Design of Bridge
over
North River
as an
Entrance to Lexington, Virginia

Thesis presented for the Degree of

Bachelor of Science
in
Civil Engineering

by
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and
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Drawings:

Number	Drawing
1.	Map of site.
2.	Profile of center line.
3.	Stress Sheet for Pratt truss.
4.	General Detail of Bridge.
5.	Pier, Abutment, and Floor Slab Loading Diagram

Contents of Text.

Bibliography

"Design of Highway Bridges" - Ketchum

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" " " Part III - " " "

"Pocket Companion" - Carnegie Steel Company

"Virginia Highway Bridge Specifications" - Virginia Highway Commission

"Ordinary Iron Highway Bridges" - Waddell

"American Highway Engineers Handbook" - Blanchard

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"Design of Masonry Structures and Foundation" - Williams

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Introduction

Location of Property:

The property upon which the bridge herein contemplated, is on the west end of the island owned by Moses, and on the north side of the North River is owned by Giter and Humphres. Other property over which the proposed road will run is owned by Moses, Bruce, Lindsay, Tankersley, and Beachbrook Chapel. The position of proposed road and bridge are indicated upon the accompanying map.

Bench Mark:

All elevations given on the drawings are based upon an assumed elevation of a bench mark (100 ft.) one hundred feet above sea level. This bench mark is on the top of the north west corner of the retaining wall, north of Moses Mill and on the east side of the present road. This retaining wall starts at Moses Mill and runs north, paralleling the present road.

Definition of Problem:

The problem of this thesis is to provide some kind of a suitable approach across North River from the Lee Highway, as an entrance to Lexington, Virginia.

Solution of Problem:

The solution of this problem as presented in this thesis, is to construct a steel Pratt truss with an eighteen foot (18 ft) clear roadway across North River, and to change the site of the present roadway entrance into Lexington, details and design of which are given in the accompanying thesis.

Necessity of Bridge and Change of Location:

The necessity of the building of a new bridge is based upon the following reasons. The present bridge is a wooden bridge built in 1871, after the bridge then in existence was washed away by the flood of 1870. It is therefore seen that the present bridge has outlived its life. Marked changes in transportation have taken place since the building of this bridge, such as the improvement of roads, increased travel upon roads and the coming of the automobile. Not only is the present bridge unable to care for the present traffic, but it is also unable to care for increased loadings that present day structures are required to support. At present the bridge is limited to a four ton capacity, nevertheless the large busses running up the valley hourly pass over this bridge.

The Lee Highway is a federal aid transcontinental highway. In 1924 the paving of this highway from Staunton to Lexington was completed, and in 1925 the paving was extended to Roanoke. This caused an immense increase in the traffic upon this highway. Some idea of this increase can be obtained from the number of people registered in the Lee Chapel, at Washington and

Lee University, given in the following table. Most of these people travel by highway, as Lexington is at the end of the railroads entering it. The busses carry most of the tourists, excepting tourists of which there are a large number, especially during the summer months.

Table: Registration at Lee Chapel, W. L. U.

Year	No. People Registered	Yearly Difference	Yearly Percentage Increase
1923	10,127		
1924	12,505	2,378	23%
1925	24,812	12,307	98%
1926	33,178	8,366	37%
1927	48,985	15,807	44%
Average yearly percentage increase - - - -			50%

A glance at the accompanying map will shop that upon entering or leaving Lexington from the North, it is necessary at present to make three right angle turns, none of which can be seen around. These turns occur at Humphres Store, at the north end of the present covered bridge, and at Moses Mill. All of these turns are extremely dangerous, besides congesting and slowing up traffic. I myself have seen three accidents recently at Humphres Store. In order for the bus to make the turn upon entering the present bridge, it has to maneuver, backing up. The possible consequences of such action is unnecessary to explain.

The present pony truss, carrying the present road over Wood's Creek is limited now to a very small cross capacity, and is much too narrow for the two lanes required by present vehicles. For these two reasons, and because it is in bad repair it is inadequate.

Curve at Humphres Store

Curve at Covered Bridge

Curve at Moses' Mill
Showing present pony truss

Site of Proposed Bridge

Preliminary Investigation

Modifying Factors:

In the location of a new bridge and road several factors enter the problem. First, it will be necessary to eliminate as far as possible all curves. Second, it will be necessary to provide as little cut and fill as possible. Third, it will be necessary to carry the road under the present Baltimore and Ohio Railroad trestle with a clearance of 14 feet. Fourth, it will be necessary to provide sufficient span, and clearance above high water of the North River. In considering these requirements, minimum grades must also not be neglected.

A 160 foot span with floor elevation of 116 feet, placed as in the accompanying map fulfills the requirements and provides more direct entrance to town. By so doing, there are minimum grades and minimum cuts and fills. At one end of the proposed road there is a curve of about 12° and at the other a curve of about 15°. However, these are much gentler than the present curves. In order to provide a 14 foot clearance under the railroad trestle it is necessary for part of the road to be below the high water mark. As it is not our thesis to design the roadway and approaches to the bridge, this will not be gone into further except to state that the portion of the roadway below high water will have to be designed to withstand being flooded. This will not greatly interfere with traffic as flooded stage only lasts at a maximum about twelve hours and there is very little traffic during such weather.

Economy:

Leaving out of consideration the 160 foot span, which is necessary, it is found that a Pratt truss with 20 foot panels and an economical depth of 24 feet is as economical as any structure, as then it fulfills the Virginia State Highway Bridge Specifications and the super-structure more nearly equals that of the sub-structure.

Beauty:

The finished bridge will present a very pleasing appearance, the entire bridge being made of steel with the exception of the concrete floor slab. As the bridge is open, it affords a beautiful view up and down North River as an entrance into Lexington, Virginia.

General Specifications for Concrete Abutments and Footings

Inspection and Approval:

All materials shall be subject to the inspection and approval of the Engineers. Defective material must be removed immediately from the property at the expense of the contractor.

Cement:

All cements shall conform to the requirements of the "Specifications and Tests for Portland Cement" as provided in the Tentative Standard Methods of Sampling and Testing of the American Association of the State Highway officials and subsequent revision thereof. The Engineer reserves the right to take check samples for the purpose of making tests to determine the stability of the product.

All cement for any given structure shall be of the same brand and produced by a single mill unless otherwise permitted by the Engineer.

Fine Aggregate:

Sand: Sand for the fine aggregate shall be made up of clean, hard, durable, uncoated particles, free from lumps of clay, soft or flaky material, loam or organic matter. In no case must frozen material be used.

Stone grit: The stone grit shall consist of particles resulting from the crushing of clean, tough, durable rock, and it must have a percentage of wear of not more than six percent. In other respects, it must conform to the requirements of Grade A or B aggregate. No more than fifty percent of this can be used in the fine aggregate. Grade A aggregate must be used in all structures. That is, 100% of the aggregate must pass a ¼ inch screen, 50-75% must pass a 20 mesh Standard sieve, 5-25% must pass a 50 mesh standard sieve, and 0-5% must pass a 100 mesh standard sieve.

Coarse Aggregate:

Coarse Aggregate shall consist of crushed rock or washed gravel, containing only clean uncoated pieces of strong and durable minerals, free from injurious amounts of soft, friable, thin, elongated, or laminated pieces, alkali, organic or other deleterious matters. In grading, the particles shall be so graded that no material will be retained on a screen with circular perforations of the diameter of the upper limit, and not less than 40% or more than 75% shall be retained on a screen with perforations one half the size of the upper limit and not more than 15% will pass a screen with perforations the size of the lower limit, and also not more than 5% of stone chips or coarser material will pass an eight mesh sieve. The quality of the stones shall be tested in accordance with the standard methods of the American Association of

State Highway Officials.

Water:

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

Mixing:

The concrete shall be mixed in a batch mixer of an approved size and type and so designed as to insure a uniform distribution of the materials. All concrete shall be mixed for a period of not less than 1½ minutes after all materials are in the mixer. The drum shall revolve at about 200. feet per minute. It shall be equipped with adequate water storage and a device for accurately measuring the amount of water used. Hand mixing shall not be permitted except in an emergency and under written permission of the Engineer.

Mixture:

Class A concrete shall be used, that is:

1 part Portland Cement.
2 " fine aggregate Grade A.
4 " coarse " " ".

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.

General Specifications for Steel Work

Design

Material:

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

Type of Truss:

Length of Span:

The length of span shall be taken as, the distance between centers of bearing in beams and girders, the distance between centers of end pins or of bearing in trusses, the distance between centers of trusses or girders in floorbeams, and the distance between centers of floorbeams in stringers.

Lateral Bracing:

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

Floor System

Floorbeams:

All floorbeams shall be rolled steel I-bars, rigidly connected to the trusses at the panel points. The floor-beams are at right angle to the trusses. The floorbeam connections are made above the bottom chord.

Stringers:

The stringers shall be rolled steel.

Loads

Dead Load:

The dead load will consist of (1) the weight of the metal, (2) weight of concrete on the bridge.

Live Load:

The bridge shall be designed to carry in addition to the dead load the vehicular traffic in the roadway. This traffic shall be assumed to be a 15 ton truck preceded and followed by $11\frac{1}{4}$ ton trucks. Special live loads will be considered later.

Wind Load:

The force due to wind and lateral vibrations shall be assumed acting in either direction horizontally, normal to the planes of the trusses and girders, at 30 lbs. per sq. ft. on the exposed surface of all girders, trusses and the floor as seen in elevation. Portals shall be designed for the full reaction of the lateral forces along the top chord, and one half this load shall be considered transferred to the trusses through the sway bracing.

Unit Stresses and Proportion of Parts.

Unit Stresses:

All parts of the structure shall be designed so that the sum of the maximum stresses shall not exceed the following: (These stress are in pounds per square inch.)

Structural Grade and Rivet Steel.

Tension

Axial tension, structural member, net section	16,000.
Rivets in tension where permitted 50% of single shear values.	
Bolts, area at root of thread	10,000.

Axial Compression

Axial compression, gross section but not to exceed 13,500.	15,000 x $\frac{50L}{r}$

L -- length of member, in inches.
r -- least radius of gyration, in inches.

Bending on extreme fiber

Rolled shapes, built sections and girders, net section.	16,000.
Pins	24,000.

Shear

Girder webs, gross section	10,000.
Pins and shop driven rivets	12,000.
Power driven field rivets and turned bolts	10,000.
Hand driven rivets and unfinished bolts	7,500.

Bearing

Pins, steel parts in contact and shop driven rivets	24,000.
Power driven field rivets and turned bolts	20,000.
Hand driven field rivets and unfinished bolts	15,000.

Expansion rollers, pounds per linear inch
600 d where d -- diameter of roller in inches.

Diagonal Tension

In webs of girders and rolled beams, at sections where maximum shear and bending occur simultaneously 16,000.

Modulus of Elasticity 30000,000.

Bearing on Bridge Seats

Bearing on Concrete Masonry, limestone masonry and better 500.

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 stress shall be design for the stress giving the largest cross-section.

Angles in Tension

When angles are in tension only 50% of the flange area shall be taken as effective.

Net Section

In members subject to tension allowance shall be made for rivet holes. In the calculations the holes are taken as one eighth greater in diameter than the rivets.

Combined Stresses

Members subject to direct and bending stresses shall be designed so that the greatest fibre 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 their gross section may be used as flange sections. The thickness of web plates shall not be less than 5/16 inch, 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. The stress pin square inch in compression flange of any beam or girder shall not exceed $19000x250\frac{l}{b}$ when flange consists of angles with flat cover plate, where l is the unsupported distance and b is width of flange. (Max. value - 16,000.)

Flange Rivets

The flanges of plate girders shall be connected to the web with a sufficient no. 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.

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 open for inspection, cleaning, and painting.

Water Pockets:

Pockets or holes which would hold water shall be filled with waterproof material or have drains.

Symmetrical Section:

The neutral axis shall be placed as nearly as possible to the center in main members and the neutral axis of the intersecting main members shall meet at a common point.

Strength of Connections:

All connections shall be designed to develop the full strength of the member, even if the computed strength is less.

Pitch of Rivets:

The minimum pitch shall be three diameters, but the distance between centers of three quarter inch rivets shall preferably be two and one half inches. The maximum pitch in the line of stress for members composed of plates and shapes shall be 6 inches. When the rivets are in two lines and are staggered the pitch may be twice as much.

In tension members composed of two angles in contact, a pitch of 12 inches will be allowed for riveting the angles together.

Edge Distance:

The distance from the center of the rivet to the edge of material shall not be less than $1\frac{1}{2}$ inches.

Compression Members:

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

Batten Plates:

The open side of all compression members shall be stayed by batten plates at the ends. They shall be as close to the end as practical. Lattice work shall be put between these plates.

Lattice Bars: To have rounded ends.

Expansion:

An allowance of 1/8 of an inch for every 10 ft. shall be made for expansion.

Expansion Bearing:

A rocker shall be placed at one end of the span to help take care of this expansion.

Fixed Bearing:

Movable bearing shall be designed to permit motion in one disection only, while fixed bearings shall be firmly anchored.

Pedestals and Bed Plates:

Pedestals and shoes shall be designed to secure rigidity and stability and to distribute the reaction uniformly over the entire bearing area. They shall be made of cast steel or structural steel. Where built pedestals and shoes are used, the web plates and angles connecting them to the base plates shall be not less than one-half inch thick. If the size allows, the web shall be rigidly connected transversely.

Camber:

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

Material and Workmanship

Material

Process of Manufacture:

All structural, rivet and eyebar steel shall comply to the requirements of the Standard Specification for Structural Steel for Bridges. Steel forgings from which pins, rollers, or other forged parts are to be fabricated, shall conform to the requirements of the Standard Specifications for Carbon-Steel Forgings for Locomotives.

Workmanship

General:

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

Straightening Materials:

All materials must be straightened in the shop by methods that will not injure it.

Size of Rivets:

The size of the rivet in the plans shall be understood to mean the size of the rivet before heating.

Rivet Holes:

When general reaming is not required, holes in material $\frac{3}{4}$ inch or less in thickness may be punched full size. Holes in material more than $\frac{3}{4}$ inch shall be sub-punched and reamed, or drilled from the solid.

Full sized punched holes shall be one sixteenth larger than the diameter of the rivet. Drilled holes shall be one sixteenth inch larger than the nominal diameter of the rivet. Sub-punched or reamed holes for rivets having diameters greater than ¾ inch shall be punched 3/16 inch less than the nominal diameter of the rivet and for rivets having diameter ¾ inch or less they shall be punched 1/16 inch less than nominal diameter of the rivet. All reaming shall be done with twist drills.

Punching:

All punching must be accurately done. If holes must be enlarged to admit the rivet, they shall be reamed. The drifting done in assembling shall be only such as to bring the parts into position and not sufficient to enlarge the holes or distort the metal.

Reaming after Assembling:

When general reaming is required, a definite provision to this effect shall be included elsewhere in the contract. When this is required, all rivet holes in main members shall be sub-punched and reamed, or drilled from the solid. Reaming shall be done after the pieces forming a built member are assembled and firmly bolted together. No interchange of reamed parts will be permitted.

Edge Planing:

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

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. All contact surfaces to be painted.

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 inch shall be cause for rejection.

Rivets and Riveting:

Refer to Page 90 - Virginia State Highway Commission.

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. In compression joints the surfaces shall be truly faced.

Screw Threads:

Screw threads shall make tight fits in the nuts and shall be U. S. Standard.

Welds:

Welds in steel will not be allowed.

Annealing:

Refer Page 93 - Va. S.H.C.

Marking and Shipping:

Refer Page 95, Va. S.H.C.

Finished Weight:

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

Shop Painting

Shop Cleaning and Painting:

Refer to Page 100 - Va. S.H.C.

Inaccessible Surfaces:

Pieces and parts which are not accessible for painting after erection shall be given a good coat of paint in the shop.

Mill and Shop Inspection

Refer to Page 94 - Va. S.H.C.

Erection

Refer to Page 95 - Va. S.H.S.

Field Painting

Refer to Page 101 - Va. S.H.C.

The Design of 160 Foot Rived Pratt Truss Bridge

General Description:

This is a riveted Pratt Truss 160 ft. long. The span is made this length so that the abutments can be placed on the river banks to afford free waterway during flood seasons. The floor is composed of reinforced concrete resting on I beam stringers, which in turn are riveted to plate girder floorbeams. The floor beams are to be riveted to the gusset plates above the lower chords.

Loads:

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

Live Load: The live load consists of one fifteen ton truck followed and preceded by two eleven and one quarter ton trucks, spaced as in accompanying loading diagram, lanes being 9 ft. wide.

Wind Load: The lateral bracing is designed for a wind load of 30 lbs. per sq. ft. or 150 lbs. per linear foot of bridge.

Impact: The impact for web and chord members is 20% of the live load.

Dimensions:

Span 160 ft. end to end of bridge; panel length 20 ft; width of roadway 18 ft; spacing of trusses 20 ft. 8 in. c. to c.; depth of truss 24 ft. from top of upper chord to bottom of lower chord.

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 concrete floor slab; (2) the live load.

Consider the outside stringers carry the same load as the inside stringer since it carries the curb.

In determining bending moments in stringers, no longitudinal distribution of the wheel loads is assumed. The lateral distribution is determined as follows.

$M = \frac{1}{2}$ bending moment produced by one truck.

$S =$ spacing of stringers in feet. - 4.5 ft.

$M_1 =$ bending moment in one interior beam, when floor system is designed for one truck.

$M_2 =$ bending moment in one interior beam when floor system is designed for two or more trucks.

$M_1 = \frac{MS}{6}$ for re-enforced concrete floors.

$M_2 = 1.2\ M_1$ for type of floor involved.

Wt. of floor slab per panel 150 x 19.5 x 20 = 58500

$$M = \frac{137920}{2} = 68960$$

$$M_1 = \frac{68960 \times 4.5}{6} = 51720$$

$$M_2 = 1.2 \times 51720 = 62064$$

In calculating bending stress due to wheel loads on concrete slabs, no distribution in the direction of the slab is assumed. In the direction perpendicular to the span of the slab, the wheel load is considered as distributed uniformly over a width of slab, known as the effective width.

Where S - span of slab in feet.

W - width of wheel in feet.

X - distance in feet from the center of the rear support to the center of wheel.

E - effective width in feet for one wheel.

$E = 2/3(2x + W)$

$8000 \times 14 + 4000X = 0$

$X = 2.8$ ft.

$-R, 20 + 8000 \times 22.6 + 32000 \times 86 = 0$

$R = 22800$

$40000 - 22800 = 17200$

$M = 17200 \times 8.6 = 137,920$

$E = 2/3(2 \times 2.8 + .5)$

$E = 4.57$ ft.

Using M_2 found above and referring to the Carnegie Pocket Companion, it is found that a 20" 65.4# I beam will be suitable.

The loads carried by the floorbeams consist of:

(1) The dead load which is the wt. of the floor system.

(2) A concentrated live load.

wt. of five stringers spaced 4 ft. 6 in:	= 6540 lbs.
" " floor beam	= 2188 "
" " floor slab	=58500 "
Dead Load	=67228 "
Live load	=77400 "
Maximum load	=144,628"

$$M = \frac{Wl}{8}$$

Where W = Maximum load.

M = " moment.

l = panel length.

Floor beam is made up of a plate girder 30 inches deep. Determination of shear is made every 30 inches.

1st position, Shear = 70.8 Kips

2nd " , " 64.9 "

3rd " , " 60.1 "

$$M = \frac{Wl}{8} = \frac{144.6 \times 20}{8} = 361.5 \text{ kip feet.}$$

From Ketchum's "Design of Highway Bridges", page 159, taking 1/8 the area of the web as available flange area.

$$M = (A'_F + \frac{A_W}{8})fh$$

Where M = resisting moment of section

A'_F = net area of tension flange

A_W = gross area of web

f = allowable unit fiber stress

h = distance between centers of gravity of flanges.

A'_F = 7.33 sq. inches.

Four angles 8" x $3\frac{1}{2}$" x $\frac{1}{2}$" are used

$$p = \frac{V}{\sqrt{w^2 + \left(\frac{A'_F + \frac{A_W}{8}}{A'_F} \cdot \frac{V}{h}\right)^2}}$$

Where p = pitch of rivets

w = concentrated load per unit length

v = allowable resistance of one rivet = 7500#

Where V = maximum shear at that joint

Pitch is changed every 2.5 ft.

p_1 = 2 9/16" for $\frac{3}{4}$" rivets in double shear.

p_2 = 2$\frac{3}{4}$" for $\frac{3}{4}$ rivets in double shear.

p_3 = 3" " " " " " "

The Plate girder the web of which is 30" x $\frac{1}{2}$" with end stiffness

Rivets required $= \frac{75000}{7500} =$ 10 rivets . $\frac{3}{4}$"

Use for end stiffness 2 angles 7" x 3$\frac{1}{2}$" x $\frac{1}{2}$" x 29" with 2 fillet plates 3$\frac{1}{2}$ x $\frac{1}{2}$ x 29"

For End Floor beams W = 105.6 kips.

Therefore $M = \frac{Wl}{8} =$ 271.9 kips ft.

By referring to the "Carnegie Pocket Companion", it is found that a I beam 24" x 105.9# is sufficient. This will have to be cut back so as to fit over the rocker.

Stresses in Trusses and Lateral System:

Dead load: The dead load is assumed to be 2699 lbs. per linear foot including trusses and all floor system, assuming 1/5 acting at top chord and 4/5 acting at bottom chord.

Thus 5.4 kips act at joint top chord.
21.6 " " " " bottom " .

The dead load web stresses are calculated by multiplying the vertical shear at the section by the secant of the angle between the vertical and the diagonal. The dead load chord stresses are calculated by the method of chord increments.

The live load web stresses are determined by placing the trucks to obtain maximum shear; the live load chord stresses are determined by placing the trucks to obtain maximum moments; the live load floor beam stresses are obtained by placing the trucks to obtain maximum reactions, all the while satisfying the criterions required of these methods.

The stresses due to impact in chord and web members is 20% of the live load stresses.

Dead load stress in floorbeams is due to weight of floorbeam, stringers and concrete floor.

Stresses in upper and lower lateral system are determined by finding wind pressure at panel points; then determining a set of cofficients for the lateral system; then multiplying W sec θ by the cofficients of the diagonals to find the stresses in the diagonals, and multiplying W tan θ, by the cofficients of the chord members. Wind load is taken as 150 lbs. per linear foot.

Overturning effect of wind.

150 x 20 = 3000# acting on each panel point.

Overturning moment about lower chord.

= 3000 x 7 x 24 = 504 kip ft.

Reaction = 504/2 x 18) = 14 kips.

End part stress is found by multiplying reaction by sec θ and the upper and lower chord stress is found by multiplying by the tan θ.

The stresses in the portal end and sway system are due to wind load and eccentric loading and are found by the usual methods.

After determining the stresses due to the various loads, the maximum stresses in the members are found and investigation made to determine if counters are necessary. It is found that counters are required at the two middle panels.

Design of Members:

All compression members are made of channels, cover plates being used upon the upper chord and end ports. Lacing and tie plates are used upon the open sides. The moment of inertia about the two axis is made as nearly equal as possible. The spacing and depth of the upper chord channels is to be constant so as to make the design of the joints easier and the appearance more uniform. Angles, plate and channel are so placed as to not collect water and to make riveting and painting easy. The lower chord is made of two to four angles; the portal, lateral and sway systems are all made of angles and the counters are to be made of round bars flattened on the ends. The area of the unriveted flange of the angles is taken as 50% of the area of that flange. Tie plates are placed as near as possible to the end and have at least three $\frac{3}{4}$" rivets in each side with a length of $1\frac{1}{2}$ times the distance c. to c. of opposite rows of rivets. In compression members single lacing, rounded ends, set at an angle of 60° is used. The gross area of the compression flanges is not to be less than the gross area of the tension flanges. The lateral unsupported length of compression flanges is not to exceed 40 times the flange width. When unsupported length exceeds 12 times the compressive stress in lbs. per square inch must not exceed

$$19000 - 250 \frac{L}{b} \text{ (maximum 16000 lbs.)}$$

Where L = Length in inches of unsupported flange.

b = flange width in inches.

The ratio of unsupported length to the least radius of gyration must not exceed 120 for main compression members and 140 for lateral and sway bracing. For main riveted tension members the ratio of length to least radius of gyration must not exceed 200. An example of the design of each type of member is given below.

Design of Top Chord B. C.:

3/8" is allowed on upper chords for camber

Maximum stress = 222.7 kips compression

Using 2,25# 10" channels with flanges turned out and $8\frac{1}{2}$" from outer face to outer face, and a cover plate 14" x 3/8"

The least moment of inertia = 230.68

" " radius of gyration = 3.4

Area of the section = 19.91 sq. in.

$$\text{Unit Stress} = 15000 - 50 \frac{L}{r}$$

Where L $=$ length in inches.

r $=$ radius of gyration.

Unit Stress $= 15000 - \frac{50 \times 20 \times 12}{3.4} = 11500$ lbs. per sq. in.

Required Area $= \frac{222700}{11500} = 19.3$ sq. in.

Since the area of the section is greater than the required area this number is satisfactory.

The cover plate is riveted with $\frac{3}{4}$" shop rivets spaced not to exceed four times diameter of rivet for $1\frac{1}{2}$ times maximum width of member gradually increasing from here to $1\frac{1}{2}$ times the width.

Testing the column for shear it is found that lacing $2\frac{1}{4}$" x $\frac{1}{4}$" is sufficient.

Where S $=$ 300 A where S $=$ transverse shear in lbs.,

A $=$ gross area.

Design of Vertical Cc:

Maximum stress $=$ 76.5 kips compression using 2, 12.25# 7" channels with flanges facing each other 7.45" inside face to inside face.

Least moment of inertia $=$ 48.2

" radius of gyration $=$ 2.59

Area of the section $=$ 7.16 sq. in.

Unit stress $= 15000 - \frac{50 L}{r} = 10800$ lbs. per sq. in.

Required area $= \frac{76500}{10800} = 7.0$ sq. in.

Therefore this member is satisfactory.

Tie plates 16" long by $\frac{1}{4}$" thick as this fills the requirement 1/50 distance between connecting lines of rivets with lacing on both sides starting from the tie plates is used. All lacing $2\frac{1}{4}$" x $\frac{1}{4}$" is to be used, as it is sufficiently strong for shear, and if the same lacing is used throughout it lessens the labor of ordering and presents a more pleasing appearance. These members are laced upon both sides.

Design of dE

Stress = 14.3 kips tension.

Unit stress = 16000 lbs. per sq. in.

Required area $= \frac{14300}{16000} = .89$ sq. in.

Since this is a counter, 2 round bases are used 1 1/8" in diameter with twinbuckles weighing 8#.

In design for tension members, allowance must be made for 50% of the unriveted flange of the angle, loss due to riveting and as in other members a even number so as not to cause an eccentricity of stress. Also it must be borne in mind that there must be enough area to place rivets about centroid so as to not cause an eccentricity of stress at the joint.

Design of Joints:

Unless otherwise provided, all connections are proportioned to develop not less than the full strength of the members connected. No connection, except for lacing bars and handrails contains less than three rivets. All joints use $\frac{3}{4}$" shop, field button head rivets with the exception of the floor beam connections where shop countersunk rivets are used. No rivets are allowed in direct tension.

Gusset plates will be used for connecting all main members. 5/6 inch is the minimum thickness of gusset plates.

Joints will be designed to withstand shear, compression and tension.

Spacing of the rivets is 3 times the diameter as a minimum, with shear edge $1\frac{1}{4}$" and rolled edge 1 1/8" to center of rivet.

Computations for all joints are not to be shown but several examples are below.

Joint C

Member BC

Area = 19.91 sq. inches.

A minimum size plate is used = 5/16". From tables in Ketchum's book, Table 33. The bearing value for 5/16" plate at 20000 lbs. per sq. in. for $\frac{3}{4}$ inch plate is 4690; shear being 4420. Therefore the shear is the controling factor in the design.

Using field rivets with unit stress of BC = 11500 lbs. per sq. in.

We find $\frac{19.91 \times 11500}{4220} = 51 +$ rivets.

Therefore 52 rivets are used, or 26 rivets on each side.

Other joints are similarly designed.

Cast Iron Rocker and Pedestal:

Cast Iron Rocker is used at the expansion end and a cast iron pedestal at the fixed end.

Vertical Reaction at end supports

$= 81728 \times \cos\theta = 62{,}745$ lbs. $= 63$ kips

diameter of pin required for shear

$$d = 1.13 \left(\frac{63000}{12000}\right)^{\frac{1}{2}} = 2.58 \text{ inches.}$$

where $d = \left(\frac{4V}{fs}\right)^{\frac{1}{2}} = 1.13 \left(\frac{V}{fs}\right)^{\frac{1}{2}}$

diameter required by bending moment.

$$d = 2.17 \left(\frac{63000 \times 1.87}{24000}\right)^{\frac{1}{3}} = 4.92$$

A clearance of $\frac{1}{2}$" is used.

A 5" pin is used since the diameter required by bending moment is the controling factor.

Thickness of the gusset plate $= \frac{63000}{5 \times 24000} = \frac{3}{4}$"

Cast Iron Rocker

Area required of the masonry plate is

$$\frac{126000}{500} = 254 \text{ sq. in.}$$

A plate 12" x 2" x 24" is used as this provides sufficient area.

Length of rocker $= 24$"

$\frac{126000}{24} = 5291$ lbs per linear in. in bearing stress.

allowable bearing stress $= 600d$

$= 600 \times 18 = 10800$ lbs. per linear inch.

Thickness of upright $= \frac{126000}{2x5x10000} = 1.26$ inches.

Therefore a thickness of $1\frac{1}{2}$" is used.

The forces on the rocker are investigated as a cantilever.

Moment at upright $= 5291\ x(\frac{5.25}{2})^2 = 72,909$ ft. lbs.

" " center $= 63000 \times \frac{10.24}{2} - 5291 \times (\frac{12}{2})^2$

$= 57400$ ft. lbs.

$S = \frac{Mc}{I} = 1650$ largest bending stress.

Using largest bending moment

with I, moment of inertia $= 101.4$

c, distance to extreme fiber $= 2.5$"

Shear at inside of upright $= 5291 \times 5 = 26455$ lbs.

Shear at outside of upright $=$

$(12 - 6.5)5291 - 63000 = 33900$

Section area $= 42.25$ sq. inches.

Largest average unit shear $= \frac{33900}{4225} = 700.+$ lbs.

A depth of five inches is sufficient for bending shear and is used.

Cast Iron Pedestal

Moment of Inertia $= 42.8$

distance to outer fibers $= 1.75$

$S = \frac{Mc}{I} = 2,400.+$

Max Shear $= \frac{33900}{3.5x12} = 810$

A bearing plate 12" x 24" x 3.5" is sufficient and is used on the fixed end. The uprights are the same as the fixed end. There is no column action. Details of rocker and pedestal are shown on general detail sheet accompanying the thesis.

Concrete Floor

E = effective width of wheel.

W = width of wheel or tire in ft.

S = span of slab in feet.

X = distance in feet from center of rear support to the center of wheel.

$$E = \frac{2}{3}(2x + W)$$

$$E = \frac{2}{3}\left(2 \times \frac{4.5}{2} + \frac{1}{2}\right) = 3\frac{1}{3}' \text{ for rear wheel.}$$

Load delivered by each wheel to 1 ft. strip

$$= 6 \text{ tons} \div 3\frac{1}{3}' = 1.8 \text{ tons per ft.}$$

Simple beam: maximum positive bending moment = 4432 ft. lbs.

Continuous beam: maximum positive bending moment

$$= \frac{8}{12} \times 4432 = 2955 \text{ ft. lbs.}$$

The maximum negative bending moment is equal to the maximum positive bending moment.

Minimum covering of re-enforcing = 2"

" " " stirrups = 1"

Steel Ratio:

Representativeletters same as customary practice

$$p = \frac{\frac{1}{2}}{\frac{fs}{fe}\left(\frac{fs}{n.fe} + 1\right)} = \frac{\frac{1}{2}}{\frac{16000}{650}\left(\frac{16000}{15 \times 650} + 1\right)} = \frac{1}{140}$$

Steel Percentage = .75

$K = .37$

$j = .87$

$d = .0965\sqrt{\frac{4432}{1}} = 6.47$

Total depth of slab allowing for covering is therefore 9".

Re-enforcing $\frac{7}{18}$" cold twisted bars spaced 3", with 2" covering turned at an angle of 45^{0} over the stringers to provide for the negative bending moment is sufficient to care for the bond, shear tension, and diagonal tension in the slab.

The road is provided with a crown of $1\frac{1}{2}$ inches with drains as shown in Drawing Number 5, spaced every 20 ft.

Pier:

Pier is made of plan concrete designed to withstand (1) total vertical load due to dead load of the span and live load on the span, and weight of the pier; (2) for wind pressure on pier and bridge (3) for longitudinal thrust due to cars stopping on bridge; (4) for sliding and bouyancy effect of water.

Weight $\frac{1}{2}$ dead load of bridge = 291000 lbs.

" $\frac{1}{2}$ live " on " = 128900 "

Pressure $\frac{1}{2}$ wind against bridge = 14100 "

Longitudinal thrust due to cars stopping

= 20% live load = 21000 "

weight of pier = 331000"

wind pressure against pier = 14580 "

batter is 1:24

Bearing value of hard rock = 20 tons/sq. ft.

In designing the pier and abutment, the depth of hard rock is assumed six ft. below the river bottom which is 11 ft. below the ground surface at that point.

Bearing value of concrete 450 lbs/sq. in.

Taking the dimensions of the top slab as 24.6' x 3' provides sufficient bearing area for the bridge. It is necessary to make this slab 2 ft. deep in order to set four anchorbolts $1\frac{1}{2}$ inch diameter $1\frac{1}{2}$ ft in the masonry. The dimensions below the top slab is $1\frac{1}{2}$ ft. by $23\frac{1}{10}$ ft. with batter on all sides of 1:24.

The center of gravity of the pier = 15.7 ft. from the base. Investigation shows that the resultant of all forces acting upon the pier fall within the middle third 4 inches from the center, and that it will withstand overturning and sliding. The pier will be 31'-7 $\frac{3}{16}$ " high.

Bridge Abutment with Wing Walls:

The abutment is designed to be safe (1) against overturning (2) against sliding (3) against crushing the material on which the abutment rests. The abutment and wing walls are made of plan concrete with drains 1" in diameter, two being placed just above the ground in the abutment proper and two in each wing wall.

Slope of fiel = $1\frac{1}{2}$:1

Character of " is clay = 100# cu. ft.

Height of Fiel = 19.6 ft.

Angle of wing walls = 45°

Outside face of abutment and wing walls is a batter of 1:24. Dimensions for the abutment and wing wall are shown on accompanying drawing number 5. The back is stepped to provide easier forms and pouring of concrete. The abutment is investigated for overturning. As the rocker is upon this end no tractive force is imparted. In this investigation the wind should not be taken as acting against the abutment. It satisfactorily meets this requirement and also does not slide. In the design of the back fill consideration of a superimposed load of trucks and concrete roadway are considered amounting to 60 tons. This is converted to a surcharge of clay to make the calculations simpler. Investigation of determining whether the resultant falls within the middle third is determined according to figures (e) and (f) from William's "Design of Masonry Structures and Foundations" page 259 for wing wall and abutment respectively.

Wing wall $y = \frac{h + h'}{3}$

$$P = \tfrac{1}{2}w(h + h')^2 \cos \theta$$

Abutment $y = \frac{h^2 + 3hh'}{3(h + 2h')}$

$$P = \tfrac{1}{2}\, wh(h + 2h')\ \frac{1-\sin\theta}{1+\sin\theta}$$

Where $y =$ distance from base to pressure P of back fill

$\theta =$ angle of repose of material

$h =$ height of wall

$h' =$ surcharge & vertical component of slope

W - wt. per cu. ft. of material

P,- pressure acting parallel to ground surface.

It is found that the resultant of the wt. of the abutment, unloaded bridge, and pressure of back fill fall within the middle third. Investigation is also separately made upon the wing walls as a separate structure.

Note: Important

All specifications meet requirements of "Virginia State Bridge Highway Commission" 1926. This is filed with this thesis.

Material

Weight of steel in truss	231,173.00	lbs.
Weight of rivets	1,425	"
Weight of Rocker and Pedestal	1,034	"
Weight of anchor bolts.	58.	"
Volume of concrete for pier & abutment	560.	cu. yd.
Volume of " for floor slab	91.2	" "
Weight of reinforcement in floor slab	6768.	lbs.

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   per hour and includes detailing, shop labor, and one coat of shop paint. The estimate 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 finish 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 costs of laying similar amounts of concrete.

Cost of Concrete Work

Excavation per cu. yd.	$7.00
Concrete in place per cu. yd.	7.00
Cement mixed in concrete	3.00
Total per cu. yd.	$17.00

Cost pier: 85.1 cu. yds x $17.00	$1446.70
" abutment : 474.9 cu. yds. x $17.00	$8073.30

Cost of Floor Slab

Re-enforcements in place per lb.	$0.04
Cost concrete in floor per cu. yd.	20.00

Cost re-enforcement 42.3 lbs. x 160 x \$.04	270.72
" concrete in floor 0.57 cu. yd. x 160 x \$20.00	1824.00
Total cost of floor	\$2114.76

Shop Cost of Steel in Truss

Average cost of steel at mill	\$2.00 per 100 lbs.
" shop cost	.72 " " "
Freight Pittsburg to Lexington	.46 " " "
Total f.o.b. to Lexington	3.18 " " "

Rivets

Average cost of rivets	\$2.50 per 100 lbs.
Freight Pittsburg to Lexington	0.46 " " "
Total f.o.b. to Lexington	\$2.96 " " "

Anchor Bolts

Cost of anchor bolts	\$4.00 per 100 lbs.
Freight Pittsburg to Lexington	0.46 " " "
Total f.o.b. to Lexington	\$4.46 " " "

Rockers and Pedestals

Cost of rockers and pedestals	\$4.00 per 100 lbs.
Freight Pittsburg to Lexington	0.46 " " "
Total f.o.b. to Lexington	\$4.46 " " "

Erection

Hauling steel from station to site at 50¢ per ton and 50¢ per ton for loading and unloading 240 tons x \$1.00 = \$120.00

Cost of erecting steel at $20.00 per ton	$4420.00
Cost of painting at $3.50 per ton	420.00
Total cost of erection	$4960.00

Summary of Cost of Entire Structure

Steel f.o.b. to Lexington 2,312 x $3.18	$7352.00
Rivets f.o.b. to Lexington 15 x $2.96	44.00
Anchor bolts f.o.b. to Lexington .5 x $4.46	2.00
Rockers and Pedestals f.o.b. to Lexington 10 x $4.46	45.00
Floor slab	2115.00
Pier	1447.00
Abutment	8073.00
Cost of structure	$19058.00
Allowing 10% for extra	1906.00
Total cost of structure	$20964.00
Allowing 10% for contractor's profit	2096.00
Estimated contractors bid	$23060.00

A 66 ft. right of way will be necessary for the proposed road and bridge. The value of the property condemned including land, buildings, machinery, etc., is estimated from Land Book of Lexington, Virginia at the Court House

Approximate value of condemned property	$12000.00
	$23060.00
Total probable cost of right of way with contractors probable bid	$35060.00