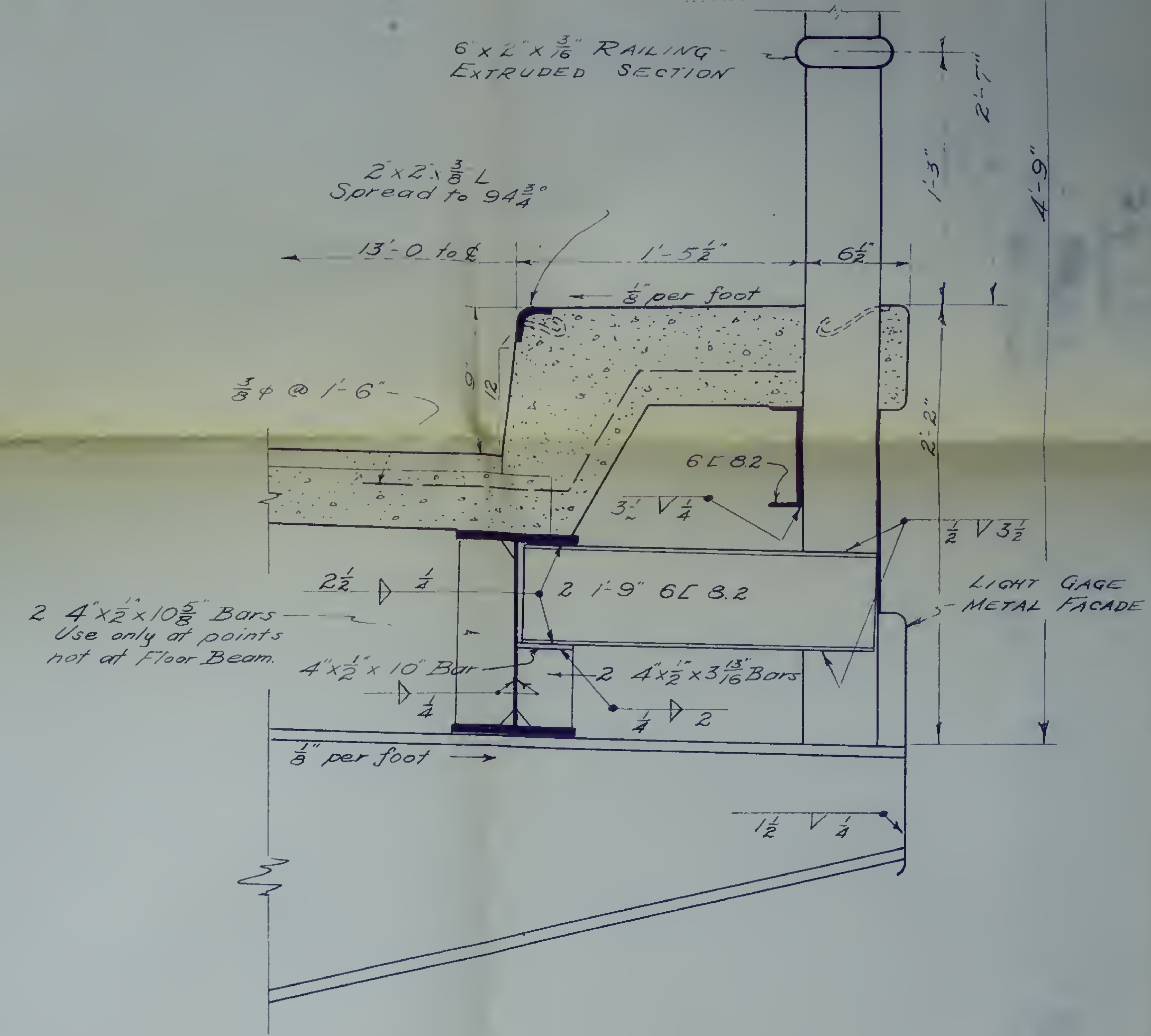


STRINGER SPLICE

Scale: 1 1/2" = 1'-0



FABRICATION of FLOOR BEAM
Scale: 1/2" = 1'-0.



DETAIL A
Scale: 1 1/2" = 1'-0

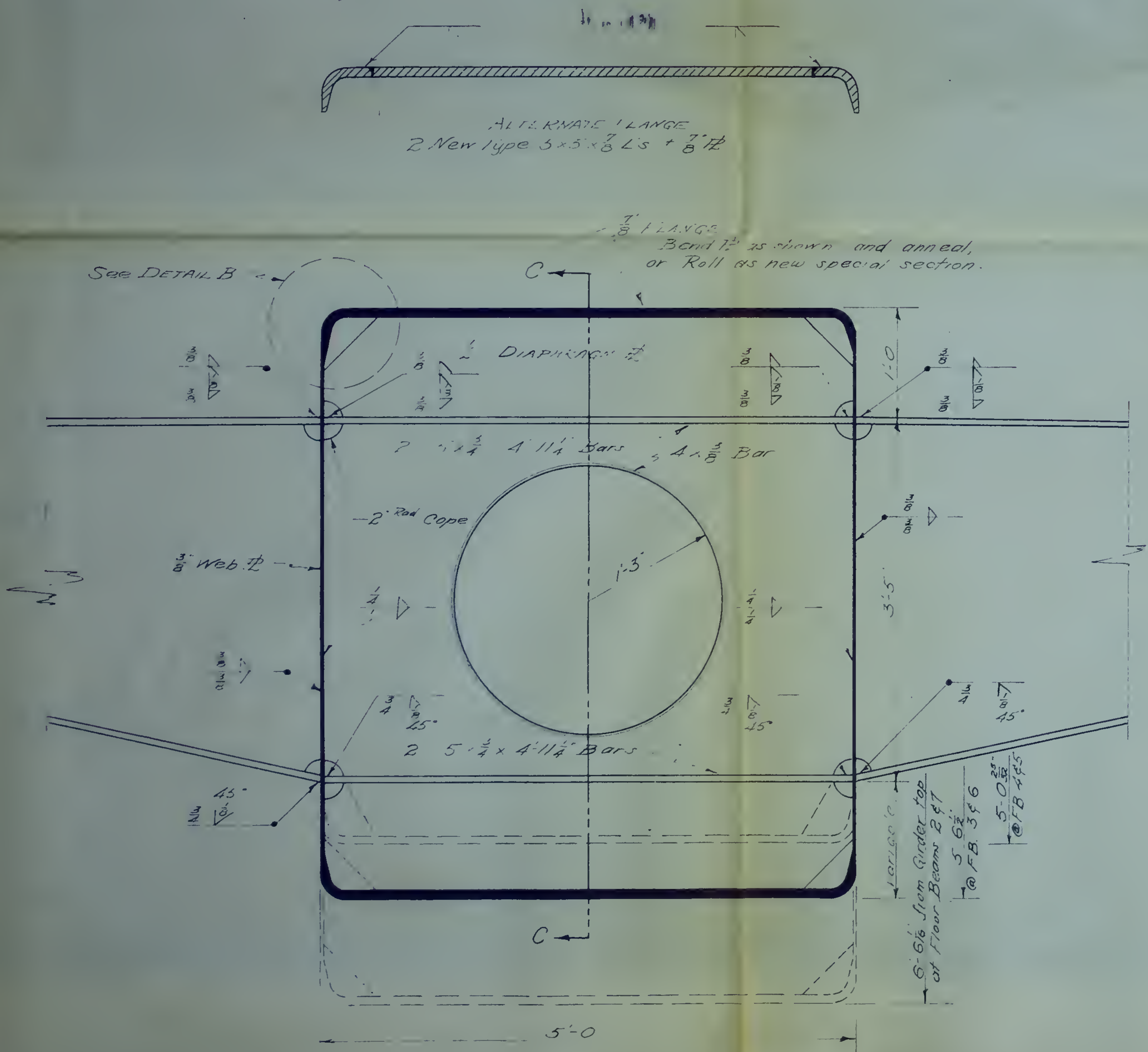
NOTE:
Standard 3" U.S.S. I-BEAM-LOK ARMORED FLOOR to be installed according to manufacturer's specifications. For details see U.S.S. publication "LIGHT-WEIGHT STEEL FLOORING," page 69.

TWO-LANE, 120' SPAN, DECK TYPE
HIGHWAY BRIDGE
SECTIONS & DETAILS

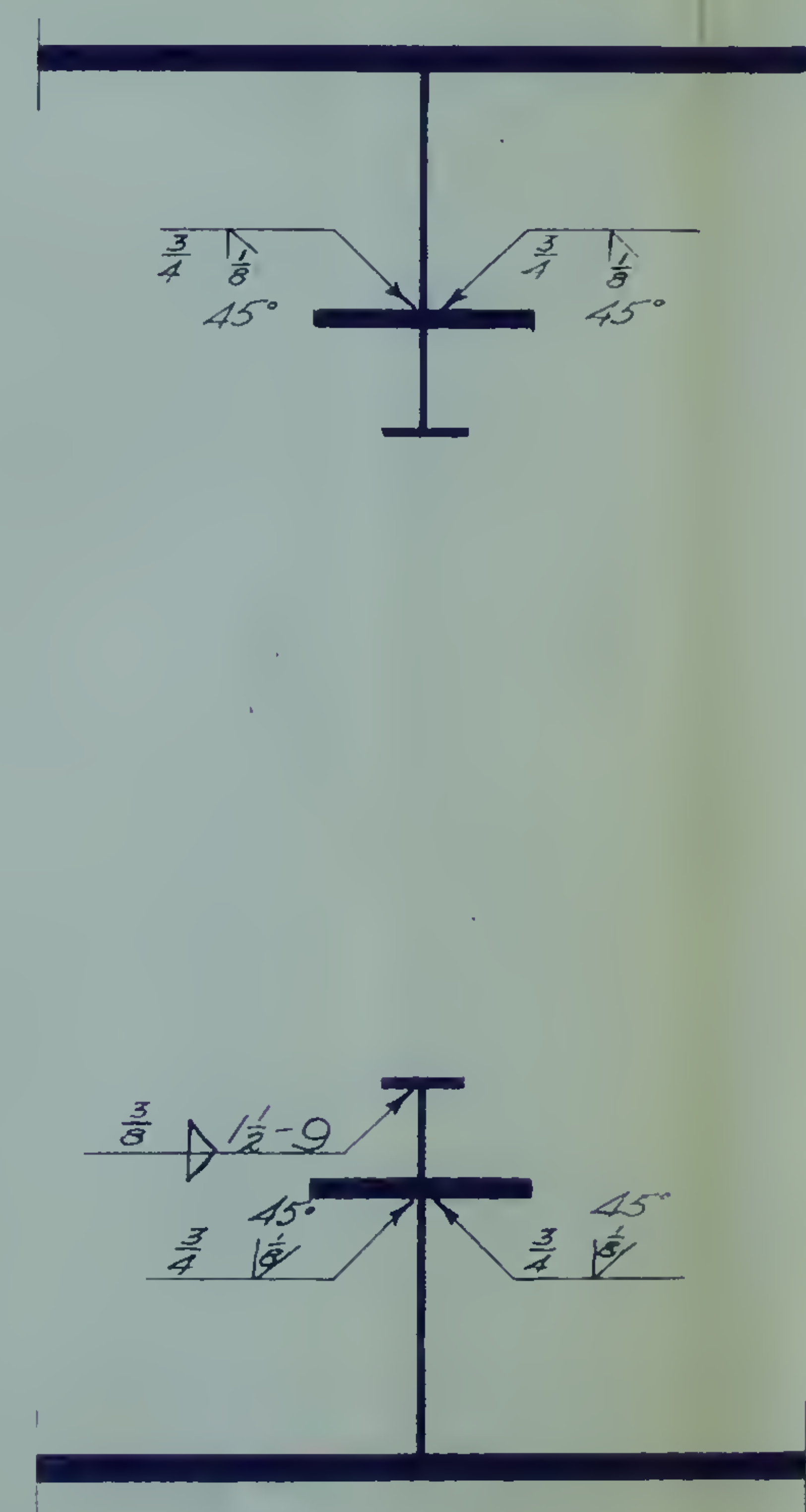
DESIGNED BY:

JUNE 1949

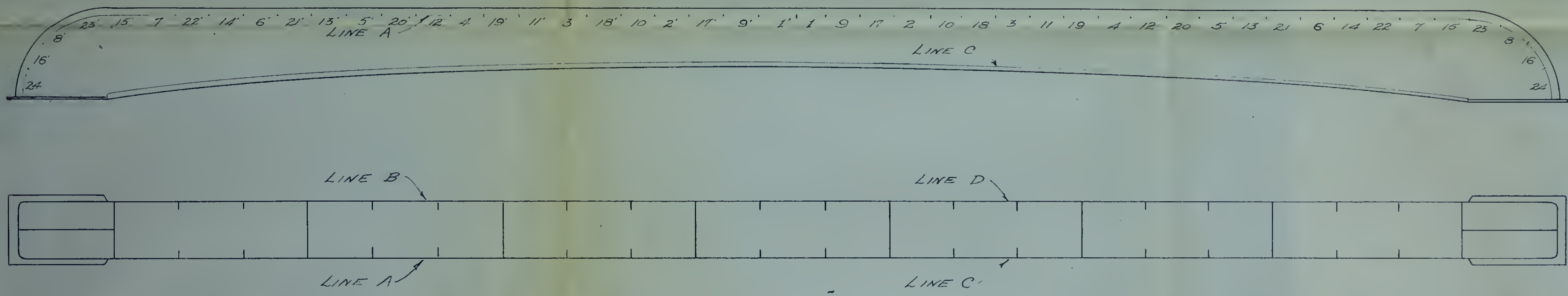
SHEET 2 of 4



TYPICAL GIRDER SECTION
Scale 1"=1'-0

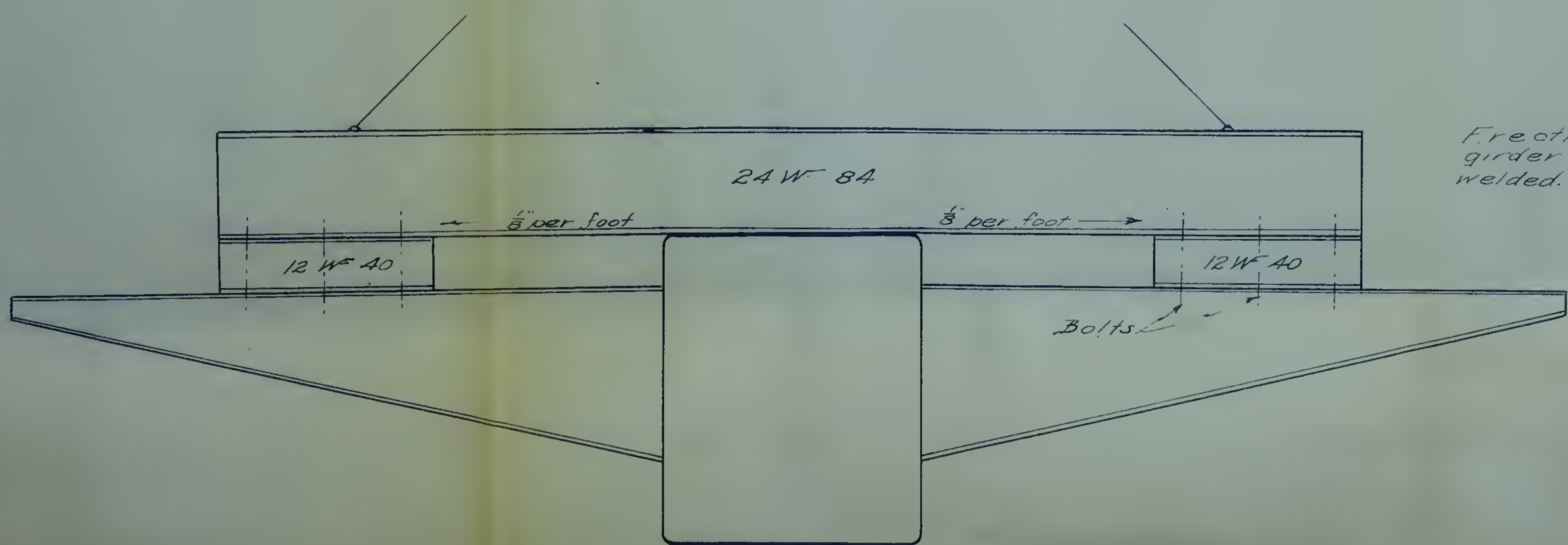


SECTION at C-C
Scale: 1"=1'-0



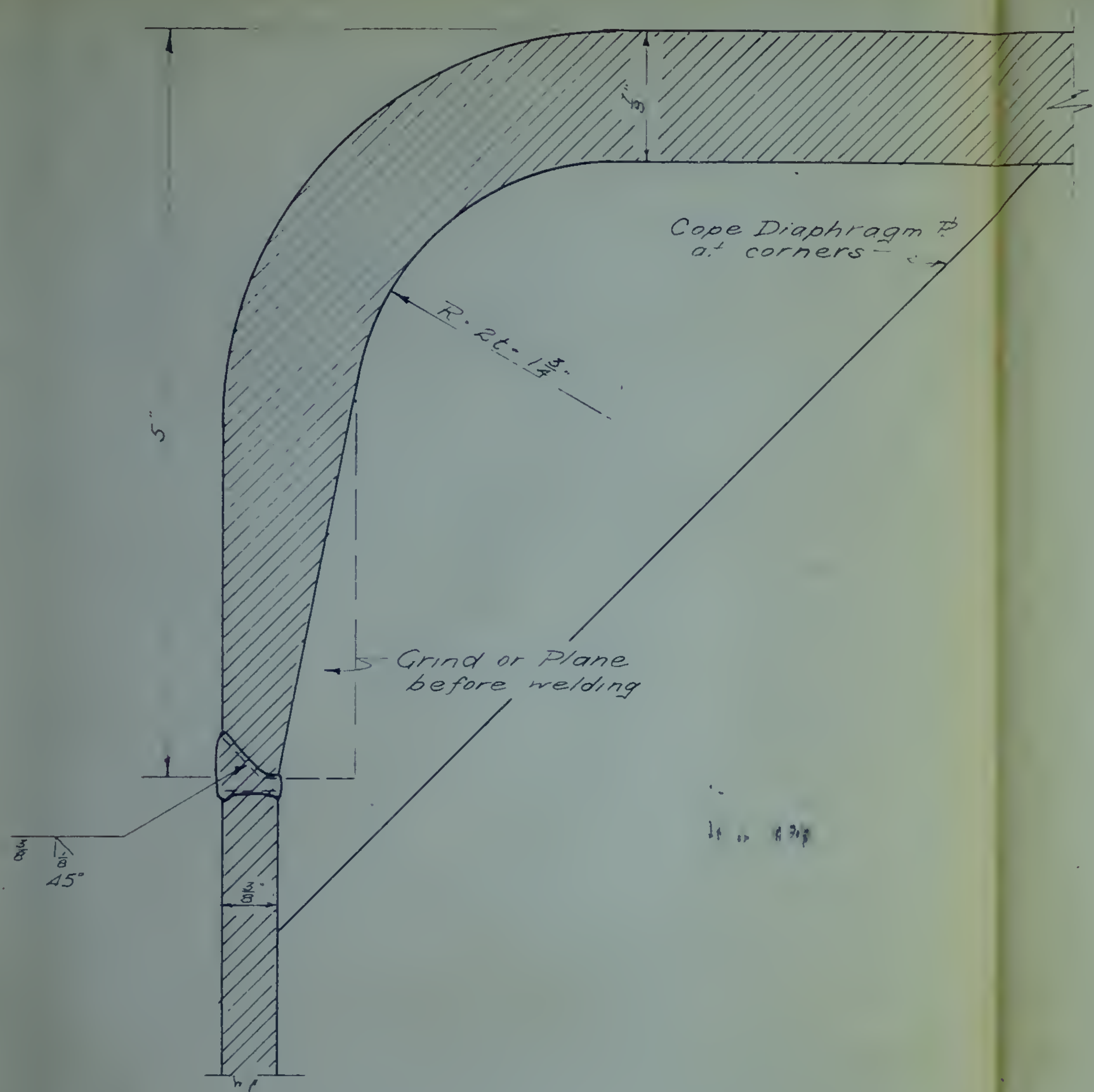
All girder welds shall be done in wandering sequence similar to that shown for LINE A above. Flanges and webs shall be completely fabricated before commencing any longitudinal welding. After proper jigging, aligning, and tack welding, LINE A shall be welded. Then diaphragm plates shall be installed as much as possible, followed by intermediate stiffeners on one side. LINE B shall then be welded and followed by completing diaphragm plates, and rest of intermediate stiffeners. Girder shall be completed by welding LINES C & D.

WELDING SEQUENCE

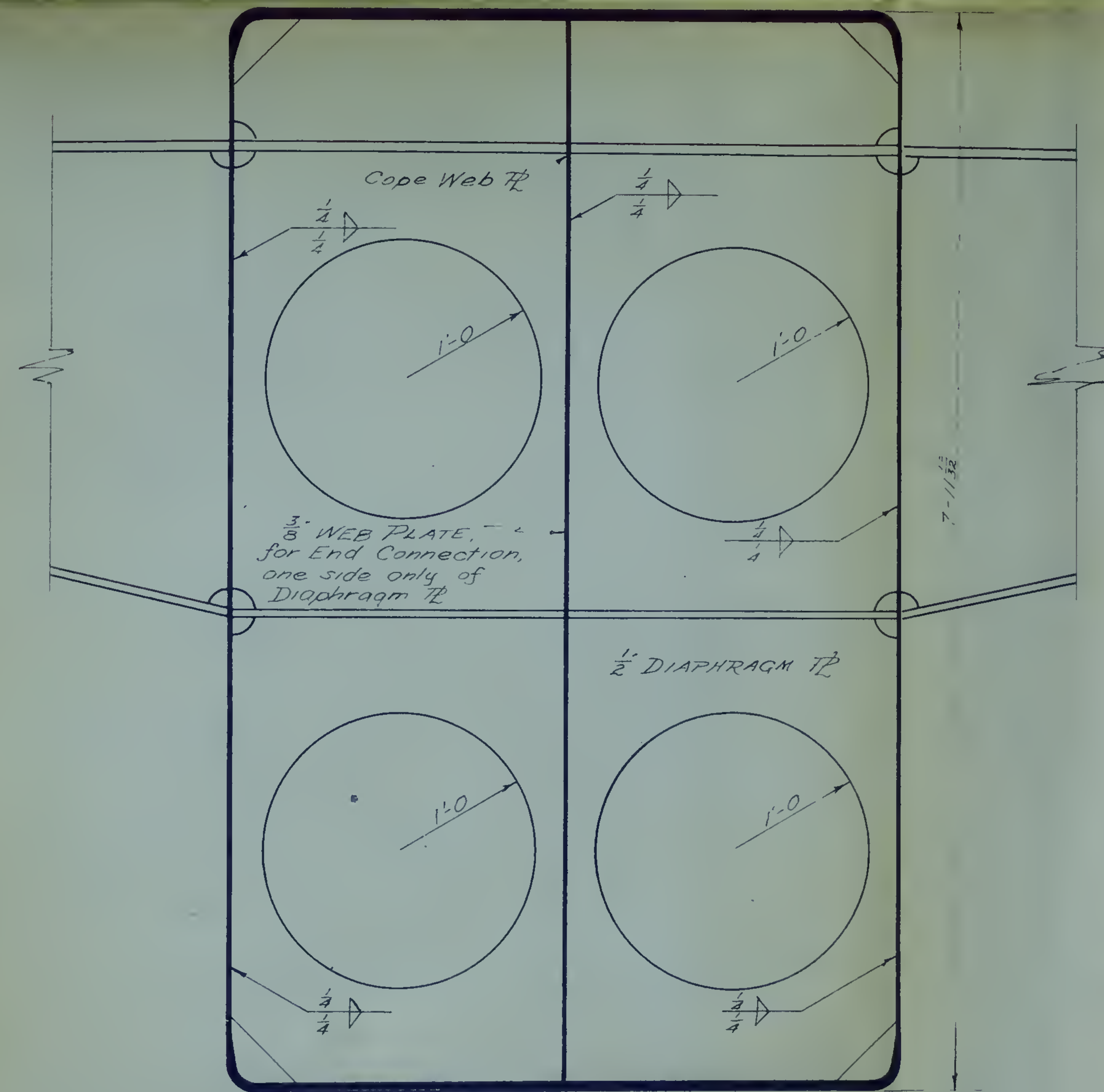


Erection piece may be left on girder while Floor Beams are being welded.

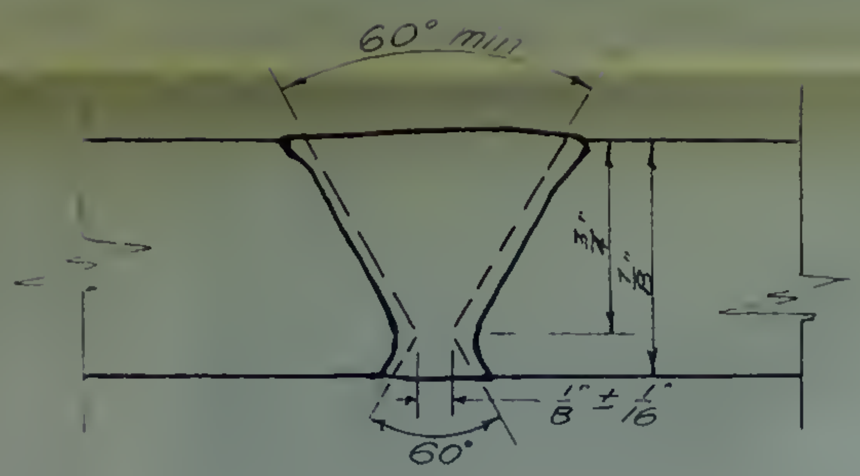
ERECTION of FLOOR BEAMS



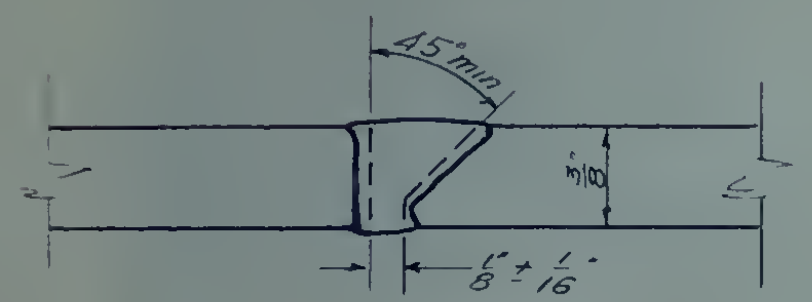
DETAIL B
Scale: Full Size



GIRDER SECTION at FLOOR BEAMS 1 & 8
Scale: 1" = 1'-0



FLANGE VEE BUTT JOINT

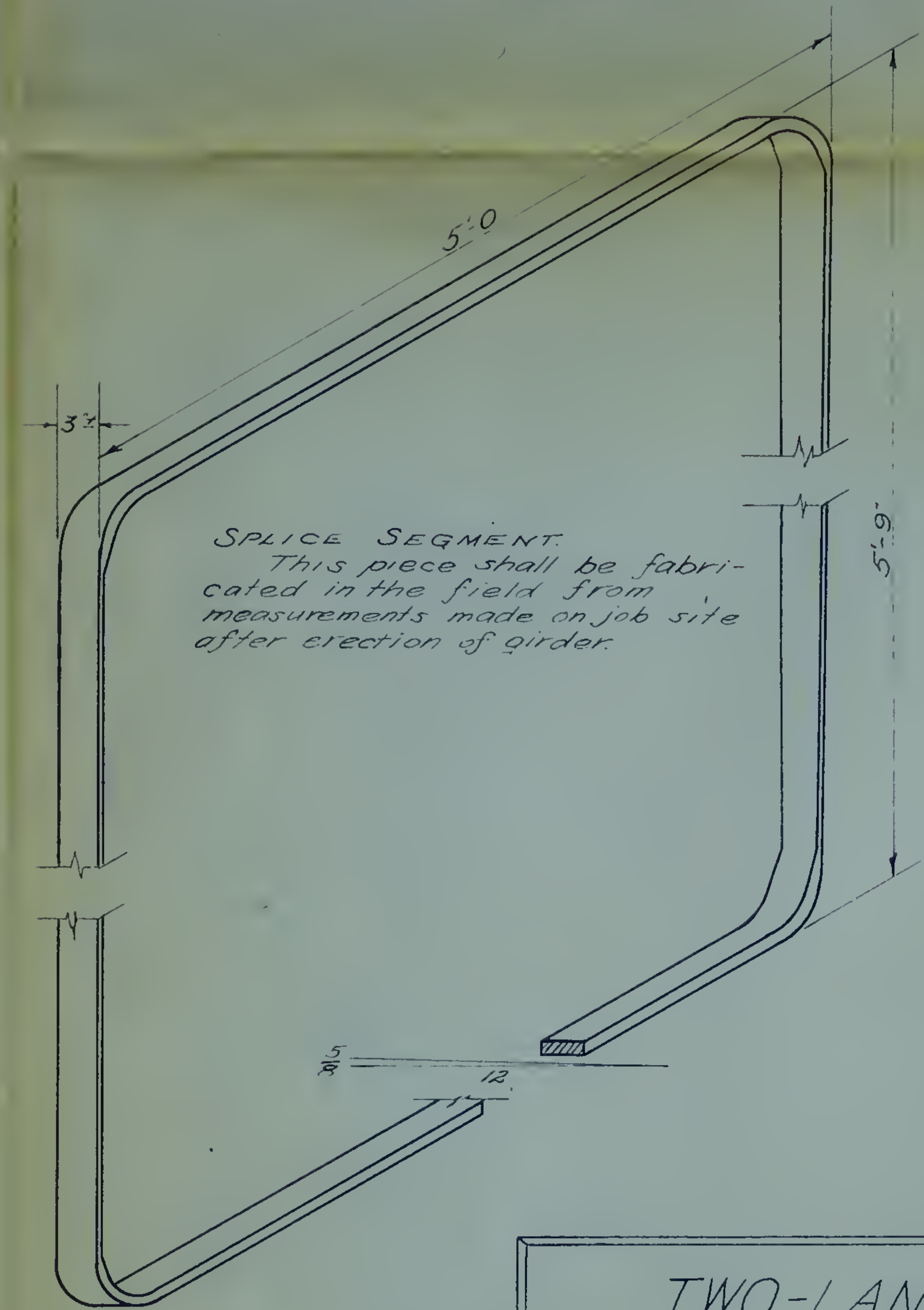


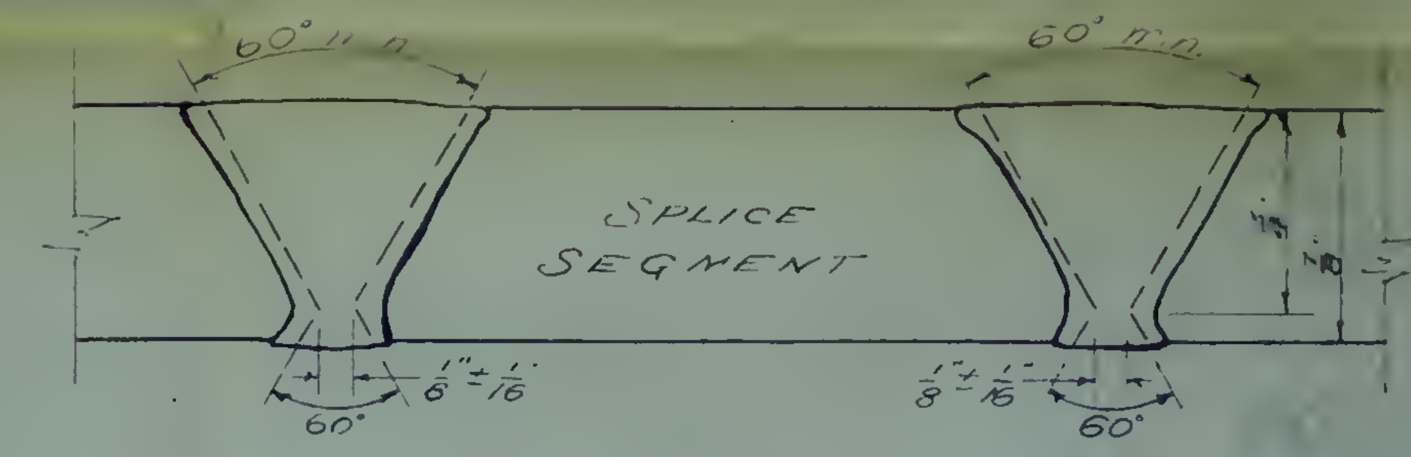
WEB BEVEL BUTT JOINT

DETAILS of BUTT SPLICE WELDS

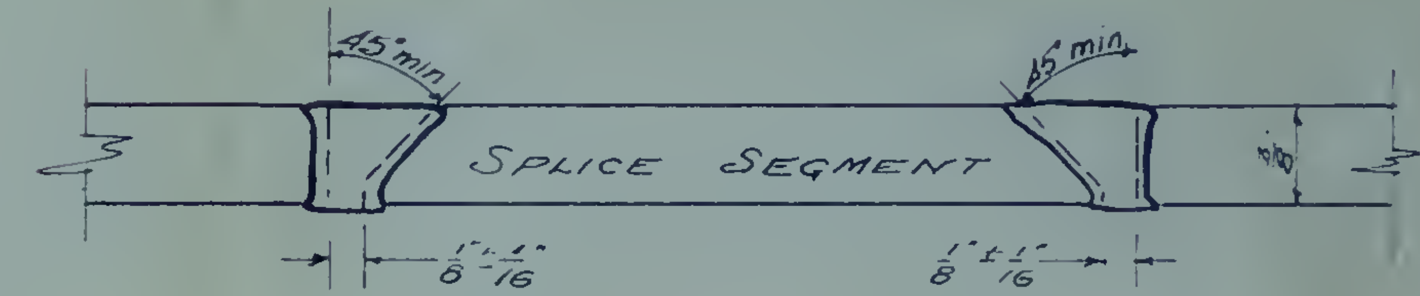
Scale: Full Size

NOTE:
Butt Splice to be made on ground prior to erection. Edges shall be prepared so that downhand welding is used on Vee Joint with only single overhead pass necessary on flanges after cleaning out root of first downhand pass, and root of Bevel Joint is on inside of girder. Weld flanges first using wandering sequence, working from center towards corners, then weld webs similarly.





FLANGE VEE BUTT JOINTS



WEB BEVEL BUTT JOINTS

DETAILS of SEGMENTAL SPLICE

NOTE:
Segmental Splice to be made after girder sections are erected. Edge preparation - see Butt Splice.

NOTE:
It is the opinion of the authors that where proper transportation is at all available, the girder should be completely shop fabricated and shipped. If, however, the job site is located where this is not possible, splices shall be used as shown on Sheet 2 and detailed above, and the three girder sections shall each be welded in a sequence similar to the one above for the whole girder.

TWO-LANE, 120' SPAN, DECK TYPE

HIGHWAY BRIDGE

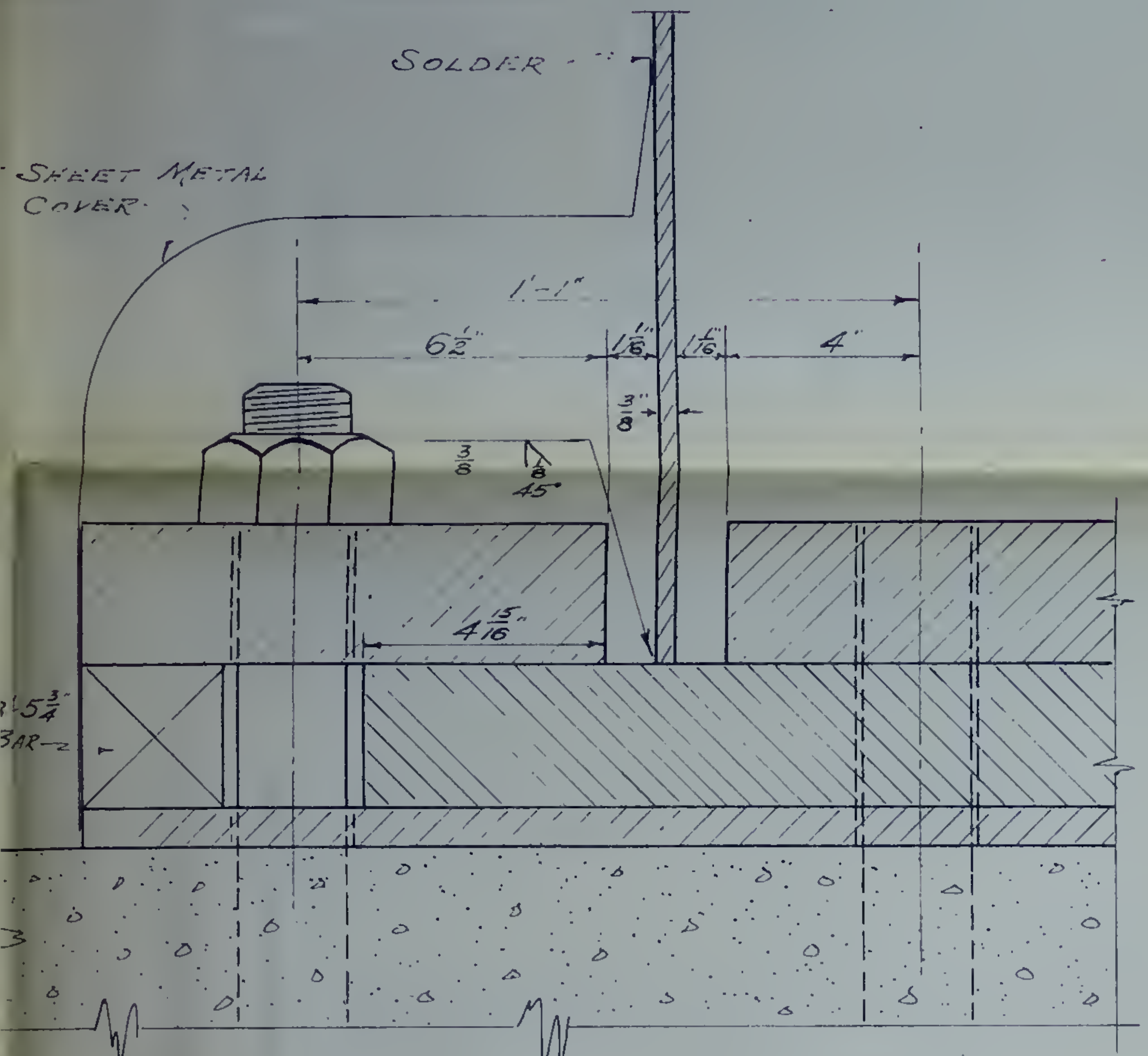
BOX GIRDER DETAILS

DESIGNED BY:

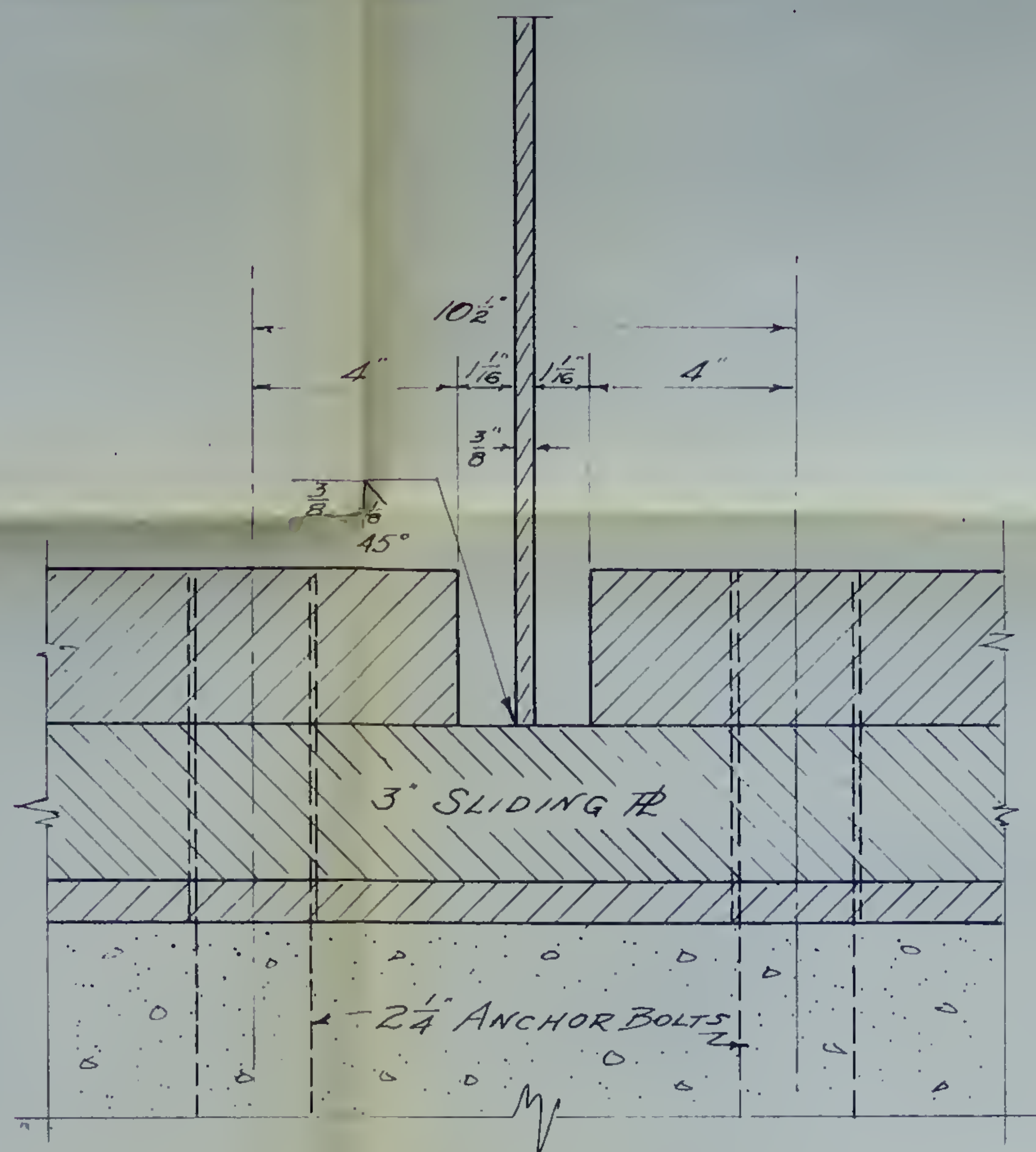
JUNE 1949

SHEET 3 of 4

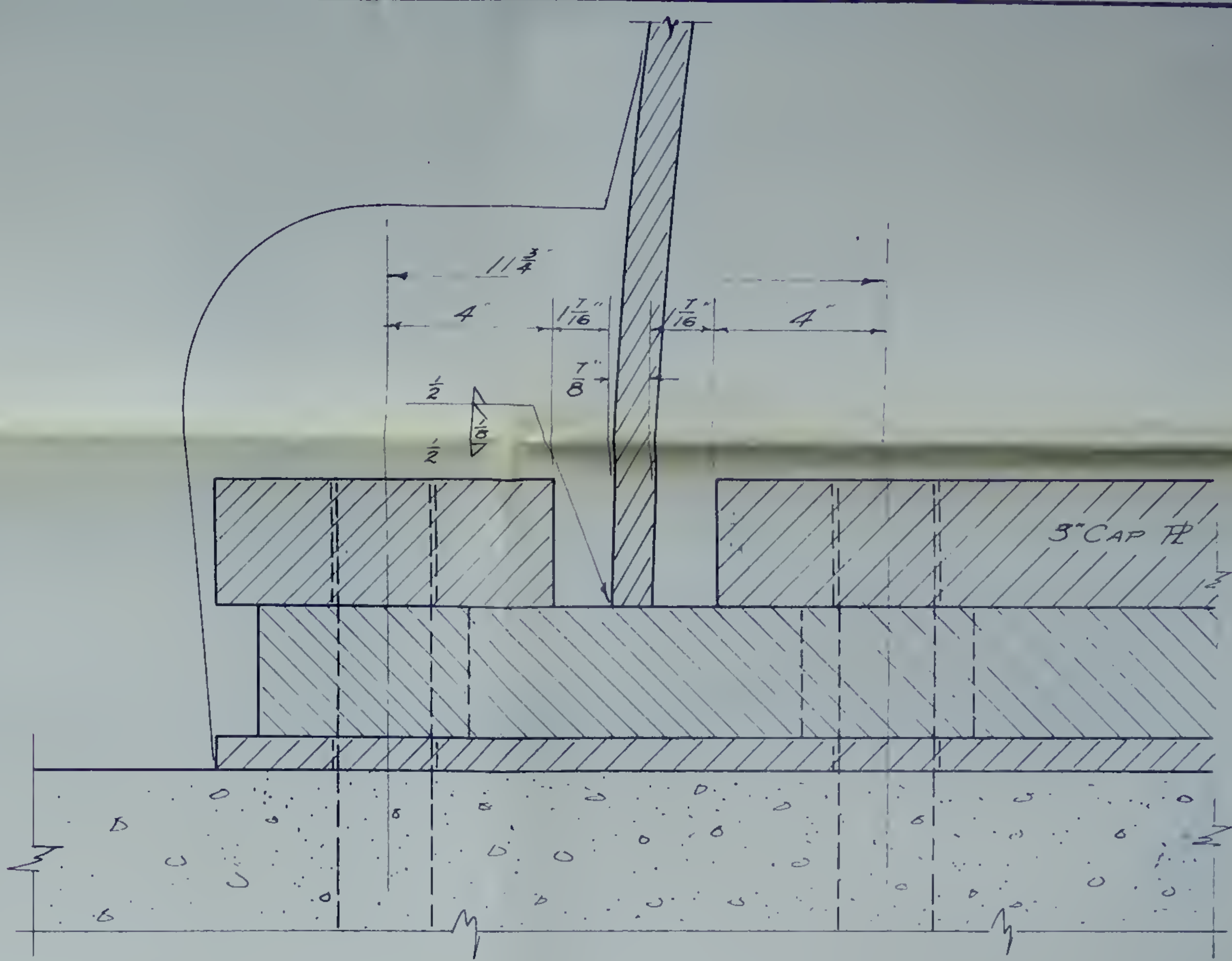

SOLDER
LIGHT GAGE SHEET METAL PROTECTIVE COVER
1'-1"
6½"
3 x 3 x 8'-5¾" FILLER BAR
SECTION at E-
Scale: ¼"=1"
WEARING COURSE
3" U.S.S. ARMORED BEAM-LOK
1" WEARING COURSE
BASE
SUB-BASE
SLIDING SURFACE
12 WF 40
60'-0 to ℄
D
WEDGE FLOOR BEAM
APPROACH FILL
8'-0¾"
8'-0 RADIUS
ACCESS DOOR 2'-0 x 3'-0
12" min ABUTMENT BACKWALL
CAP PLATE
SLIDING PLATE
BEARING PLATE
ELEVATION
Scale: ¾"=1'-0
2½"
60° min
SLIDING ℞
45° min



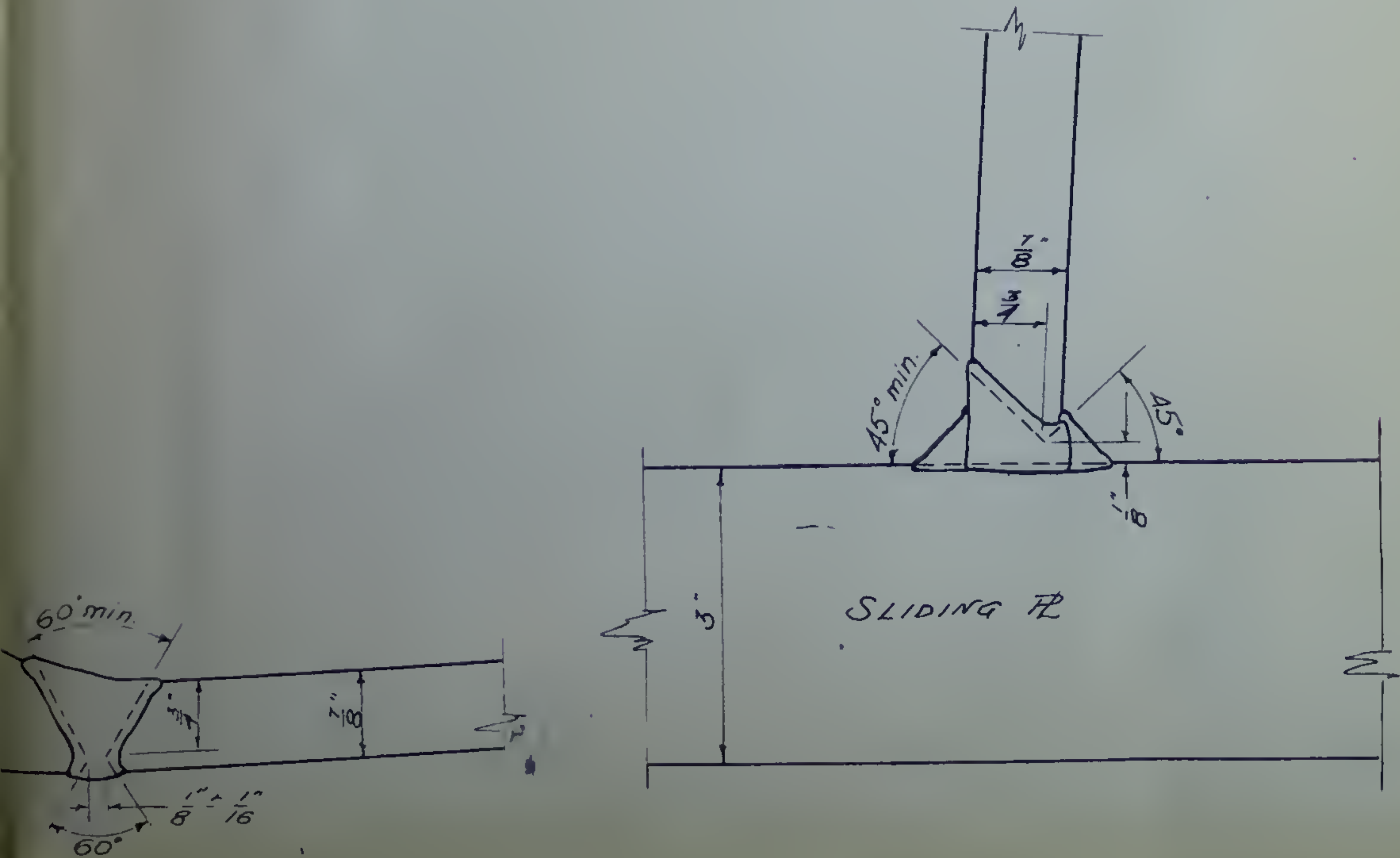
SECTION at E-E
Scale: 1/4" = 1"

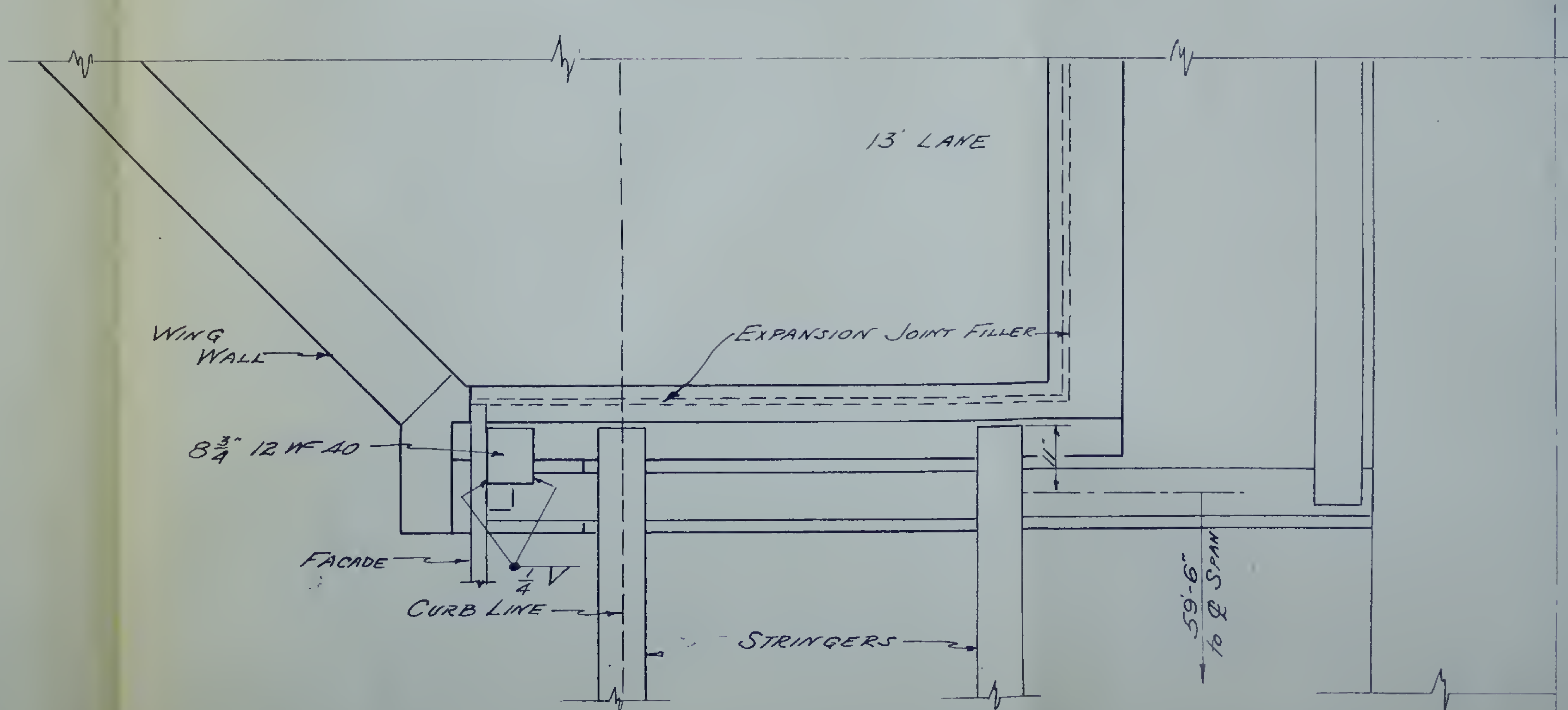


SECTION at F-F
Scale: 1/4" = 1"



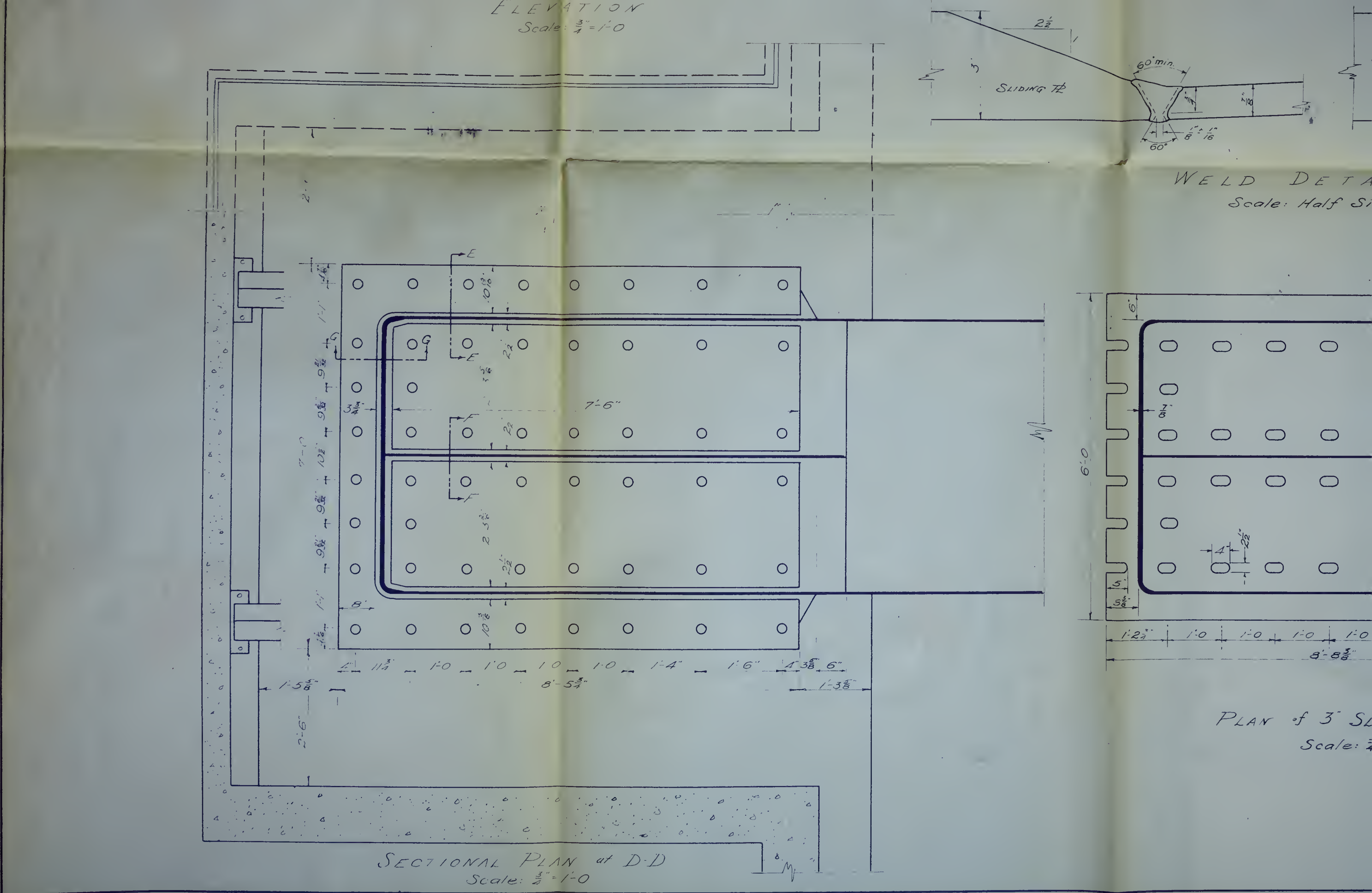
SECTION at G-G
Scale: 1/4" = 1"





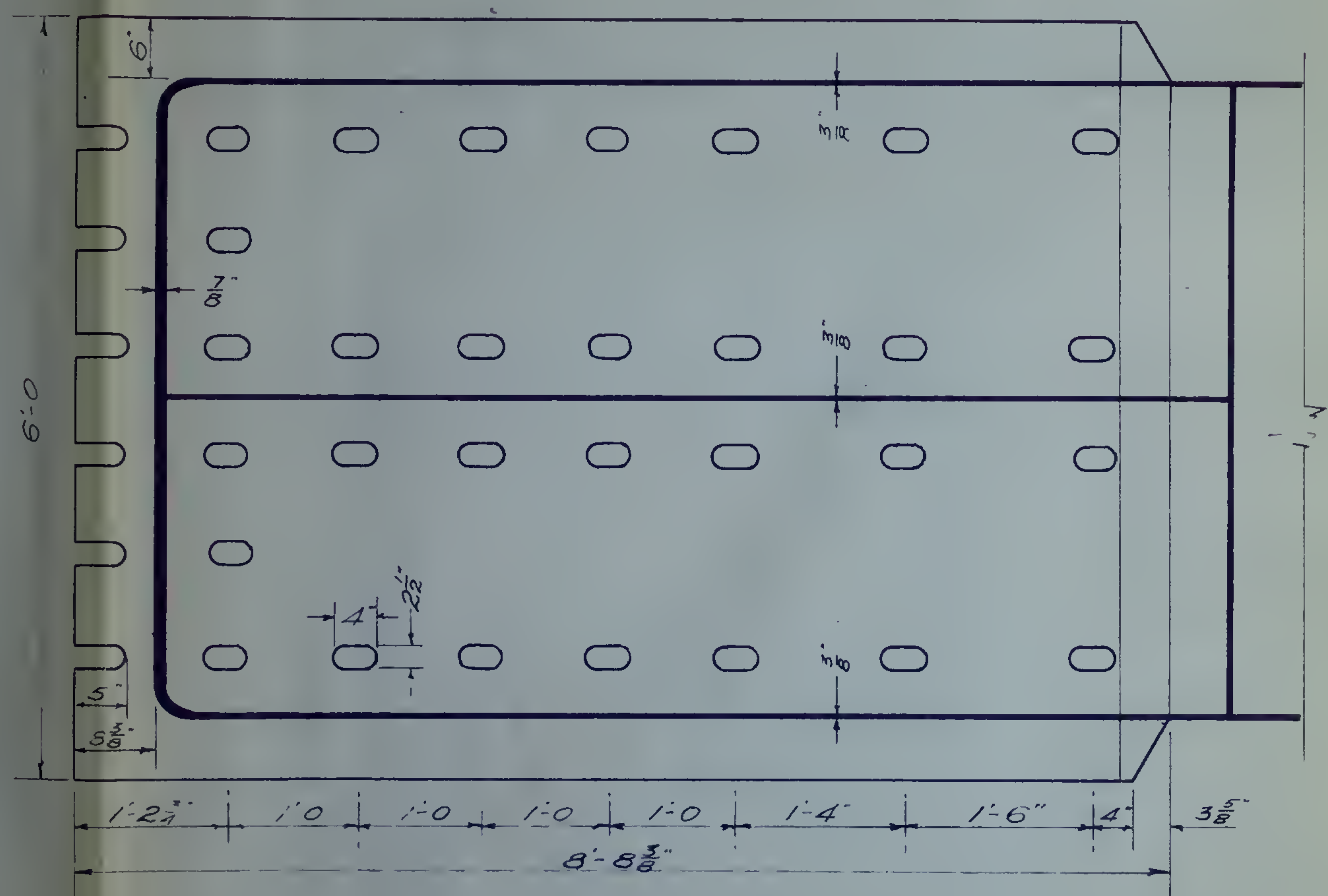
PLAN of BRIDGE to ABUTMENT CONNECTION
Scale: 1/2" = 1'-0

Sym. ℄

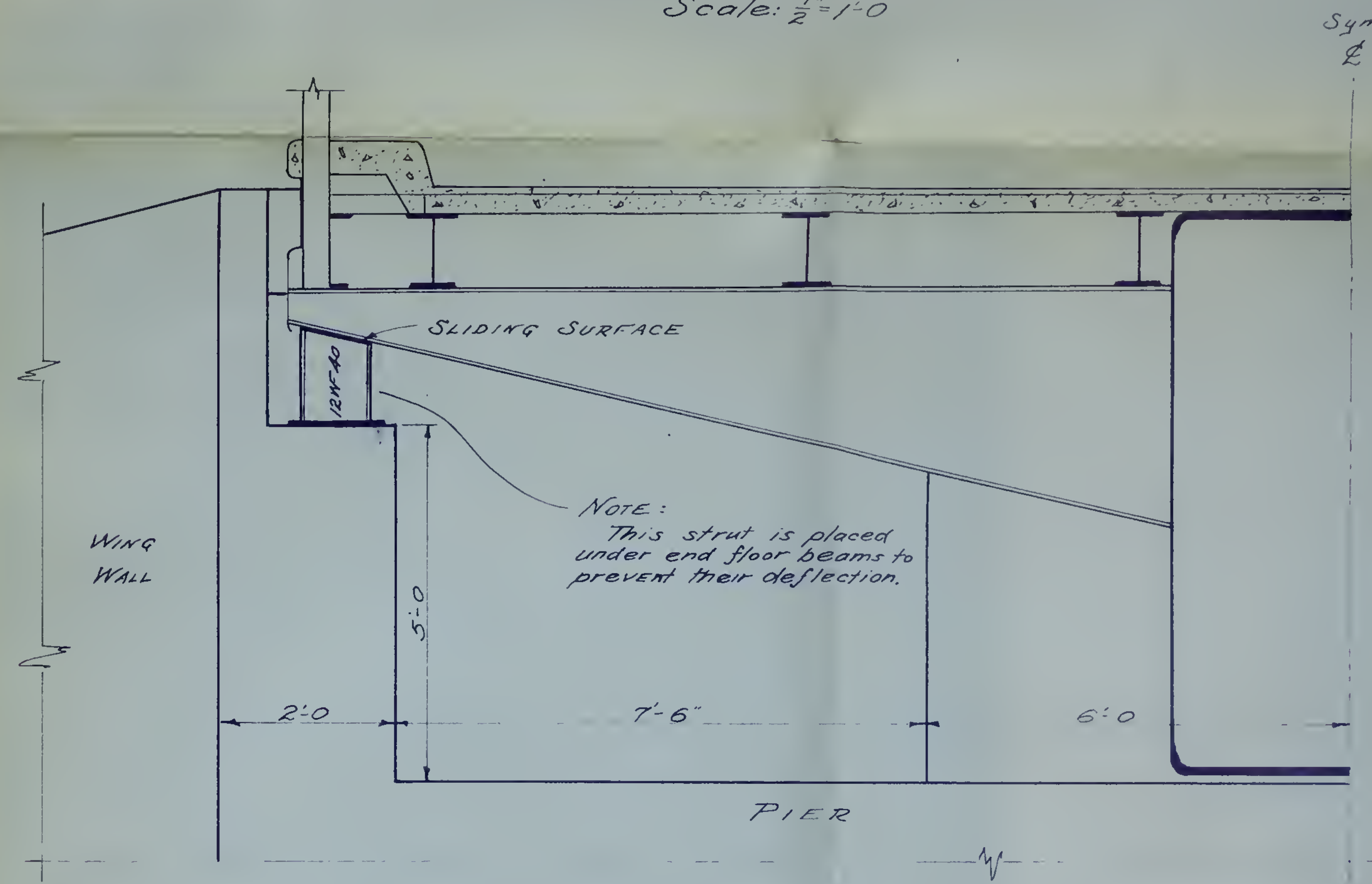

ELEVATION
Scale: 3/4"=1'-0
SLIDING PL
60° min.
60°
WELD DETA
Scale: Half Si
7'-6"
8'-5 3/4"
1'-5 5/8"
1'-3 5/8"
6'-0
8'-8 3/8"
PLAN of 3" SL
Scale:
SECTIONAL PLAN at D-D
Scale: 3/4"=1'-0

WELD DETAILS
Scale: Half Size

PLAN of BRIDGE to ABUTMENT CONNECTION
Scale: 1/2"=1'-0



PLAN of 3" SLIDING PLATE
Scale: 3/4"-1'-0



ELEVATION of BRIDGE to ABUTMENT CONNECTION
Scale: 1/2"=1'-0

TWO-LANE, 120' SPAN, DECK TYPE

HIGHWAY BRIDGE

END CONNECTION

DESIGNED BY:

JUNE 1949

SHEET 4 of 4