DESIGN OF A VIADUCT

from

DOREMUS MEMORIAL GYMNASIUM

to

WILSON ATHLETIC FIELD

Thesis Presented for the Degree of

BACHELOR OF SCIENCE

in

CIVIL ENGINEERING

by

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Lexington, Virginia.

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BIBLIOGRAPHY.

Design of Highway Bridges-- Ketchum

Structural Engineer's Handbook--Ketchum

Roofs and Bridges--Merriman and Jacoby

Modern Framed Structures--Johnson, Bryan, Turneaure

Pocket Companion--Carnegie Steel Co.

Engineering News Record.

Handbook of Building Construction--Hool and Johnson.

LOCATION OF PROPERTY.

The property of Washington and Lee University, Lexington, Virginia, upon which the work herein is contemplated, is situated to the North and West of the Doremus Memorial Gymnasium, as indicated on the accompanying map.

BENCH MARK.

All elevations given on the drawings are actual elevations above sea level. All work here is referred to a bench mark on top of wheel guard on North corner of Doremus Memorial Gymnasium, which has an elevation above sea level of 1061.29 feet.

DEFINITION OF PROBLEM.

The problem in this thesis is to provide some kind of suitable approach to the stadium on Wilson Athletic Field from the front of Doremus Memorial Gymnasium, for pedestrians.

SOLUTION OF PROBLEM.

The solution of this problem, as presented in this thesis, is to construct a steel viaduct with a 10 ft. plank walkway between the above mentioned points, details and design of which are given in the accompanying thesis.

NECESSITY OF BRIDGE.

This bridge is necessary as an approach to the athletic field for pedestrians, as it alleviates the two main bad features of the present approach. To understand these improvements, an idea of the present approach must be had. Leaving the gymnasium, the pedestrian descends approximately 50 ft. on narrow and dangerous steps to the Baltimore and Ohio Railroad track, walks approximately 300 ft. on railroad property, which is an exceedingly narrow fill, leaves the track to walk across the Washington and Lee Tennis Courts, and then climbs a steep and slippery bank approximately 20 ft. high, and then enters the stadium.

In the first place, this viaduct will eliminate the discomfort and lost time incident to descending or climbing the steps, to walking either on the railroad ties or on the narrow cinder paths on each side, to crossing the tennis courts and climbing the steep path to the stadium, especially in wet weather, and above all, to the congestion existing at various points along this route after a large attended athletic contest.

In the second place, this viaduct will eliminate danger to pedestrians existant in two places along this route. First, the danger of trains passing when the railroad fill is packed from edge to edge with people, and second, the danger from automobiles while crossing the earth road between the stadium and the top of the bank above the tennis courts.

Again, there is the fact that there exists a law in Virginia about trespassing on railroad property, which is being flagrantly violated every day by people going to and from the athletic field. The proposed viaduct will provide sufficient clearance to satisfy the requirements of the B. & O. R. R. Co. and as the tower abutments rest on Washington and Lee University property, the law would be fulfilled and there could be no possible objection to this viaduct being constructed, although the Chief Engineer of the B. & O. R. R. Co. would have to approve the design of the span crossing the B. & O. R. R. right-of-way.

PRELIMINARY INVESTIGATION.

MODIFYING FACTORS.

Several peculiarities of the site make this design somewhat out of the ordinary. The proposed structure will have to cross the 100 ft. right-of-way of the Baltimore and Ohio Railroad at approximately an angle of 58 degrees. This necessitates the use of one span at least 140 feet. Also, to provide proper clearance of the railroad track and to continue the bridge floor level with the top step of the proposed addition to the stadium, some of the towers must be of uneconomical height. This bridge floor level also provides clearance for the earth road next to the stadium. Also, the fact that the University students may at some time cross the bridge in lock-step, necessitates designing the bridge for a large live load in order to properly care for the vibration due to this.

ECONOMY.

Leaving out of consideration the 140 foot span, which is necessary, it is found that alternate 50 foot and 20 foot plate girder spans resting on towers, provide the most economical structure, as then the cost of the superstructure more nearly equals that of the substructure. In the design of the 50 foot girder, the economical depth was found to be 42 inches. For the sake of uniformity of appearance, the 20 foot girder was made the same depth. To be in keeping as far as possible with the rest of the structure, the 140 foot truss was arbitrarily given a depth of 7 feet.

BEAUTY.

The finished viaduct will present a very pleasing appearance, the depth being uniform with the exception of the truss (the handrailing on the girders practically equalling this, however), and the entire structure being made of steel except the footings and the floor.

GENERAL SPECIFICATIONS FOR CONCRETE ABUTMENT AND FOOTINGS.

Inspection and Approval.

All materials shall be subject to the inspection and approval of the Engineers, and all materials rejected must be removed immediately from the property.

Cement.

All cement must be a recognized brand of American Portland Cement which has been in general for a period of at least two years immediately prior hereto, of uniform quality, color and wieght, and shall comply with the Standard Specifications for Portland Cement, C 9-21, adopted by the American Society for Testing Materials.

Fine Aggregate.

Fine aggregate shall consist of a washed natural sand having clean grains of strong durable materials. Sand containing shale or soft or flaky particles totaling more than 2 % by wieght, or sand containing more than 5 % by weight of silt, or sand containing strong alkali, shall not be used. The sand shall be of such size and grading that 95 % or more shall pass a # 4 sieve and that at least 30 %, but not more than 65 %, shall pass the No. 20 sieve.

Coarse Aggregate.

Coarse aggregate shall consist of crushed rock or washed gravel, containing only clean uncoated pieces of strong and durable minerals. Aggregates containing soft, friable, thin, elongated or laminated pieces, totaling more than 3 % by weight, or containing shale or soapstone in excess of 1 % by weight, or containing organic matter or strong alkali, shall not be used. The coarse aggregate shall have a maximum size of 1½", at least 40 %, but not more than 75 % will pass a screen having ¾ " openings, and not more than 5 % will pass a No. 4 sieve.

Water.

All water used in mixing material shall be free from oil, acid, alkali, or organic matter.

Mixing.

The concrete shall be mixed in a batch mixer of a type to be approved by the Engineers, except where special permission is given by the Engineers. The mixing of each batch shall be continued not less than one and one-half minutes after all the materials, including the water, are in the mixer. The mixer shall rotate at a peripheral speed of about 200' per minute. When hand-mixing is authorized by the Engineers, it shall be done on a water-tight platform. The cement and fine aggregate shall first be mixed dry until the whole is of a uniform color. The water and coarse aggregate shall then be added and the entire mass turned at least three times or until a homogenous mixture is obtained.

Mixture.

The plain concrete footings and abutment shall be composed of one (1) part of cement, two and one-half (2½) parts of sand and five (5) parts of coarse aggregate. This proportion shall be subject to such changes as the Engineers may determine after testing the aggregates to be used therein. The concrete shall be of such consistency when placed that a light ramming will be necessary to fully flush the mortar to the surface. Such concrete shall be placed in layers not over twelve inches thick. The concrete footings shall finish perfectly smooth and level at the top and to the elevations shown on the drawings, properly prepared to receive the superstructure.

GENERAL SPECIFICATIONS FOR STEEL WORK.

(Taken largely from specifications by Milo S. Ketchum.)

DESIGN.

Material.

All parts of the structure shall be of rolled steel, except the flooring and nailing strips. Cast iron or cast steel may be used for bed plates.

Type of Truss.

Warren truss with sub-verticals.

Length of Span.

In calculating the stresses the length of span shall be taken as the distance between centers of end bearing plates for the riveted truss and the girders, and center to center of trusses for floorbeams.

Lateral Bracing.

All lateral bracing shall have riveted connections. The truss shall be braced by a bracket at each floorbeam and the plate girders shall be stayed by gusset plates at each floorbeam.

Handrailing.

A strong and suitable handrailing shall be placed at each side of the bridge and be rigidly attached to the superstructure.

Trestle Towers.

Trestle bents shall preferably be composed of two supporting columns, two bents forming a tower; each tower thus formed shall be thoroughly braced in both directions and have struts between the feet of the columns. The feet of the columns must be secured to an anchorage capable of resisting one and one-half times the specified wind forces.

FLOOR SYSTEM.

Floorbeams.

All floorbeams shall be rolled steel I-beams, rigidly connected to the trusses at the panel points. Floorbeams shall be square to the trusses and girders.

Stringers.

All stringers shall be rolled steel channels placed on edge. Steel stringers shall be riveted to the webs of floorbeams by means of standard angle connections. Outside stringers shall be designed for the same live loads as the intermediate stringers.

Floor Plank.

For single thickness the footwalk planks shall not be less than 2 in. thick nor more than 6 in. wide, spaced with $\frac{1}{2}$ in. openings. All planks shall be laid with heart side down. Nailing strips shall be provided bolted to the stringers to insure that the planks will have full and even bearing on and be firmly attached to the stringers.

LOADS.

Dead Load.

The dead load will consist of (1) the weight of the metal, and (2) the weight of the timber in the floor, or of the material other than steel. In determining the dead load the weight of the wood shall be taken as 4 lbs. per foot board measure.

Live Load.

The entire bridge shall be designed to carry, in addition to its own weight and that of the floor, a moving load of 140. lbs. per sq. ft. , either uniform or concentrated, placed so as to give the greatest stress in each member.

Wind Loads.

The lateral bracing in the unloaded chords of the truss shall be designed for a lateral wind load of 150 lbs. per lineal foot of bridge, considered as a moving load. The lateral bracing in the loaded chords of the truss shall be designed for a lateral wind load of 300 lbs. per lineal foot of bridge, considered as a moving load.

In trestle towers the bracing and columns shall be designed to resist the following lateral forces, in addition to the stresses due to dead and live loads: The truss or girders loaded or unloaded, the lateral pressures specified above; and a lateral pressure of 100 lbs. for each vertical lineal foot of trestle bent.

UNIT STRESSES AND PROPORTION OF PARTS.

Unit Stresses.

All parts of the structure shall be proportioned so that the sum of the maximum stresses shall not exceed the following amounts in lb. per sq. in.

Tension. Axial tension on net section 16000.

The lengths of riveted tension members in horizontal or inclined positions shall not exceed 200 times their radius of gyration about the horizontal axis. The horizontal projection of the unsupported portion of the member is to be considered as the effective length.

Compression. Axial compression on gross section 16000-70l/r with a maximum of 14000. lb.; where"l" is the length of the member in inches and "r" is the least radius of gyration in inches.

No compression member shall have a length not exceeding 125 times its least radius of gyration for main members or 150 times for laterals.

Bending.

Bending: on extreme fibers of rolled shapes, built sections and girders;

Net section	16000
on extreme fibers of pins	24000

Shearing.

Shearing: shop driven rivets and pins	12000.
field driven rivets and turned bolts	10000.
plate girder webs; gross section	10000.

Bearing.

Bearing: shop driven rivets and pins	24000.
field driven rivets and turned bolts	20000.
granite masonry and Portland cement concrete	600.
expansion rollers; per linear inch	600.d

where "d" is the diameter of the roller in inches.

Rivets shall not be used in direct tension; except for lateral bracing where unavoidable; in which case the value for direct tension on the rivet shall be taken the same as for single shear.

Alternate Stresses.

Members subject to alternate stresses shall be proportioned for the stresses giving the largest section.

Angles in Tension.

When single-angle members subject to direct tension are fastened by one leg, only seventy-five percent of the net area shall be considered effective.

Net Section.

In members subject to tensile stresses full allowance shall be made for reduction of section by rivet-holes. In calculating net area the rivet-holes shall be taken as having a diameter 1/8 in. greater than the normal size of the rivet.

Wind Stresses.

The stresses in truss members or trestle posts from assumed wind forces need not be considered except as follows:

1. When the direct wind stresses in any member per sq. in. exceed 25% of the stresses due to dead and live loads in the same member.

2. When the wind stress alone or in combination with a possible temperature stress can neutralize or reverse the stresses in the member.

When both direct and flexural stresses due to wind are considered 50 percent may be added to allowable stresses for dead and live loads, provided the area thus obtained is not less than required for dead and live loads alone.

Combined Stresses.

Members subjected to direct and bending stresses shall be designed so that the greatest fiber stress shall not exceed the allowable unit stress on the member.

Design of Plate Girders.

Plate girders shall be proportioned by assuming that the flanges are concentrated at their centers of gravity, in which case one-eighth of the gross section of the web may be used as flange section. The thickness of web plates shall not be less than 5/16 in., nor less than 1/160 of the unsupported distance between flange angles.

Compression Flanges.

In beams and plate girders the compression flanges shall have the same gross section as the tension flanges. Through plate girders shall have their top flanges stayed at each end of every floorbeam by gusset plates. The stress per sq. in. in compression flange of any beam or girder shall not exceed 16000.-200 l/b. when flange consists of angles with a flat cover plate, where "l" is the unsupported distance and "b" is the width of flange.

Web Stiffeners.

There shall be web stiffeners, in pairs, over bearings, at points of concentrated loading, and at other points where the thickness of the web is less than 1/60 of the unsupported distance between flange angles. The distance between stiffeners shall not exceed the clear depth of web.

Flange Rivets.

The flanges of plate girders shall be connected to the web with a sufficient number of rivets to transfer the total shear at any point in a distance equal to the effective depth of the girder at that point combined with any load that is applied directly on the flange.

Low Truss.

The riveted low truss shall have top chords composed of a double web member with a cover plate. The top chords shall be stayed against lateral bending by means of brackets rigidly connected to each floorbeam. The floorbeams shall be riveted above the lower chord.

Rolled Beams.

Rolled beams shall be designed by using their moment of inertia. The web shall be assumed to take care of all the shear.

DETAILS OF DESIGN.

Open Sections.

The structure shall be so designed that all parts will be accessible for inspection, cleaning and painting.

Water Pockets.

Pockets or depressions which would hold water shall have drain holes, or be filled with waterproof material.

Symmetrical Sections.

Main members shall be so designed that the neutral axis will be as nearly as practicable in the center of section, and the neutral axes of intersecting main members of trusses shall meet at a common point.

Strength of Connections.

The strength of connections shall be sufficient to develop the full strength of the member, even though the computed stress is less, the kind of stress to which the member is subjected being considered.

Pitch of Rivets.

The minimum distance between centers of rivet holes shall be three diameters of the rivet; but the distance shall preferably be not less than 3 in. for 7/8 in. rivets and $2\frac{1}{2}$ in. for $\frac{3}{4}$ in. rivets. The maximum pitch in the line of stress for members composed of plates and shapes shall be 6 in. For angles with two gage lines and rivets staggered, the maximum shall be twice the above in each line. In tension members composed of two angles in contact, a pitch of 12 in. will be allowed for riveting the angles together.

Edge Distance.

The minimum distance from center of any rivet hole to a sheared edge shall be $1\frac{1}{2}$ in., and to a rolled edge $1\frac{1}{4}$ in. for all rivets.

Maximum Diameter.

The diameter of the rivets in any angle carrying calculated stresses shall not exceed one-quarter of the width of the leg in which they are driven.

Pitch at Ends.

The pitch of rivets at the ends of built compression members shall not exceed four diameters of the rivets, for a length equal to one and one-half times the maximum width of member.

Compression Members.

In compression members the metal shall be concentrated as much as possible in webs and flanges. The thickness of each web shall be not less than one-thirtieth of the distance between its connections to the flanges. Cover plates shall have a thickness not less than one-fortieth of the distance between rivet lines.

Minimum Angles.

Flanges of girders and built members without cover plates shall have a minimum thickness of one-twelfth of the width of the outstanding leg.

Batten Plates.

The open sides of all compression members shall be stayed by batten plates at the ends and diagonal lattice-work at intermediate points. The batten plates must be placed as near the ends as practicable.

Lattice Bars.

The latticing of compression members shall be proportioned to resist the shearing stresses corresponding to the allowance for flexure for uniform load provided in the column formula by the term 70 l/r. They must not be less in width than $1\frac{1}{2}$ in. for 6 in. members, $1\frac{3}{4}$ in. for 9 in. members, 2 in. for 12 in. members, $2\frac{1}{4}$ in. for 15 in. members, nor $2\frac{1}{2}$ in. for members 18 in. and over in width. Double lattice bars connected by a rivet at the intersection shall have a thickness not less than one-sixtieth of the distance between the rivets connecting them to the members. They shall be inclined at an angle not less than 45° for double latticing with riveted connections. The pitch of the lattice bars must not exceed the width of the channel plus nine inches. $\frac{3}{4}$ in. rivets are to be used in latticing flanges from $2\frac{1}{2}$ to $3\frac{1}{2}$ in. wide.

Splices.

In compression members joints with abutting faces planed shall be placed as near the panel points as possible, and must be spliced on all sides with at least two rows of rivets on each side of the joint. Joints with abutting faces not planed should be fully spliced. Joints in tension members should be fully spliced.

Bolts.

Where members are connected by bolts, the turned body of these bolts shall be long enough to extend through the metal. A washer at least $\frac{1}{4}$ in. thick shall be used under the nut. Bolts shall not be used in place of rivets except by special permission. Heads and nuts shall be hexagonal.

Expansion.

Provision for expansion to the extent of 1/8 of an inch for each 10 ft. shall be made for the bridge.

Expansion Bearings.

Spans of 60 ft. and over shall have turned rollers or rockers at one end; and those of less length shall be arranged to slide on smooth surfaces.

Fixed Bearings.

Movable bearings shall be designed to permit motion in one direction only. Fixed bearings shall be firmly anchored.

Pedestals and Bed Plates.

Built pedestals shall be made of plates and angles. All bearing surfaces must be planed. The vertical webs must be secured to the base by angles having two rows of rivets in the vertical legs. No base plate or web connecting angle shall be less in thickness than $\frac{1}{2}$ in. The vertical webs shall be of sufficient height and must contain material and rivets enough to practically distribute the loads over the bearings or rollers. All the bed plates and bearings under fixed and movable ends must be fox-bolted; for trusses, these bolts must not be less than $1\frac{1}{4}$ in. diameter; for plate and other girders not less than 7/8 in. diameter.

Wall Plates.

Wall plates may be cast or built up; and shall be so designed as to distribute the load uniformly over the entire bearing.

Anchorage.

Anchor bolts for viaduct towers shall be long enough to engage a mass of masonry the weight of which is at least one and one-half times the uplift.

Camber.

Truss spans shall be given a camber by making the panel length of the top chords, or their horizontal projections, longer than the corresponding panels of the bottom chord in the proportion of 3/16 in. in 10 ft.

MATERIALS AND WORKMANSHIP.

MATERIAL.

Process of Manufacture.

Steel shall comply with the standard specifications of the Am. Ry. Eng. Assoc.

Timber.

The timber shall be strictly first-class cyprus, Southern yellow pine, or white oak bridge timber; sawed true, full size, free from wind shakes, large or loose knots, decayed or sapwood, wormholes or other defects impairing its strength or durability.

WORKMANSHIP.

General.

All parts forming the structure shall be built in accordance with approved drawings. The workmanship and finish shall be equal to the best practice in modern bridge works.

Straightening Material.

Material shall be thoroughly straightened in the shop, by methods that will not injure it, before being laid off or worked in any way.

Finish.

Shearing shall be neatly and accurately done and all portions of the work exposed to view neatly finished.

Size of Rivets.

The size of rivets, called for on the plans, shall be understood to mean the actual size of the cold rivet before heating.

Rivet Holes.

When general reaming is not required the diameter of the punch shall not be more than 1/16 in. greater than the diameter of the rivet; nor the diameter of the die more than 1/8 in. greater than the diameter of the punch. Material more than ¾ in. thick shall be sub-punched and reamed or drilled from the solid.

Punching.

All punching shall be accurately done. Drifting to enlarge unfair holes will not be allowed. If the holes must be enlarged to admit the rivet, they shall be reamed. Poor matching of holes will be cause for rejection.

Sub-punching and Reaming.

Where reaming is required, the punch used shall have a diameter not less than 3/16 in. smaller than the nominal diameter of the rivet. Holes shall then be reamed to a diameter not more than 1/16 in. larger than the nominal diameter of the rivet. All reaming shall be done with twist drills.

Reaming after Assembling.

When general reaming is required it shall be done after the pieces forming one built member are assembled and firmly bolted together. If necessary to take the pieces apart for shipping and handling, the respective pieces reamed together shall be so marked that they may be reassembled in the same position in the final setting up. No interchange of reamed parts will be allowed.

Edge Planing.

Sheared edges or ends shall, when required, be planed at least 1/8 in.

Burrs.

The outside burrs on reamed holes shall be removed.

Assembling.

Riveted members shall have all parts well pinned up and firmly drawn together with bolts, before riveting is commenced. Contact surfaces to be painted.

Lattice Bars.

Lattice bars shall have neatly rounded ends.

Web Stiffeners.

Stiffeners shall fit neatly between flanges of girders. The ends of stiffeners shall be faced and shall be brought to a true contact bearing with the flange angles.

Splice Plates and Fillers.

Web splice plates and fillers under stiffeners shall be cut to fit within 1/8 in. of flange angles.

Web Plates.

Web plates of girders shall have ¼ in. clearance from the backs of flange angles.

Connection Angles.

Connection angles for floorbeams and stringers shall be flush with each other and correct as to position and length of girder. In case milling is needed or required after riveting, the removal of more than 1/16 in. from their thickness will be cause for rejection

Rivets.

Rivets shall be driven by pressure tools wherever possible. Pneumatic hammers shall be used in preference to hand driving.

Riveting.

Rivets shall look neat and finished, with heads of approved shape, full and of equal size. They shall be central on shank and grip the assembled pieces firmly. Recupping and caulking will not be allowed. Loose, burned or otherwise defective rivets shall be cut out and replaced. In cutting out rivets, great care shall be taken not to injure the adjacent metal. If necessary, they shall be drilled out.

Members to be Straight.

The several pieces forming one built member shall be straight and fit closely together, and finished members shall be free from twists, bends or open joints.

Finish of Joints.

Abutting joints shall be cut or dressed true and straight and fitted close together, especially where open to view. In compression joints, depending on contact bearing, the surfaces shall be truly faced, so as to have even bearings after they are riveted up complete and when perfectly aligned.

Field Connections.

Holes for floorbeams and stringer connections shall be sub-punched and reamed to a steel templet one inch thick.

Pin-Holes.

Pin-holes shall be bored true to gages, smooth and straight; at right angles to the axis of the member and parallel to each other.

Pins and Rollers.

Pins and rollers shall be accurately turned to gages and shall be straight and smooth and free from flaws.

Screw Threads.

Screw threads shall make tight fits in the nuts and shall be U. S. standard, except above the diameter of 1 3/8 in., when they shall be made with six threads per inch.

Annealing.

Steel, except in minor details, which has been partially heated, shall be properly annealed.

Steel Castings.

All steel castings shall be annealed.

Welds.

Welds in steel will not be allowed.

Bed Plates.

Expansion bed plates shall be planed true and smooth. Cast wall plates shall be planed top and bottom. The cut of the planing tool shall correspond with the direction of expansion.

Pilot Nuts.

Pilot nuts and driving nuts shall be furnished for each size of pin, in such numbers as may be ordered.

Field Rivets.

Field rivets shall be furnished to the amount of 15 % plus ten rivets in excess of the nominal number required for each size.

Shipping Details.

Pins, nuts, bolts, rivets and other small details shall be boxed and crated.

Weight.

The weight of every piece and box shall be marked on it in plain figures.

Finished Weight.

Payment for pound price contracts shall be by weight scale. No allowance over 2 per cent of the total weight of the structure as computed from the plans will be allowed for excess weight.

SHOP PAINTING.

Cleaning.

Steel work, before leaving the shop, shall be thoroughly cleaned and given one good coating of pure linseed oil, or such paint as may be called for, well worked into all joints and open spaces.

Contact Surfaces.

In riveted work, the surfaces coming in contact shall each be painted before being riveted together.

Inaccessible Surfaces.

Pieces and parts which are not accessible for painting after erection, shall have a good coat of paint before leaving the shop.

Condition of Surfaces.

Painting shall be done only when the surface of the metal is perfectly dry. It shall not be done in wet or freezing weather, unless protected under cover.

Machine-finished Surfaces.

Machine-finished surfaces shall be coated with white lead and tallow before shipment or before being put out into the open air.

INSPECTING AND TESTING AT THE SHOP AND MILL.

Facilities for Shop Inspection.

The manufacturer shall furnish all facilities for inspecting and testing the weight and quality of workmanship at the shop where material is manufactured. He shall furnish a suitable testing machine for testing full-sized members, if required.

Starting Work in Shop.

The purchaser shall be notified well in advance of the start of the work in the shop, in order that he may have an inspector on hand to inspect material and workmanship.

Copies of Mill Orders.

The purchaser shall be furnished complete copies of mill orders, and no material shall be rolled, nor work done, before the purchaser has been notified where the orders have been placed, so that he may arrange for the inspection.

Facilities for Mill Inspection.

The manufacturer shall furnish all facilities for inspecting and testing the weight and quality of all material at the mill where it is manufactured. He shall furnish a suitable testing machine for testing the specimens, as well as prepare the pieces for the machine, free of cost.

Access to Mills.

When an inspector is furnished by the purchaser to inspect material at the mills, he shall have full access, at all times, to all parts of mills where material to be inspected by him is being manufactured.

Accepting Material or Work.

The inspector shall stamp each piece accepted with a private mark. Any piece not so marked may be rejected at any time, and at any stage of the work.

Shop Plans.

Complete shop plans shall be furnished.

Shipping Invoices.

Complete copies of shipping invoices shall be furnished with each shipment.

ERECTION.

If the contractor erects the bridge, he shall furnish all staging and falsework, erect and adjust all metal work, and shall frame and put in place all floor timbers, etc., complete ready for use.

The contractor shall drill all holes in footings and in the abutment and shall put in place all anchor bolts and set them in neat Portland cement.

The erection will also include all necessary hauling, unloading of materials and their proper care until erection is completed.

The contractor shall so conduct his work so as not to interfere with any kind of traffic.

The contractor shall assume all risks of accidents and damages to persons and properties prior to the acceptance of the work.

The contractor must remove all falsework and other obstructions or unisghtly material produced by his operations.

PAINTING AFTER ERECTION.

After the bridge is erected the metal work shall be thoroughly cleaned of mud, grease or other material, then thoroughly and evenly painted with two coats of paint of the kind specified by the engineer, mixed with linseed oil. All recesses which may retain water, or through water can enter, must be filled with thick paint or some waterproof cement before the final painting. The first coat must be allowed to dry thoroughly before the second coat is applied. All painting shall be done with best quality round brushes. The paint shall be delivered on the work in the original packages and is subject to inspection. The paint shall not be thinned with anything whatsoever; in cold weather the paint may be thinned by heating under the direction of the inspector. No turpentine or benzine shall be allowed on the work, except by permission of the inspector, and in such quantity as he shall allow. The inspector shall be notified when any painting is to be done by the contractor, and no painting shall be done until the inspector has approved the surface to which the paint is to be applied. Paint shall not be applied out of doors in freezing, rainy, or misty weather, and all surfaces to which paint is to be applied shall be dry, clean, and warm.

THE DESIGN OF THE 140 FOOT PONY RIVETED WARREN TRUSS BRIDGE.

GENERAL DESCRIPTION.

This is to be a pony truss bridge with riveted Warren trusses, It is the long span bridge in the viaduct crossing the hollow between Doremus Memorial Gymnasium and Wilson athletic field. The span is made this length so that the towers at both ends will clear the 100 foot right of way of the B. and O. R. R. The floor is to be composed of 3 in. planks resting on channel stringers, which in turn rest on I-beam floorbeams. The floorbeams will be riveted to the posts above the lower chords.

LOADS.

Dead Load. The dead load consists of the weight of the floor, the stringers, the floorbeams, the trusses and the lateral bracing.

Live Load. The live load is taken as 140 lbs. per sq. ft. of floor surface.

Wind Load. The lateral bracing is designed for a moving load of 300 lbs. per lineal foot of bridge.

Impact. No allowance was made separately for impact, as the live load was taken large enough to include it.

DIMENSIONS.

Span 140 ft. c. to c. of bearings; panel length 7 ft.; width of walk 10 ft.; spacing of trusses 11 ft. 1 in. c. to c. about; depth of truss 7 ft. c. of g. to c. of g. chords.

DESIGN OF FLOOR SYSTEM.

The loads carried by the stringers are (1) the dead load which is made up of the weight of the stringers and the floor planking; (2) a uniform live load.

Considering that the outside stringers carry one-half the load on the inside stringers, the total maximum uniform load per linear foot per inside stringer is,

507 lbs. per linear foot.

Referring to the tables, it is seen that a 5 in. channel @ 6.7 lbs. per ft. will be satisfactory and will be used. The inside and outside stringers will be the same.

The loads carried by the floorbeams consist of (1) The dead load which is the weight of the floor system; (2) A uniform live load.

Each stringer reaction is 3594 lbs. and these stringer reactions act as concentrated loads on the floorbeams at 4 points. From the symmetry of these concentrated loads, it is seen that the dangerous section is at the middle of the beam. The maximum bending moment at this dangerous section is found to be 172512 in. lbs. Taking the unit stress as 13000 lbs. per sq. in. the section modulus required is 13.27 in. For an 8 inch I-beam, c = 4 in. and the moment of inertia required for an 8 inch I-beam is 53.08 in. Referring to the tables it is seen that an 8 in. I-beam @ 20.5# is satisfactory and will be used. End floorbeams are made the same as intermediate floorbeams.

The following floor system will be used:

Surface. 3 in. plank

Stringers. 5 in. channels @ 6.7 lbs. spaced about 3'-0" c. to c.

Floorbeams. 8 in. I-beams @ 20.5 lbs. spaced 7'-0" c. to c.

Weight of floor system per panel:

Surface.	$10 \times 3 \times 7 \times 4$	=	840 lbs.
Stringers.	$4 \times 7 \times 6.7$	=	188 lbs.
Floorbeam.	10×20.5	=	205 lbs.

Weight of floor per panel = 1233. lbs.

Total weight of floor system = 20×1233 = 24660. lbs. for the entire bridge.

STRESSES IN TRUSSES AND LATERAL SYSTEM.

Dead Load. The dead load is assumed to be 700. lbs. per linear foot including floor sustem and all steel.

Total dead load = 140×700 = 98000. lbs.

Dead joint load = 98000. / 20. = 4900. lbs.

Dead joint load per truss = 2450. lbs.

One-third of the dead joint load was considered as applied at the upper chord joints and the two-thirds at the lower chord joints. The dead load web stresses were calculated by multiplying the vertical shear at the section by the secant of the angle between the vertical and the diagonals. The dead load chord stresses were calculated by the method of chord increments.

Live Load. The live load was assumed to be 140 lbs. per sq. ft.

Total live load = $140 \times 10 \times 140$	=	196000. lbs.
Live joint load 196000. / 20	=	9800. lbs.
Live joint load per truss	=	4900. lbs.

The live load chord stresses are determined by use of a multiplication factor (a ratio between live and dead loads), and occur when the bridge is fully loaded. The live load web stresses are calculated by the same method as the dead load web stresses, the bridge being loaded at all times so as to give the greatest vertical shear at the section.

Wind Load. The wind load was considered as a moving load and was taken as 300 lbs. per lineal ft. The wind load was considered as moving so the maximum stresses in the laterals will occur with the longer segment loaded and in the chords with all joints loaded. The total wind load was considered as concentrated at the joints of the windward truss. The total shear in any panel was assumed as taken by the member which would carry it in tension. The effect of the floor was not considered.

Stress Sheet. Before proceeding with the design of members the stresses due to the various loadings were collected on the stress sheet. As soon as the size of a member was determined, it was recorded on the stress sheet.

DESIGN OF MEMBERS.

The upper chord and end-post sections will be made of two channels, flanges turned out, with a cover plate on top and batten plates on the under side. The section will be made wide to provide lateral rigidity. The width of cover plates given by the following formula represents conservative practice, $b = L/10 + 6$ in.; where b = minimum width in inches, and L = span in feet. The width of the cover plate should and will be in integral inches.

The lower chord sections will be made of four angles, battened, two with legs turned out placed outside of gusset plate, and two with legs turned in placed inside of gusset plate.

The inclined web members and vertical members will be made of two angles battened with legs turned in and placed inside of gusset plates. Where there are no gusset plates at one end of a vertical, a fill will be provided.

The web members of the lateral system will consist of single angles for the diagonals and two angles for the verticals, being fastened to the lower chord by plates.

Complete calculations for one of each type of member will be given. The size of most of the truss members will depend upon the width of the top chord sections so the top chord carrying the largest stress will be designed first. The top chord will be supported horizontally by brackets at each floorbeam.

DESIGN OF THE TOP CHORD "JL".

Dead load stress	122200. lbs.	(compression)
Live load stress	245300. lbs.	(compression)
Wind load stress	0	
Total	367500. lbs.	(compression)

Length for bending in a horizontal plane 168. in.

Length for bending in a vertical plane 84. in.

The width of the cover plate should be at least:

$b = L/10 + 6\text{in.} = 140/10 + 6 = 20.$ in.

Since the member is supported at the center against bending in a vertical plane, bending in a horizontal plane will probably control. The approximate radius of gyration for an axis perpendicular to the cover plate is 0. 32 *b* for this type of section or $r_B = 0.32 \times 20 = 6.4$ in.

Using this radius of gyration and the corresponding length, the approximate allowable unit stress is ;-

$S = 16000.-70\ l/r = 16000-70 \times 168/6.4 = 14260.$ lbs. per sq. in. But 14000. is maximum stress from this formula and will be used.

The approximate area required is:-

$A = P/S = 367500/14000. = 26.25$ sq. in.

The gage lines of the cover plate will be 17 inches apart, allowing $1\frac{1}{2}$ in. edge distance on each side. The thickness of the cover plate must be at least $1/40 \times 17 = .425$ in. so a 7/16 in. plate will be used. Deducting the area of the 20 in. × 7/16 in. cover plate from the total area required, the area to be provided by the two channels is 17.50 sq. in. or 8.75 sq. in. for each channel. Referring to a table of the properties of channels it was seen that a 12 in. channel @ 30 lb. will provide sufficient area for this case and also allow considerable reduction in area. The 12 in. channel @ 30 lb. will therefore be used.

The width of flange of a 12 in. channel @ 30 lbs. is $3\frac{1}{4}$ in., and the gage is 2 in., leaving $1\frac{1}{4}$ in. of flange projecting beyond the gage. The rivet line of the cover plate should therefore not be less than $1\frac{1}{4}$ in. from the edge of the plate. The maximum rivet allowed in the flange of this channel is 7/8 in. and the minimum edge distance for a 7/8 in. rivet is 1 5/16 in. An edge distance of $1\frac{1}{2}$ in. will be used making the rivet lines of the plate 17 in. apart, and the distance back to back of the channels 13 in.

DETAILS OF DESIGN OF "JL".

The properties of the section composed of 2 channels 12 in. @ 30 lbs. placed 13 in. back to back and a cover plate 20 in $\times$ 7/16 in. will now be calculated.

Area of channels $= 2 \times 8.79 = 17.58$ sq. in.

Area of plate $= 20 \times 7/16 = \underline{8.75}$ sq. in.

Total area $=$ 26.33 sq. in.

The distance from the horizontal axis to the centroid of the section is found by taking moments about the horizontal axis and dividing by the area:-

Moments of channel 0

Moment of plate $8.75 \times 6.22 = \underline{54.5}$

Total 54.5

eccentricity $= e = 54.5/26.33 = 2.06$ in.

The moment of inertia about the horizontal axis is first determined;-

Channels, $2 \times I = 2 \times 161.2 =$ 322.4

Plate, $8.75 \times (6.22)^2 =$ $\underline{339.0}$

Total I_H about horizontal axis $=$ 661.4 in.^{4}

The moment of inertia about the centroid of the section is next determined:-

$$I_A = I_M - Ae^2 = 661.4-(26.33 \times \overline{2.06}^2) = 549.8 \text{ in.}^4$$

The moment of inertia about the vertical axis of the section is next determined:-

Channels, $2(5.2+8.79(.68+6.50)^2) = 919.1$

Plate, $1/12bd^3 = 1/12 \times 7/16 \times 20 \times 20 \times 20 = \underline{292.0}$

Total I_B $= 1211.1$ in.^{4}

The radii of gyration are next determined:-

Centroid axis, $r_A = \sqrt{I_A/A} = \sqrt{\frac{549.8}{26.33}} = 4.56$ in.

Vertical axis, $r_B = \sqrt{I_B/A} = \sqrt{\frac{1211.1}{26.33}} = 6.78$ in.

For bending in a horizontal plane, $l/r = \frac{168}{6.78} = 24.8$

For bending in a vertical plane, $l/r = \frac{84}{4.56} = 18.5$

The maximum l/r allowed for main compression members is 125 so this section is well below the limit. The maximum l/r occurs for bending in a horizontal plane and is 24.8 The allowable unit stress then is :-

$S = 16000-70\ l/r = 16000-70 \times 24.8 = 14264.$ lb. per sq. in.

$A = P/S = 367500./14264 = 25.8$ sq. in.

The actual area is 26.33 sq. in., so the section assumed is satisfactory as sufficient area is provided and no reduction is possible.

DESIGN OF LOWER CHORD "ik".

The lower chord carrying the maximum stress is ik so this member should be designed first.

Dead load stress	121100. lb. (tension)
Live load stress	243500. lb. (tension)
Total	364600. lb. (tension)

Wind 70600 lb. tension or 73600. lb. compression. The wind load tension is less than the dead load tension, the wind load tension is less than 25% of the sum of dead load and live load tension, and the wind load compression is less than the dead load tension, so the wind stress may be neglected.

The net area required for this member is:-

$A = P/S = 364600./16000. = 22.79$ sq. in.

Deducting one less holes than there are gage lines on the angles and considering a 1" hole for a 7/8" rivet, the following section can be used:-

4 angles $7'' \times 3\frac{1}{2}'' \times \frac{3}{4}''$, net area $= 4(7.31 - 2\times\frac{3}{4}\times 1) = 23.24$ sq. in.

No material thicker than $\frac{3}{4}''$ should be used on account of the difficulty of punching. The thickness of the gusset plates will not exceed $\frac{1}{2}''$. The distance back to back of the channels of the top chord is 13 in. so the clear distance between gusset plates is at least 12 in. With the $3\frac{1}{2}''$ legs of the angles turned in, sufficient allowance is provided for clearance, overrun of angles, and drainage.

The required net area is 22.79 sq. in. and the net area provided by 4 angles $7'' \times 3\frac{1}{2}'' \times \frac{3}{4}''$ is 23.24 sq. in., so this section will be adopted if the ratio of l/r does not exceed 200. The 7 in. leg will be placed vertical. The radius of gyration about the horizontal axis is 2.22 in. and the length is 168 in., making $l/r = 168/2.22 = 76$. which is satisfactory.

All compression members are designed on the same principles as the top chord, and the design will not be repeated here.

All tension members are designed on the same principles as the lower chord and the design will not be repeated here.

DESIGN OF JOINTS.

All joints will be designed to develop the full strength of the members and not simply the calculated stress.

The gusset plates will be made at least thick enough to develop in bearing the strength of the rivets in single shear. This thickness will be made $\frac{1}{2}$ in. The plates must be made of sufficient size to contain the necessary rivets and to carry the stresses transmitted from the members.

The arrangement of the members at the joints is shown on the general plan drawings. The centers of gravity of all members are placed on the center line of the truss. The size of gusset plates is determined by the space required for the rivets necessary to connect the members to the plate. To secure uniformity of stress the rivets will be symmetrically spaced.

JOINT "B".

This joint should be designed first. The gusset plates will be shop riveted to BC and field riveted to all other members.

Bearing controls the number of rivets in a B. The number of field rivets required will be:

$$\frac{297000}{(.875 \times .28 \times 20000)} = 61 \text{ or } 31 \text{ on each side.}$$

The number of shop rivets in BC is determined by bearing, and is $\frac{297000.}{.875 \times .28 \times 24000} = 51$ or 26 on each side.

The number of field rivets required for Bc is determined by shear, and is $\frac{2 \times 45900}{4420} = 21$ or 11 on each side.

The number of field rivets required for Bb is determined by bearing, and is $\frac{2 \times 10400}{2810} = 8$ or 4 on each side.

Joint "g".

The $\frac{1}{2}$" gusset plates will be shop riveted to gh and field riveted to all other members.

The number of 7/8 in. shop rivets required by gh is determined by bearing, and is $\frac{22.2 \times 16000}{10500} = 34$ or 17 on a side.

The number of field rivets required by fg is determined by bearing, and is $294000/8750 = 34$ or 17 on each side.

The number of field rivets required by Fg is determined by shear, and is $\frac{16000 \times 2 \times 1.85}{4420} = 14$ or 7 on a side.

The number of field rivets required by gH is determined by shear, and is $\frac{2.48 \times 9220 \times 2}{4420} = 11$ or 6 on each side.

The number of field rivets required by Gg is determined by bearing, and is $\frac{.61 \times 2 \times 8370}{1875} = 6$ or 3 on each side.

The bottom lateral plate is field connected to fg and shop connected to gh. This bottom lateral plate is 3/8" thick. It assists in transmitting stress from fg to gh. The maximum stress which can be carried in this way is equal to the stress value of the legs riveted to the plate. This stress value will be determined for fg and gh and the lesser value used. It is evident that fg controls. The portion of the net area furnished by the attached legs is:-

$$4(3.5 \times 9/16 - .27) = 6.80 \text{ sq. in.}$$

which carries a stress of 6.80×16000 lbs.

The number of shop rivets required to develop this stress in single shear is $\frac{6.80 \times 16000}{5300} = 20$.

So the number of rivets between the lateral plate and each angle is 5. This enables the rivets in the gusset plate to be reduced by 5 on each side. This reduction can also be applied to the gusset plate on the other side of the joint. The difference between shop rivets and the field rivets is not considered in designing the bottom lateral plates and in reducing the number of rivets in the gusset plates.

JOINT "A".

Cast iron shoes will be used at the fixed end and cast iron rockers at the expansion end. The pin should be made as large as the channels of the end post will permit. A pin $3\frac{1}{2}$ in. in diameter will be used if the following investigation shows it to have sufficient strength. The forces acting on the pin are all vertical and are equal to one-half of the maximum pedestal reaction,

pedestal reaction 73500. lbs.

The thickness of the gusset plate is determined by the bearing area required, and is

$$t = \frac{73500.}{4 \times 24000.} = .385 \text{ in.}$$

The thickness of the channel web is .28 in., making the thickness of the gusset plate (.385 - .28) = .10 in. A $\frac{1}{2}$ in. plate will be used to insure uniformity in the truss.

The maximum bending moment on the pin is:

$73500/2 \times 1.97 = 72398$ in lbs.

The maximum shear on the pin is 36750. lbs.

The diameter required by the bending moment is

$$d = \left(\frac{32\ M}{\pi f}\right)^{\frac{1}{3}} = 3.25 \text{ in.}$$

The diameter required by the shear is

$$d = \sqrt{\frac{4\ V}{\pi f_s}} = 1.98 \text{ in.}$$

A $3\frac{1}{2}$ in. pin is therefore satisfactory and will be used.

The endpost will be shop riveted to the gusset plate and the lower chord and floorbeam will be field riveted.

The number of shop rivets in the endpost is determined by bearing, and is $\frac{297000.}{.875 \times .28 \times 20000.} = 61$ or 31 on each side.

The number of field rivets in the lower chord at a is determined by bearing, and is $\frac{88400}{8750} = 10$ or 5 on each side.

The number of field rivets required for the bottom lateral plate is 6 or 3 on each side.

JOINT "J".

The gusset plates will be shop riveted to JK and field riveted to all other members.

The number of field rivets required in IJ is determined by shear, and is $\frac{26.39 \times 14264}{6010} = 63$ or 32 on each side

The number of shop rivets required in JK is determined by bearing and is $\frac{26.39 \times 14264}{7220} = 52$ or 27 on each side.

The number of field rivets required in iJ is determined by bearing, and is $\frac{1.56 \times 9560 \times 2}{3750} = 8$ or 4 on each side.

The number of field rivets required in JK is determined by bearing, and is $\frac{17400 \times 2}{3750} = 9$ or 5 on a side

The number of field rivets required in J_j is determined by bearing and is $\frac{2 \times 10400}{2810} = 8$ or 4 on each side.

The cover plate on top chord will be spliced for its full stress value of (8.75×14264) lbs. The number of field rivets required at one side of the splice is

$$\frac{8.75 \times 14264}{6010} = 21 \text{ rivets.}$$

So the number of rivets required in the gusset plates can be reduced by 10 on each side.

The number of shop rivets required to develop the cover plate is $\frac{8.75 \times 14264}{7220} = 18$ rivets, so the number of rivets in gusset plates can be reduced by 9 on each side.

END BEARINGS.

Rocker or roller bearings are required on spans of 70 ft. or more, so must be used for this bridge.

DESIGN OF CAST IRON ROCKERS.

The maximum pedestal reaction will occur with the bridge fully loaded and will equal one-half of the sum of the dead and live load joint loads multiplied by the number of panels, and is, $R = \frac{1}{2}(2450 + 4900)20 = 73500.$ lbs.

The area of each steel plate must be:-

73500./20000. = 3.68 sq. in.. where 20000. is the allowable bearing stress on steel.

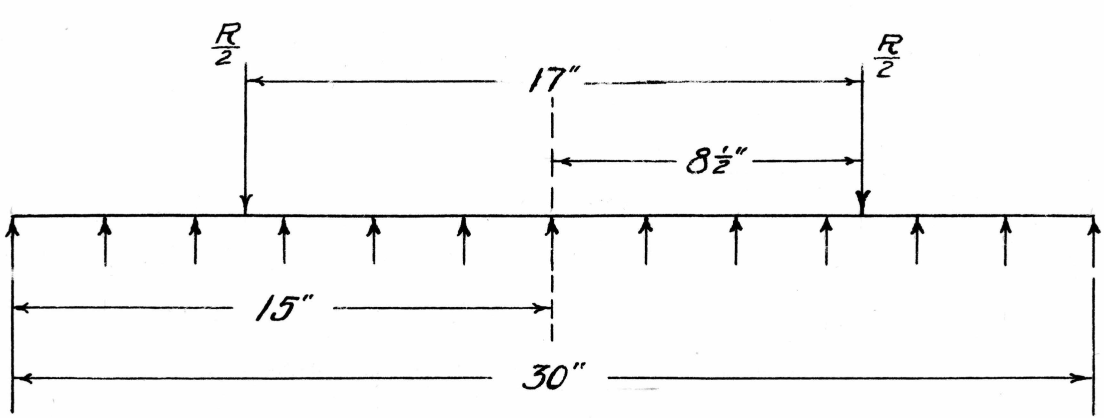
The length of the rocker will be taken as 30. in. so the bearing stress between the rocker and the plate is:-

p = 73500./30. = 2450. lb. per lineal inch.

The allowable bearing stress is 300.x d pounds per lineal inch where d is diameter of rocker, and is :-

300. x 18. = 5400. lb. per lineal inch.

FORCES ON ROCKER AND PEDESTAL.



The forces acting on the rocker are shown above. The section will be investigated as a cantilever beam with an effective length of $15-(7\frac{1}{2}+2) = 5\frac{1}{2}$ inches.

$$M_2 = 2450. \times (5.5)^2/2 = -37056. \text{ in. lb.}$$

$$M_3 = 2450. \times (15)^2/2 + 36750 \times 8.5 = 36750. \text{ in. lb.}$$

The moment of inertia of this section is 28.88 in.^{4}, c = 2 in., so the largest bending stress is:-

$$S = \frac{Mc}{I} = \frac{37056 \times 2}{28.88} = 2567. \text{ lbs. per sq. in.}$$

which is safe, so a depth of 4 in. is sufficient for bending.

The shear to the left of the upright is:-

$$5.5 \times 2450 = 13475. \text{ lb.}$$

and the shear to the right is:-

$$(15-7.5)2450.-36750 = 18375. \text{ lb.}$$

The section area is 31.04 sq. in. so the largest average unit shear is:- $18375/31.04 = 592$ lbs per sq. in.

The depth of 4 in. is sufficient for bending and shear so will be used.

The thickness of the upright will be determined by the bearing area on the pin. Using an allowable bearing stress of 9000. lb. per sq. in. for cast iron, for a $3\frac{1}{2}$ in. pin, we have, $2 \times 3\frac{1}{2} \times t \times 9000. = 73500.$

or $t = 1.17$ in. thickness of 2 in. will be used.

The unsupported length of the upright is 5 in. and with a thickness of 2 in. there will be no column action.

The type of pedestal shown on the general drawing will be used at the fixed end. The same bearing stress on the steel exists here as at the expansion end. The forces acting will be the same as for the rocker and are shown above. The maximum bending moment is -37056. in. lbs. The moment of inertia of the section is $1/12 \times 12 \times (2.5)^3 = 15.6$ in.^{4} and $c = 1.25$ in.

$$S = \frac{-37056 \times 1.25}{15.6} = 2969. \text{ lb. per sq. in.}$$

The maximum shear is 18375. lb., and the maximum average unit shear is:-

$$\frac{18375}{2\frac{1}{2} \times 12} = 612.5 \text{ lb. per sq. in.}$$

The uprights will be the same as at the expansion end. The unsupported length is $7\frac{1}{4}$ in. which is not sufficient to require an investigation as a short column.

DESIGN OF 50 FOOT PLATE GIRDER SPAN.

GENERAL DESCRIPTION.

This is to be a 50 ft. through plate girder bridge with a 10 ft. walkway. The floor is to consist of 3 in. planks laid upon stringers, the stringers in turn resting upon floorbeams.

LOADS.

Dead Load.

The dead load consists of the weight of the girders, floorbeams, stringers, and planking.

Live Load.

The live load is assumed to be 140. lbs. per sq. ft. of walkway. The live load is taken large enough to include impact.

Wind Load.

The lateral bracing is designed for a moving wind load of 300. lbs. per lineal foot of bridge.

GENERAL DIMENSIONS.

Span 50 ft. out to out or about 49 ft. center to center of bearings. Width of walkway 10 ft. Spacing of girders 10' 0 5/16" about, center to center. Depth of girder 42½ in. back to back of flange angles. Rivets ¾ in. in diameter will be used throughout.

Gusset plates for lateral bracing will come at the floorbeams and will be connected to the stiffeners. The minimum web thickness is 5/16 in. and will be used, so stiffeners will be required. The spacing of the stiffeners will be 40 in. Floorbeams are spaced 10 ft. apart and a gusset plate is used at every floorbeam.

DESIGN OF FLOOR SYSTEM.

The surface of the walkway will be 3 in. planks.

The outside stringers will be spaced 6 in. from the web of the girder and the inside stringers 3 ft. from them and from each other, thus giving a total of four stringers. These stringers have a span of 10 ft. and the outside stringers are designed to carry one-half as much as the inside stringers. The total uniform load per stringer per ft. is found to be 507. lbs. Referring to tables, it is seen that a 7 in. channel at 9.8 lbs. per ft. is satisfactory. Both inside and outside stringers will be 7 in. channels @ 9.8 lbs.

The floorbeams will be spaced 10'-0" center to center, the end floorbeams being of the same section as the intermediate ones. The stringer reactions of 5170. lbs. each act as concentrated loads on the floorbeams. From the symmetry of these loads, it is seen that the dangerous section is at the middle and that the maximum bending moment occurs there and is 20680. ft. lbs.

Assuming the weight of the floorbeam to be 22. lbs. per foot, the bending moment due to this load is 275. ft. lbs.

The total bending moment in the floorbeam is then the sum of these two, and is 20955. ft. lbs.

Referring to tables, it is seen that a 9 in. I-beam @ 21.8 lbs. is satisfactory and will be used.

Standard connections of two angles will be used to connect the floorbeams with the girders.

Nailing strips 3" x 4" will be placed on top of each stringer and bolted to them.

STRESSES IN GIRDERS.

The girder section will be made uniform throughout the entire length for there is no economy in varying the light section required for this length and span. A cover plate will be used on the top flange for the full length of span to keep out water and to improve the appearance of the girder. No cover plate will be used on the bottom flange. The floor loads will be considered as concentrated at the floorbeam points.

Each floorbeam reaction 10340. lb.

Maximum vertical shear due to floor loads 20680. lb.

Weight of girders assumed 200. lb. per sq. ft. per girder. Then the maximum vertical shear due to the weight of the girder is 5000. lb. Then the total end shear for each girder is 25680. lb.

Due to the symmetry of the concentrated loading on the girder the dangerous section is at the middle and the maximum bending moment due to the floor loads is 310200. ft. lb. The maximum bending moment due to weight of girder is :-

$1/8 \times 200 \times (50)^2 = 62500.$ ft. lb. The total maximum bending moment then is 372700. ft. lb.

Total bending moment for one girder is 372700. ft. lb.

Total end shear for one girder is 25680. lb.

DESIGN OF GIRDERS.

The depth of the web plate will be taken as 42 in. The thickness of the web plate must be large enough to take care of the shearing stresses and to insure a practicable rivet spacing in the flanges, at the end of the girder. Neither of these factors are likely to exert much influence on a girder as small as the one under consideration, but must nevertheless be considered.

Using a web plate 42 in. $\times$ 5/16 in. the actual unit shear on the gross section of the web is:-

$25680/(42 \times 5/16) = 1960.$ lb. per sq. in.

The allowable unit stress is 10000 lb. per sq. in.

It is assumed for the present that the distance between gage lines of top and bottom flanges is 3 in. less than the distance back to back of flange angles and that there will be but one row of rivets connecting the flange to the web plate, the rivet spacing in the flanges at the end of the girder will be:-

$$p = \frac{rh}{V} = \frac{5630 \times 39}{25680} = 8.2 \text{ in.}$$

where p = pitch of rivet in inches, r = allowable stress on rivet, and V = shear at section. From this calculation it is seen that a 5/16 in. web plate is satisfactory as far as the flange rivet spacing is concerned. A web plate 42 in. x 5/16 in. will be adopted.

The gross area of the compression flange should not be less than the gross area of the tension flange. The allowable fiber stress in the bottom flange is 16000.-150 l/b, where l = distance between lateral supports of the top flange *and* = 10 x 12 = 120 in., and b = the width of the cover plate. Assuming the effective depth to be $1\frac{1}{2}$ in. less than the distance back to back of flange angles the net area required for the tension flange is:-

$$A = \frac{372700. \times 12.}{16000 \times 41} = 6.84 \text{ sq. in.}$$

Allowing one-eighth of the area of the web as flange area, the required net area of the tension flange is:-

$$6.84 - 1/8 \times 13.13 = 5.20$$

Two angles $4\frac{1}{2}$ in. x 3 in. x $\frac{1}{2}$ in. provide a net area of 5.24 sq. in., deducting one 7/8 in. hole from each angle. This section will be used with the $4\frac{1}{2}$ in. legs outstanding.

The width of the cover plate must be 1/12 x 120 = 10 in. for the compression flange. The width of the tension flange is about 9 5/16 in., so the cover plate will be made 10 in. wide.

The allowable unit stress in the compression flange is:-

$$16000.-150\ l/b = 16000.-150 \times \frac{120}{10} = 14200. \text{ lb. per sq. in.}$$

The gross area required for the compression flange is:-

$$A = \frac{12 \times 372700}{14200. \times 41.} = 7.78 \text{ sq. in.}$$

Allowing one-eighth of the flange area as web area, the required gross area for the compression flange is:-

$$7.78 - 1/8 \times 13.13 = 6.14 \text{ sq. in.}$$

The area provided by a 10 in. $\times$ 5/16 in. cover plate and two angles 4" $\times$ 3" $\times$ 5/16" is 7.31 sq. in., and this section will be used. The edge distance on the cover plate is about 2 $\frac{11}{12}$ in.

The distance back to back of flange angles will be 42 in. + $\frac{1}{2}$ in. = $42\frac{1}{2}$ in. The distance between gage lines of the flange angles will be $42\frac{1}{2} - (2 \times 1\frac{3}{4}) = 39$ in.

The portion of the moment carried by the top flanges is:-

$$6.51/8.15 = 0.80$$

The required spacing of the rivets between the web and flanges is:

$$p = \frac{rh}{0.80\ V} = \frac{5630 \times 39}{0.80 \times 25680} = 10.70 \text{ in.}$$

The maximum spacing allowed is 6 in. but this is longer than should be used. A spacing of about 5 in. will be used throughout the entire length of the top and bottom flanges. This spacing will also be used for the rivets between the cover plate and flange angles.

The end stiffeners should have an area sufficient to carry the total end shear by column action. There will be two pairs of end stiffeners at each end of each girder. Sliding bearings are to be used so the pair of stiffeners towards the center of the girder should be designed for $\frac{3}{4}$ of the shear or $\frac{3}{4} \times 25680. = 19260.$ lb. Only the area of the outstanding legs should be considered as effective at the ends of the stiffeners.

because of the poor bearing of the other leg on the fillet of the flange angle. The area required is:-

$$19260./16000. = 1.20 \text{ sq. in.}$$

The value 16000. is used without reduction because there is no column action at the ends of stiffeners. At other points along the stiffeners the full section can be used, but the allowable stress must be reduced by the column formula. The case just figured will evidently control, however.

Two angles $3\frac{1}{2}" \times 2\frac{1}{2}" \times 5/16"$ provide an area of $7 \times 5/16 = 2.18$ sq. in. so will be used for the end stiffeners nearer the center of the girder. The other pair of end stiffeners will be made the same. A $10" \times \frac{1}{4}"$ plate will be riveted to these to improve the appearance of the end of the girder.

The outstanding leg of the intermediate stiffeners must not be less than $1/30 \times 42 \times 2 = 3.4$ in. All intermediate stiffeners will be made of angles $3\frac{1}{2}" \times 2\frac{1}{2}" \times 5/16"$ placed in pairs, with the $3\frac{1}{2}"$ leg outstanding. Intermediate stiffeners will be spaced 40 in. apart.

The stress carried by the end stiffeners nearest the center of the girder is 19260. lb. as determined above. Enough rivets must be used between the angles and the web to transmit this stress to the web in double shear or bearing. The number required is $\frac{19260.}{5630.} = 4$

The rivets will be spaced about 5 in. This will provide about 8 rivets for this case. The rivets in all stiffeners will have the same spacing.

The web plate, flange angles, and cover plate can be obtained long enough for the entire span, so they will not have to be spliced.

DESIGN OF END BEARINGS.

Roller bearings are not required for spans less than 70 ft., so sliding bearings will be used for this girder. Slotted holes will be used in the sole plates to allow for a movement of at least 5/8 in. at one end. The area of the wall plate must be at least,

$$A = \frac{v}{f} = \frac{25680}{600} = 42.5 \text{ sq. in. for concrete}$$

and,

$$A = \frac{v}{f} = \frac{25680}{9000} = 2.85 \text{ sq in. for cast iron}$$

where v = maximum end reaction and f = allowable bearing. The sizes of the sole plates and masonry plates will probably be determined by the detail adopted. The thickness of the sole plate will be taken as 5/8 in. and the masonry plate as $\frac{3}{4}$ in.

The anchor bolts will be hacked bolts, $1\frac{1}{4}$ in. in diameter and 1 ft. 3 in. long.

DESIGN OF 20 FOOT PLATE GIRDER SPAN.

GENERAL DESCRIPTION.

This is to be a 20 ft. plate girder bridge with a 10 ft. walkway. The floor is to consist of 3 in. planks laid upon stringers, the stringers in turn resting upon floorbeams. This girder bridge is to span the distance between the bents of each tower.

LOADS.

The loads are taken the same as for the 50 ft. girder.

GENERAL DIMENSIONS.

Span 20 ft. out to out or about 19 ft. center to center of bearings. Width of walkway 10 ft. Spacing of girders 10'-0 5/16" about, center to center. Depth of girder $42\frac{1}{2}$" back to back of flange angles. Rivets $\frac{3}{4}$" in diameter will be used throughout.

Gusset plates for lateral bracing will come at the floorbeams and will be connected to the stiffeners. The minimum web thickness is 5/16" and will be used, so stiffeners will be required. The spacing of the stiffeners will be 40 in. Floorbeams are spaced 10 ft. apart and a gusset plate is used at every floorbeam.

DESIGN OF FLOOR SYSTEM.

The loading of the floor system being the same as for the 50 ft. girder, the design of the floor system is the same and the floor system will be the same.

Surface of walkway	3" plank
Stringers	7" channels @ 9.8 lbs.
Floorbeams	9" I-beams @ 21.8 lbs.
Nailing strips on stringers	3" x 4" timbers.

STRESSES IN GIRDERS.

For the sake of uniformity of appearance, the girders will be made the same depth as the 50 ft. girders. The top chord will be covered with a plate to keep out water and to improve the appearance of the girder.

DESIGN OF GIRDERS.

As the depth of the 20 ft. girder is made the same as that of the 50 ft. girder for the sake of uniformity of appearance, the web thickness must be the same, as 5/16" is the minimum thickness for this depth. Also, good engineering practice requires that the rest of the members of the 20 ft. girder section be made the same as the depth and thickness of web are the same, so the same section will be used.

Compression flange (2 angles $4'' \times 3'' \times 5/16''$
(cover plate $10'' \times 5/16''$

Tension flange 2 angles $4\frac{1}{2}'' \times 3'' \times \frac{1}{2}''$

All stiffeners in pairs of angles $3\frac{1}{2}'' \times 2\frac{1}{2}'' \times 5/16''$.

DESIGN OF END BEARINGS.

Sliding bearing will be used for this girder. Slotted holes will be used in the sole plates to allow for a movement of at least 5/8" at one end. The area of the wall plate must be at least

$$A = \frac{v}{f} = \frac{12340}{9000} = 1.37 \text{ sq. in.}$$

where v = maximum end reaction, and f = allowable bearing. The sizes of the sole plates will probably be determined by the detail adopted. The thickness of the sole plate will be taken as 5/8 inch.

THE DESIGN OF BENT NO. 3.

General Description.

This is a simple bent placed in a vertical plane at right angles to the line of the bridge and braced as shown on stress sheet. This bent supports one end of the Warren truss and one end of a 20 ft. girder.

LOADS.

Dead Load.

The dead load consists of one-half of the weight of the entire truss span and one-half of the weight of the 20 ft. girder. Also, the dead load includes the weight due to the entire bridge being fully loaded. The vertical dead load per joint at the top of the bent is found to be 83000. lbs.

Wind Load.

The wind load is taken as 300 lbs. per lineal foot of bridge, treated as a moving load. It is taken as applied $4\frac{1}{2}$ ft. above the top of the bent. In addition, a lateral pressure of 100 lbs. for each vertical lineal foot of trestle bent is considered. The wind load acting $4\frac{1}{2}$ ft. above the top of this bent is found to be 22750. lbs.

DIMENSIONS.

Height, 5 stories @ 15'-0" = 75'-0"; width of top 11'-0"; width of base, 48'-6".

STRESSES IN BENT.

Dead Load Stresses.

The diagonals cannot act under a symmetrical vertical load, therefore the weights at the apexes are decomposed directly into the direction of the columns and the horizontal braces. These stresses are calculated by use of the formulae:-

stress in column = $P \sec\theta$

stress in brace = $P \tan\theta$

where θ is angle between column and vertical, and P is total load coming on apex immediately above member considered.

Wind Load Stresses.

To find the wind stress in the columns, the section was cut by a horizontal plane and the center of moments taken at the opposite vertex. For the horizontal struts, the principle was used that the sum of the horizontal components of the stresses in a section and of the forces above it must vanish. These were checked by moments. For the diagonals, the method of resolution of forces was used.

Stress Sheet.

The stresses due to the various loadings were collected on the stress sheet, and the sizes of members were also recorded there.

DESIGN OF MEMBERS.

The columns will be made of two channels, flanges turned out, with a cover plate on top and lacing on the under side. Ease in fabrication and simplicity in connections are of greater weight than to have as large a radius of gyration as possible, although this is very important. The horizontal struts and diagonals will be composed of two angle sections, legs turned in. All horizontal struts will be laced to provide rigidity. The diagonals will be battened. Vertical ties will be provided at the intersections of each pair of diagonals to support the horizontal strut beneath and thus reduce the ratio of l/r in it, so a smaller member may be used.

DESIGN OF COLUMN.

There is no economy in varying the section because of the slight stress variation, so the same section will be used for the entire length of column.

Dead load stress due to truss and girder	86000. lbs.
Wind load stress (total)	54200. "
Dead load stress due to assumed tower weight	3800. "
Total compression stress	144000. lbs.

The maximum value of l/r is 125, so the minimum value of the radius of gyration is $r = \frac{1.03 \times 15 \times 12}{125} = 1.48$

Using this radius of gyration and the corresponding length, the approximate allowable unit stress is

$S = 16000 - 70\ l/r = 16000 - (70 \times 125) = 7250$ lbs./sq. in.

The approximate area required is $\frac{144000}{7250} = 19.8$ sq. in.

This area is much too large, so a section with a larger radius of gyration will be used, thus reducing value of l/r (which is desirable) and at the same time reducing required area. A section composed of 2 channels 9" @ 13.25 lbs. and a cover plate 14" x 5/16" will be investigated. This section gives an area of 12.16 sq. in. and a radius of gyration of 3.57. Using this radius of gyration and the corresponding length, the allowable unit stress is

$$S = 16000-70\ l/r = 16000-70\frac{(1.03 \times 15 \times 12)}{3.57} = 12400.\ \text{lb. sq. in.}$$

The required area then is $\frac{144000.}{12400.} = 11.6$ sq. in.

This section therefore proves satisfactory and will be used.

The horizontal struts were designed in the same manner as the column, vertical ties being inserted at the center of each to reduce the effective bending length.

The diagonals were designed in the same manner as the lower chord of the truss, and the design will not be repeated here.

DESIGN OF JOINTS.

All joints will be designed to develop the full strength of the members and not simply the calculated stress.

The gusset plates will be made at least thick enough to develop in bearing the strength of the rivets in single shear. This thickness is 5/16 in.

All rivets will be made ¾ inch.

The arrangement of the members at the joints is shown on the general drawings. The gage lines of angles are placed on the center line of the bent. Where an angle has two gage lines, the center of gravity is used. The centers of gravity of the columns are placed on center line of bent. The size of gusset plates are determined by space required for the rivets necessary to connect the members to the plate. To secure uniformity of stress, the rivets are spaced symmetrically whenever possible.

The design of joints was done exactly in same manner as for the truss and the design will not be repeated here.

BEARING PLATES.

The bearing plates at the bottom of the tower are large enough to transmit the weight coming upon them to the footings without exceeding the allowable bearing unit-stress.

The bearing plates at the top of the tower are large enough to provide sufficient area for the end bearing plates of the girder and truss to rest upon.

GENERAL DRAWINGS.

All details are given upon the general drawing.

OTHER BENTS.

The design of the other bents will not be given here, as the same general principles were used in their design. The stresses and size of members are shown on the stress sheet.

MATERIAL.

Weight of steel in truss	76800.	lb.
Weight of steel in each 50' girder	15750.	lb.
Weight of steel in each 20' girder	6300.	lb.
Weight of steel in all towers	112950.	lb.
Oak lumber (board measure)	22000.	ft.
Weight of rivets	3600.	lb.
Weight of nails for floor	500.	lb.
Weight of bolts for nailing strips	300.	lb.
Weight of anchor bolts	2200.	lb.
Volume of concrete for footings, steps and abutment	84.	cu. yd.

ESTIMATE OF COST.

These estimates are based on current prices of steel as given in the Engineering News Record. The shop cost of steel is based on labor at 40 cents per hour and includes detailing, shop labor, and one coat of shop paint. The estimated cost of erection of steel per ton is taken from actual costs of similar structures and includes all falsework, rivet driving, labor, etc. The cost of laying the floor is determined in the same manner and includes nailing and all other incidentals to a finished floor. The estimated cost of painting includes the cost of the paint and the labor of painting. The cost of concrete in place is taken from actual total cost of laying similar amounts of concrete.

Shop Cost of Steel in Truss.

Average cost of steel at mill	1.92	cts.	per	lb.
Average shop cost	.70	"	"	"
Freight- Pittsburg to Lexington	.405	"	"	"
Total cost f. o. b. Lexington	3.025	"	"	"

Shop Cost of Steel in 50 Ft. Girders.

Average cost of steel at mill	1.92	cts.	per	lb.
Average shop cost	.85	"	"	"
Freight- Pittsburg to Lexington	.405	"	"	"
Total cost f. o. b. Lexington	3.175	"	"	"

Shop Cost of Steel in 20 Ft. Girders.

Average cost of steel at mill	1.92	cts. per lb.
Average shop cost	.90	" " "
Freight- Pittsburg to Lexington	.405	" " "
Total cost f. o. b. Lexington	3.225	" " "

Shop Cost of Towers.

Average cost of steel at mill	1.92	cts. per lb.
Average shop cost	1.15	" " "
Freight- Pittsburg to Lexington	.405	" " "
Total cost f. o. b. Lexington	3.475	" " "

Total Shop Cost of Steel.

Truss	(76800 x .03025)	$2323.20
50' Girders	(7 x 15750 x .03175)	3500.44
20' Girders	(8 x 6300 x .03225)	1625.40
Towers	(112950 x .03475)	3925.01
Total cost of all steel f. o. b. Lexington		$11374.05

Erection.

Hauling steel from station to site at 50 cts. per ton and 50 cts per ton for loading and unloading	$ 175.00
Hauling and handling lumber at $1.50 per M ft.	33.00
Cost of erecting steel at $20.00 per ton	3500.00
Cost of laying floor at $10.00 per M ft. B. M.	220.00
Cost of painting at $3.50 per ton	612.50
Total cost of erection	$4540.50

Summary of Cost of Entire Structure.

Steel f. o. b. Lexington	$11374.00
Oak lumber- 22000 ft. at $50.00 per M B. M.	1100.00
Rivets - 3600 lbs. at 2.25 cts.	97.80
Nails- 500 lbs. at 2.65 cts.	13.25
Bolts- 300 lbs. at 4.00 cts.	12.00
Anchor bolts- 2200 lbs. at 4.00 cts.	88.00
Cost of concrete in place- 84 cu. yd. at $10.00	840.00
Erection	4540.50
Total cost of structure	$18065.60