ECONOMIC ANALYSIS AND TYPE SELECTION FOR
A HIGHWAY BRIDGE OVER TALLULAH GORGE

A THESIS

Submitted for the Degree of
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ENGINEERING

By

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B. S. in C. E., 1928

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Approved by:

Professor of Civil Engineering

Date:
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The purpose of this thesis is to give the design for a three-hinged steel arch and a reinforced concrete arch highway bridge, both for the same crossing, and finally to make an economic comparison of these bridges, together with a steel cantilever bridge, the design of which is given in a thesis prepared by Mr. S. E. Braswell, and a copy of which may be found in the appendix of this paper. It has been attempted to present the material in such a manner as to be of the most assistance to a student or inexperienced engineer beginning his first design of an arch highway bridge, either steel or concrete.

For information concerning design, the following reference books have been the most used: "Movable and Long Span Bridges", Vol. III, Hool and Kinne; "Roofs and Bridges", Part IV, Merriman and Jacoby; "Modern Framed Structures", Parts I, II, and III, Johnson, Bryan and Turneaure; "Notes On The Design of Concrete Structures", Professor F. C. Snow, Georgia School of Technology; "Design of Concrete Structures", Urquhart and O'Rourke; "Reinforced Concrete Construction", Part II, Hool; and "Economics of Highway Bridge Types", McCullough.

It is wished to express sincere appreciation to the following men and organizations: Professor F. C. Snow and Professor J. M. Smith, of the Georgia School of Technology, for their kind and willing assistance given throughout the preparation of this thesis; Bridge Department of The State
Highway Department of Georgia, for field notes and other information concerning the bridge location; Mr. F. A. Blackwell, Chief Engineer of The Virginia Bridge and Iron Co., for information concerning weights of details and unit prices for structural steel; The American Institute of Steel Construction and The American Bridge Co., for information concerning the weights of steel details; and to Mr. S. E. Braswell for cooperation in planning his thesis and this one so that the results of both could be combined to give an economic comparison of three types of bridges for the same crossing. Most of the design computations are given as slide-rule results, but while there are necessarily small errors, the results are believed to be accurate enough for all practical purposes.

When the subject of these theses was chosen it was the object of the two participants first to obtain knowledge, and second to put what was learned in presentable form so that others might know what had been accomplished. Since neither of the participants had any experience in designing either of the three types of bridges, it was decided that more personal knowledge could be obtained by each of the individuals studying the theories and making the necessary design computations together for each type of bridge.

This method was found to be very beneficial and entirely satisfactory. It resulted in the correction of many numerical errors which otherwise would have gone unnoticed. Also many points of doubt in theory were found, especially where there was more than one possible solution, and in each case the question was debated thoroughly between the designers. Relative merits
of different methods were discussed, and if an agreement could not be reached, the question was taken up and discussed with the professors before further computations were made. No design computations on either type of bridge were made by one unless the other was present and giving assistance. So it is believed that the work has been of much more value to each individual than if the work had been divided and done separately.

After the designs had been completed, work was divided as equally as possible into two parts. One took the three-hinged steel arch and the concrete arch by Cochrane's method, while the other took the steel cantilever with suspended span and the concrete arch by the elastic theory. The problem of getting the material into presentable form was done by each individual independently of the other. This work included writing the thesis and making the necessary tracings for drawings and tables. This latter part of the work, which was done independently, proved to be far more laborious than the actual design.

In order that the design of all three types of bridges may be easily referred to, a complete copy of the thesis submitted by Mr. S. E. Braswell has been included in the appendix of this one.
INTRODUCTION

The bridges referred to in this thesis have been designed for a proposed crossing over Tallulah Gorge about sixty feet below the Georgia Power Company's existing dam. Traffic is now carried across the gorge by a bridge, having a roadway width of only 12 feet, located on top of the dam. The site is on one of Georgia's most important highways, Route 15, between Atlanta, Ga. and Asheville, N. C. It is evident, without going into traffic details, that such a bridge as the present one is entirely inadequate.

Officials of the Highway Department have for more than two years been aware of the fact that a new structure is needed, but, due to lack of funds, they have been compelled to prolong the use of the present bridge. In the meantime, however, investigations have been made concerning the proper location for a future structure. The first plan was to build a wider bridge directly over the present one by using columns supported by the sides of the dam, but on account of very poor alignment that idea was abandoned in favor of a new crossing. A survey was then made by the State Highway Department of the site below the dam. Thus, with a definite profile given, the problem was to determine the most suitable type of structure to be used. And it is that problem with which this thesis is concerned.

A study of the profile, Fig. 1, shows that the finished grade elevation of the bridge has been fixed at about 150 feet above the lowest point in the gorge. This fact alone
suggested that some type of bridge requiring no falsework for the main span would be the most economical. The solid rock outcropping at the surface suggested almost perfect foundation conditions, and therefore a steel arch seemed suitable. Also, it was apparent that by using concrete piers of reasonable height a steel-arched cantilever with a suspended span might be economical. It was impossible to say without further investigation which of the two types mentioned above would prove to be the most suitable; a possible saving in steel in the cantilever, due chiefly to a shorter main span, being partly or entirely offset by the large quantity of concrete in the piers. A simple steel truss, which would most likely contain less steel than either the arch or cantilever, was undesirable on account of the excessive falsework which would be required. If the length had been more than 700 or 800 feet a suspension bridge might have been considered, but for this crossing such a bridge would undoubtedly be found uneconomical.

In addition to the question of economics there was the important problem of aesthetics. In many cases it has been necessary to give more attention to the aesthetics of a bridge than to the actual cost. A study of the local conditions surrounding the bridge site determined to what extent economy should be sacrificed for aesthetics. It happens that the site of the bridge treated in this thesis is located in the midst of the most beautiful mountain scenery in Georgia. The
small town of Tallulah is well known as a summer resort, and extensive scenic developments in the vicinity of the bridge have been proposed. It was for this aesthetic reason that a concrete arch bridge has been designed; and an attempt has been made to give justice to both economy and aesthetics in the comparison of the three types of bridges.

Designs complete enough only to determine the most economical structure have been given, and complete detailed designs have not been attempted. The steel arch and the concrete arch have first been treated separately and later comparisons have been made together with the steel cantilever.

After finding that it would be proper to investigate a steel arch it had to be decided which of the various types to use. There are solid ribs, braced ribs, and spandrel-braced arches; and each of these types may have one, two, or three hinges, or it may be hingeless. The spandrel-braced arch with three hinges is the type most used in this country. Authorities differ as to whether a three-hinged arch contains more or less steel than a fixed arch. The true answer may depend largely upon the ratio of rise to span length; for with a low rise, temperature stresses in a fixed arch are much larger than in one with a high rise. For a three-hinged arch temperature stresses are comparatively small for any practical rise ratio. In most cases such stresses are entirely neglected in a three-hinged arch. However, the change in the rise of the crown due to a change in temperature affects the magnitude of the horizontal thrust, which in turn affects nearly all the
members in the arch.

The chief advantage of the fixed arch is its stiffness, but with a concrete floor and moderate live load there should not be objectionable vibration even in an arch with three hinges. The chief advantage of the three-hinged arch is its more correct theory of analysis and less labor in design computations.

In view of the above discussion it was decided that a three-hinged spandrel braced arch would be the proper type to select for spanning Tallulah Gorge. The stresses in a bridge of this type are statically determinate, and there are no unusual problems in determining stresses except for those caused by wind. In the following pages a brief discussion of the theory is given along with the design. It is hoped that in this way the thesis will be made of maximum value to anyone seeking information concerning methods of determining stresses in a three-hinged steel arch.

THREE-HINGED STEEL ARCH

The arch was laid out for this profile with a view of obtaining a ratio of span to rise somewhere between 6 and 10. A span of 280 ft. 0 in. with a rise of 40 ft. 0 in. makes this ratio exactly 7, and gives a main span which very well fits the profile. In addition to the main arch span, three 45 ft. approach spans have been laid out to complete the bridge proper.

Usually the floor system of steel bridges consists of some sort of road surface supported by stringers which are in turn supported by floor beams at the panel points. Due to
Total length 415'-0"

14 panels at 20'-0" = 280'-0" c.e.c. of end pins

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HIGHWAY BRIDGE
OVER
TALLULAH GORGE
TALLULAH FALLS, GEORGIA
Scale 1/2" = 1'-0"

Fig. No. 1
erection conditions, however, it was decided to omit the
stringers in the arch span and support the road slab directly
by floor beams spaced 5 ft. apart. The floor beams are support-
ed directly by the main top chord members of the arch. This
arrangement caused large bending stresses in the top chord.
The full dead-load stresses from arch action, however, were
theoretically zero for the top chord members with the lower
chord constructed as a parabola. Further, it was evident that
these members must be designed strong enough to carry the high
tensile stresses caused by the cantilever action during erection.
Thus it was thought advisable to make the top chord large
enough to carry the erection stresses and to omit stringers.

When a concrete road slab is used its weight will
constitute the greater part of the dead load of the structure.
For this reason the floor system was designed before any com-
putations were made for stresses in the truss members. In
order to make the concrete slab as thin as practicable and
thus reduce the dead load stresses in the arch, the floor
beams were spaced 5 ft. 0 in. apart. The slab was made con-
tinuous for seven panels, thus giving an expansion joint every
35 ft. It was designed, however, as if it were continuous for
only three panels. A 15-ton truck with loads distributed
according to specifications was used in determining live load
stresses. Live-load bending moments were determined by the
use of influence lines for three continuous spans. (See "Re-
inforced Concrete Construction", Hool, Vol. II). Dead-load
moments were taken as .078 wL² for positive moment and
.085 \text{WL}^2 \text{ for negative moment. Impact percentage was determined by the formula, } I = \frac{50}{125/L} \text{ and the effective width was determined by the formula } E = .75/W, \text{ for main reinforcing parallel to the direction of traffic. These formulas are explained in the specifications given in the appendix of this thesis. The bending moments found by the above method are tabulated below:}

<table>
<thead>
<tr>
<th>D.L.W.</th>
<th>End Span</th>
<th>Inter. Span</th>
<th>Inter. Support</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>221</td>
<td>221</td>
<td>236</td>
</tr>
<tr>
<td>L.L. / I.M.</td>
<td>3570</td>
<td>3050</td>
<td>1780</td>
</tr>
<tr>
<td>Total ft. lbs.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>per ft. strip</td>
<td>3791</td>
<td>3271</td>
<td>2016</td>
</tr>
</tbody>
</table>

Using \( f_c = 650 \text{ lb. sq. in.} \) and \( f_s = 16,000 \text{ lb. sq. in.} \), the effective depth required in the end span is, \( d = \frac{3791}{107.7} = 5.3 \text{ in.} \). Then allowing 1.7 in. for cover, \( t = 7.0 \text{ in.} \). This thickness was used throughout the bridge. Using the above bending moments, the area of steel required per ft. strip is 0.612 sq. in. for the end spans, 0.535 sq. in. for the intermediate spans, and 0.333 sq. in., over the supports. 5/8" \( \phi \) bars @ 6" ctrs. gives an area of 0.610 sq. in. and this spacing was used in the bottom of the slab for the entire length of bridge. 1/2" \( \phi \) bars @ 7" ctrs. were used in the top of the slab for the entire length of bridge. In addition to this steel, 1/2" \( \phi \) bars @ 12" ctrs. in alternate faces of the slab were placed perpendicular to the
center line of roadway to prevent cracking from changes in

temperature. The ends of the temperature bars were extended

upward into the concrete curbs. Concrete curbs 91 in. high and

8 in. wide were used on each side of the roadway.

With the top chord of the arch spaced 16' - 0"
c. to c. and with a 20' - 0" roadway, the ends of the floor

beams overhang about three feet on either side. For this

reason, and also because of the rigid connection between the

floor beam and the top chord, the floor beams were designed

for a span of 16' - 0" and with fixed ends. It was found that

a section modulus of 67.5 was required, and 16" Beth. I-beams

@ 45#/ having a section modulus of 73.76 were used.

After the floor system had been designed, the next

step was to determine the dead load to be used in designing

the arch trusses. This load is composed of the weights of the

floor slab and curbs, floor beams, hand rails, and the steel

in the arch itself. The weight of the steel had to be estimat-

ed before design computations were begun. Several formulas

for the weights of steel in simple truss spans were applied

and the general average was found to be about 800 lbs. per

foot of bridge or 400 lbs. per foot of truss, including all

bracings. A list of dead loads is given below:

(over)
Concrete slab and curbs 2070# per foot of bridge.
2" Bituminous mat 500# " " " "
Steel railings (est.) 90# " " " "
2 trusses and bracings 800# " " " "
Total 3460# " " " " or 1730# " " " truss.

In reality the panel point loads increase from the crown toward the abutment, but since the uniform load from the slab, paving, and railing was about three times as much as that from the trusses, and since the weight of the trusses was only estimated, it seemed most practical to use equal panel loads in the design. With 20 ft. panels and a load of 1730 lbs. per foot of truss, the panel load was 34600 lbs.

Dead-Load Stresses

The easiest and quickest way to determine the dead-load stresses is by the use of a Maxwell diagram. With equal panel loads and parabolic curve for the lower chord, dead-load stresses in the top chord and diagonals are zero and the only stress in the columns is that due to the direct upper panel point load. If the crown hinge had not been placed in the lower chord, however, all the truss members would be called upon to carry dead-load stresses. The dead-load stress diagram for this bridge is shown in Fig. 2. A full explanation of the construction of such a diagram may be found in "Movable and Long-Span Steel Bridges", Hool and Kinne, Page 375. The dead-load stresses shown in Table II were scaled from the dead-load diagram.
Note: Loads are 50,000# including loads from approach decks.

- Loads F1 to F6, inclusive = 34,000#
- Assumed of the following items:
  - 2' bituminous mat paving = 500#
  - 6# concrete slab = 1740#
  - 2.5# concrete curb = 250#
  - Steel rails and posts (assumed) = 90#
  - Truss load per ft. of bridge = 880#
  - Total = 34,000#
  - Total load per panel = 17,000#
  - or 20 x 17,000 = 34,000# per panel.

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DIAGRAM FOR D.L. STRESSES
3-HINGED STEEL ARCH
HIGHWAY BRIDGE
YEAR 1929
G.T. PARKIN

Fig. No. 2
Live-Load Stresses

The next procedure in order, was to find the live-load stresses due to vertical loads. There are several methods by which these stresses may be determined. Loads may be placed so as to give the maximum stress in each member and the result found graphically in the same manner in which the dead-load stresses were found. Several problems would need to be solved, however, and a more correct and shorter method is by the use of influence lines.

An influence diagram for any member is a graphical representation of the stress in that particular member caused by a vertical load of one pound placed at any point on the truss. To a beginner, an influence line is often confused with a bending moment diagram which represents graphically the bending moment in a beam or truss at all points, caused by a fixed condition of loading. From an influence diagram it can be seen at a glance just what parts of the truss to load to get stresses all of a like sign, and also at just what point to place a concentrated live-load to produce the maximum stress in the member for which the influence line is drawn.

The method of constructing influence lines has been given further on, but for purposes of explanation the influence line for L-4, L-5, Fig. 3, has been referred to in this paragraph. The shaded and lined areas are the parts with which we are concerned. A vertical ordinate terminated by lines inclosing the shaded area represents total stress in the
Fig. No. 3
member in pounds, caused by a vertical load of one pound placed at the point at which the ordinate is measured. The horizontal line represents to scale the span of the bridge in feet. Any convenient scale may be used for plotting an influence line. In Fig. 3 the horizontal scale was taken as 1" = 50' and the vertical scale as 1" = 6#. The ordinate at U-8 scales 2.8#. This means that with 1# placed at U-8, and with no other load whatever on the bridge, there will be produced a stress of 2.8# in member L-4, L-5. Special care should be taken to label the diagram tension or compression whichever it might be. If it is not certain as to which sign the stress should be, an algebraic equation should be set up with the unit load at the point considered. In this case, however, it is quite evident that compression is predominant throughout the lower chord, and therefore the largest area must represent compression.

Referring again to the influence line for member L-4, L-5, it is seen that at U-4 the ordinate is 0.8# and that tension is indicated. It is also noted that at some point between U-4 and U-5 the vertical ordinate is zero. This means that with a unit load at that point no stress will be produced in L-4, L-5, provided there are no other loads on the structure. Likewise, the diagram shows that zero stress will be produced in member L-4, L-5, by loads placed at U-1, or U-15. If a single concentrated load is to be used, it is evident that the maximum compression will occur in L-4, L-5, with the load at
U-8, and the maximum tension with the load at U-4. To determine the value of the stress due to a concentrated load it is necessary only to multiply the load in pounds by the value of the ordinate in pounds over which the load is placed. For a uniform load per foot of truss the stress in the member can be determined by summing up the values found after multiplying ordinates for each foot across the bridge by the load per foot. Fortunately, however, the same result is obtained by multiplying the area of the diagram by the load per foot of truss. For a maximum compressive stress in member L-4, L-5 the bridge should be loaded over the 203 feet measure from U-15, and the 77 feet measure from U-1 should remain unloaded.

Throughout this design influence lines for the various members have been constructed similar to the method given in "Movable and Long-Span Steel Bridges", Hool and Kinne. By cutting a section through panel U-4, U-5 it is evident that the bending moment about U-4 is a function of the stress in L-4, L-5. It is this function which has been plotted in the book mentioned above. But it is also evident that if the bending moment about U-4 be divided by the perpendicular distance from U-4 to the line of the member L-4, L-5, the actual stress in member L-4, L-5 is obtained. It seemed preferable, on account of time saved, to plot the influence lines for direct stress rather than for the bending moment, which is a function of the stress. For that reason all influence lines for this bridge were drawn to represent the actual stress in the members.
The method of construction used is based on the fact that the horizontal component of the reaction causes a stress of unlike sign from that caused by the vertical component. An influence line for one component of the reaction is superimposed upon the influence line for the other. The difference between the two lines, therefore, represents the net effect or the stress caused by the total load. All the necessary computations required for determining the influence line for L-4, L-5 are given in the following pages. Table I, consisting of ordinates for influence lines, has been included chiefly to show a convenient tabular form which may be used.

In case of a simple beam the maximum moment at any point due to a concentrated load occurs when the load is placed at the point. It is also true that the maximum moment at say panel point U-4 due to the vertical component of the reaction, together with one vertical load on the bridge, will occur when a load is placed at U-4. Since U-4 is the point about which moments must be taken to find the stress in L-4, L-5 and since the vertical component of the reaction causes tension in L-4, L-5 it follows that a load placed at U-4 will cause more tension in L-4, L-5 than the same load will cause if placed at any other panel point. The vertical component of the reaction at R₁ caused by an unit load at U-4 is \( a/L \) and the moment about U-4, due to that component, is \( aKb / L = 220 \times 20 / 220 = 47.2 \text{ ft. lbs.} \). The stress in L-4, L-5 is \( 47.2 / d = 47.2 / 17.75 = 2.66 \text{ lbs. tension.} \)
### Tables of Ordinates for Influence Lines

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<tr>
<th>Member</th>
<th>a</th>
<th>b</th>
<th>(\frac{ab}{L})</th>
<th>c</th>
<th>(\frac{Lc}{4y}) or (\frac{Lc'}{4y})</th>
<th>d</th>
<th>C'</th>
<th>(\frac{ab}{Ld})</th>
<th>(\frac{Lc}{4yd})</th>
<th>(\frac{Lc'}{4yd})</th>
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<tr>
<td>U₂L₂</td>
<td>260</td>
<td>20</td>
<td>5200</td>
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<td>10.811</td>
<td>18.6</td>
<td>35.388</td>
<td></td>
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<td>1.526</td>
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<td>U₂L₃</td>
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### Table N° I

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<th>(\frac{b}{t})</th>
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<th>(\frac{c}{t})</th>
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<tr>
<td>U₂L₂</td>
<td>86</td>
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<td>2.73</td>
<td>46.75</td>
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Table N° I
A vertical ordinate is then plotted under U-4 equal to 2.66 lbs. and one point on the influence line is located. Loads at either U-1 or U-15 cause zero stress in all members except the vertical end posts. Therefore, the point plotted under U-4 is connected to ordinates of zero value at the ends of the bridge.

Now, it is evident that the greatest compressive stress in L-4, L-5 will occur when the horizontal component of the reaction at $R_1$ is greatest. This condition will exist when the unit load is placed over the center hinge. The value of $R_h$ for an unit load at U-8 is found by taking moments about the crown hinge. Then, $R_h = L / 4y$ and the moment about U-4 is $Lc' / 4y = 230 \times 46.505 / 4 \times 40 = 80.00$ ft. lbs. The compressive stress in L-4, L-5 is $Lc' / 4yd = 80 / 17.75 = 4.50$ lbs. This value is plotted under U-8, and the point is connected to zero ordinates at the ends of the bridge. The area under the first line represents tension, and that under the second line represents compression. Therefore, the difference between the two lines represents the actual stress in the member.

For a vertical truss member such as U-5, L-5 the center of moments must be taken at point 0 where the line L-5, L-6 intersects U-4, U-5, extended (see Fig. 4). The mechanical procedure in constructing the influence line for member U-5, L-5 is outlined here.

(a). Lay off $hc / t = Lc / 4yt$ at the center, thus determining point d.
Fig. No. 4
(b). Lay off the distance a/t on a vertical through the left support, thus determining the point e vertically under U-5.

(c). Lay off the distance b/t on a vertical through the right support, thus determining the point f vertically under U-4.

The area which has been shaded represents the influence line for member U-5, L-5. The results may be verified by determining algebraically the effect of an unit load placed at the center of the bridge, at U-5, and at U-4 respectively; each load being considered separately.

For member U-5, L-6 the influence line was determined in the same manner as for U-5, L-5, except that the critical points are U-3, U-6, and U-5, instead of U-5, U-5, and U-4, and t' was substituted for t. If the top chord members had had appreciable camber, then the intersection of L-5, L-6 with the top chord would have been changed, but the same method could have been used.

In order to obtain stresses caused by uniform loads, a planimeter was used to find the areas of the influence diagrams. Another and perhaps better method of constructing influence lines for any of the various members is to place an unit load at each of the critical points and determine the actual stress in the member for each position of the load. As indicated in Fig. 5, that part of the diagram above the axis represents tension, and that part below the axis represents compression. It is also noted that the areas of each part may be determined by scaling only the bases of the triangles; and the
INFLUENCE LINE FOR L4L5

Note: Tension plotted above axis and compression below axis.
Scale: 1" = 2# vertically
1" = 40' horizontally.
For influence lines shown in this thesis in connection with the 3-hinged arch, $K$ represents a constant, according to the scales used, in changing sq.in. of area to stress in lbs. Ordinates ended by arrows are values which must be multiplied by the concentrated load to obtain stress in lbs. Shaded areas represent compression. Lined areas represent tension.
INFLUENCE LINES

Comp. = 1.23
Total comp = 49.2
Stress = ± 9.20

Comp. = 1.0
Total Comp = 54.80
Stress = ± 12.0

Comp. = 1.20
Total " = 48.0
Stress = ± 16.8

Comp. = 1.02
Total " = 40.8
Stress = ± 19.6

Scale this sheet: 1" = 1 vertcally
1" = 40 horizontally
k = 40
INFLUENCE LINES

Comp.: 0.79
Total comp.: 31.60
Stress = 18.40

Ten.: 0.33
Total " = 13.20

Comp.: 0.68
Total " = 27.20
Stress = 17.60

Ten.: 0.24
Total " = 9.60

Comp.: 1.01
Total " = 40.40
Stress = 20.40

Scale for this sheet: 1" = 1# vertically
1" = 40' horizontally
K = 40

U₅ L₅
U₆ L₆
U₇ L₇
INFLUENCE LINE

Scale 1" = 1 ft
K = 40
use of a planimeter is not necessary. The bases of the triangles also represent the loaded lengths to be used in determining the percentages of impact for each member.

**Stresses Due To Lateral Loads**

Wind stresses may be divided into two groups: (a) those in bracings and (b) those in main truss members. Since wind loads and lateral forces due to moving loads produce the same effect, they may be combined, and stresses caused by the combined load may for convenience be called wind stresses.

Lateral loads on the top chord have two paths which may be followed. They may be transferred to the lower chord through the cross frames and thence to the abutments, or they may be carried by the upper chord lateral system to the end bents and thence to the abutments. It is evident that the largest part of the load is carried through the cross frames, because that path is the most rigid; but it is present practice to design the members to carry the total load through each path. This method gives added rigidity to the structure as a whole because of the increased size of members; but where a more exact method would result in considerable savings in material, the increased rigidity obtained in current practice does not seem justified.

Stresses in the upper lateral system were found by assuming that the lateral truss acts as a cantilever supported by a rigid end bent. The diagonals were assumed to carry tension only. Floor beams act as struts, and wind stresses in them were omitted because they were very small compared to the
bending stresses. The stress in member U-5, U'-6 is 
\[(2.5W/2.5W')/\sin A\]. Likewise the stress in any other diagonal is equal to the sum of the forces to the right of the member divided by the sine of the angle.

Stresses in the main upper chord were found by the usual section method. For instance, to find the stress in member U-5, U-6 a section was cut through panel U-5, U-6 and moments were taken about U'-6 dealing with forces to the right of the section.

In the cross frames at the panel points, as shown in Fig. 7, the diagonals were again assumed to carry no compression. Then the tensile stress in U'-6, L'-6 was \[(W/W')/\cosine B\]. Taking moments about L-6 the stress in U'-6, L'-6 may be found by the equation \[(W/W')h/d_6\], but the value \(p'_6\) was needed for further computations, so the stress in U'-6, L'-6 was taken as \[(W/W')h/c_6\], which is very nearly correct.

The lower lateral system was designed as if it were a truss in a single plane in the same manner in which the upper lateral system was designed. Due to the batter of the columns, however, the lower lateral truss becomes deeper towards the abutment and each diagonal makes a different angle with the center line of the truss. Also, in computing stresses in the main members a different lever arm had to be used in each case. (See Fig. 8.) Care was taken in investigating stresses in the lower lateral system to include the forces which are transferred through the cross frames from the upper panel.
UPPER LATERAL SYSTEM

FIG. NO. 6

CROSS FRAME

FIG. NO. 7
points to the lower panel points. For example, the shear in panel L-5, L-6 is 2.5W / 2.5W1 / 2.5W1 / 2.5W1.

In addition to direct wind stresses which have already been mentioned, there are vertical reactions produced on the arch ring which cause stresses in the entire arch truss in exactly the same manner as live-load forces. First, there is an overturning action caused by lateral forces on moving loads applied 6 ft. above the roadway according to specifications. The total distance above the center line of the top chord was assumed to be 8.5 ft., and the load per ft. of bridge was taken as 200#. At panel point L-7 the vertical reaction produced on the arch ring is p17 = 200 X 20 X 8.5 / c7 = 2040# where c7 = 17.136, the distance between trusses at the lower chord panel point. The total lateral force from the moving load was then assumed to be concentrated on the upper chord as a part of the terms W and W1, and thus the lateral load causes an additional vertical load on the arch ring of p17 = (W / W1) X h7 / c7 where h7 is the depth of truss at panel point L-7. The result was found to be 7600 X 6.317 / 17.136 = 3030#.

There is still another vertical load produced on the arch ring—that caused by the diagonals of the lower lateral system. The load is equal to the vertical component of the stress in the diagonal, and its value can be found after determining the angle which the diagonal makes with the vertical. But that method requires much labor in trigonometry dealing with two planes, and a shorter method which gives the same
**Fig. No. 8  1"=30'**

**PLAN—LOWER LATERALS**

Dotted lines indicate idle members under assumed condition that diagonals carry no compression. \( W \) & \( W' \) indicate total windward and leeward lateral forces on lower chord including those transferred from the upper chord.

**Fig. No. 9  1"=20'**

**END ELEV. — LOWER LATERALS**

Arrows indicate direction wind. Dotted arrows indicate tendency of truss to rotate about each panel point. Dotted lines indicate idle members under assumed condition that diagonals carry no compression.
result is explained in "Movable and Long-Span Steel Bridges", Holc and Kinne, page 391. A sketch is shown in Fig. 9 illustrating this method. To find the vertical load on the arch ring at L-7 the shear in panel L-6, L-7, which is the lateral load at L-7, was multiplied by the vertical distance $h_{v6}$ between L-6 and L-7 and divided by the distance between trusses at L-6. The shear in panel L-6, L-7 includes the lateral loads transferred from the upper chord as well as the direct wind load on the lower chord. The value of the vertical load on the arch ring at L-7 caused by the diagonal L-6, L-7 was found to be $P_7 = 16800 \times \frac{2.435}{17.136} = 2450\#$. The total vertical load acting at L-7 caused by lateral forces was $P_7 = \frac{P_{v7}}{P_{v7}} \times \frac{P_{v7}}{P_{v7}} = 3030 / 2040 / 2450 = 7520\#$. Loads at each of the other panel points were computed in the same manner. Stresses caused by the summation of these vertical wind loads were determined by means of a Maxwell diagram as shown in Fig. 10.

There is one other type of stress which had to be determined before the design of top chord members was begun. The cross beams of the floor system rest directly on the top chord members, and thereby produce bending. The dead load from each cross beam was found to be 6970\#. A live load of 24000\# concentrated midway between panel points was used, thus being equivalent to the total rear axle load of one truck. With 34\% impact, the total bending stress in a top chord member was found to be 2,766,000 in. lbs.
After finding all the necessary stresses for arch action, the question arose as to whether erection stresses should be determined before selection of members was started, or whether the members should be designed for arch action and checked for cantilever action during erection, after determining more exactly the panel point dead loads of the truss members and bracings. It was finally decided to follow the second procedure because it was reasonable to believe that only a few, if any, members would be governed by erection stresses.

Tables of stresses for various members have been made up and presented together in Tables II, III, IV, and V. In selecting members care was taken to fix dimensions so that good joints could be made between chord members and the web system. Sample computations have been reproduced below to show the manner in which the tables were built.

**Design of L-2, L-3**

Total design stress = 766,900# compression; length = 21.92 ft., or 263". A section of 2-20" X 1/2" plls. and 4 angles 6" X 6" X 5/8" was selected as a trial. A = 53.76 sq. in.; I (min) = 3202 ; r = 7.72 ; and w = about 235# per ft. including details; L/r = 272 / 7.72 = 35.2. Using the formula provided in the specifications for determining the allowable stress in compression members, the allowable unit stress was found to be 14,300# per sq. in. This was based on a value L/r = 40.0 as that is the limit set by the specifications.
### TABLE OF STRESSES - MAIN MEMBERS OF ARCH

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*Note: Stresses shown in kips. Governing design stresses indicated by *.

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*Numbers indicate stress values in kips.*
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<th>γ-y</th>
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<th>Total B.M. in Kips</th>
<th>C inches</th>
<th>Unit Stress Direct</th>
<th>Total Unit Stress</th>
<th>Allowable Unit Stress</th>
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<th>Unsupported Length Inches</th>
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**DESIGN - MAIN MEMBERS OF ARCH**
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<th>Gross A</th>
<th>Net A</th>
<th>r</th>
<th>Section used</th>
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Short legs outstanding in above members

Take above weights 4 times

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Take above weights 4 times

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</tr>
<tr>
<td>L,L'</td>
<td>5.6</td>
</tr>
</tbody>
</table>

Take above weights twice

<table>
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<tr>
<th>Mem.</th>
<th>Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>L,L'</td>
<td>5.24</td>
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</table>

Take above weight 16 times

Table No. IV
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<tr>
<th>Member</th>
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<th>Net A</th>
<th>r</th>
<th>ed length inches</th>
<th>Section Used</th>
<th>Comp. Ten.</th>
<th>Comp. Ten.</th>
<th>Length Feet</th>
<th>Membr. Weight times</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>U-3</td>
<td>75.5</td>
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<td>6.20</td>
<td>146</td>
<td>4.63# X 2.2# X 9/16</td>
<td>14000</td>
<td>1600</td>
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<td>2.85</td>
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<tr>
<td>a-f</td>
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<td>&quot;</td>
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<td>2.85</td>
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<td>d-f</td>
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<td>10.2</td>
<td>&quot;</td>
<td>&quot;</td>
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<td>&quot;</td>
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<td></td>
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<td>11-14</td>
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<td>118</td>
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<td>&quot;</td>
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<td></td>
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<td></td>
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<tr>
<td>11-14</td>
<td>148</td>
<td>148</td>
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<td>6.48</td>
<td>6.48</td>
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<td></td>
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<tr>
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<td>6.48</td>
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<td>&quot;</td>
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<td>1050</td>
<td></td>
<td>2.85</td>
<td>3.50</td>
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</table>

The following members are horizontal bracings seen in an Elevation of the Arch Truss:

<table>
<thead>
<tr>
<th>A-B</th>
<th>6.48</th>
<th>6.48</th>
<th>6.48</th>
<th>4.63# X 2.2# X 9/16</th>
<th>1050</th>
<th>1050</th>
<th>2.85</th>
<th>3.50</th>
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</thead>
<tbody>
<tr>
<td>B-C</td>
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<td>&quot;</td>
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<td>1050</td>
<td>2.85</td>
<td>3.50</td>
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<tr>
<td>C-D</td>
<td>74.5</td>
<td>74.5</td>
<td>74.5</td>
<td>&quot;</td>
<td>1050</td>
<td>1050</td>
<td>2.85</td>
<td>3.50</td>
</tr>
<tr>
<td>D-E</td>
<td>177.5</td>
<td>177.5</td>
<td>177.5</td>
<td>&quot;</td>
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<td>1050</td>
<td>2.85</td>
<td>3.50</td>
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<tr>
<td>E-F</td>
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<td>60</td>
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<td>1050</td>
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<td>3.50</td>
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<tr>
<td>F-G</td>
<td>250</td>
<td>250</td>
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<td>&quot;</td>
<td>1050</td>
<td>1050</td>
<td>2.85</td>
<td>3.50</td>
</tr>
</tbody>
</table>

Note: It was found desirable not to use any angle smaller than 3# X 2.2# X 9/16 on account of connections. Thus it was not necessary to investigate thoroughly the members where stresses are not shown, for in those cases the stresses were known to be lower than those that are shown. The sections used for horizontal bracings were made large enough to keep the / of 140 or under.

BRACINGS OTHER THAN LATERALS

Table No. V
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
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<th></th>
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</thead>
<tbody>
<tr>
<td>1</td>
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<td>3010</td>
<td>509</td>
<td>313</td>
<td>7803</td>
<td>3902</td>
<td>171</td>
<td>165</td>
<td>339</td>
<td>675</td>
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<tr>
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<td>480</td>
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<td>3381</td>
<td>513</td>
<td>313</td>
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<td>3983</td>
<td>195</td>
<td>167</td>
<td>347</td>
<td>709</td>
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<tr>
<td>3</td>
<td>711</td>
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<td>3431</td>
<td>523</td>
<td>313</td>
<td>8178</td>
<td>4089</td>
<td>265</td>
<td>170</td>
<td>370</td>
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<tr>
<td>4</td>
<td>774</td>
<td>3200</td>
<td>3493</td>
<td>539</td>
<td>313</td>
<td>8319</td>
<td>4160</td>
<td>280</td>
<td>177</td>
<td>420</td>
<td>377</td>
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<tr>
<td>5</td>
<td>972</td>
<td>3200</td>
<td>3584</td>
<td>663</td>
<td>313</td>
<td>9247</td>
<td>4624</td>
<td>611</td>
<td>186</td>
<td>502</td>
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<td>6</td>
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<td>3320</td>
<td>3676</td>
<td>723</td>
<td>369</td>
<td>9541</td>
<td>4771</td>
<td>847</td>
<td>353</td>
<td>990</td>
<td>1620</td>
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<tr>
<td>7</td>
<td>1308</td>
<td>3480</td>
<td>4229</td>
<td>827</td>
<td>369</td>
<td>902</td>
<td>1115</td>
<td>1244</td>
<td>379</td>
<td>1188</td>
<td>2530</td>
</tr>
</tbody>
</table>

Note: Panel point load L₅ found by adding 0.9% for L₆, 0.9% for L₅, 0.9% for L₄, and 1% for L₃.

Note: Truss Dead loads to be used for determining erection stresses.

These loads consist of weights of steel members in truss including bracings, but not including floor beams.

Table No. VI
\[ \frac{1}{8} wL^2 = 141,000 \text{ in. lbs.} \] bending moment in the member due to its own weight. With the crown pin placed 3/16 in. under the c. of g. of the member, the negative moment of \(-\frac{3}{16} \times 766,900 = -143,800\) in. lbs., was theoretically produced in member L-2, L-3. This reduced the net bending moment to only 2800 in. lbs. \(c = 10.25\) in. (or half the depth of the member), and \(f_b = 2800 \times 10.25 / 3202 = 1.8\) lbs. per sq. in. bending stress. The direct stress was found to be \(766,900 / 53.76 = 14,269\) lbs. per sq. in. thus giving a total unit stress of 14,269 lbs. per sq. in. The section selected satisfied the requirements. Other members of the truss were designed in a similar manner and the various steps may be followed by referring to Tables III, IV, and V. Most of the designs were made by making tabulations directly to the tables without recording results elsewhere. Sometimes several trial sections were investigated before a suitable one was obtained, but in such cases changes were made in the tables at that time. Tables IV and V show stresses and designs for bracings.

**Erection Stresses**

Table VI shows the method by which the panel point dead-loads were determined after the arch had been designed. One-fourth of each panel point load was assumed to be concentrated at the upper chord and the remainder at the lower chord. These loads do not include the weight of the floor beams. It will be noted that in Table VII there is a superimposed load of \(8,080\) lbs. per panel point distributed wholly to the upper chord. This superimposed load consists of the following items:
Floor beams = 22.67 X 52 / 2 X 10 = 59# per ft. of truss.
Assumed timber planking = 1265# " " " "
Working material, laboring men, and unforseen loads = 220# " " " "
Total 404# " " " "
404 X 20 = 8,080# per panel point. This load was omitted at U-8.

Panel Loads for Erection Stresses

<table>
<thead>
<tr>
<th>Panel Point</th>
<th>U-1</th>
<th>U-2</th>
<th>U-3</th>
<th>U-4</th>
<th>U-5</th>
<th>U-6</th>
<th>U-7</th>
<th>U-8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Truss Load</td>
<td>2960</td>
<td>3540</td>
<td>3020</td>
<td>2780</td>
<td>2550</td>
<td>2440</td>
<td>2370</td>
<td>1260</td>
</tr>
<tr>
<td>Superimposed</td>
<td>8080</td>
<td>8080</td>
<td>8080</td>
<td>8080</td>
<td>8080</td>
<td>8080</td>
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<td></td>
<td></td>
<td>10000</td>
</tr>
<tr>
<td>Total Upper</td>
<td>11040</td>
<td>11620</td>
<td>11100</td>
<td>10860</td>
<td>10630</td>
<td>20520</td>
<td>20450</td>
<td>1260</td>
</tr>
<tr>
<td>Total Lower</td>
<td>8860</td>
<td>10610</td>
<td>9100</td>
<td>8320</td>
<td>7630</td>
<td>7320</td>
<td>7080</td>
<td>3780</td>
</tr>
<tr>
<td>Total Upper &amp; Lower</td>
<td>19900</td>
<td>22230</td>
<td>20200</td>
<td>19180</td>
<td>18260</td>
<td>27840</td>
<td>27530</td>
<td>5040</td>
</tr>
</tbody>
</table>

A 20-ton crane was assumed to be distributed equally between panel points U-6 and U-7 and equally between the two trusses, thus giving a concentrated load of 10,000# each at U-6 and U-7. Erection stresses were determined by means of a Maxwell diagram. (See Fig. 11.) Referring to the tabulated erection stresses, Table VIII, it may be seen that erection stresses did not govern in a single case. The specifications do not cover allowable unit erection stresses, but since 20,000 lb. per sq. in. is allowed for wind stresses which are of short duration, it was considered safe to use the same unit stress during erection. It is noted that for member U-3, U-4 the total
GA SCHOOL OF TECHNOLOGY

DIAGRAM FOR ERECTION STRESSES
CANTILEVER ACTION

YEAR 1929-1930
G.T. PARKIN
<table>
<thead>
<tr>
<th>Member</th>
<th>Cantilever Stress</th>
<th>Arch Stress</th>
<th>Design Stress</th>
<th>Net A. Reqd.</th>
<th>Gross A. Used</th>
<th>Net A. Used</th>
<th>( \frac{T}{A} )</th>
<th>Mc I</th>
<th>( \frac{T}{A} + \text{Mc I} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>L L l</td>
<td>259.0 + 849.5</td>
<td>Arch Stress governed</td>
<td>238.5</td>
<td>14.95</td>
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<td>37.05</td>
<td>6440</td>
<td>9050</td>
<td>15490</td>
</tr>
<tr>
<td>L L l</td>
<td>239.5 + 766.9</td>
<td>Arch Stress governed</td>
<td>202.5</td>
<td>12.70</td>
<td>46.72</td>
<td>35.27</td>
<td>5750</td>
<td>9780</td>
<td>15530</td>
</tr>
<tr>
<td>L L l</td>
<td>218.5 + 730.9</td>
<td>Arch Stress governed</td>
<td>166.5</td>
<td>10.82</td>
<td>45.00</td>
<td>33.05</td>
<td>5040</td>
<td>10600</td>
<td>15640</td>
</tr>
<tr>
<td>L L l</td>
<td>190.0 + 728.5</td>
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<td>45.00</td>
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<td>4810</td>
<td>15410</td>
<td>15070</td>
</tr>
<tr>
<td>L L l</td>
<td>148.5 + 720.5</td>
<td>Arch Stress governed</td>
<td>156.5</td>
<td>9.78</td>
<td>45.00</td>
<td>33.05</td>
<td>4470</td>
<td>15070</td>
<td>15070</td>
</tr>
<tr>
<td>L L l</td>
<td>81.0 + 725.7</td>
<td>Arch Stress governed</td>
<td>110.3</td>
<td>6.90</td>
<td>43.24</td>
<td>31.76</td>
<td>3390</td>
<td>11300</td>
<td>14690</td>
</tr>
<tr>
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<td>13.5 + 684.9</td>
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<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
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<table>
<thead>
<tr>
<th>Member</th>
<th>Cantilever Stress</th>
<th>Arch Stress</th>
<th>Design Stress</th>
<th>Net A. Reqd.</th>
<th>Gross A. Used</th>
<th>Net A. Used</th>
<th>( \frac{T}{A} )</th>
<th>Mc I</th>
<th>( \frac{T}{A} + \text{Mc I} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>L L l</td>
<td>204.5 + 207.5</td>
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<td>5750</td>
<td>9780</td>
<td>15530</td>
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<td>5040</td>
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<td>45.00</td>
<td>33.05</td>
<td>4470</td>
<td>15070</td>
<td>15070</td>
</tr>
<tr>
<td>L L l</td>
<td>14.0 + 110.3</td>
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<td>110.3</td>
<td>6.90</td>
<td>43.24</td>
<td>31.76</td>
<td>3390</td>
<td>11300</td>
<td>14690</td>
</tr>
<tr>
<td>L L l</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
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</table>

<table>
<thead>
<tr>
<th>Member</th>
<th>Cantilever Stress</th>
<th>Arch Stress</th>
<th>Design Stress</th>
<th>Net A. Reqd.</th>
<th>Gross A. Used</th>
<th>Net A. Used</th>
<th>( \frac{T}{A} )</th>
<th>Mc I</th>
<th>( \frac{T}{A} + \text{Mc I} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>L L l</td>
<td>20.0 + 82.5</td>
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<td>5.16</td>
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<td>5.16</td>
<td>5.16</td>
<td>5.16</td>
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<td>5.16</td>
<td>5.16</td>
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<td>8.38</td>
<td>8.38</td>
<td>8.38</td>
</tr>
<tr>
<td>L L l</td>
<td>30.0 + 87.6</td>
<td>Arch Stress governed</td>
<td>87.6</td>
<td>8.76</td>
<td>11.72</td>
<td>8.38</td>
<td>8.38</td>
<td>8.38</td>
<td>8.38</td>
</tr>
</tbody>
</table>

*Note: An allowable unit stress of 20,000 psi was used for erection stresses & 16,000 psi for arch stresses. Therefore the arch stresses controlled in all cases.
erected stress was 182,500 - 166,500 = 16,000 lbs. more than for arch action, but the total unit erected stress was only 
\[
\frac{T}{A/Mc/I} = \frac{182,500}{33.05/10,600} = 16,120 \text{ lbs. per sq. in., which is far below the unit stress allowed.}
\]

Approach Spans

On the approach spans the concrete slab was made 9 inches thick with 5/8" φ bars @ 13" ctrs. in both top and bottom of the slab, and other 5/8" φ bars between them bent alternately from top of slab over stringers to bottom of slab between stringers. Twenty lines of 3/4" φ longitudinal steel were used. Four lines of stringers spaced 6'-2" c. - c. were framed to floor beams placed at U-1, U-3, U-4, and U-5 only. For the long stringers between U-1 and U-3 the stresses were found to be as follows:

<p>| | | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>D. L. M.</td>
<td>L. L. M.</td>
<td>I. M.</td>
<td>Total M.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>---------------</td>
<td>---------------</td>
<td>---------------</td>
<td>---------------</td>
<td></td>
</tr>
<tr>
<td>30,200 ft. lbs.</td>
<td>74,000 &quot; &quot;</td>
<td>22,200 &quot; &quot;</td>
<td>126,400 &quot; &quot;</td>
<td></td>
</tr>
</tbody>
</table>

An 18" Beth. I-Beam @ 52# was required.

For the short stringers between U-3 and U-4 the stresses were:

<p>| | | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>D. L. M.</td>
<td>L. L. M.</td>
<td>I. M.</td>
<td>Total M.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>---------------</td>
<td>---------------</td>
<td>---------------</td>
<td>---------------</td>
<td></td>
</tr>
<tr>
<td>7,550 ft. lbs.</td>
<td>37,000 &quot; &quot;</td>
<td>11,100 &quot; &quot;</td>
<td>55,650 &quot; &quot;</td>
<td></td>
</tr>
</tbody>
</table>

A 14" Beth. I-Beam @ 30# was required.
The stresses in the floor beam at U-3 or U-4 were:

D. L. M.  --------------------------  51,860 ft. lbs.
I. L. M.  --------------------------  85,000 " "
I. M.  --------------------------  25,500 " "
Total M.  --------------------------  162,360 " "

A 22" Beth. I-Beam @ 58# was required.

Since the end floor beams carry the same live load and almost as much dead load as the floor beam at U-3, the same section was used for both.

The dead load per foot of truss is composed of the following items:

Concrete slab and curb  --------------------------  1,275 lbs.
2" Bituminous mat paving  --------------------------  225 "
Stringers  --------------------------  104 "
Floor beams  --------------------------  43 "
Rail (assumed)  --------------------------  45 "
Total, dead load per foot of truss  --------------------------  1,722 "

Concentrated truck loads were used in determining live-load stresses. The rear axle load was taken as 24,000 lbs. and the front axle load as 6,000 lbs., the axles being 14 ft. apart. Stresses were determined by use of influence lines as shown in Fig. 12. It will be noted that the influence lines are not the same as they would have been if floor beams had been used at every panel point. Table IX shows the design of compression members in the truss. Channel sections were used instead of angles in order to make better details. The design of tension members has not been included in the table because they consist of eye-bars designed to be
Fig. No. 12

Approach Span Influence Lines

Note: Floor beams at \( U_1 \), \( U_2 - U_4 \), \& \( U_6 \) only.
Areas used for D.L.
Ordinates used for L.L.
<table>
<thead>
<tr>
<th>Member</th>
<th>D.L.</th>
<th>L.L. Concentrated</th>
<th>I. = 30%</th>
<th>D.H. + I</th>
<th>Length ft.</th>
<th>Gross A</th>
<th>Net A</th>
<th>I</th>
<th>Sect. Used</th>
<th>Unit Stress Allowable</th>
<th>Unit Stress Final</th>
</tr>
</thead>
<tbody>
<tr>
<td>U₁ U₂</td>
<td>+34.9</td>
<td>+24.2</td>
<td>+7.5</td>
<td>+66.6</td>
<td>9</td>
<td>8.04</td>
<td>2.99</td>
<td>2-8&quot;[9] @ 13.75#</td>
<td>14,300</td>
<td>8,300</td>
<td></td>
</tr>
<tr>
<td>U₂ U₃</td>
<td>+69.8</td>
<td>+48.4</td>
<td>+14.5</td>
<td>+132.7</td>
<td>9</td>
<td>9.52</td>
<td>2.89</td>
<td>2-8&quot;[9] @ 16.25#</td>
<td>14,300</td>
<td>13,100</td>
<td></td>
</tr>
<tr>
<td>U₃ U₄</td>
<td>+69.8</td>
<td>+48.4</td>
<td>+14.5</td>
<td>+132.7</td>
<td>9</td>
<td>8.04</td>
<td>2.99</td>
<td>2-8&quot;[9] @ 13.25#</td>
<td>14,300</td>
<td>13,100</td>
<td></td>
</tr>
<tr>
<td>U₂ L₂</td>
<td>+23.3</td>
<td>+16.1</td>
<td>+4.8</td>
<td>+44.2</td>
<td>6</td>
<td>5.26</td>
<td>1.83</td>
<td>2-5&quot;[9] @ 9#</td>
<td>14,300</td>
<td>8,400</td>
<td></td>
</tr>
<tr>
<td>U₃ L₃</td>
<td>+23.3</td>
<td>+16.1</td>
<td>+4.8</td>
<td>+44.2</td>
<td>6</td>
<td>5.26</td>
<td>1.83</td>
<td>2-5&quot;[9] @ 9#</td>
<td>14,300</td>
<td>8,400</td>
<td></td>
</tr>
<tr>
<td>U₁ L₂</td>
<td>-41.8</td>
<td>-29.1</td>
<td>-8.2</td>
<td>-79.1</td>
<td>10.8</td>
<td>Eye bars</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>U₂ L₃</td>
<td>-41.8</td>
<td>-29.1</td>
<td>-8.2</td>
<td>-79.1</td>
<td>10.8</td>
<td>Eye bars</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>U₃ L₄</td>
<td>0</td>
<td>+18.3</td>
<td>+5.5</td>
<td>+23.8</td>
<td>10.8</td>
<td>5.26</td>
<td>1.83</td>
<td>2-5&quot;[9] @ 9#</td>
<td>11,700</td>
<td>4,530</td>
<td></td>
</tr>
<tr>
<td>L₂ L₃</td>
<td>-34.9</td>
<td>-24.2</td>
<td>-7.5</td>
<td>-66.6</td>
<td>9</td>
<td>Eye bars</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>L₃ L₄</td>
<td>-69.8</td>
<td>-48.4</td>
<td>-14.5</td>
<td>-132.7</td>
<td>9</td>
<td>Eye bars</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**TABLE OF DESIGN FOR APPROACH SPAN**

**TABLE IX**
used in the anchorage system for erection of the main span. One angle 3"x 3"x 5/16" has been used for lateral bracings.

Erection of Arch Span

Usually, in simple structures, the design of a bridge does not include the design of necessary falsework required for erection. The contractor is paid only for materials actually placed in the bridge, and the contractor must allow for the cost of form work in fixing his unit bid prices. In the case of a steel arch which may be erected as a cantilever, however, the design of the anchorage system is usually made before the contract is let for building it, and the quantities required for erection purposes are paid for at unit bid prices, even though some of the materials may be discarded after the bridge has been completed.

The anchorage system has been made up of eye-bars strung from the end of the arch at the upper chord to the base of a concrete bent at one end of the bridge, or to an abutment at the other end. The eye-bars are fastened to an I-beam set perpendicular to the line of stress and imbedded in the concrete. The end bent has been made heavy enough to balance a little more than twice the vertical component of the stress in the eye-bars, and the concrete base has been keyed into the rock so as to develop a great deal more in shear than the value of the horizontal component of the stress in the eye-bars.

Computation are shown here to indicate the manner in which the anchorage system was designed. (See Fig. 13 for sketches.)
<table>
<thead>
<tr>
<th>Quantities in bent:</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Parapet</td>
<td>1.5</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>X 20</td>
<td></td>
</tr>
<tr>
<td></td>
<td>=</td>
<td>300</td>
</tr>
<tr>
<td>Cap</td>
<td>3</td>
<td>X 2</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td></td>
</tr>
<tr>
<td></td>
<td>=</td>
<td>180</td>
</tr>
<tr>
<td>Three columns</td>
<td>5</td>
<td>X 10</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>=</td>
<td>300</td>
</tr>
<tr>
<td>Base</td>
<td>4</td>
<td>X 10</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td></td>
</tr>
<tr>
<td></td>
<td>=</td>
<td>800</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>=</td>
<td>1,580</td>
</tr>
<tr>
<td></td>
<td>or</td>
<td>52.5</td>
</tr>
<tr>
<td></td>
<td>cu.</td>
<td>yds</td>
</tr>
</tbody>
</table>

**Weight of bent = 237,000 lbs. = W.**

Total tension in eye-bars from both trusses = T =

\[ 228,000 \times 92.3 \times 2 / 90 = 468,000 \text{ lbs.} \]

\[ T_v = 20 \times 468,000 / 92.3 = 101,500 \text{ lbs.} \]

237,000 - 101,500 = 135,500 lbs. = net vertical reaction on rock from bent. Net shearing area of concrete base = 6 X 20 = 120 sq. ft. = 17,280 sq. in. Unit allowable shear in concrete = 40 lb. per sq. in.

\[ T_h = 228,000 \times 2 = 456,000 \text{ lbs.} \]

Actual shearing stress in concrete = 456,000 / 17,280 = 26.4 lb. per sq. in.

Quantities in the abutment at the other end of the bridge have been estimated to be 65.2 cu. yds., which was found to be sufficient to anchor that part of the bridge during erection.

The beginning and end of the project have been fixed so that the ends of either of the three types of bridges would be included in the project. This gives a definite length of project to use in making an economic comparison of the cost of the three types. Quantities for earth-work include only that required between the ends of the bridge and
Two cables each consisting of 2-5"x2" eye bars in 10.5' lengths
ANCHORAGE SYSTEM AT SOUTH END.

10" Beth. I @ 21# 18'-0" long.
10-1" Steel wire cables wrapped around I-beam and pin.

NOTE: Same details to be used at abutment on North End of bridge.
the ends of the project.

Summary of quantities

Structural steel:

Main span

Main members of arch 242,304 lbs.
Top and bottom laterals 34,188
Bracings other than laterals 44,190
Sub-total 320,682
33 1/3 % for details 106,984
Railing : 74 X 280 20,720
Shoes and pins (Est.) 16,000
Floor beams 68,375
Total for main span 532,699

One approach span exclusive of eye-bar members

Main members 4,297 lbs.
Bracings 2,955
Stringers 8,568
Sub-total 15,820
33 1/3 % for details 5,273
Railing 74 X 45 3,530
Floor beams 5,259
Total for one approach span 29,682
Total for three approach spans 89,046
Total for one steel bent (estimated) 10,670
Total for eye-bars and pins in anchor-age system 23,800
Grand total for bridge 656,145
Reinforcing steel:

<table>
<thead>
<tr>
<th>Description</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>In floor slab of main span</td>
<td>25,380 lbs.</td>
</tr>
<tr>
<td>In floor slab of approach spans</td>
<td>10,593 lbs.</td>
</tr>
<tr>
<td>In one end bent and one abutment</td>
<td>12,741 lbs.</td>
</tr>
<tr>
<td><strong>Total for bridge</strong></td>
<td><strong>46,714 lbs.</strong></td>
</tr>
</tbody>
</table>

Concrete Class "D"

<table>
<thead>
<tr>
<th>Description</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>In slab of main span</td>
<td>139.5 cu. yds.</td>
</tr>
<tr>
<td>In slab of approach spans</td>
<td>83.0 cu. yds.</td>
</tr>
<tr>
<td><strong>Total for bridge</strong></td>
<td><strong>222.5 cu. yds.</strong></td>
</tr>
</tbody>
</table>

Concrete Class "A"

<table>
<thead>
<tr>
<th>Description</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>In one end bent</td>
<td>58.5 cu. yds.</td>
</tr>
<tr>
<td>In one end abutment</td>
<td>65.2 cu. yds.</td>
</tr>
<tr>
<td><strong>Total for bridge</strong></td>
<td><strong>123.7 cu. yds.</strong></td>
</tr>
</tbody>
</table>

Concrete Class "B"

<table>
<thead>
<tr>
<th>Description</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>In two pedestals</td>
<td>11.8 cu. yds.</td>
</tr>
<tr>
<td>In arch abutments</td>
<td>113.0 cu. yds.</td>
</tr>
<tr>
<td><strong>Total for bridge</strong></td>
<td><strong>124.8 cu. yds.</strong></td>
</tr>
</tbody>
</table>

Excavation

<table>
<thead>
<tr>
<th>Description</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>For end bent</td>
<td>48.9 cu. yds.</td>
</tr>
<tr>
<td>For end abutment</td>
<td>86.2 cu. yds.</td>
</tr>
<tr>
<td>For pedestals</td>
<td>19.0 cu. yds.</td>
</tr>
<tr>
<td>Arch abutments</td>
<td>149.3 cu. yds.</td>
</tr>
<tr>
<td><strong>Total for bridge</strong></td>
<td><strong>303.4 cu. yds.</strong></td>
</tr>
</tbody>
</table>
1" Ø steel wire cable ------------------ 530 lin. ft.
Embankment ----------------------------- 200 cu. yds.
2" bituminous mat paving ----------------- 923 sq. yds.

Estimate of cost

<table>
<thead>
<tr>
<th>Item</th>
<th>Quantity</th>
<th>Unit</th>
<th>Rate</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>656,145# struct. steel in place @ $ .0775</td>
<td></td>
<td></td>
<td></td>
<td>$50,681.24</td>
</tr>
<tr>
<td>46,714# reinf. steel</td>
<td></td>
<td></td>
<td>$ .06</td>
<td>2,802.84</td>
</tr>
<tr>
<td>224.5 cu. yds. conc. cl.&quot;P&quot; @ 22.00</td>
<td></td>
<td></td>
<td></td>
<td>4,939.00</td>
</tr>
<tr>
<td>123.7 &quot; &quot; &quot; &quot; A&quot; @ 21.00</td>
<td></td>
<td></td>
<td></td>
<td>2,597.70</td>
</tr>
<tr>
<td>124.2 &quot; &quot; &quot; &quot; B&quot; @ 20.00</td>
<td></td>
<td></td>
<td></td>
<td>2,484.00</td>
</tr>
<tr>
<td>303.4 &quot; &quot; excavation @ 5.00</td>
<td></td>
<td></td>
<td></td>
<td>1,517.00</td>
</tr>
<tr>
<td>530.0 lin. ft. 1&quot; Ø steel wire cable</td>
<td></td>
<td></td>
<td>$ 1.25</td>
<td>639.60</td>
</tr>
<tr>
<td>200.0 cu. yds. embankment @ 1.55</td>
<td></td>
<td></td>
<td></td>
<td>70.00</td>
</tr>
<tr>
<td>923 sq. yds. 2&quot; bit. mat paving</td>
<td></td>
<td></td>
<td>$ 1.50</td>
<td>1,384.50</td>
</tr>
<tr>
<td>Sub-total</td>
<td></td>
<td></td>
<td></td>
<td>$66,778.78</td>
</tr>
<tr>
<td>10% Engineering &amp; Contingencies</td>
<td></td>
<td></td>
<td></td>
<td>6,677.88</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td>$73,456.66</td>
</tr>
</tbody>
</table>
DESIGN OF CONCRETE ARCH BRIDGE

Although it was realized at the beginning that the first cost of a concrete arch bridge would most likely be more than for a steel structure, the concrete arch has been designed because the location demands a consideration of aesthetics. It was also thought that the concrete arch might even prove to be the most economical after considering the cost over a period of years equal to the estimated length of life of the structure.

In making preliminary estimates or comparisons of costs, Cochrane's approximate method of designing concrete arches is accurate enough. But in large arches, designs by the approximate method should be checked by the elastic theory before the bridge is built. It is believed that the most satisfactory way to design a concrete arch is to assume its shape and thickness according to Cochrane's method and then investigate stresses by the elastic theory.

There are different methods of arch analysis based on the elastic theory. The "Bureau of Public Roads" has published a method in which the origin of coordinates is taken at the springing, and another method in which the origin of coordinates is taken at the crown is given in "Concrete Structures", Urquhart and O'Rourke. Either of these two methods may have different modifications.

The arch treated in this thesis has been designed
by Cochrane's approximate method, by the method explained in "Concrete Structures", and also by the "Bureau of Public Roads Method". The design by the second method may be found in the thesis presented by Mr. S. E. Braswell, a copy of which has been included in the appendix of this thesis. A design of the same bridge by the "Bureau of Public Roads Method" was made by the State Highway Department of Georgia. Cochrane's approximate method showed unit stresses somewhat lower than either of the other two, but since it was necessary to extrapolate some of Cochrane's curves, the low stresses found in this case do not condemn the use of the method.

Stresses in the floor system of the superstructure were determined graphically by the method outlined in the class notes on "Reinforced Concrete" prepared by Professor F. C. Snow. The same floor system was designed by the State Highway Department of Georgia by use of influence lines for three continuous spans. (See "Reinforced Concrete Construction", Hool.) Detailed plans for the entire bridge, including the approach decks, have been drawn up by the Highway Department, and since the designs made for this thesis compared so closely with those made by the State, the quantities determined by the Highway Department have been used with the exception of the concrete and steel in the arch ring itself.

DESIGN OF FLOOR SYSTEM.

A half section of the floor system has been shown in Fig. 15. The slab was made continuous over the entire arch span with expansion joints between the main span and the
approach decks. The slab and girders both, however, were designed as if they were continuous over three spans. A diagram for the maximum positive moment in the outside girder has been included in Fig. 16. For sake of brevity, the diagrams for stresses in other parts of the superstructure have been omitted, but they were all similar to the one shown. It is seen that the maximum moment is about 112,500 ft. lbs. as determined by scaling the maximum ordinate on the diagram.

**DESIGN OF ARCH RING BY COCHRANE'S METHOD**

A complete discussion of Cochrane's method of arch analysis is given in "Concrete Engineers' Handbook", Hool and Johnson. For that reason, very little discussion of that method has been included in this thesis. In case of the steel arch, which has already been treated, no one textbook was found which contained all the information necessary to complete the design, so it was thought desirable to include a discussion of the theories.

In order that the design of the concrete arch made for this thesis may be followed step by step, all the numerical computations have been given in order of procedure. In connection with these computations the diagrams in "Concrete Engineers' Handbook" have been used freely. In some cases it was necessary to extrapolate the curves in order to get conditions applicable to the given problem.

**COMPUTATIONS**

\[
\begin{align*}
L &= 210' \\
to &= 2.667' \\
R &= 68' \\
te &= 6.667' \\
\rho &= 0.3236 \\
\theta &= 2.5
\end{align*}
\]
Live Loads used —— Equivalent loadings for H-16
Equivalent loading = concentrated load for shear distributed over the length of span gives 1145 lbs. per ft. bridge.

Uniform L. L. on roadway ------------------ 1145# per ft.
Impact = 50/125/210) = 16% ----------------- 172# " "
U.L.L. = Impact ---------------------- 1317# " "

WL = 1317 x 210 ------------------ 276,570 ft.lbs. L.L.
WL2 = 1317 x (210)2 ------------------ 53,079,700 ft.lbs. L.L.

DEAD LOAD AT CROWN

Wt. of paving & slab ------------------ 3000# per ft. of br.
Wt. of railing ------------------------ 400# " " " "
Wt. of curb -------------------------- 562# " " " "
4" fillets ------------------------------ 33# " " " "
6" " ------------------------------- 38# " " " "

Cross beams and curtain wall per ft. of span ------------------------------ 795# " " " "
Cantilever brackets ------------------- 124# " " " "
Inter. Girders ------------------------ 223# " " " "
" Brackets -------------------------- 24# " " " "
Outside Girders ----------------------- 406# " " " "
Spandrel Walls ------------------------- 990# " " " "

Sub-total ---------------- 6595# " " " "

6595/2 = 3298 # = dead weight on one arch ring.
Wt. of arch ring = 5x2.67x150 = 2000# per ft.
Total dead Wt. = 3298/2000 = 5298, or say
Total Uo = 5300#

WL = 1,113,000 ft. lbs. } Dead Load
WL2 = 233,730,000 " " 
D.L. from Diag. 14a,  

\[ T_0 \text{ or } T_0 = 0.493 \times 5300 \times 210 \quad \text{--------} \quad 548,708\# \]

\[ V_0 = 0.770 \times 1,115,000 \quad \text{--------} \quad 857,016\# \]

\[ T_0 = 0.918 \times 1,115,000 \quad \text{--------} \quad 1,021,754\# \]

L. L. Max. Pos. M. at crown; Diag. 15a  

\[ T_0 = 0.218 \times 276,570 \quad \text{--------} \quad 59,463\# \]

\[ M_0 = 0.00535 \times 58,079,700 \quad \text{--------} \quad 310,726 \text{ ft. lbs.} \]

L. L. Max. Neg. M. at crown  

\[ T_0 = 0.185 \times 276,570 \quad \text{--------} \quad 51,165\# \]

\[ M_0 = -0.00378 \times 58,079,700 \quad \text{--------} \quad 319,542 \text{ ft. lbs.} \]

L. L. Max. Pos. M. at springing  

\[ T_0 = 0.295 \times 276,570 \quad \text{--------} \quad 81,588\# \]

\[ T_0 = 0.301 \times 276,570 \quad \text{--------} \quad 83,247\# \]

\[ M_0 = 0.0305 \times 58,079,700 \quad \text{--------} \quad 1,771,430 \text{ ft. lbs.} \]

L. L. Max. Neg. M. at springing  

\[ T_0 = 0.340 \times 276,570 \quad \text{--------} \quad 94,034\# \]

\[ T_0 = 0.105 \times 276,570 \quad \text{--------} \quad 29,040\# \]

\[ M_0 = -0.081 \times 58,079,700 \quad \text{--------} \quad 1,219,673 \text{ ft. lbs.} \]

The arch was reinforced with 10-1" Ø bars at the extrados and 10-1½" Ø bars at the intrados placed 3" from each face.  

\[ I_0 = 169 \times 20/(144 \times 144) \times 15 \times 2730.667 \times 60/(144 \times 144) \times 2.44 = 7.91 \times 10.35 \text{ ft.}^4 \]

Equivalent area at crown = \( A_0 = 15.43 \text{ sq. ft.} \)

Using a fall of temperature = 40 F.  

\[ t_0 = 0.0000065 \times 40 \times 288,000,000 = 53,400 \text{ lbs. per sq. ft.} \]

From Diag. (19)a  

\[ T_0 = \frac{90}{1} \times t_0 t d E I_0/\pi^2 = -24.4 \times 63400 \times 10.35/(68 \times 68) \times -3,463\# \]
\[ M_0 = -e_2 x h \frac{T_0}{100} = \frac{-21.0 \times 68 \times (-3,463)}{100} < 49,451 \text{ ft. lbs.} \]

\[ T_0 = (1.09 - 1.75 \times 0.325)(20,463) = -1,312\# \]

\[ M_0 = M_0 h T_0 = 49,451 \times 68(-3,463) = -186,038 \]

* See "Concrete Engineers' Handbook", Hool & Johnson

**Average Stresses - From Diag. 20 & 21**

For D.L., Diag. 20w,

\[ f_a = 0 \frac{f_{ac} \times 1.040 \times 548,709/15.43}{548,709/15.43} = 36,985\# \text{ per sq.ft.} \]

For L. L. Producing Max. Pos. M. at crown

\[ f_a = 9245 \times 59463/15.43 = 3,563\# \]

For L. L. Producing Max. Neg. M. at crown

\[ f_a = 1.05 \times 51163/15.43 = 3,482\# \]

For L. L. Producing Max. / mom. at springing

\[ f_a = 0.995 \times 81588/15.43 = 5,206\# \]

For L. L. Prod. Max. / mom. at springing

\[ f_a = 1.045 \times 94034/15.43 = 6,369\# \]

For Temp. drop of 40°F

\[ f_a = .705 \times -3463/15.43 = -158\# \]

* See "Concrete Engineers' Handbook", Hool & Johnson.
### SUMMARY FOR MAX. - MOM. AT CROWN
#### (a) D.L., L.L., & Arch Shortening

**TABLE X**

<table>
<thead>
<tr>
<th></th>
<th>THRUST</th>
<th>MOMENT</th>
<th>AV. STRESS</th>
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<tr>
<td>D. L.</td>
<td>$548,709</td>
<td>$310,726</td>
<td>$36,933</td>
</tr>
<tr>
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<td>$310,726</td>
<td>$3,863</td>
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<td>- 2,210</td>
<td>$31,500</td>
<td>- 101</td>
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<tr>
<td>TOTAL</td>
<td>$609,962</td>
<td>$342,226</td>
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#### (b) D.L., L.L., Temp. Variation and Arch Shortening

<table>
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<td>- 2,210</td>
<td>$49,451</td>
<td>- 158</td>
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<td>Arch S.</td>
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<td>TOTAL</td>
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<td>$391,577</td>
<td>$40,287</td>
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### SUMMARY FOR MAX. - MOMENT AT CROWN
#### (a) D.L., L.L., & Arch Shortening

**TABLE XI**

<table>
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<th>AV. STRESS</th>
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<tr>
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<td>$3,862</td>
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#### (b) D.L., L.L., Temp. Variation and Arch Short.

<table>
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<th>AV. STRESS</th>
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<td>Temp.</td>
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<td>$49,451</td>
<td>- 158</td>
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<td>Arch S.</td>
<td>- 2,220</td>
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### SUMMARY FOR MAX. MOM. AT SPRINGING
(a) D.L., L.L., & Arch Shortening

<table>
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<td><strong>D. L.</strong></td>
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<td><strong>L. L.</strong></td>
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<td>$5,208$</td>
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<td><strong>Arch S.</strong></td>
<td>$1,210$</td>
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<td>$105$</td>
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<tr>
<td><strong>TOTAL</strong></td>
<td>$1,102,112$</td>
<td>$1,647,450$</td>
<td>$42,086$</td>
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</table>

(b) D.L./L. L. Temp. Variation and Arch Short.

<table>
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<th>AV. STRESS</th>
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<td><strong>Temp.</strong></td>
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<td>$158$</td>
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<tr>
<td><strong>Arch S.</strong></td>
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<td>$105$</td>
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<td><strong>TOTAL</strong></td>
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<td>$1,935,465$</td>
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### SUMMARY FOR MAX. -MOMENT AT SPRINGING
(a) D.L., L.L., & Arch Shortening

<table>
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<th>THRUST</th>
<th>MOMENT</th>
<th>AV. STRESS</th>
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<tbody>
<tr>
<td><strong>D. L.</strong></td>
<td>$1,021,734$</td>
<td></td>
<td>$36,925$</td>
</tr>
<tr>
<td><strong>L. L.</strong></td>
<td>$94,034$</td>
<td>$1,219,673$</td>
<td>$6,366$</td>
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<tr>
<td><strong>Arch S.</strong></td>
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<td>$126,800$</td>
<td>$108$</td>
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<tr>
<td><strong>TOTAL</strong></td>
<td>$1,114,536$</td>
<td>$1,346,473$</td>
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<tbody>
<tr>
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<tr>
<td><strong>Temp.</strong></td>
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<td>$186,933$</td>
<td>$158$</td>
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<tr>
<td><strong>Arch S.</strong></td>
<td>$1,240$</td>
<td>$126,800$</td>
<td>$107$</td>
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<tr>
<td><strong>TOTAL</strong></td>
<td>$1,112,710$</td>
<td>$1,532,506$</td>
<td>$43,384$</td>
</tr>
</tbody>
</table>
MAX. UNIT STRESS AT CROWN

For Max. Positive Moment = 391,577
(Including Temp.) Thrust = 602,504

\[ F_0 = \frac{20(50 \times 32)}{602,504 \times 32} = 0.01042 \]
\[ e = \frac{391,577 \times 12}{602,504 \times 32} = 0.244 \]
\[ d' = \frac{3}{32} = 0.094 \]
\[ x = 0.82; \ \frac{M}{ba^2} f_o = 0.123 \]
\[ f_o = \frac{M(0.123 \times ba^2)}{391,577 \times 12(0.123 \times 60 \times 32 \times 32)} \]
\[ 622\# \text{ per sq. in.} \]
\[ f' = 15 \times 622 \times \left[ 1 - 3(0.82 \times 32) \right] = 8270\# \text{ per sq in.} \]
\[ f_b = 15 \times 622 \times \left[ \frac{29}{0.82 \times 32} - 1 \right] = 1030\# \text{ per sq. in.} \]
MAX. UNIT STRESSES

For Max. Pos. Moment at Springing

Considering Temp.

Max. / Nom. = 1,833,463 ft. lbs.
Thrust = 1,102,720 lbs.

\[ \frac{P_0}{a} = \frac{20(60 \times 80)}{80} = 0.00417 \]
\[ \frac{e}{a} = \frac{1,833,463 \times 12}{(1,102,720 \times 80)} = 0.249 \]
\[ d' = 3/80 = 0.0375 \]

\[ k = 0.88 \text{, N/ha} \]
\[ f_e = \frac{1,833,463 \times 12}{(0.112 \times 60 \times 60 \times 80)} = 512\# \text{ per sq. in.} \]
\[ f'_{e} = 15 \times 512 \times \left[ 1 - \frac{3}{455 \times 80} \right] = 7750\# \text{ per sq. in.} \]
\[ f_{s} = 15 \times 512 \times \left[ 77/455 \times 80 - 1 \right] = 9470\# \text{ per sq. in.} \]

Unit stresses for maximum negative moments at the
crown and springing were not determined because it was evident
from inspection of Tables X to XIII that maximum positive
moments gave the critical conditions.
Before a concrete arch can be constructed some definite shape of both the extrados and intrados must be assumed. The usual practice is to make a large scale drawing of the arch ring as determined by Cochrane's curve, by the shape of the dead load thrust diagram, or by some other trial curve, and then draw in multi-centered circular curves to fit the original curve as closely as possible. In the following pages, a method has been given by which the shape of the arch ring does not change from the original shape if made according to Cochrane's equation. The method also affords an easy way to determine vertical thicknesses of the arch ring at any point and to determine the area of the vertical face of the rib.

The procedure is outlined below:

(a) Determine shape of neutral axis by Cochrane's formula. (See Table XIV.)

(b) Determine thicknesses at 10 points by use of tables from Cochrane's method of arch analysis.

(c) Draw up half the arch ring to a large scale. (See Fig. 17.)

(d) From the drawing choose any two convenient points x distance from the crown (points other than x = 0) and measure the values of y for the intrados and extrados at the points chosen.

The equation of the neutral axis for an open spandrel arch is \[ y = \frac{8r L}{L+5r}(3c^2 - 10c^4 r), \] or \[ y = \frac{8r L}{E+5r}(3c^2 + 10c^4 r), \] where a and b are constants for a given
\[ r = \frac{h}{L} = \frac{680}{210} = 0.3238 \quad j = \frac{E}{L} \quad j = 210; \quad L^2 = 44,100 \]

For the Neutral axis:

\[ y = \frac{8rL^4}{(6 + 5r)(3c^2 + 10c^4)} = \frac{8(3238)(210)}{(6 + 1.6190)(3(\frac{2}{5})^2 + 10(\frac{2}{5})^4)(0.3238)} = 
\]

\[ 71.39835 \{ 0.0006802721 \chi^2 + 0.000000016649442 \chi^4 \} = 
\]

\[ \chi^2(0.0486570305 + 0.000001887427 \chi^2) \]

### Values of y for neutral axis

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<th>( \chi^2 )</th>
<th>( 0.0000001887427 \chi^2 )</th>
<th>( \text{Col. 3} + 0.004857031 )</th>
<th>( \text{Col. 4} / \chi^2 )</th>
<th>( \text{Col. 4} )</th>
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<td>0</td>
<td>0</td>
<td>0</td>
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<tr>
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<td>68.000</td>
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</tbody>
</table>

**Table XIV**
Formula for vertical face of arch rib terminated by a vertical through the springing (formula includes both halves of one rib).

\[ y = 65.90' \quad \text{for } x = 100' \]
\[ y = 55.55' \quad \text{for } x = 100' \]
\[ y = 28.70 \quad \text{for } x = 70' \]
\[ y = 24.60 \quad \text{for } x = 70' \]

Equation of curve for inflexions:

\[ y = e \left( \frac{a}{b} \right) \left( e^{-b} - e^{-a} \right) \]

Equation of curve for extrados:

\[ y = \frac{-e}{4} \left( b - a \right) \left( e^{-b} - e^{-a} \right) \]

Extrados:

\[ l = 105' \]

Fig. No. 17

\[ \theta = 2.9^\circ \]
rise and span.

(e) Assume that the intrados and extrados are of the same type of curve as the neutral axis and take the origin of coordinates on the neutral axis at the crown.

The formula for the intrados is \( y = a_1x^2 / b_1x^4 \sim t_0/2 \), and for the extrados \( y = a_2x^2 / b_2x^4 \sim t_0/2 \), where \( t_0 \) is the crown thickness and \( y \) is assumed positive in a downward direction from the neutral axis at the crown.

(f) Determine the values of \( a_1, b_1, a_2 \) and \( b_2 \) by substituting the known values of \( x \) and \( y \), as determined from the drawing, into the algebraic equations.

(g) Determine other values of \( y \) for both the intrados and extrados at points where vertical thicknesses of the arch ring are desired.

The algebraic difference in the corresponding values of \( y \) for the intrados and extrados is the vertical thickness of the arch ring.

(h) Determine the area of the arch rib by substituting in the formula:

\[
A = (a_1 - a_2)(L^3/12) \sim (b_2 - b_1)(L^5/80) \sim t_0 L
\]

where \( A \) represents sq. ft. and \( L \) and \( t_0 \) are in ft.

(i) Multiply the area by the width of the arch ring to obtain the volume in one rib.

The derivation of the formula for the area of an arch rib is given here:

The area between a horizontal line through the neutral axis at the crown and the intrados of the arch ring is,
\[ A_1 = 2 \int_0^{L/2} (a_1 x^2 / b_1 x^4 / t_0/2) \, dx = 2 \int_0^{L/2} a_1 x^2 / 2 \int_0^{L/2} b_1 x^4 / 2 \int_0^{L/2} t_0/2 \, dx = 2 (a_1 x^3 / 3)_{x=0}^{x=L/2} / 2 (b_1 x^5 / 5)_{x=0}^{x=L/2} / 2 (t_0 x/2)_{x=0}^{x=L/2} = a_1 L^3/12 / b_1 L^5/80 / t_0 L/2. \]

Similarly, the area between a horizontal line through the neutral axis at the crown and the extrados is \( A_2 = a_2 L^3/12 / b_2 L^5/80 - t_0 L/2 \).

Subtracting \( A_2 \) from \( A_1 \), we get

\[ A = (a_1-a_2) L^3/12 / (b_1-b_2) L^5/80 / t_0 L. \]

**Volume of Arch Rib**

As stated above, the equation for the intrados is:

\[ y = a_1 x^2 / b_1 x^4 / t_0/2. \]

By scaling ordinates from Fig. 17

\[ y = 28.7 \text{ when } x = 70, \text{ and } y = 65.9 \text{ when } x = 100. \]

For this arch, \( t_0 = 2.667 \). Therefore,

1. \( a_1(70)^2 / b_1(70)^4 / 1.333 = 28.7 \) and

2. \( a_1(100)^2 / b_1(100)^4 / 1.333 = 65.9 \)

Multiplying equation (2) by .49 and subtracting (1) from (3),

1. \( 4,900 a_1 / 49,000,000 b_1 = 31.638 \)

or \( b_1 = .00000017 \times 0.008 \)

Substituting this value of \( b_1 \) in equation (2),
10,900 a₁ / 17.0906 = 64.587

or a₁ = .00474762

Therefore, y = x²(.00474762 / .00000170906 x²) / 1.333

Similarly, for the extrados a₂ = .004912;

b₂ = .000000077631; and y = x²(.004912 / .000000077631 x²) / 1.333

Area of arch rib = (a₁-a₂) L³/12 / (b₁-b₂) L⁵/80 / t₁L

v = (.00474762 - .004912)(210)³/12 / (.000000170906 - .000000077631)(210)⁵/80 / (2.667)(210) = 1126.96 / 476.19 / 560.07 = 909.4 sq. ft.

The volume of 2 ribs each 5 ft. wide, is V = 2 x 5 x 909.4 = 9094 cu. ft. = 355.8 cu. yds.

Each arch rib was reinforced with 20 lines of 1" @ steel bars and 5/8" @ hoops at 24" ctrs. This steel amounts to 39,580 lbs. Earth work for this bridge has been estimated at 400 cu. yds. of embankment.

The remainder of the quantities for the entire bridge were taken from computations made by the State Highway Department as listed below.

<table>
<thead>
<tr>
<th>Class</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class A concrete</td>
<td>1022.2 cu. yds.</td>
</tr>
<tr>
<td>Class B concrete</td>
<td>141.4 cu. yds.</td>
</tr>
<tr>
<td>4&quot; concrete paving</td>
<td>923 sq. yds.</td>
</tr>
<tr>
<td>Concrete handrail</td>
<td>846 lin. ft.</td>
</tr>
<tr>
<td>Reinforcing steel</td>
<td>155,000 lbs.</td>
</tr>
<tr>
<td>Excavation</td>
<td>207.6 cu. yds.</td>
</tr>
</tbody>
</table>

*Note: These quantities do not include materials in the arch rib. Quantities for excavation were divided by the Highway Department into 138.4 cu. yds. No. 1 and 69.2 cu. yds. No. 2.*
## TOTAL QUANTITIES AND ESTIMATE OF COST

<table>
<thead>
<tr>
<th>Description</th>
<th>Quantity</th>
<th>Rate</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,359 cu. yds. concrete A</td>
<td></td>
<td>$35.00</td>
<td>$47,565.00</td>
</tr>
<tr>
<td>141.4 cu. yds. concrete B</td>
<td></td>
<td>$20.00</td>
<td>2,828.00</td>
</tr>
<tr>
<td>925 sq. yds. 4&quot; concrete paving</td>
<td></td>
<td>$2.00</td>
<td>1,846.00</td>
</tr>
<tr>
<td>846 lin. ft. concrete handrail</td>
<td></td>
<td>$2.50</td>
<td>2,115.00</td>
</tr>
<tr>
<td>192,680 lbs. reinf. steel</td>
<td></td>
<td>$0.06</td>
<td>11,554.80</td>
</tr>
<tr>
<td>207.6 cu. yds. excavation</td>
<td></td>
<td>$5.00</td>
<td>1,038.00</td>
</tr>
<tr>
<td>400 cu. yds. embankment</td>
<td></td>
<td>$0.35</td>
<td>140.00</td>
</tr>
</tbody>
</table>

**Sub-total**  
$67,086.80

**10% engineering and contingencies**  
$6,708.66

**Total**  
$73,795.46
ECONOMIC COMPARISON
ECONOMIC COMPARISON

After the preliminary estimates of quantities and first costs have been determined for the competing types of bridges, the next step is to find out which type is the most economical to build.

A lowest first cost does not necessarily mean the most economical. A bridge may be compared with any other useful article. For instance, let us suppose that a man wishes to purchase a pair of shoes. If he has only $2.00 to invest, he will purchase the pair costing $2.00 or less, which he thinks will give him the most satisfactory service. Similarly, if a Bridge Engineer wishing to build a bridge, has a very limited amount of money available to invest he will have to build a bridge at the lowest first cost even though he may not expect it to give long service.

Now suppose that the man wishing to purchase a pair of shoes has $4.00 which he can invest. He will then probably select a pair that will give him more service than the $2.00 shoes would give. Likewise, a Bridge Engineer will build a treated timber bridge rather than an untreated one, unless it is for a temporary purpose.

Let us now say that the man wishes to purchase a pair of shoes to be used for dress occasions. He will more than likely be willing to pay an extra amount just to have a better looking pair of shoes even though they might not be as durable as a cheaper pair. It is for a similar reason that a
bridge having high aesthetic value is worth more to the state than one which is just as durable but having a poor appearance. Thus it can be correctly said that a bridge does have an aesthetic value. In the following pages an attempt has been made to investigate fairly and thoroughly all items entering into an economic equation used for selecting the most economical type of bridge to be used.

In addition to the first cost of a bridge there are other items entering into an economic equation, such as maintenance, interest on first cost, operating cost, insurance, and depreciation. Thus, the true cost of a bridge is represented by the total average annual cost over a period of years equal to the length of life of the structure.

The cost of maintenance should be determined from actual cost records for similar structures built previously by the same organization, but if such data are not available, some published information may be used. Maintenance costs include items that are common to all types of bridges, and others that are necessary only for the special type of construction. The items of the first class include repairs on the roadway surfacing, protection of approach embankment shoulders, protection of earth slopes at ends of bridge, and the cleaning of drains in the roadway of the structure. Also such items as, engineering supervision for maintenance work, rental on idle maintenance equipment, and cost of annual inspection of bridge structures are common to all types of bridges.

Maintenance costs common to steel bridges include repainting all steel parts, repairing expansion joints and
hinges, and other small items.

Maintenance costs common to concrete bridges are mostly concentrated in the repairing of damage done to wheel guards and handrails. The total cost of maintenance of concrete structures is somewhat lower than for any other type. However, if concrete construction is not carefully supervised, deterioration may soon set in and maintenance costs will be much greater.

Interest on the first cost of a structure represents an annual expense, to the owner, which will probably amount to more than the cost of maintenance unless the structure is built of timber. If the money is borrowed, the interest on the first cost will be the amount of interest which must be paid to the lender. If cash is paid for the bridge without borrowing money, the interest charge on the first cost will be equal to the interest which could be earned by lending the money, or by some investment. In either case six per cent may be taken as a probable value, but the amount may vary under different conditions or in different localities.

Operating costs for bridges may be divided into two classes: those resulting in an expense to the state and those causing an expense to the public which uses the structure. And since the public is a part of the state, both classes should be considered in an economic analysis. For instance the floor on the present bridge over Tallulah Gorge is very rough and it is evident that an automobile traveling over the floor will suffer more depreciation than while passing over a smooth floor of the same length. It is true that the
depreciation on one automobile traveling across the bridge may not amount to much, but when traffic reaches a magnitude of a thousand or more vehicles per day, the combined depreciation is considerable. In "Economics of Highway Bridge Types", by Mr. C. B. McCullough, Bridge Engineer for the State Highway Commission of Oregon, information is given concerning average transportation costs per vehicle mile on five different types of surfaces. Since all of the bridges treated in this thesis have a concrete road surface, the operation costs to vehicles need not be considered in making an economic comparison.

The other class of operation costs are common only to movable span bridges and need not be further considered here.

Insurance costs are necessary only in case of timber construction on account of fire hazards, or in case of bridges built in flood areas where the cost of adequate flood protection is prohibitive. Evidently insurance costs need not be considered for a steel or concrete bridge over Tallulah Gorge.

Annual depreciation is another expense which the state must meet when a bridge is built. This cost depends upon the economical length of life of the structure, the first cost, and probably upon a rate of compound interest earned upon a yearly deposit. There are two methods of arriving at the annual depreciation method. In order to determine the annual depreciation by the annuity method, the amount of money is computed which is necessary to deposit at the end of each year, at a given rate of compound interest, to amount to
enough to rebuild the structure at the end of its economical life. If the bridge has no scrap value at the end of its life, the amount necessary to replace the bridge is assumed to be the same as the first cost. If it does have a scrap value, the amount necessary to replace the bridge is assumed to be equal to the first cost minus the scrap value; or, in other words, equal to the total depreciation. The other method of determining the annual depreciation is to neglect any compound interest which might be earned on yearly deposits and divide the first cost minus the scrap value by the number of years of economic life of the structure.

The economic length of life of a structure occurs when the total combined average annual cost is a minimum. Mr. McCullough shows that for concrete structures there is very little difference in the total average annual cost between 40 and 80 years of age, and that for steel structures the same is true for a period of between 35 and 65 years. Since the average yearly costs are practically the same for periods between the limits mentioned above, Mr. McCullough concludes that it does not make a great deal of difference what economical length of life is assumed as long as it is kept within those limits. Professor F. C. Snow, of the Georgia School of Technology, in his class notes on "Engineering Economics", gives formulae by which the economical lengths of life may be computed for various types of construction.

Before giving any definite computations it may be well to collect the different items together which enter into the economic equation and to give each a symbol which will
be used in making comparisons.

\[ N = \text{Economical length of life of the structure in years.} \]

\[ D = \text{Average annual depreciation.} \]

\[ M = \text{Average annual cost of maintenance.} \]

\[ I = \text{Annual interest on first cost expressed in percentage.} \]

\[ O = \text{Average annual operating cost.} \]

\[ I = \text{Annual insurance cost.} \]

\[ C = \text{Total first cost of structure.} \]

\[ W = C \text{ minus the salvage value of the structure at the end of its economical life.} \]

\[ \text{A.C.} = \text{Total average annual cost.} \]

\[ \text{A.C.} = \frac{C (D/C + M/C + I + O/C + I/C)} {N} - \frac{(C-W)} {N} \]

\[ D, M, O, \text{ and } I \text{ are divided by } C \text{ because information concerning these terms are expressed as percentages of the first cost. For example, instead of finding a value } N \text{ to substitute in the equation, we find the value } \frac{M}{C} \text{ directly.} \]

A concrete bridge does not have a salvage value except in cases where the old bridge may be broken up and used as rip rap to protect a new structure. Even then, its value is so small that no allowance is made for it in an economic comparison. A steel bridge, however, may be torn down and parts of it used again in smaller structures. The remainder of the steel may be sold as scrap. The salvage value of steel truss construction depends upon the location of the structure and upon the difficulty of tearing down the bridge. In many cases steel structures are so far away from a shipping point
that the cost of reclaiming the steel would be more than its value, so the structures are left in place.

The Atlantic Steel Company of Atlanta is paying $7.75 per ton for scrap steel F.O.B. Atlanta, the steel being hauled only about five miles. There is approximately 523,000 lbs. of structural steel in the cantilever type of bridge included in the appendix of this thesis. Using $7.75 per ton, the scrap value is $2,082.75; say $2,000. The cost of tearing down the bridge and delivering it to Atlanta has been estimated at $1,000. This value is approximate, and is probably a little under-estimated. In addition to the scrap value, some of the members might be used again in short spans as stringers. Thus considering all factors, a total salvage value of $3,000 may be considered a fair amount.

Aesthetics is an item which is always considered, either consciously or unconsciously, in the design of any bridge. For instance, if a timber bridge is used care is taken to make the handrail have a pleasing appearance. Probably a cheaper rail of the same strength could be used, but a sacrifice is made in cost for sake of aesthetics. Just as a pair of nice looking shoes have more value than a pair of brogans, so does a nice looking bridge have more value than one of poor aesthetics. It has an advertising value to the community in which it is located, just as a well kept residential section has to a city. It attracts the public. Further, it has an advertising value to the state as a whole. Motorists from other states see it, admire it, and talk of "that beautiful bridge I saw in Georgia." Thus it can be definitely
said that the aesthetics of a bridge does have a true value.

The amount of the aesthetic value depends upon the distance from a center of population, the landscape in the vicinity of the bridge, the architectural treatment, and the chance which passing people will have in getting an unobstructed view of the structure. For example, a bridge paralleling a railroad bridge will be seen by passengers on trains, and will have more aesthetic value than the same bridge would have in some remote swamp. A bridge near a city has more aesthetic value than one in a sparsely populated district.

Evidently the weight given to aesthetics must be determined by the Bridge Engineer after he has studied the bridge location. The values below are given as a suggested means of determining what weight to give to aesthetics in selecting a bridge type.

Values of Aesthetics, P, Per Year Expressed as Percentages of First Cost.

<table>
<thead>
<tr>
<th>Type of Construction</th>
<th>In National Parks or Within City Limits</th>
<th>Highway Bridges with Beautiful Surroundings</th>
<th>Highway Bridges in Remote Districts</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber Construction</td>
<td>-</td>
<td>-</td>
<td>0.1</td>
</tr>
<tr>
<td>Steel through trusses</td>
<td>1.0</td>
<td>0.5</td>
<td>0.25</td>
</tr>
<tr>
<td>Steel deck trusses</td>
<td>2.0</td>
<td>1.0</td>
<td>0.5</td>
</tr>
<tr>
<td>Plate girder construction</td>
<td>2.0</td>
<td>1.0</td>
<td>0.5</td>
</tr>
<tr>
<td>Steel arches or arched cantilevers</td>
<td>3.0</td>
<td>1.5</td>
<td>0.75</td>
</tr>
<tr>
<td>Concrete through girders</td>
<td>2.0</td>
<td>1.0</td>
<td>0.5</td>
</tr>
<tr>
<td>Concrete deck girders</td>
<td>3.0</td>
<td>1.8</td>
<td>0.75</td>
</tr>
<tr>
<td>Concrete arches</td>
<td>4.0</td>
<td>2.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>
A summary of the first costs for the three types of bridges is given below:

<table>
<thead>
<tr>
<th>Type of Bridge</th>
<th>Structural Steel</th>
<th>Concrete and Other Parts</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>STEEL CANTILEVER</strong></td>
<td>$44,615.16</td>
<td>$20,471.33</td>
<td>$65,086.49</td>
</tr>
<tr>
<td><strong>STEEL ARCH</strong></td>
<td>$55,936.36</td>
<td>$17,520.50</td>
<td>$73,456.86</td>
</tr>
<tr>
<td><strong>CONCRETE ARCH</strong></td>
<td>$73,795.48</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: The costs listed above include 10% for engineering and contingencies.

Let us first assume that maintenance costs vary directly as the age of the structure, and that no interest is made on yearly deposits for renewal purposes. Further, let us neglect aesthetics entirely. Referring to class notes on "Engineering Economics" by Professor F. C. Snow, we find that the economical length of life of a structure can be determined by the formula \( N = (2W/a)^{1/3} \) where, \( a \), is a constant depending upon the type of construction, which if multiplied by the age of the bridge in years and by the total first cost of the bridge in dollars will give the annual maintenance for that year. Or in other words \( y = ax \) where \( y \) = annual maintenance in percentage of first cost and \( x \) = years of age. The value of \( a \), given for steel structures is 0.00075 and for con-
crete structures .00025.

Then assuming the interest rate on borrowed money to be 6%, and the salvage value of the steel cantilever to be $5,000, or net cost of steel = W = $61,986.49 - $5,000 = $56,986.49, we arrive at the following comparison.

The values of ,a, stated above are for a first cost of $1.00. Therefore, in order to obtain the correct values of ,a, to use in the equations, the values given above should be multiplied by the total first cost minus the salvage value.

Then, for the concrete arch, a, = .00025 x $73,793.48 = $18.48 and for the steel cantilever, a, = .00075 x $61,986.49 = $46.49. The steel arch need not even be considered because the cost is greater than for the cantilever having the same type of construction.

<table>
<thead>
<tr>
<th></th>
<th>Steel Cantilever</th>
<th>Concrete Arch</th>
</tr>
</thead>
<tbody>
<tr>
<td>Economic Life 3 (2L/a)² = I</td>
<td>52 yrs.</td>
<td>90 yrs.</td>
</tr>
<tr>
<td>Av. An. Maint. N x aN/E</td>
<td>$1209</td>
<td>$ 830</td>
</tr>
<tr>
<td>An. Int. I = (C) 6%</td>
<td>3999</td>
<td>4488</td>
</tr>
<tr>
<td>An. Dep. = D x W/F</td>
<td>1182</td>
<td>820</td>
</tr>
<tr>
<td>An. Operating Expense x 0</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>An. Insurance 1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Total Average Annual Cost</td>
<td>$6,300</td>
<td>$5,070</td>
</tr>
</tbody>
</table>

From the above comparison it seems that the concrete arch would be the most economical type to build even with aesthetic values neglected entirely. Annual operating expenses were assumed to be the same for each bridge on account of the first class pavement used for both. The values of ,a,
used above are based on data given in "The American Highway Magazine" for July, 1925, Vol. IV, No. 3. Professor F. C. Snow used that date in determining the values of ,a, given in his class notes on "Engineering Economics".

A recently published book, "Economics of Highway Bridge Types", by Mr. C. B. McCullough, Bridge Engineer for the Oregon State Highway Commission, includes numerous curves and tables, based so far as possible on actual data, showing costs and quantities for various bridge types. Since this book represents a very thorough study of costs of bridges it is believed that the data in it comes nearer representing true conditions in practice than the information given in "The American Highway Magazine."

In either case, the fundamental principals underlying the economic equations are the same. Mr. McCullough gives a table on page 173 of his book summarizing the data necessary for making an economic comparison after the first costs have been determined. A constant ,K, may be selected from that table for several types of construction, for different interest rates, and for different values of aesthetics, which when multiplied by the first cost, C, gives the total average annual cost. In the table, K, represents \( \frac{R\Delta t/c}{D/c + I/c-P} \) where P is the aesthetic value (see Table XIV, this thesis).

Tables included in Mr. McCullough's book are based on 5\% compound interest for annual deposits to replace the structure at the end of its life. The comparison given below is based on 5\% compound interest, 6\% straight interest on
the cost of the bridge, \( P = 2.0 \) for the concrete arch, and
\( P = 1.5 \) for the steel cantilever.

### STEEL CANTILEVER

<table>
<thead>
<tr>
<th>Cost</th>
<th>Annual Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural steel</td>
<td>7.7%</td>
</tr>
<tr>
<td>Concrete and</td>
<td></td>
</tr>
<tr>
<td>other parts</td>
<td>6.6%</td>
</tr>
<tr>
<td>Sub-total annual</td>
<td></td>
</tr>
<tr>
<td>cost</td>
<td></td>
</tr>
<tr>
<td>For economical life</td>
<td>50 yrs.,</td>
</tr>
<tr>
<td>annual salvage</td>
<td>$3000/50 x</td>
</tr>
<tr>
<td>Net total</td>
<td></td>
</tr>
</tbody>
</table>

### CONCRETE ARCH

<table>
<thead>
<tr>
<th>Cost</th>
<th>Annual Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>All parts</td>
<td>6.1%</td>
</tr>
</tbody>
</table>

The above comparison shows a net difference of
\$217.26 per year in favor of the concrete arch.

To illustrate the weight given to aesthetics by the
above method, the comparison below is given neglecting aesth-
etics entirely. All other conditions are assumed to be the same
as in the comparison given above.

### STEEL CANTILEVER

<table>
<thead>
<tr>
<th>Cost</th>
<th>Annual Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural steel</td>
<td>9.2%</td>
</tr>
<tr>
<td>Concrete and</td>
<td></td>
</tr>
<tr>
<td>other parts</td>
<td>8.1%</td>
</tr>
<tr>
<td>Sub-total</td>
<td></td>
</tr>
</tbody>
</table>

Annual salvage value for
economic life = 50 yrs.

Net Total          | $5,693.57   |
This comparison shows a net difference of $233.86 per year in favor of the steel cantilever.

The final decision depends upon whether the aesthetic value of a concrete arch is worth as much as $233.86 per year more than the steel cantilever.

If the bridge location had been in some remote district where aesthetics could not have much value to the surroundings nor to the state, the steel cantilever would make a wise selection. But it must be remembered that Tallulah Falls is already well known as a summer resort, principally because of its cool summer weather and beautiful mountain scenery.

The frontpiece of this thesis does not do justice to the concrete dam just above the bridge site. The face of the dam appears to be much rougher in the picture than is actually the case. So taking all things in consideration it seems that the concrete arch would be the wisest selection for a highway bridge over Tallulah Gorge.
APPENDIX - A

SPECIFICATIONS
SECTION 67

CONCRETE STRUCTURES

67.01. General. Concrete Masonry referred to in these Specifications shall consist of a mixture of Portland cement, aggregate and water mixed in such proportions and handled in such a manner as to obtain the specified quality of finished masonry.

MATERIALS

CEMENT

67.02. Portland Cement: All cement used in the work shall conform to the requirements of the “Standard Specifications and Tests for Portland Cement” adopted by the American Society for Testing Materials—serial designation C-9-26, as modified and amended from time to time by the American Association of State Highway Officials.

Each shipment or lot of cement used on the work shall be sampled and tested in accordance with the amended “Standard Specifications and Tests” above designated. All cement used shall be shipped in cotton cloth bags.

67.03. Storage of Cement: The Contractor shall provide suitable means for storing and protecting the cement against dampness. When cement is furnished by the State, any that has been damaged because of the failure of the Contractor to provide suitable storage shall be paid for by the Contractor. Different brands or grades of cement shall be stored separately.

67.04. Damaged Cement: Bags of cement, which for any reason have become partially set, or which contain lumps or caked cement, shall be rejected. In no instance will any portion of a bag of damaged cement, or a bag containing lumps of caked cement, be used.

Cement salvaged from discarded or used sacks will not be permitted to be used.

Cement shall not be used while its temperature is more than 100 degrees Fahrenheit.

67.05. Ordering Excess Cement: The Contractor shall be held responsible for ordering the correct amount of cement, and if the cement is being furnished by the State, the Contractor shall be charged the net price paid by the State for the Excess cement after the completion of any contract, which excess shall become the property of the Contractor.

67.06. Mixing Different Cements: Different brands of cements, even if tested and approved, shall not be mixed during use, or used alternately, in any one class of construction.

HYDRATED LIME

67.07. Hydrated lime shall be used only when required by Special Provision or ordered by the Engineer. When so used the material shall conform to the requirements of the “Specifications for Hydrated Lime” of the American Society for Testing Materials, serial designation C-6-24.

WATER

67.08. The water used in mortar or concrete shall be subject to the approval of the Engineer, and shall be fresh, reasonably clear, free from oil, acid, salt, strong alkali or vegetable matter. Tests of water shall be made in accordance with the requirements of United States Department of Agriculture, Bulletin 1216 of May 1924 with subsequent revisions.

FINE AGGREGATES

67.09. (a) General: Fine aggregate shall consist of sand, stone screenings, or other inert materials with similar characteristics, or a combination thereof, having clean, hard, strong, durable uncoated grains, free from injurious amounts of clay lumps, soft or flaky particles, shale, alkali, organic matter, loam or other deleterious substances.
67.10. **Grading:** Stone screenings either alone or in combination with sand, shall not be used as fine aggregate except when approved by the Engineer.

Fine aggregate shall preferably be graded from fine to coarse with the coarser particles predominating within the following limits:

- Passing No. 4 sieve: 100%
- Passing No. 20 sieve: 50 to 75%
- Passing No. 50 sieve, not more than: 30%
- Passing No. 50 sieve, not less than: 5%
- Passing No. 100 sieve, not more than: 5%
- Weight removed by elutriation test, not more than: 3%

Sieves shall conform to the requirements specified in the “Standard Method of Test for Sieve Analysis of Aggregates for Concrete,” serial designation C-41-24-T of the American Society for Testing Materials.

67.11. **Strength in Concrete:** The fine aggregate shall be tested in combination with the coarse aggregate and the cement with which it is to be used in the proportions, including water, in which they are to be used on the work, in accordance with the requirements specified in Paragraph 67.14. In case the test provided in Paragraph 67.14 shows the strengths specified therein, the fine aggregate shall be considered acceptable.

67.12. **Mortar Strength:** With the approval and consent of the Engineer, on less important work, the following requirements may be substituted:

(a) Mortar briquettes, cylinders or prisms, consisting of one part by weight of Portland cement and three parts by weight of fine aggregate, mixed and tested in accordance with methods described in the Standard Specifications and test for Portland Cement, serial designation C-9-26 of the American Society for Testing Materials, shall show a tensile or compressive strength at the age of 7 and 28 days not less than that of 1:3 Standard Ottawa sand and mortar of the same consistency made with the same cement.

(b) Upon failure to meet the requirements set forth in the foregoing paragraph, material for Class “A,” concrete having a test ratio of 85% or above and material for Class “B” and “C” concrete having a test ratio of 75% or above, when compared with Standard Ottawa sand, may have the proportion of cement increased so as to produce 100% strength ratio for material to be used in Class “A” concrete and 90% strength ratio for material to be used in Class “B” and “C” Concrete. The exact proportions shall be determined by Laboratory tests. No extra compensation shall be allowed the Contractor for additional cement required.

(c) A mixture of sand and stone screenings, when permitted, in proportion of not over fifty (50) per cent stone screenings shall have a strength ratio of not less than 112 per cent when tested as outlined above.

67.13. **Organic Matter:** No fine aggregate showing a color darker than the standard color when tested in accordance with the Standard Method of Test for Organic Impurities in Sands for Concrete, serial designation C-40-22 of the American Society for Testing Materials shall be used unless the strength requirements of Paragraph 67.14 is fulfilled.

67.14. **Grade and Strength:** The grade of concrete required will generally be specified and shall meet the following minimum requirements as to compressive strength:

<table>
<thead>
<tr>
<th>Class</th>
<th>Mixture</th>
<th>7 Day Test</th>
<th>28 Day Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1-2</td>
<td>1300 lbs. per sq. inch</td>
<td>2200 lbs. per sq. inch</td>
</tr>
<tr>
<td>B</td>
<td>1-2½-5</td>
<td>1000 lbs. per sq. inch</td>
<td>1700 lbs. per sq. inch</td>
</tr>
<tr>
<td>C</td>
<td>1-3</td>
<td>900 lbs. per sq. inch</td>
<td>1500 lbs. per sq. inch</td>
</tr>
<tr>
<td>D</td>
<td>1-2</td>
<td>1500 lbs. per sq. inch</td>
<td>2500 lbs. per sq. inch</td>
</tr>
</tbody>
</table>

The Concrete materials including cement, fine aggregate, coarse aggregate and water, mixed in the proportions in which they are to be used in the work, and tested in accordance with the standard methods of testing, shall at twenty-eight (28) days develop a strength of not less than that specified for the grade.
of concrete required. Upon failure to meet the requirements the proportion of cement, fine aggregate, coarse aggregate and water shall be changed in such way as to produce the specified strength, and the contractor shall receive no extra compensation for such change.

67.15. Storage and Handling: The method of storage and handling fine aggregate on the work shall be such as to avoid segregation of sizes, and the material becoming mixed with mud, dust or trash. When deemed necessary, the Engineer may order the use of platforms for the storage of fine aggregate.

COARSE AGGREGATES

67.16. General: The coarse aggregates for all classes of concrete shall consist of clean, hard, durable crushed stone, gravel or slag free from any coating of clay or slime. Material containing organic matter, thin elongated particles, clay or other deleterious substance, may be rejected by the Engineer without testing.

67.17. Crushed Stone: Crushed stone shall be obtained from clean tough, sound durable rock having a French Coefficient of wear of not less than six (6). The material shall be free from dust and an excess of flat and elongated pieces. Stone shall meet the accelerated soundness test described in U. S. Department of Agriculture, Bulletin No. 1216, of May 1924 with subsequent additions.

67.18. Gravel: Shall consist of clean, tough, durable stone of high resistance to abrasion, free of clay or coatings of any character. “Run of Bank” gravel or gravel which contains disintegrated or soft stone or shale, or excess of flat pieces shall not be used. The loss by abrasion shall not be more than twenty (20) per cent, when tested as described in U. S. Department of Agriculture Bulletin No. 1216.

67.19. Slag: Broken slag aggregate shall consist of clean, tough, durable pieces of copper or iron furnace slag, reasonably uniform in density and quality, non-glassy and free from thin elongated pieces, or any deleterious substance. The dry slag when shaken to refusal shall have a weight per cubic foot of not less than seventy-five (75) pounds, and a French coefficient of wear of not less than five and five-tenths (5.5).

67.20. Methods of Tests: Method of sampling and testing shall be in conformance with the practice recommended by the “Committee on Tests and Investigations” of the American Association of State Highway Officials or U. S. Department of Agriculture, Bulletin 1216.

67.21. Grading: Coarse aggregate shall be uniformly graded between the limits specified and shall meet the following requirements:

For Class “A” Concrete:

<table>
<thead>
<tr>
<th>Screen Size</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 1/2-inch</td>
<td>95–100%</td>
</tr>
<tr>
<td>3/4-inch</td>
<td>35–70%</td>
</tr>
<tr>
<td>1/4-inch</td>
<td>0–5%</td>
</tr>
</tbody>
</table>

For Class “B” and “C” Concrete:

<table>
<thead>
<tr>
<th>Screen Size</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 1/2-inch</td>
<td>100%</td>
</tr>
<tr>
<td>2-inch</td>
<td>80–100%</td>
</tr>
<tr>
<td>1-inch</td>
<td>40–75%</td>
</tr>
<tr>
<td>1/4-inch</td>
<td>0–5%</td>
</tr>
</tbody>
</table>

For Class “D” Concrete, the coarse aggregate shall be graded from particles passing a 3/4-inch circular opening to particles retained on a 1/4-inch screen as specified above.

67.22. Abrasive Test: For Class “A” and Class “D” concrete, the French coefficient shall not be less than six (6) when tested in accordance with Paragraph 67.20. Coarse aggregate for Class “B” and “C” concrete shall be subject to such tests as may be necessary to insure the selection of material suitable for the construction involved.

67.23. Cyclopean Aggregate: With the approval of the Engineer, boulders and large fragments of rock may be imbedded in Class “C” concrete used in footings. Each stone before being imbedded or placed shall be washed clean of all clay, dirt or other deleterious matter, and only sound, tough stone shall be used. No boulders shall be placed closer to each other, or to any part of the forms, than six (6)
inches, and no stone shall be used the greatest dimensions of which is more than one-half (½) the least dimension of the footing. Each stone shall be thoroughly soaked before being placed.

67.24. **Storage and Handling**: The method of storage and handling aggregate on the work shall be such as to avoid segregation of sizes, and the material becoming mixed with mud, dust or trash. When deemed necessary, the Engineer may order the use of platforms for the storage of coarse aggregate. Stock piles shall be built in layers not exceeding three (3) feet in height. Mixing of different types of aggregates in one storage pile will not be permitted.

**CONSTRUCTION METHODS**

67.25. **Classification of Mixtures.** In general four classes of concrete will be specified having the following proportions of cement and aggregates:

- **Class A concrete**: 1 part of cement and 6 parts of fine and coarse aggregates, to approximate a 1:2:4 mixture.
- **Class B concrete**: 1 part of cement and 7½ parts of fine and coarse aggregates, to approximate a 1:2½:5 mixture.
- **Class C concrete**: 1 part of cement and 9 parts of fine and coarse aggregates to approximate a 1:3:6 mixture.
- **Class D concrete**: 1 part of cement and 5 parts of fine and coarse aggregates to approximate a 1:2:3 mixture.

The ratio of fine and coarse aggregate shall be determined by the Engineer for the particular aggregates to be used, but the ratio of fine aggregate to cement shall not be more than the following:

- **Class A concrete**: 1 part of cement to 2½ parts of fine aggregate.
- **Class B concrete**: 1 part of cement to 3 parts of fine aggregate.
- **Class C concrete**: 1 part of cement to 3½ parts of fine aggregate.
- **Class D concrete**: 1 part of cement to 2 parts of fine aggregate.

When the class of concrete is not indicated on the plans the following requirements will control:

1. **For superstructure, arch rings, walls or other parts of a structure having a least dimension less than one (1) foot, and for all heavily reinforced concrete and concrete deposited under water, Class "A" concrete shall be used.** Concrete deposited under water shall have ten per cent (10%) excess cement.

2. **For substructures having a minimum thickness of at least one (1) foot, and in which all steel is embedded at least three (3) inches, Class "B" concrete shall be used.**

3. **For unreinforced footings, not deposited under water, Class "C" concrete shall be used.**

4. **For railing, rail posts, paving on bridges and other very thin sections Class "D" concrete shall be used, except that for paving the maximum size of the coarse aggregate may be one and one-half (1½) inches.**

67.26. **Quality of Concrete.** In case the desired density or strength cannot be obtained from the concrete mixes previously described the Engineer may vary the relative proportions of fine and coarse aggregates or increase the amount of cement to obtain the desired results. No allowance will be made to the Contractor for additional cement required.

The minimum strength of concrete expressed in pounds per square inch developed by concrete test specimens made and tested in accordance with the requirements of the American Society for Testing Materials, Serial designation C-31-27 shall be as follows:

<table>
<thead>
<tr>
<th>Approx. Mix.</th>
<th>Age 7 days</th>
<th>Age 28 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class A concrete</td>
<td>1300</td>
<td>2200</td>
</tr>
<tr>
<td>Class B concrete</td>
<td>1000</td>
<td>1700</td>
</tr>
<tr>
<td>Class C concrete</td>
<td>900</td>
<td>1500</td>
</tr>
<tr>
<td>Class D concrete</td>
<td>1500</td>
<td>2500</td>
</tr>
</tbody>
</table>

6
67.27. **Consistency.** The quantity of mixing water to be used shall be determined in each case by the Engineer and no changes shall be made without his consent. In general a mixture shall be used which contains the minimum amount of water consistent with the required workability.

In general, the consistency of concrete mixtures shall be such that:

1. The mortar clings to the coarse aggregate.
2. The concrete is not sufficiently fluid to segregate when transported to the place of deposit.
3. The concrete, when dropped directly from the discharge chute of the mixer, shall flatten out at the center of the pile but shall stand up and not flow at the edges.
4. The mortar shall show no free water when removed from the mixer.
5. The concrete shall settle into place when deposited in the forms and, when transported in metal chutes at an angle of thirty (30) degrees with the horizontal, it shall slide and not flow in place.
6. The upper layer of the set concrete shall show a cement film upon the surface but shall be free from laitance.

If the slump test is used for measuring the consistency of concrete, due consideration shall be given the character of the aggregates as affecting this test. The measured slump shall be as follows:

- **Mass Concrete.** .......................................................... 1½ to 3 inch slump.
- **Reinforced Concrete.** .................................................. 3 to 5 inch slump.
- **Beams and Thin Sections.** ............................................ 2 to 4 inch slump.
- **Heavy Sections.** ........................................................... 2 to 4 inch slump.

67.28. **Measuring Materials.** Fine and coarse aggregates shall be measured separately by volume or by weight with an approved measuring device. The amount of aggregate used shall be the equivalent of loose volume measurement, correction being made for the water in the aggregate and for the bulking of damp sand.

The accurate measurement of each of the materials composing, and the production of a uniform mixture of the concrete are essential. The Contractor shall furnish and use approved timing devices, a water measuring and discharging device, also a batch measuring equipment which will give the exact amount of aggregate required by the Engineer.

The cement shall be measured as packed by the manufacturer, a sack containing not less than ninety-four (94) pounds being considered one cubic foot.

67.29. **Mixing Concrete.** **Machine Mixing.** Concrete shall be thoroughly mixed in a batch mixer of an approved type. Class A and Class D concrete shall be mixed for a period of not less than one and one-half (1½) minutes; Class B and Class C concrete shall be mixed for a period of not less than one (1) minute after all the materials are in the drum. During the time of mixing the drum shall revolve at the speed for which the machine was designed, but not less than fourteen (14) nor more than twenty (20) revolutions per minute. The entire contents of the drum shall be discharged before recharging.

The mixer shall be equipped with a device for accurately measuring the water and shall be equipped with a device for measuring the time of mixing or recording the number of revolutions of the drum for each batch.

**Hand Mixing.** Hand mixing shall not be permitted except with the specific permission of the Engineer in case of an emergency or on very small jobs. Hand mixing shall be done on water tight platforms. The sand shall be spread evenly over the platform and the cement spread upon it. The sand and cement shall then be thoroughly mixed while dry by means of shovels, until it has a uniform color, after which it shall be formed into a “crater” and water added to give it the proper consistency.

The coarse aggregate shall be thoroughly wetted and added to the mortar and the entire mass turned and returned at least six (6) times until all the stone particles are coated with mortar and the mixture is uniform in color. Batches shall not exceed one-half (½) cubic yard in volume.
67.30. **Retempering.** Retempering of concrete or mortar which has reached its initial set or partially hardened, by remixing with or without additional materials, shall not be permitted.

67.31. **Handling.** The concrete shall be placed in the forms immediately after mixing and in such manner as to avoid the separation or segregation of the aggregate. The mixing plant shall be equipped and arranged so as to permit the mixing and placing of the concrete quickly and uniformly. In case concrete is handled by chutes the resulting concrete shall be satisfactory to the Engineer. In depositing concrete all of the following precautions shall be observed.

(a) In handling the concrete from the mixer to the place of deposit, care shall be taken to avoid any separation of the materials.

(b) When concrete is deposited through chutes, the angle of the same with the horizontal shall be such as will allow the concrete to flow slowly and without separation of the aggregate. The delivery from the spout shall be as close as possible to the point of deposit. Dropping the concrete a distance of more than five (5) feet will not be permitted.

(c) Chutes shall preferably be of metal, but if of wood, metal lined. They shall be kept clean and free from material adhering to their sides and shall be thoroughly flushed with water before and after each run.

(d) Depositing large quantities at one point in the forms, and running and working it along the forms, will not be permitted.

(e) In depositing the concrete, care shall be taken to entirely fill the form but not to bulge or distort the forms or to disturb their alignment.

(f) Special care shall be taken in filling the forms, to work the coarser aggregate away from the face of the forms and to force the concrete under and around the reinforcement. The concrete shall be worked with a spade, pointed steel rod, or other satisfactory implement, in such a manner as to bring a thick layer of mortar in contact with the forms and reinforcement, and to prevent the formation of pockets of stone.

(g) Concrete shall be placed in continuous horizontal layers, the thickness of which generally shall not exceed ten (10) to twelve (12) inches. When it is necessary by reason of an emergency to place less than a complete horizontal layer at one operation, such layer shall terminate at a vertical bulkhead. In any given layer the separate batches shall follow each other so closely that each one shall be placed and compacted before the preceding one has taken initial set, in order that the green concrete shall not be injured and that there shall be no line of separation between the batches. Each layer of concrete shall generally be left somewhat rough to secure efficient bonding with the next layer above. A succeeding layer placed before the under layer has become set shall be compacted in a manner that will entirely break up and obliterate the tendency to produce a construction joint between layers.

67.32. **Depositing Under Water.** In general, concreting under water shall be avoided and it will be permitted only when provided for on the plans, or when specifically authorized by the Engineer and under his direct supervision.

The cofferdams shall be sufficiently tight to prevent any current passing through the space in which the concrete is to be deposited. Pumping will not be permitted in the cofferdams while the concrete is being placed nor until it has reached its initial set.

All concrete deposited under water shall conform to the requirements of Class “A” concrete to which shall be added ten (10) per cent of excess cement.

The flow of concrete shall be continuous, or in case the flow is interrupted the cofferdam shall be pumped out, and all laitance removed before proceeding with the work. The concrete shall be deposited as nearly as practicable in horizontal layers.

The method used in depositing the concrete shall be such as will not permit the washing of the cement from the concrete. The following approved methods may be used under the direct supervision of the Engineer.

(a) **Tremie.** If a tremie is used, it shall consist of a tube twelve (12) to sixteen (16) inches in diameter, constructed in sections, with flanged couplings with gaskets and so placed as to permit the initial
and all subsequent charging to take place without the concrete being dropped through the water. In operating the tremie it shall be kept filled at all times and the discharge end shall be raised only an amount sufficient to permit the concrete to discharge. Provisions shall be made in supporting the tremie so that it may be readily lowered when necessary to "choke off" or retard the flow.

(b) **Dump bucket.** A dump bucket may be used if so designed that it may be opened when it rests upon the surface of the concrete which is to receive the charge. The bucket shall be filled level full and in lowering and raising the bucket, care shall be taken to prevent any unnecessary movement of the water in the cofferdam.

67.33. **Protection and Curing.** All concrete shall be properly protected from extremes of heat and cold until well seasoned. Surfaces exposed to premature drying shall be kept covered and shall be kept damp for from at least five (5) to ten (10) days according to the weather conditions, and as directed by the Engineer.

67.34. **Cyclopean or Rubble Concrete.** Rubble or cyclopean aggregate may be used in unreinforced sections two (2) or more feet in thickness. The stone shall be set in place by hand (not cast or thrown into the concrete) for one-half (1/2) the depth of the stone, and shall be not less than four (4) inches apart in the concrete and not less than six (6) inches from the face of any form. No course of stone shall extend within two (2) feet of the top surface of piers or the surface upon which the super-structure rests. The stone shall be thoroughly wetted before placing in concrete.

67.35. **False Work.** False work for supporting concrete work shall be built on foundations of sufficient strength to carry the load without appreciable deformation. False work which cannot be founded on solid footings must be supported by ample false work piling. False work shall be designed to carry the full loads coming upon it.

For single span bridges false work shall be given a permanent camber equal to one-fortieth (1/40) inch per foot of clear span. Multiple span bridges shall be given the amount of camber specified on the plans.

In general, double wedges or other suitable means shall be provided for constructing and maintaining false work and forms to correct lines.

On important structures, when requested by the Engineer, the contractor shall submit plans for false work and forms for checking and approval before the false work is constructed.

67.36. **Forms.** Wood or metal forms shall be constructed of materials sufficient in strength to hold the concrete without bulging between supports. If the forms bulge or sags at any point when the concrete is placed in them, the portion of concrete causing the distortion shall be immediately removed and the forms properly repaired and strengthened before continuing the work.

In designing forms and centering the concrete shall be treated as a liquid weighing one hundred fifty (150) pounds per cubic foot for vertical loads and eighty-five (85) pounds per cubic foot for horizontal pressure. The unsupported length of the wooden columns and compression members shall not exceed thirty (30) times the diameter or least side.

The material to be used in wood forms for exposed surfaces shall be sized and dressed lumber, free from knot holes, loose knots, cracks, splits, or other defects affecting its strength or the accuracy or the appearance of the finished concrete surfaces. Tongue and grooved material may be required by the Engineer. If metal forms are used all bolt and rivet holes shall be counter-sunk so that a plane smooth surface will be obtained.

Forms shall be so designed and constructed that they may be removed without injury to the concrete. Blocks and bracing shall be removed with the forms and in no case shall any portion of the wood forms be left in the concrete. Special attention shall be paid to the ties and bracing, and where the forms appear to be insufficiently braced, or unsatisfactorily built, either before or during construction, the Engineer shall order the work to be stopped until the defects have been corrected to his satisfaction. The forms shall be so constructed that the finished concrete shall be of the form and dimensions as shown on the plans and true to line and grade.
Forms shall be filleted at all sharp corners and should be given a bevel in the case of all projections such as girders, copings, etc., sufficient to insure their easy removal.

To insure a first-class surface finish on the concrete, the forms shall be painted with a colorless oil, or some other satisfactory means taken to prevent the concrete adhering to them. The forms should be thoroughly drenched with water immediately before the concrete is placed in them. Form lumber which is used a second time shall be thoroughly cleaned and shall be free from bulges, splits or warps.

67.37. Removal of Forms. To permit proper surface finish, forms shall in general, be removed as soon after the concrete has set as practicable and safe.

Forms for ornamental work, railing, parapets, and vertical surfaces that do not carry loads, shall remain in place at least fifteen (15) hours after the last concrete is placed, and a greater length of time in cool or unfavorable weather.

Forms under slabs, beams, girders, arches and structures or parts of structures carrying loads, except culverts or slabs of eight (8) foot span or less, shall remain in place a minimum of twenty-one (21) days in warm weather or greater length of time in cool or unfavorable weather at the discretion of the Engineer. The forms for slabs having a span of eight (8) feet or less shall remain in place from ten (10) to fourteen (14) days.

67.38. Joints. Unless otherwise provided in the detailed plans, the joints in concrete masonry shall be constructed in the following manner.

(a) Construction Joints. Construction joints, except where shown on the plans are to be avoided, but when necessary they shall be made under the direction of the Engineer in such places and in such manner that they will have the least possible effect on the strength of the structure. Construction joints shall be perpendicular to the principal lines of stress and in general at points of minimum shear.

In placing concrete in successive horizontal layers in walls, piers, abutments, etc., construction joints between layers of concrete placed intermittently shall be properly bonded. The surface of each layer shall be left rough and bond provided as shown on the plans or directed by the Engineer. Bond stones shall be placed as prescribed in paragraph 67.34 for “Cyclopean or Rubble Concrete.”

When new concrete is to be placed in contact with old concrete or concrete which has already reached its final set, the surface of the concrete shall be roughened and cleaned of all laitance, dirt or other foreign materials and thoroughly wetted. The amount of coarse aggregate in the batches of concrete placed directly against the old concrete, shall be reduced, so as to give an excess of mortar at the contact surfaces.

(b) Sliding Joints. Where sliding joints are to be provided at the ends of slabs, girders or beams or between walls, etc., the surface of the supporting concrete shall be given a smooth finish and covered with two layers of three ply roofing felt to separate the concrete.

(c) Expansion Joints. Unless otherwise shown on the plans expansion joints shall be filled with an approved asphalt and felt material. The thickness of the joints shall be one-quarter (¼) inch where the length of the moving concrete is twenty (20) feet or less, one-half (½) inch for lengths of twenty-one (21) to thirty-six (36) feet, three-quarters (¾) of an inch for lengths of thirty-seven (37) to fifty (50) feet.

(d) Water Tight Joints. Special water tight and flashed joints shall be constructed as shown on the plans or prescribed in the special provisions.


(a) General. The external surface of all concrete masonry shall be thoroughly worked, while the concrete is being placed, by using a fork or concrete spade of an approved type. The working shall be such as to force coarse aggregate particles from the surface and work the mortar against the forms and around the reinforcing.

As soon as the forms are removed all wires or fastenings shall be cut and recessed, or pushed back with a centering punch, so that the ends will be at least three-eighths (3-8) inch from the finished surface. “Honey comb” or stone pockets shall be cut out as directed by the Engineer, and after the surface has been thoroughly wetted, all holes and depressions shall be carefully pointed with a mortar of sand and cement mixed in the same proportion as was used in the concrete.
Unless otherwise provided on the plans or set forth in the special provisions, railings, posts, curbs, copings, pier caps and pilasters under arches, exposed sides of arch rings, outside of girders, brackets, faces of abutments, and wing walls or other portions of the structure exposed to frequent close view shall be given a rubbed finish.

Panel work and spandrel walls of arches shall be given a rough finish.

The surface of piers, bents, walls and the underside of slabs, girders, arches and bents shall be pointed up without further finish; except when the surface will be exposed to view, excessive form marks and irregularities shall be removed and sufficient finishing done to present a reasonably uniform or pleasing appearance.

(b) Rubbed Finish. A rubbed finish shall be obtained by rubbing the surface to be finished, as soon as practicable after it has been pointed up, with a wood float or mortar block and clear water until all form marks or irregularities of the surface are removed. A No. 16 carborundum brick shall be used to remove irregularities which cannot be removed with the wood float or mortar brick. The use of plaster or neat cement wash shall not be allowed.

(c) Special Finish. As soon as the pointing has been completed and set so as to permit, the entire surface to be treated shall be thoroughly wet by means of a brush and clear water and immediately thereafter thoroughly rubbed with a No. 16 carborundum stone to bring the surface to a lather. All form marks and projections shall be removed by the use of a carborundum stone. The entire surface shall then be lightly brushed with a coat of sand and cement in the proportion of one (1) part of cement to two (2) parts of sand. The sand used shall pass a twenty (20) mesh sieve. The final finish shall be obtained by rubbing with a No. 30 carborundum stone until the entire surface is of a smooth texture. After the final rubbing, the entire surface shall be drenched with water and kept wet for a period of seven (7) days unless otherwise directed by the Engineer. Handrail spindles or other members which have become disfigured by the drip from the carborundum stone, shall be thoroughly cleaned by means of a dilute solution of muriatic acid.

(d) Rough Finish. To obtain this finish the forms must be removed within forty-eight hours after concrete is poured. After the surface has been pointed up it shall be thoroughly rubbed with stiff wire brushes and clear water until the surface skin of mortar has been removed, partially exposing the coarse aggregate. The finish shall be used only when called for on the plans.

67.40. Basis of Payment: The contract price per cubic yard for the various classes of concrete will be payment in full for all concrete materials, false work, forms, equipment, labor, finishing and other materials and incidental expenses entering into concrete construction, which shall include placing weep holes, pipes, conduits, anchor bolts, etc. Reinforcing steel, expansion plates or other metal shall be paid for separately unless otherwise provided on the plans.

The cost of excavation shall not be included in the price of concrete unless specifically so stated on the plans or in the proposal.

If a bid is asked for on railing, that portion of the structure above the top of the curbs shall not be included in the yardage of concrete, but shall be paid for as railing. The contract price per linear foot for railing shall include the cost of all materials, reinforcing steel, labor, finishing, forms and other items necessary to complete the railing as shown.

Payment will be made under:
- Item No. 66, Class “A” Concrete (cubic yard).
- Item No. 67, Class “B” Concrete (cubic yard).
- Item No. 68, Class “C” Concrete (cubic yard).
- Item No. 69, Class “D” Concrete (cubic yard).
- Item No. 70, Concrete Hand Rail (per linear foot).
SECTION 68

REINFORCEMENT IN STRUCTURES

68.01. Description: Under this item reinforcing steel, consisting of deformed bars shall be furnished and placed as called for on the plans or as directed. When deformed bars are specified, the form of the bar used must be approved by the Engineer and shall be such as to provide a net section at all points equivalent to that of a plain square or round bar of equal nominal size. Cold twisted bars shall be used only with the permission of the Engineer.

MATERIALS

68.02. (a) Bar reinforcement: Reinforcement metal for concrete structures shall be manufactured in accordance with and shall in all respects fulfill the physical and chemical requirements of the American Society for Testing Materials, Standard Specifications for Billet Steel Reinforcement Bars serial designation A-15-14 with any subsequent amendments and additions thereto adopted by the Society. Unless otherwise designated upon the plans, all bar reinforcement shall be of open hearth steel of the structural or intermediate grade.

When deformed bars are specified, the form of the bar used must be such as to provide a net section at all points equivalent to bars that of a plain square or round bar of equal nominal size. Twisted bars will be used only by permission of the engineer.

For small single culverts, spans not exceeding five (5') feet, reinforcement material may be of rail steel reinforcement bars, meeting the requirements of the specifications of the American Society for Testing Materials, serial designation A-16-14.

68.03. (b) Mesh Reinforcement: Reinforcing materials shall consist of a steel fabric manufactured from steel meeting the requirements of the American Society for Testing Materials, serial designation A-92-91-T.

Reinforcement shall be free from excess rust, scale or coating of any character which will impair its bond with the concrete. Fabric reinforcement shall consist of members rigidly attached at all joints, or points of intersection and shall have an effective weight of approximately forty-four (44) pounds per one hundred (100) square feet. The spacing of the main members of the fabric shall not be more than six (6) inches nor less than four (4) inches. Tie members to be spaced not more than twelve (12) inches nor less than eight (8) inches. The effective cross sectional area of the main members shall not be less than twenty-five hundred (2500) sq. in. per foot of the fabric. The effective cross sectional area of the tie members shall not be less than twenty-five hundred (0.025) sq. in. per foot of the fabric. Unless otherwise noted on plans reinforcing conforming to the above specifications will be the standard.

CONSTRUCTION METHODS

68.04. Handling and Placing: Metal reinforcement, before being placed, shall be thoroughly cleaned of mill and rust scale and of coatings of any character that will destroy or reduce the bond. Reinforcement appreciably reduced in section shall be rejected. In case reinforcement which has been placed, becomes spattered with mortar which dries out before concrete is placed around it, such reinforcement shall be thoroughly cleaned before being covered with concrete.

The reinforcement shall be bent to the required shapes. The radius of bend shall be four (4) or more times the least diameter of the bar.

The reinforcement shall be accurately placed, and secured against displacement before any concrete is placed, by using annealed iron wire of not less than No. 16 gauge, or suitable clips at intersections, and shall be supported by concrete blocks.

68.05. Spacing Reinforcement: Where the spacing of reinforcement is not definitely shown on the plans, the following general rule shall govern:

Beams and Girders. The distance between the centers of bars shall be not less than D plus A plus ½ inch, not less than 3D, where A equals maximum screen openings specified for coarse aggregate, D equals nominal diameter of bar.
Distance from side form to center of nearest bar shall not be less than \( D + A + \frac{3}{8} \) inch, nor a clear distance of less than 1 inch.

Distance from bottom form to centers of bars in bottom layer shall not be less than \( \frac{3}{4}D + A + \frac{3}{4} \) inch, nor less than 2D.

**Slabs:** The distance from bottom and top surface of concrete to center of bars shall not be less than \( \frac{3}{4}D + 1 \) inch.

68.06. **Splicing:** Splices of tension reinforcement at points of maximum stress shall be avoided. Bars to be spliced shall be securely wired together and unless otherwise shown on the plans shall have a lap of not less than fifty times the nominal diameter of the bar for plain bars or forty times the nominal diameter for deformed bars.

68.07. **Method of Measurement:** The weight of steel to be paid for shall be the theoretical weight of the steel placed as shown on the plans and accepted. The unit weight used for deformed bars shall be the weight of plain square or round bars, as the case may be, of equal nominal size.

The weight of the steel actually placed shall not vary more than two per cent from the calculated weights.

68.08. **Basis of Payment:** This item will be paid for at the contract unit price per pound for "Reinforcing Steel" described on plans, which price shall be full compensation for furnishing the material, equipment, tools, labor and incidentals necessary to complete the item. No allowance will be made for the clips, wire, separators, or other material used for fastening the reinforcing in place.

When there is no separate item for structural steel metal drains or expansion plates used in the construction of concrete bridges, these will be paid for at the same price bid for steel reinforcement unless otherwise provided.

Payment will be made under
   Item No. 71 (a)—Bar Reinforcing Steel (per pound).
   Item No. 71 (b)—Steel Mesh Reinforcing (per pound).
SECTION 69

STEEL STRUCTURES

69.01. Description: In these Specifications, "Steel Structures" embraces all construction composed of structural steel, together with the appurtenant metal details regardless of the composition of the latter.

Unless otherwise directed, these structures shall be constructed to the design, dimensions, lines, and grades, indicated on the plans, and in accordance with the methods and with the materials of the quality fully described in these Specifications.

MATERIALS

STRUCTURAL RIVET AND EYEBAR STEEL

69.02. General: All Structural, rivet and eyebar steel shall conform to the requirements of the Standard Specifications for structural steel for Bridges. Serial Designation A7-24, of the American Society for Testing Materials, with subsequent amendments and additions there-to adopted by the Society, and supplemented by the following paragraphs:

69.03. Character of Fracture: Test specimens of structural, rivet or eyebar steel shall show a fracture having a silky or fine granular structure throughout with a bluish gray or dove color, and shall be entirely free from granular, black and brilliant specks.

69.04. Defects in Material: Finished rolled material shall be free from cracks, flaws, injurious seams, laps, blisters, ragged and imperfect edges, and other defects. It shall have a smooth, uniform finish and shall be straightened in the mill before shipment.

Material shall be free from loose mill scale, rust pits, or other defects affecting its strength and durability.

69.05. Full Size Tests: When full-size tests of built up structural members and eyebars are required by the contract the Contractors shall supply testing machines of the proper type and capacity and shall provide all facilities and labor incidental to the making of tests. In all tests involving the determination of tensile and compressive strengths the ultimate strength, deformation and other pertinent data shall be recorded.

69.06. Payment for Full-Size Tests: Any full-size member tested to destruction shall be paid for by the Purchaser at the unit contract price, less its scrap value, if the test proves satisfactory. If the test proves the member to be unsatisfactory, the members represented by it will be rejected. The expense of conducting tests shall be borne by the Contractor unless otherwise provided.

EYEBARS

69.07. Full Size Tests: When tests of full-size bars are required the following conditions and requirements shall supplement the general provision of Paragraph 3.15.04.

69.08. Number and size of Test Bars: The number and size of the bars tested shall be designated by the Engineer before the mill order is placed. The number shall not exceed five (5) per cent of the whole number of bars ordered, with a minimum of two (2) bars on small orders.

69.09. Selection of Test Bars: The test bars shall be of the same section as the bars to be used in the structure and of the same length if within the capacity of the testing machine. They shall be selected by the Inspector from the finished bars, preferably after annealing. Test bars representing bars too long for the testing machine shall be selected from the full-length bar material after the heads on one end have been formed and shall have the second head formed upon them after being cut to the greatest length which can be tested.

69.10. Physical Requirements: Full-size tests of eyebars shall show a yield point of not less than twenty-nine thousand (29,000) pounds per square inch, an ultimate strength of not less than fifty-four thousand (54,000) pounds per square inch, and an elongation, including fracture, of not less than
ten (10) per cent in a length of twenty (20) feet measured in the body of the bar. The fracture shall show a uniform silky or fine granular structure throughout.

69.11. **Failure to Meet Requirements:** If a bar fails to fulfill the specified requirements, two additional bars of the same size and from the same melt shall be tested. If the failure of the first bar tested is on account of the character of the fracture only, the bars represented by the test may be re-annealed before the additional bars are tested. If two of the three test bars fail to give satisfactory results, the bars of that size and mill heat shall be rejected. A failure in the head of the bar shall not be cause for rejection if the other requirements are fulfilled. The bars shall be so selected that every melt of material entering into the various sizes of bars shall be represented by at least one test.

69.12. **Record of Annealing:** A failure in the head of a bar shall not be cause for rejection if the other requirements are fulfilled. A record of the annealing charges shall be furnished the Engineer showing the bars included in each charge and the treatment they received.

**STEEL FORGINGS**

69.13. **General:** Steel forgings from which pins, rollers, trunions, or other forged parts are to be fabricated, shall conform to the requirements of the Standard Specifications for Carbon-Steel Forgings for locomotives, Serial Designation A20-21, of the American Society for Testing Materials, with subsequent amendments and additions thereto adopted by the Society.

69.14. **Annealing:** All forgings shall be thoroughly annealed prior to being machined to form finished parts.

**WROUGHT-IRON**

69.15. Wrought-Iron shall conform to the requirements of the Standard Specifications for Refined Wrought-Iron Bars, Serial Designation A41-18, of the American Society for Testing Materials with subsequent amendments and additions thereto adopted by the Society.

**STEEL CASTINGS**

69.16. **General:** Steel castings shall conform to the requirements of the Standard Specifications for Steel Castings, Serial Designation A27-24, of the American Society for Testing Materials with subsequent amendments and additions thereto adopted by the Society, and supplemented by the following:

Unless otherwise specified all castings shall be Class B, Medium Grade.

69.17. **Annealing:** All steel castings shall be thoroughly annealed unless otherwise provided.

69.18. **Structural Defects:** Steel castings shall be true to pattern in form and dimensions, free from pouring faults, sponginess, cracks, blow holes and other defects in positions affecting their strength and value for the service intended. Blow holes appearing upon finished castings shall be so located that a straight line laid in any direction will not cut a total length of cavity greater than one (1) inch in any one (1) foot, nor shall any single blow hole exceed one inch in any dimension or have an area greater than one-half (½) square inch. Blow holes shall not have a depth injuriously affecting the strength of the casting. Minor defects which do not impair the strength may, with the approval of the Engineer, be welded by an approved process. The defects shall be removed to solid metal by chipping, drilling or other satisfactory methods and, after welding, the castings shall be annealed, if required by the Engineer. Castings which have been welded without the Engineer's permission shall be rejected. Large castings, if required by the Engineer, shall be suspended and hammered all over. No cracks, flaws or other defects shall appear after such treatment. No sharp unfiled angles or corners will be allowed.

**GRAY-IRON CASTINGS**

69.19. **General:** Iron castings shall conform to the requirements of the Standard Specifications for Gray-Iron Castings, Serial Designation A48-18, of the American Society for Testing Materials with subsequent amendments and additions thereto adopted by the Society. Castings shall be boldly filleted at angles and the arrises shall be sharp and perfect.
69.20. **Structural Defects**: Iron castings shall be true to pattern in form and dimensions, free from pouring faults, sponginess, cracks, blow holes and other defects in positions affecting their strength and value for the service intended.

**MALLEABLE CASTINGS**

69.21. **General**: Malleable castings shall conform to the requirements of the Standard Specifications for Malleable Castings, Serial Designation A47-24, of the American Society for Testing Materials, with subsequent amendments and additions thereto adopted by the Society. The castings shall be boldly filleted at angles and the arrises shall be sharp and perfect. The surfaces shall have a workmanlike finish.

69.22. **Structural Defects**: Malleable castings shall be true to pattern in form and dimensions, free from pouring faults, sponginess, cracks, blow holes and other defects in positions affecting their strength and value for the service intended.

**PHOSPHOR-BRONZE**

69.23. **General**: Phosphor-bronze shall conform to the requirements of the Standard Specifications for Bronze-Bearing Metals for Turn-tables and Moveable Railroad Bridges, Serial Designation B22-21-7-B of the American Society for Testing Materials, with subsequent amendments and additions thereto adopted by the Society. Grade B metal shall be used.

69.24. **Structural Defects**: Bronze castings shall be free from inclusions of foreign material, casting faults, injurious blow holes and other defects rendering them unsuitable for the service intended.

**PAINT**

69.25. **General**: Paint for structural steel work shall consist of an approved pigment mixed with an approved vehicle in accordance with the requirements hereinafter set forth.

1. Unless otherwise provided the materials entering into the composition of paints shall conform to the specifications of the Federal Specifications Board issued by the Bureau of Standards and in the examination of paints the methods specified herein shall be used.

2. All pigments, oils, thinners, driers, etc., shall be of the highest quality, free from adulterations, and in compliance with the requirements herein set forth. The various mixtures of pigment and vehicle shall be in the proportions herein enumerated for the particular paint specified.

3. All paints shall be shipped in strong, substantial containers plainly marked, with the weight, color, and volume, in gallons of the paint content and with the name and address of the manufacturer, together with a statement of the percentage composition of pigment and the proportions of pigment to vehicle. Any package or container not so marked will not be accepted for use under these specifications.

4. Where dry pigments are to be used, the dry pigment shall first be thoroughly and uniformly mixed by grinding with oil in a suitable grinding machine to reduce it to a paste consistency, after which additional oil, thinners and driers shall be added to produce the working consistency specified herein below. The oil shall be added a little at a time and well stirred in. In no case shall dry powdered pigments be mixed directly with the total vehicle. Where lamp black or other tinting pigments are used, they shall be used in paste form only. In no case shall dry lamp black or other powdered tinting pigments, be mixed directly with oil.

69.26. **Paint Paste**: Where paint pigments are purchased in paste form, the paste shall consist of the specified pigment or pigments ground to the required consistency in linseed oil. The paste so prepared shall be uniform in consistency and composition and shall not cake or segregate in the containers. When additional vehicle is added, the paint paste shall be such as to readily break up, to form a smooth uniform liquid of the proper brushing consistency and which will not run or sag.

**VEHICLE.**

69.27. **Definition**: The term vehicle shall be used to designate the mixture of linseed oil with the necessary amount of drier or thinner to produce an acceptable drying coefficient and a workable con-
consistency. The total amount of volatile vehicle shall in no case exceed ten (10%) of the total vehicle, nor shall
the vehicle contain in excess of one (1) per cent of water.

69.28. Raw Linseed Oil. The linseed oil shall conform to the requirements adopted by the Fed­
eral Specifications Board issued by the Bureau of Standards in Circular No. 82.

The requirements for Raw Linseed Oil are as follows:

<table>
<thead>
<tr>
<th>Property</th>
<th>Maximum</th>
<th>Minimum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loss on heating at 105 to 110°C. (per cent)</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td>Foots by volume (per cent)</td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td>Specific gravity 15.5°C.</td>
<td>0.936</td>
<td>0.932</td>
</tr>
<tr>
<td>Acid number</td>
<td>6.0</td>
<td></td>
</tr>
<tr>
<td>Saponification number</td>
<td>193.0</td>
<td>189.0</td>
</tr>
<tr>
<td>Unsaponifiable matter (per cent)</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>Iodine number (hanus) a.</td>
<td></td>
<td>170.0</td>
</tr>
<tr>
<td>Color</td>
<td>Not darker than a freshly prepared solution of 10G potassium bichro­mate in 100 cc pure strong (1.84 sq. gr.) sulphuric acid.</td>
<td></td>
</tr>
</tbody>
</table>

69.29. Boiled Linseed Oil: Boiled Oil shall be well settled linseed oil that has been boiled with
oxides of manganese and lead. It shall conform to the requirements adopted by the Federal Specifications
Board in Circular No. 82, which are as follows:

<table>
<thead>
<tr>
<th>Property</th>
<th>Maximum</th>
<th>Minimum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loss on heating at 105 to 110°C. (per cent)</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td>Specific gravity at 15.5°/15/5°C.</td>
<td>0.945</td>
<td>0.937</td>
</tr>
<tr>
<td>Acid number</td>
<td>8.0</td>
<td></td>
</tr>
<tr>
<td>Saponification number</td>
<td>195.0</td>
<td>189.0</td>
</tr>
<tr>
<td>Unsaponifiable matter (per cent)</td>
<td>1.50</td>
<td></td>
</tr>
<tr>
<td>Iodine number (Hanus)</td>
<td></td>
<td>168.0</td>
</tr>
<tr>
<td>Ash (per cent)</td>
<td>.7</td>
<td>.2</td>
</tr>
<tr>
<td>Manganese (per cent)</td>
<td></td>
<td>.03</td>
</tr>
<tr>
<td>Lead (per cent)</td>
<td></td>
<td>.1</td>
</tr>
<tr>
<td>Time of drying on glass (hours)</td>
<td>20.0</td>
<td></td>
</tr>
</tbody>
</table>

69.30. Vehicle for Aluminum Paint: The vehicle for aluminum paint shall be either an exterior
varnish or a bodied oil meeting the following requirements:

(a) Exterior Varnish Material: The vehicle for use with aluminum bronze powder may either
be a specially prepared spar mixing varnish or a bodied linseed oil. The spar mixing varnish for exterior
use should fulfill the following requirements:

The varnish shall contain not less than fifty (50) per cent by weight of non-volatile oils and gums,
and the ratio of oil to gum shall be not less than three (3) to one (1).

The varnish shall pass a sixty (60) per cent Kauri reduction test, as specified in Federal Board Stan­
dard Specification No. 18.

The varnish shall set to touch in not less than two (2) nor more than six (6) hours, and dry hard and
tough in not more than twenty-four (24) hours.

The varnish shall be of such consistency that when thoroughly mixed with aluminum bronze powder
in the proportion of two (2) pounds per gallon of vehicle, the paint shall show satisfactory spreading quali­
ties, and shall not run or sag when applied to a vertical surface.

(b) Bodied Oil Vehicle: The bodied linseed oil vehicle shall be a linseed oil bodied by heat alone,
known to the trade as a "kettle bodied" linseed oil or lithographic oil.
The paint shall be of such consistency that when diluted with a volatile thinner in the ratio of six (6) parts linseed oil to four (4) parts thinner and then mixed with two (2) pounds of aluminum bronze powder per gallon of vehicle, the paint shall show satisfactory spreading qualities and shall not run or sag when applied to a vertical surface.

It shall set to touch in not more than ten (10) hours and dry hard in not more than thirty (30) hours.

69.31. Mixing Aluminum Paint: Aluminum paint shall be mixed on the job and only enough for one day's use shall be mixed at one time. The paint shall be mixed in the proportion of two (2) pounds of aluminum bronze powder per gallon of vehicle. This makes a paint containing twenty-one (21) per cent pigment and seventy-nine (79) per cent vehicle. The weighed amount of powder shall be placed in a suitable mixing container and the measured volume of vehicle then poured over it. The powder shall be incorporated in the paint by vigorous stirring with a paddle. The powder will readily disperse in the vehicle. Before removing any paint from the mixing container, the paint shall be thoroughly stirred to insure a uniform mixture and the paint shall be suitably stirred during use.

THINNER

69.32. Turpentine: The turpentine used shall either be distillate commonly known as “Gum Turpentine” or “Spirits Turpentine” which is distilled from pine oleoresins or the product secured from resinous wood by extraction with volatile solvents, by steam or by destructive distillation, and shall meet the following requirements as given in Circular No. 86 of the Bureau of Standards.

The turpentine shall be clear and free from suspended matter and water.  
The color shall be “Standard” or better.  
The specific gravity shall be not less than 0.862 nor more than 0.875 at 13.5°C.  
The refractive index at 20°C. shall be not less than 1.463 nor more than 1.478.  
The initial boiling point shall not be less than 150° nor more than 160°C. at 760 mm. pressure.  
Ninety (90) per cent of the turpentine shall distill below 170°C. at 760 mm. pressure.  
The polymerization residue shall not exceed two (2) per cent and its refractive index at 20°C. shall not be less than 1.500.

69.33. Mineral Spirits: Mineral Spirits shall meet the following requirements as given in Circular No. 98 Bureau of Standards:

Appearance—Shall be clear and free from suspended matter and water.  
Color—Shall be “water white.”  
Spot Test—Shall evaporate completely from filter paper.  
Flash Point—Shall be not lower than 30°C. (86°F.) when tested in a closed cup tester.  
Sulphur—Shall be absent, as determined by the white-lead test.  
Distillate below 130°C. (266°F.) shall not exceed 5 per cent.  
Distillate below 230°C. (446°F.) shall not be less than 97%.  
Reaction—Shall be neutral.

DRIER

69.34. The drier shall meet the following requirements as given in Circular No. 105 of the Bureau of Standards.

The drier shall be composed of lead, manganese, or cobalt, or a mixture of any of these elements combined with a suitable fatty oil, with or without rosin or “gums” and mineral spirits or a mixture of these solvents. It shall be free from sediment and suspended matter. The drier when flowed on metal and baked for two (2) hours at 100°C. (212°F.) shall leave an elastic film. The flash point shall be not lower than 39°C. (88°F.) when tested in a closed-cup tester. It shall mix with pure raw linseed oil in the proportion of one (1) volume of drier to nineteen (19) volumes of oil without curdling, and the resulting mixture when flowed on glass shall dry in not more than eighteen (18) hours. When mixed with pure raw linseed oil in the proportion of 1 volume of drier to eight (8) of oil, the resulting mixture shall be no darker than a solution 6g. of potassium dichromate in 100cc. of pure sulphuric acid of specific gravity 1.84.
PAINT PIGMENTS

69.35. **Red Lead:** The pigment shall conform to the requirements of the Federal Specifications Board or the A. S. T. M. Standard Specifications for red lead (Serial Designation Ds3-24 with subsequent amendments and additions thereto) for the ninety-five (95) per cent grade.

69.36. **Graphite:** The pigment in both semipaste and ready mixed paint shall consist of finely ground graphitic carbon and insoluble siliceous material. The graphitic carbon may be derived from either natural or artificial graphite and the insoluble siliceous matter may be either the naturally occurring insoluble impurities of the graphite or added insoluble siliceous matter. The pigment shall show on analysis not less than fifty (50) per cent graphitic carbon and not less than thirty (30) per cent insoluble siliceous matter. The sum of the graphitic carbon and insoluble siliceous matter shall be not less than eighty-five (85) per cent. Nor more than five (5) per cent of calcium and magnesium carbonate and sulphates shall be present.

The pigment shall be ground so that it will all pass through a 200-mesh sieve.

69.37. **Carbon Black:**

(a) Carbon black shall be either gas carbon or lamp black. Its color and tinting strength shall be equal to a "standard" sample submitted and approved.

(b) Gas carbon shall be obtained by burning natural gas. It shall not contain more than a minute trace of grease or other benzine soluble impurities, nor more than 0.5% of ash.

(c) Lamp black shall be obtained by burning oils. It shall contain no more than a minute trace of grease or other benzine soluble impurities, nor more than 0.5% of ash.

69.38. **Basic Carbonate White Lead:** This pigment shall conform to the requirements of the Federal Specifications Board or the A. S. T. M. standard specifications for basic carbonate white lead (Serial Designation D81-24 with subsequent amendments and additions thereto.)

69.39. **Basic Sulphate White Lead:** This pigment shall conform to the requirements of the Federal Specifications Board or A. S. T. M. standard specifications for Basic Sulphate White Lead. (Serial Designation D81-24 with subsequent amendments and additions thereto.)

69.40. **Pure Zinc Oxide:** This pigment shall conform to the requirements of the Federal Specifications Board or the A. S. T. M. standard specifications for zinc oxide. (Serial Designation D79-24 with subsequent amendments and additions thereto.)

69.41. **Leaded Zinc Oxide:** This pigment shall conform to the requirements of the Federal Specifications Board or the A. S. T. M. standard specifications for leaded zinc oxide. (Serial Designation D80-24 with subsequent amendments and additions thereto.)

69.42. **Yellow Ochre:** All yellow ochre shall be pure French Ochre, and shall not contain less than nineteen (19) per cent of Iron Oxide nor more than five (5) per cent of Lime in any form.

69.43. **Asbestine:** Asbestine (Magnesium silicate) shall be finely ground, free from grit, adulterants or impurities other than silicate of iron, alumina, lime and manganese. It shall show under a microscope the characteristic crystalline structure of asbestine.

69.44. **Red Oxide of Iron:** Red Oxide of Iron shall contain not less than sixty-five (65) per cent of oxide of iron (Fe₂O₃) the remainder being silica and silicates. It may contain sulphur as calcium sulphate not to exceed 0.10%.

69.45. **Tinting Pigments:** Lamp Black, lead chromate, Prussian Blue, or Ochre may be employed as tinting pigments. Other pigments may be submitted to the Engineer and if approved may be used under these specifications, subject to such limitations as the Engineer may prescribe. All tinting pigments shall be added in paste form.

Lamp black paste shall be lamp black mixed with linseed oil in the following proportions:

- Dry lamp black: 20%
- Linseed oil: 80%
- Weight of one gallon of paste: 9 1/4 lbs.
Lead Chromate and Prussian Blue (or a mixture of the two—Chrome Green) shall be of the highest commercial quality, free from adulterants or any impurities in excess of two (2) per cent. The total amount of water soluble material shall not exceed one-half (½) of one per cent.

Ochre Paste shall conform to the requirements of the Federal Specifications Board or the A. S. T. M. standard specifications for Ochre (Serial Designation D85-24 with subsequent additions and amendments thereto.)

Inert pigments, where used, shall be silica, magnesium silicate, aluminum silicate, barium sulphate, pure tinting colors or any mixture thereof. Inerts shall in no case contain organic coloring matter, soap or emulsifying products.

No pigment containing coal tar or asphaltic products shall be used.

69.46. **Aluminum, Powdered:** Powdered Aluminum shall be made by the best commercial methods from metallic aluminum having a minimum aluminum content of ninety-nine (99) per cent for iron, silicon, and copper. The total weight of these impurities shall not exceed one (1) per cent of the weight of the sample after deduction of the acetone soluble portion.

(a) The powder shall conform to the following composition limits:

- Acetone extract (2 hrs.) maximum: 3.0%
- Lead and zinc maximum: 0.06%
- Copper maximum: 0.2%

(b) Powdered Aluminum shall be powdered in the forms of flakes and shall be polished. It shall possess the property of "leafing," when suspended in varnish or airplane dope.

(c) The fineness shall be such that it will pass through a standard 100 mesh screen.

(d) Powdered Aluminum shall contain no adulterants such as powdered mica.

**PAINT FORMULAS**

69.47. **Shop Coat:** For the first or priming coat or for any maintenance coat placed on bare metal the following paint shall be used:

<table>
<thead>
<tr>
<th>Ingredient</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Red lead paste (95% grade)</td>
<td>33-1/3 lbs.</td>
</tr>
<tr>
<td>Linseed oil</td>
<td>1.0 gal.</td>
</tr>
<tr>
<td>Total volume of paint by above quantities approximately</td>
<td>1.75 gals.</td>
</tr>
<tr>
<td>Weight of resulting paint per gallon (before addition of any thinner or drier) approximately</td>
<td>24.0 lbs.</td>
</tr>
</tbody>
</table>

To each gallon of the paint prepared as above, may be added such amounts of thinner or drier as are necessary to secure work-ability and a good drying coefficient, but in no case shall the amounts so added be such as to cause a separation of pigment and vehicle, nor shall they exceed the following:

Maximum amount of drier per gallon of paint, 1/3 pint.

Maximum amount of thinner per gallon of paint, 1/3 pint.

For ordinary bridge work, the linseed oil used shall consist of a mixture of raw and boiled oils in the proportion of from 1/3 to ½ boiled oil, the balance being raw oil.

For work exposed to water action, boiled linseed oil shall be used and the amount of drier used shall be not more than 50% of the value specified above.

The above formula percentages are in terms of the paste pigment which contains about seven (7) per cent of oil. Reduced to equivalent dry pigment percentages, the formula is as follows:

<table>
<thead>
<tr>
<th>Ingredient</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Red Lead (95% grade)</td>
<td>24.0 lbs.</td>
</tr>
<tr>
<td>Linseed Oil</td>
<td>1.0 gal.</td>
</tr>
<tr>
<td>Total gallons of paint produced from above quantities approximately</td>
<td>1.33 gals.</td>
</tr>
</tbody>
</table>
69.48. **First Field Coat:** First Field Coat after all abrasions of the shop coat have been repaired shall be the same as the shop coat with approved Carbon Black added as may be needed for tinting.

- **Red Lead Paste** (95% grade) .......................................................................................... 33-1/3 lbs.
- **Approved Carbon Black Paste** (as noted for tinting) from ......................................... 1/8 to 1/4 lbs.
- **Linseed Oil** ..................................................................................................................... 1.0 gal.
- **Weight per gallon of paint (without drier or thinner) approximately** ......................... 24.0 lbs.

**Raw Linseed Oil** or a mixture of raw and boiled oils containing not to exceed twenty-five (25) per cent of boiled oil, shall be used as a vehicle, except for work exposed to water action, in which case boiled oil shall be used as noted herein above under shop coat.

69.49. **Second Field Coat:**

(a) **Black Graphite Paint:**

- **Graphite Pigment** ........................................................................................................... 5.25 lbs.
- **Linseed Oil Vehicle** ....................................................................................................... 1.0 gals.
- **Total volume of paint produced approximately** .............................................................. 1.24 gals.
- **Weight of paint per gallon before addition of any thinner or drier approximately** ....... 10.5 lbs.

A paint conforming to the following requirements with or without tinting may be used for the second field coat.

This paint may be tinted to an olive green or dark green by adding Chrome Yellow or Chrome Green until the desired shade of color is obtained.

(b) **Alternate Second Field Coat:**

- **Carbon Black** ................................................................................................................ 3/4 lb.
- **Lead Carbonate** ............................................................................................................. 3/4 lb.
- **Linseed Oil** ..................................................................................................................... 1.0 gal.
- **Total volume of paint produced approximately** .............................................................. 1.06 gals.
- **Weight of paint per gallon before addition of any thinner or drier approximately** ....... 8.7 lbs.

(c) **Alternate White and Grey Coats:**

This paint may be used for either first or second field coat or both if tinted so that the coats applied can be distinguished.

(1) **White Lead Carbonate:**

- **Basic Lead Carbonate Paste** ......................................................................................... 19.0 lbs.
- **Linseed Oil** ..................................................................................................................... 1.0 gal.
- **Total volume of paint produced approximately** .............................................................. 1.534 gals.
- **Weight of paint per gallon before addition of any thinner or drier approximately** ...... 17.44 lbs.

**Note:** Basic Sulphate White Lead may be used in place of the Carbonate if approved by the Engineer.

(d) **Alternate White Lead Paint:**

(2) **White Lead and Zinc Oxide:**

- **Basic Lead Carbonate Paste** ......................................................................................... 12.0 lbs.
- **Zinc Oxide Paste** ............................................................................................................ 83 4/4 lbs.
- **Linseed Oil** ..................................................................................................................... 1.0 gal.
- **Total volume of paint produced approximately** .............................................................. 1.68 gals.
- **Weight of paint per gallon before addition of any thinner or drier approximately** ...... 17.78 lbs.

**Note:** The above white paints may be tinted to produce a grey by the addition of lamp black paste or carbon black, if desired by the Engineer.

21
69.50. **Maintenance Paint:**

<table>
<thead>
<tr>
<th>Pigment</th>
<th>Weight (lbs)</th>
<th>Pigment (lbs)</th>
<th>Linseed Oil (lbs)</th>
<th>Volume (gallons)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Red Lead Paste (95% grade)</td>
<td>17.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Graphite</td>
<td>2.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Zinc Oxide</td>
<td>1.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Linseed Oil</td>
<td>1.0</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Total volume of paint produced by the above quantities approximately 1.49 gals.

Weight of resulting paint per gallon (before addition of any thinner or drier) approximately 1.86 lbs.

To the above formulas may be added such amount of thinner and drier as is necessary to secure workability and a suitable drying coefficient but in no case to exceed the requirements specified for shop coat.

The formulas above are based on using the pigments in the paste which is assumed to be made up as follows:

**PASTE PROPORTIONS—WEIGHTS AND VOLUME**

<table>
<thead>
<tr>
<th>Pigment</th>
<th>Weight (lbs)</th>
<th>Pigment (lbs)</th>
<th>Linseed Oil (lbs)</th>
<th>Volume (gallons)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Red Lead, 95% grade</td>
<td>100</td>
<td>93</td>
<td>7</td>
<td>2.16</td>
</tr>
<tr>
<td>White Lead Carbonate</td>
<td>100</td>
<td>92</td>
<td>8</td>
<td>2.75</td>
</tr>
<tr>
<td>White Lead Sulphate</td>
<td>100</td>
<td>90</td>
<td>10</td>
<td>2.80</td>
</tr>
<tr>
<td>Zinc Oxide</td>
<td>100</td>
<td>84</td>
<td>16</td>
<td>3.75</td>
</tr>
<tr>
<td>Graphite</td>
<td>100</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Carbon Black (drop)</td>
<td>100</td>
<td>50</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>Lamp Black</td>
<td>100</td>
<td>20</td>
<td>80</td>
<td></td>
</tr>
<tr>
<td>Chrome Green</td>
<td>100</td>
<td>70</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>Chrome Yellow</td>
<td>100</td>
<td>70</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>Ochre</td>
<td>100</td>
<td>70</td>
<td>30</td>
<td></td>
</tr>
</tbody>
</table>

**Note:** One gallon of linseed oil weighs 7.76 pounds.

**TESTS AND ACCEPTANCE**

69.51. **Manufacturer's Guarantee:** The manufacturer of each brand of paint submitted for acceptance under these specifications, or any contractor desiring to use any particular paint for work to be done under these specifications, shall file with the State a certificate of analysis and manufacturer's guarantee, setting forth the trade name or brand of the paint to be furnished together with a facsimile copy of the label (if the material is of the ready mixed type) and a typical analysis showing the percentage of each of the chemical elements in the pigment and vehicle. The manufacturer's guarantee shall provide that all paint furnished under these specifications shall conform to the certified analysis as filed and to the statement of the various percentage of the ingredients on the receptacle or container. The manufacturer's guarantee shall be of the form furnished by the engineer and shall be sworn to by a person having legal authority to bind the manufacturing company by his acts.

69.52. **Sampling and Testing:** At least thirty (30) days before shop painting starts, the Contractor shall submit for examination and test samples of the particular brand of paint which he proposes to use.

Field paint shall also be submitted for examination in sufficient time to allow at least thirty (30) days for test. No paint shall be used until it has been accepted for use, in writing by the engineer.

In sampling paint for test, the engineer shall take at least one (1) quart sample from every five (5) barrels in the consignment. These samples may be tested individually or as a composite representing not more than twenty-five (25) barrels.

From time to time during the progress of the work, samples of paint used on the work will be taken and subjected to laboratory tests, as may be deemed advisable. A material difference in composition or working quality of these samples, as compared with samples originally furnished by the contractor, or as compared with the manufacturer's guarantee analysis may be considered sufficient reason for can-
cellation of the contract or for the suspension of any further payment for work done with the defective materials.

69.53. Inspection: The Contractor or manufacturer, shall allow the engineer, or his inspector, free access to all parts of his shops while work on these paints is being carried out; and shall give him every reasonable facility to enable him to insure that the paints are being made in accordance with this specification.

CONSTRUCTION METHODS

69.54. Storage of Materials: Structural material delivered at the bridge shop receiving yard shall be stored above the surface of the ground upon platforms, skids or other supports and shall be protected as far as practicably from surface deterioration by exposure to conditions producing rust. It shall be kept free from accumulations of dirt, oil or other foreign matter.

Fabricated material stored prior to shipment shall be subject to the same conditions of storage as the unfabricated material in the shop receiving yard.

69.55. Straightening Material: All deformed structural material shall be properly straightened by methods which are non-injurious prior to being laid off, punched or otherwise worked in the shop. Sharp kinks and bends shall be cause for rejection.

69.56. Workmanship and Finish: The workmanship and finish shall be first class and equal to the best practice in modern bridge shops. Shearing and chipping shall be neatly and accurately done and all portions of the work exposed to view shall be neatly finished.

69.57. Changes and Substitutions: No changes shall be made in any drawing after it has been approved except by the consent or direction of the Engineer in writing.

Substitutions of sections having different dimensions than those shown on the plans shall be made only when approved in writing by the Engineer.

69.58. Rivet Holes: When general reaming is not required, holes in material three-quarters (3/4) inch or less in thickness may be punched full-size. Holes in material more than three-quarters (3/4) inch in thickness shall be sub-punched and reamed, or drilled from the solid.

69.59. Punched Holes: Full-size punched holes shall be one-sixteenth (1/16) inch larger than the nominal diameter of the rivet. The diameter of the die shall not exceed the diameter of the punch by more than three thirty-seconds (3/32) inch. Holes must be clean cut, without torn or ragged edges. If any holes must be enlarged to admit the rivets, they shall be reamed.

69.60. Accuracy of Punched Holes: The punching of holes shall be so accurately done that, after assembling the component parts of a member, a cylindrical pin one-eighth (1/8) inch smaller than the nominal diameter of the punched hole may be passed through at least seventy-five (75) of any group of one hundred (100) contiguous holes in the same surface or in like proportion for any group of holes. If this requirement is not fulfilled the badly punched pieces shall be rejected. If any holes will not pass a pin three-sixteenth (3/16) inch smaller than the nominal diameter of the punched hole, this shall be cause for rejection.

69.61. Drilled Holes: Drilled holes shall be one-sixteenth (1/16) inch larger than the nominal diameter of the rivet. Burrs on the outside surfaces shall be removed with a tool producing a one-sixteenth (1/16) inch fillet around the edge of the hole.

69.62. Sub-Punched and Reamed Holes: Sub-punched and reamed holes for rivets having diameters greater than three-quarters (3/4) inch shall be punched three-sixteenth (3/16) inch smaller than the nominal diameter of the rivet, and for rivets having diameters three-quarters (3/4) inch or less the holes shall be punched one-sixteenth (1/16) inch less than the nominal diameter of the rivet. The punch and die shall have the same relative sizes as specified for full-size punched holes. After punching, the holes shall be reamed to a diameter one-sixteenth (1/16) inch larger than the nominal diameter of the rivet. Burrs resulting from reaming shall be removed with a tool producing a one-sixteenth (1/16) inch fillet around the edge of the hole.
Reaming of rivet holes shall be done with twist drills or with short taper reamers. Reamers preferably shall not be directed by hand. No oil or grease shall be used as a lubricant.

69.63. **Accuracy of Reamed and Drilled Holes:** Reamed or drilled holes shall be cylindrical and perpendicular to the member and their accuracy shall be the same as specified for punched holes except that, after reaming or drilling, eighty-five (85) of any group of one hundred (100) contiguous holes in the same surface, or in like proportion for any group of holes, shall not show an offset greater than one-thirty-second (1/32) inch between adjacent thicknesses of metal.

69.64. **Drifting of Holes:** The drifting done during assembling shall be only such as to bring the parts into position, and not sufficient to enlarge the holes or distort the metal.

69.65. **General Reaming:** General reaming may be required, in which case a definite provision to this effect shall be included elsewhere in the contract.

When general reaming is required, all rivet holes in main members shall be sub-punched and reamed or drilled from the solid. This requirement shall not apply to rivet holes in top and bottom chord lateral members, lateral hangers, truss and girder sway bracings, and to the lateral plates, connection angles, etc., connecting these members to the main members of the structure. Connection plates or other parts acting both as main member material and secondary (lateral, sway bracing, etc.) member material shall generally have sub-punched and reamed holes in locations engaging similar holes in main members.

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.

69.66. **Reaming of Field Connections:** When general reaming is required, or in punched work when specifically required by the Engineer, holes for field connections, except those in lateral, longitudinal and sway bracing, shall be reamed or drilled. Riveted trusses shall be assembled in the shop, the parts adjusted to line and fit, and the holes for field connections reamed or drilled while so assembled. Holes for other field connections shall be reamed or drilled with the connecting parts assembled, or else reamed or drilled to a metal templet not less than one (1) inch thick.

69.67. **Shop Assembling:** All surfaces of metal to be in contact when assembled shall be carefully painted with one coat of the paint specified for the shop coat. The paint shall be applied upon surfaces free from dirt, loose mill scale or other foreign matter and the parts shall be assembled while the paint is plastic.

The component parts of a built member shall be assembled, drift-pinned to prevent lateral movement, and firmly bolted to draw the parts into close contact before reaming, drilling or riveting is begun. Assembled parts shall be taken apart, if necessary, for the removal of burrs and shavings produced by the remaining operation.

The member shall be free from twists, bends or other deformations.

Preparatory to shop riveting full-size punched material, the rivet holes shall be cleared for the admission of the rivets by reaming.

End connection angles, stiffener angles, etc., shall be carefully adjusted to correct locations and rigidly bolted, clamped or otherwise firmly held in place until riveted.

69.68. **Match-Marking:** Connecting parts assembled in the shop for the purpose of reaming or drilling holes in field connections shall be match-marked, and a diagram showing such marks shall be furnished to the Engineer.

69.69. **Rivets:** The diameter of rivets indicated upon the plans shall be understood to mean their diameter before heating.

Heads of driven rivets shall be of approved shape, concentric with the shanks, true to size, full, neatly formed, free from fins and in full contact with the surface of the member.

69.70. **Field Rivets:** Field rivets, for each size and length, shall be supplied in excess of the actual number to be driven to provide for losses due to misuse, improper driving or other contingencies. Rivets shall be free from furnace scale on their shanks and from fins on the under side of the machine formed heads.
69.71. **Bolts and Bolted Connections:** Bolted connections shall not be used unless specifically authorized. Where bolted connections are permitted the bolts furnished shall be unfinished bolts (ordinary rough or machine bolts) or turned bolts, as specified or directed by the Engineer.

**Unfinished Bolts:** Unfinished bolts shall be standard bolts with hexagonal heads and nuts. The use of "button head" bolts will not be permitted. Bolts transmitting shear shall be threaded to such length that not more than one thread will be within the grip of the metal. The bolts shall be of lengths which will extend entirely through their nuts but not more than one-quarter (¼) inch beyond them. The diameter of the bolt holes shall be one-sixteenth (1/16) inch greater than the diameter of the bolts used.

**Turned Bolts:** Holes for turned bolts shall be carefully reamed or drilled and the bolts turned to a driving fit by being given a finishing cut. The threads shall be entirely outside of the holes and the heads and nuts shall be hexagonal. Approved nut-locks shall be used on all bolts unless permission to the contrary is secured from the Engineer. When nut-locks are not used, round washers having a thickness of one-eighth (1/8) inch shall be placed under the nuts.

69.72. **Riveting:** Rivets shall be heated uniformly to a light cherry red color and shall be driven while hot. The heating of the points of rivets more than the remainder will be permitted. When ready for driving they shall be free from slag, scale and other adhering matter and when driven they shall completely fill the holes. Burned, buried or otherwise defective rivets, or rivets which throw off sparks when taken from the furnace or forge shall not be driven.

Loose, burned, badly formed or otherwise defective rivets shall be cut out. Caulking and re-cupping of rivet heads will not be allowed. In cutting out defective rivets care shall be taken not to injure the adjacent metal and, if necessary, the rivet shanks shall be removed by drilling.

Countersinking shall be neatly done and counter-sunk rivets shall completely fill the holes.
Shop rivets shall be driven by direct-acting riveters where practicable. The riveting machine shall retain the pressure for a short time after the upsetting is complete.

Pneumatic hammers shall be used for field riveting except when the use of other hand tools for riveting is permitted by the Engineer.

69.73. **Edge Planing:** Sheared edges of material more than five-eighths (5/8) inch in thickness shall, when required by the Engineer, be planed to a depth of not less than one-eighth (1/8) inch. Reentrant cuts shall be filleted before cutting.

69.74. **Planing of Bearing Surface:** Ends of columns taking bearing upon base and cap plates shall be milled to true surfaces and correct bevels after the main section of these members and the end connection angles have been fully riveted.

Caps and base plates of columns and the sole plates of birders and trusses shall have full contact when assembled. The plates, if warped or deformed, shall be hot-straightened, planed or otherwise treated to secure an accurate, uniform contact. After being riveted in place the excess metal of countersunk rivet heads shall be chipped smooth and flush with the surrounding metal and the surfaces which are to come in contact with other metal surfaces shall be planed or milled, if necessary, to secure proper contact. Correspondingly, the surfaces of base and sole plates which are to come in contact with masonry shall be rough finished, if not free from warps or other deformations.

Surfaces of cast pedestals and shoes which are to come in contact with metal surfaces shall be planed and those which are to take bearing upon the masonry shall be rough finished.

In planing the surfaces of expansion bearings the cut of the tool shall be in the direction of expansion. Surfaces of bronze bearing plates intended for sliding contact, shall be carefully milled and polish finished.

69.75. **Abutting Joints:** Abutting ends of compression members shall, after the members have been riveted, be accurately faced to secure an even bearing when assembled in the structure.

Ends of tension members at splices shall be rough finished to secure close and neat but not contact fitting joints.
69.76. **End Connection Angles:** End connection angles or floor beams and stringers shall be flush with each other and accurately set as to position and length of member. In general, and connection angles shall not be finished unless required by the terms of the contract. However, faulty assembling and riveting may be cause for requiring them to be milled, in which case their thickness shall be reduced not to exceed one-sixteenth (1/16) inch, nor shall their rivet bearing value be reduced below design requirements.

69.77. **Built Members:** The several pieces forming one built member shall be straight and close fitting. Such members shall be true to detailed dimensions and free from twists, bends, open joints or other defects resulting from faulty fabrication and workmanship.

69.78. **Lacing Bars:** The ends of lacing bars shall be neatly rounded unless otherwise indicated.

69.79. **Plate Girders—Web Plates:** Web plates of girders having no cover plates may be detailed with the top edge of the web flush with the backs of the flange angles. Any portion of the plate projecting beyond the angles shall be chipped flush with the backs of angles. Web plates of girders having cover plates may be one-half (1/2) inch less in width than the distance back to back of flange angles.

When web plates are spliced, not more than three-eighths (3/8) inch clearance between ends of plates will be allowed.

**Web Stiffeners:** End stiffener angles of girders and stiffner angles intended as supports for concentrated loads shall be milled or ground to secure a uniform, even bearing against the flange angles. Intermediate stiffener angles shall fit sufficiently tight to exclude water after being painted.

**Web Splices and Fillers:** Web splice plates and fillers under stiffeners shall fit within one-eighth (1/8) inch at each end.

69.80. **Eye-Bars:** Eye-bars shall be straight and true to size, and shall be free from twists, folds in the neck or head, or any other defect affecting their service strength. Heads shall be made by upsetting, rolling or forging. Welds in the body portions or in the heads of bars will not be permitted. The form of the heads may be determined by the dies in use at the works where the eye-bars are to be made, if satisfactory to the Engineer. The thickness of head and neck shall not overrun more than one-sixteenth (1/16) inch.

**Boring:** Before boring, each eye-bar shall be properly annealed and carefully straightened. Pin holes shall be located on the center line of the bar and in the centers of the heads. The holes in the ends of bars shall be bored simultaneously and shall be so accurately located that when the bars of the same truss panels are placed in a pile the pins may be completely inserted in the pin-holes without driving. All eye-bars intended for the same locations in the trusses shall be interchangeable.

69.81. **Annealing:** All eye-bars shall be annealed by heating uniformly to the proper temperature followed by slow and uniform cooling in the furnace. The temperature of the bars shall be under full control at all stages.

Forged pins, and other steel parts requiring their full strength, which have been partially heated shall be subsequently annealed. Slight bends in pieces of secondary importance may be made without heating the metal. Crimped web stiffeners need not be annealed.

69.82. **Pins and Rollers:** Pins and rollers shall be accurately turned to detailed dimensions and shall be smooth, straight and free from flaws. The final surface shall be produced by a finishing cut.

**Forged Pins:** Pins having a diameter greater than six (6) inches shall be forged and annealed.

**Bored Pins:** Pins larger than eight (8) inches in diameter shall have a hole not less than two (2) inches in diameter bored longitudinally through their centers. Pins showing defective interior conditions shall be rejected.

69.83. **Boring Pin Holes:** Pin holes shall be bored true to detailed dimensions, smooth and straight, at right angles with the axis of the member and parallel with each other unless otherwise required. A finished cut shall always be made.

The length outside to outside of holes in tension members and inside to inside of holes in compression members shall not vary from detailed dimensions more than one-thirty-second (1/32) inch. Boring of holes in built up members shall be done after the riveting is completed.
69.84. **Pin Clearances**: The difference in diameter between the pin and the pin hole shall be not more than one-thirty-second (1/32) inch.

69.85. **Welds**: Welding of steel shall not be permitted except to remedy minor defects and then only with the approval of the Engineer.

69.86. **Screw Threads**: Screw threads shall make close fits in the nuts and shall be U. S. Standard, except that for diameters greater than one and one-half (1½) inch, they shall be made with six (6) threads to the inch.

69.87. **Pilot and Driving Nuts**: Two pilot nuts and two driving nuts shall be furnished for each size of pin, unless otherwise specified.

**MILL AND SHOP INSPECTION**

69.88. **Notice of Rolling and Fabrication**: The Contractor shall give ample notice to the Engineer of the beginning of work at the mill and shop, so that inspection may be provided. No material shall be rolled or fabricated before the Engineer has been notified where the orders have been placed.

69.89. **Facilities for Inspection**: The Contractor shall furnish all facilities for the inspection of material and workmanship in the mill and shop and Inspectors shall be allowed free access to the necessary parts of the premises.

69.90. **Inspector’s Authority**: The Inspector shall have the power to reject materials or workmanship which do not fulfill the requirements of these specifications; but in cases of dispute the Contractor may appeal to the Engineer, whose decision shall be final.

Inspection at the mill and shop is intended as a means of facilitating the work and avoiding errors, and it is expressly understood that it will not relieve the Contractor from any responsibility in regard to imperfect material or workmanship and the necessity for replacing the same.

69.91. **Mill Orders and Shipping Statements**: The Contractor shall furnish the Engineer with as many copies of mill orders and shipping statements as the Engineer may direct. The weights of the individual members shall be shown.

69.92. **Cost of Testing**: Unless otherwise provided, the Contractor shall furnish without charge, test specimens as specified herein, and all labor, testing machines and tools necessary to prepare the specimens and to make the full-size tests.

69.93. **Rejections**: The acceptance of any material or finished members by the Inspector shall not be a bar to their subsequent rejection, if found defective. Rejected material and workmanship shall be replaced promptly or made good by the Contractor.

69.94. **Marking and Shipping**: Members weighing more than three (3) tons shall have the weight marked thereon. Bolts and rivets of one length and diameter and loose nuts or washers of each size, shall be packed separately. Pins, small parts, and small packages of bolts, rivets, washers and nuts shall be shipped in boxes, crates, kgs or barrels, but the gross weight of any package shall not exceed three-hundred (300) pounds. A list and description of the contained material shall be plainly marked on the outside of each shipping container.

The weight of all tools and erection material shall be kept separate.

Anchor-bolts, washers, and other anchorage or grillage materials, shall be shipped to suit the requirements of the masonry construction.

**Loading and Unloading**: The loading, transportation, unloading and piling of structural material shall be so conducted that the metal will be kept clean and free from injury by rough handling.

**ERECTION**

69.95. **Field Inspection**: All work of erection shall be subject to the inspection of the Engineer who shall be given all facilities required for a thorough inspection of workmanship.
Material and workmanship not previously inspected will be inspected after its delivery to the site of the work.

69.96. Storage: All material shall be stored in such manner as to prevent deterioration by rust or loss of minor parts. No material shall be piled so as to rest upon the ground or in water but must be placed on suitable skids or platforms.

69.97. Preparation of Bearing Area: Column bases, truss and girder pedestals and shoes shall have a full and uniform bearing upon the substructure masonry. Masonry bearing plates shall not be placed upon the bridge seat areas of piers or abutments which are improperly finished, deformed or irregular.

The shoes and pedestals of truss and girder spans, the bases of columns, and the center and end bearings of swing spans shall be rigidly and permanently located to correct alignments and elevations. Unless otherwise provided they shall be placed on a layer of canvas and red lead applied as follows:

Thoroughly swab the top surface of the bridge seat bearing area with red lead paint and place upon it three layers of twelve (12) ounces to fourteen (14) ounces duck, each layer being thoroughly swabbed on its top surface with red lead paint. Place in position the superstructure shoes or pedestals while the paint is plastic.

69.98. Handling Members: The field assembling of the component parts of a structure shall involve the use of methods and appliances not likely to produce injury by twisting, bending or otherwise deforming the metal. No member slightly bent or twisted shall be put in place until its defects are corrected, and members seriously damaged in handling shall be rejected.

69.99. Alignment: Before beginning the field riveting the structure shall be adjusted to correct grade and alignment and the elevation of panel points (ends of floorbeams) properly regulated. For truss spans a slight excess camber will be permitted while the bottom chords are being riveted, but the correct camber and relative elevations of panel points shall be secured before riveting the top chord joints, top lateral system and sway bracing.

69.100. Straightening Bent Material: The straightening of bent edges of plates, angles and other shapes shall be done by methods not likely to produce fracture or other injury. The metal shall not be heated unless permitted by the Engineer, in which case the heating shall not be to a higher temperature than that producing a dark cherry red color. After heating, the metal shall be cooled as slowly as possible.

Following the completion of the straightening of a bend or buckle, the surface of the metal shall be carefully inspected for evidence of incipient or other fractures.

69.101. Assembling and Riveting: All field connections and splices shall be securely drifted, pinned and bolted before riveting. Important connections in trusses, girders, floor system, etc., shall at least have fifty (50) per cent of the holes filled. An ample number of drift pins shall be used to prevent slipping at joints and splices.

The results obtained in the field assembling and riveting of the members of a structure shall conform to the requirements for shop assembling and riveting. Field driven rivets shall be inspected and accepted before being painted.

Field riveting of tension chord members shall be done before the false work is removed; but compression chord members shall not be riveted until the span is released sufficiently from the false work to bring the compression chord joints into full bearing.

Railings shall not be riveted until the false work is removed.

69.102. Adjustment of Pin Nuts: All nuts on pins shall be thoroughly tightened and the pins so located in the holes that the members shall take full and even bearing upon them.

69.103. Setting Anchor Bolts: Anchor bolt holes shall be drilled in correct locations vertically to the plane of the bridge seat, and the anchor bolts set in Portland cement mortar therein. The mortar shall consist of one part cement to one part clean, fine grained sand mixed sufficiently wet to flow freely.

Anchor bolts shall be first dropped into the dry holes to assure their proper fit after setting. They shall then be set as follows:
Fill the hole about two-thirds full of mortar and by a uniform, even pressure or by light blows with a hammer, (flogging and ramming will not be permitted) force the bolt down until the mortar rises to the top of the hole and the anchor bolt nut rests firmly against the metal shoe or pedestal. Remove all excess mortar which may have flushed out of the hole, to permit proper field painting of the metal surfaces.

The location of the anchor bolts in relation to the slotted holes in expansion shoes shall be varied with the prevailing temperature. The nuts on anchor bolts at the expansion ends of spans shall permit the free movement of the span.

Anchor bolts which are to be set in the masonry prior to the erection of the superstructure shall be carefully set to proper location and elevation with templets or by other suitable means.

PAINTING

69.104. General Conditions: The painting of metal structures shall include, unless otherwise provided in the contract, the proper preparation of the metal surfaces, the application, protection and curing of the paint coatings, the protection of pedestrians, vehicular or other traffic upon or underneath the bridge structure, the protection of all portions of the structure (superstructure and substructure) against disfigurement by spatters, splashers and smirches of paint or of paint materials, and the supplying of all tools, tackle, scaffolding, labor, workmanship and materials necessary for the entire work. Materials to conform to the requirements set forth in Paragraphs 69.25 to 69.50.

APPLICATION OF PAINT

69.105. Number of Coats. All new structural steel work shall, unless otherwise especially provided upon the plans or in the contract, be painted three coats of paint. The first coat is to be applied immediately after the shop fabrication is complete except that all surfaces coming into contact are to be painted before being assembled. The second and third coats are to be applied after all erection is complete, except that immediately following the field riveting of the members, the heads of filed rivets, and all abrasions of the shop coat due to handling at the shop, shipment, erection, etc., and all field erection marks shall be thoroughly covered with one coat of shop paint and permitted to become thoroughly dry before the first field coat is applied.

69.106. Colors of Coats: The color of each succeeding coat shall be sufficiently different from that previously applied to readily permit the discovery of an incomplete application of the paint coat. The colors of the coats shall be determined by the Engineer.

69.107. Weather Conditions: Paint shall be applied only when the air temperature is at or above forty degrees Fahrenheit (40 degrees F.). It shall not be applied upon damp surfaces or upon metal containing frost, nor shall it be applied when the air is misty, or otherwise, in the opinion of the Engineer, unsatisfactory for the work.

Material painted under cover in damp or cold weather shall remain under cover until dry or until weather conditions permit its exposure in the open. Painting in open yards or upon erected structures shall not be done when the metal has absorbed sufficient heat to cause the paint to blister and produce a porous paint film.

69.108. Application: No wide, flat brushes shall be used. All brushes shall be either round or oval in shape.

The paint when applied shall be so manipulated under the brush as to produce a uniform even coating in close contact with the metal or with previously applied paint. In general, the primary movement of the brush shall describe a series of small circles to thoroughly fill all irregularities in the surface, after which the coating shall be smoothed and thinned by a series of parallel strokes.

To secure a maximum thickness of paint film upon rivet heads, and edges of plates, angles or other rolled shapes these areas shall be "striped" in advance of the general painting, and shortly afterward shall be given a second or "wash" coat when the general coat is applied. The paint shall be well worked into all joints and open spaces.

Paint shall be thoroughly stirred preferably by means of mechanical mixers before being removed from the containers, and to keep the pigments in suspension shall be kept stirred while being applied.
All painting must be done in a neat and workmanlike manner.

On all surfaces which are inaccessible for paint brushes, the paint shall be applied with sheepskin daubers specially constructed for the purpose.

69.109. **Removal of Improper Paint**: All metal coated with impure or unauthorized paint shall be thoroughly cleaned and repainted to the satisfaction of the Engineer, at the expense of the Contractor.

69.110. **Thinning**: If it is necessary in cool weather to thin the paint in order that it shall spread more freely, this shall be done only by heating in hot water or on steam radiators.

**SHOP PAINTING STEEL STRUCTURES**

69.111. **Shop Cleaning**: All surfaces of metal to be painted shall be thoroughly cleaned of rust, loose mill scale, dirt, oil or grease, and all other foreign substances. The removal of rust, scale and dirt shall generally be done by the use of metal brushes, scrapers, chisels, hammers or other effective means. Oil and grease may be removed by the use of gasoline or benzine. Bristle or wood fibre brushes shall be used for removing loose dust.

69.112. **Shop Painting**: Surfaces to be riveted in contact either in the shop or field shall not be painted. Surfaces not in contact, but which will be inaccessible after assembly or erection, shall be painted two coats.

When all fabrication work is complete and has been accepted as such, all surfaces not painted before assembling shall be painted a good shop coat. Materials shall not be loaded for shipment until thoroughly dry. No painting shall be done after loading material on cars.

69.113. **Erection Marks**: Erection marks for the field identification of members shall be painted upon previously painted surfaces.

69.114. **Machine Finished Surfaces**: With the exception of abutting chord and column splices, column and truss shoe bases, machine finished surfaces shall be coated as soon as practicable after being accepted, with a hot mixture of white lead and tallow before removal from the shop. Surfaces of iron and steel castings milled for the purpose of removing scales, scabs, fins, blisters or other surface deformations shall generally be given the shop coat of paint.

The composition used for coating machine finished surfaces shall be mixed in the following proportions:

- 4 lbs. pure tallow.
- 2 lbs. pure white lead.
- 1 qt. linseed oil.

**FIELD PAINTING STEEL STRUCTURES**

69.115. **Field Cleaning**: When the erection work is complete including all riveting, straightening of bent metal, etc., all adhering rust, scale, dirt, grease or other foreign matter shall be removed as specified under Shop Cleaning, Paragraph 69.111.

69.116. As soon as the field cleaning is done to the satisfaction of the engineer the heads of field rivets and bolts and any surfaces from which the shop coat of paint has become worn off or has otherwise become defective and all shipping and erection marks shall be thoroughly covered with one coat of the same paint as used in the shop and permitted to become thoroughly dry before the first field coat is applied.

When the paint applied for “touching up” rivet heads and abraded surfaces has become thoroughly dry the first and second field coats may be applied. In no case shall a succeeding coat be applied until the previous coat has dried throughout the full thickness of the paint film.

All small cracks and cavities which have not become sealed in a watertight manner by the first field coat shall be filled with a pasty mixture of red lead and linseed oil before the second field coat is applied.
MAINTENANCE PAINTING STEEL STRUCTURES

69.117. Scope of Work: Unless otherwise provided maintenance painting shall consist of the removal of the rust, scale, dead paint, dirt, grease or other foreign matter from the metal parts or portions of existing bridge structures and the application of paint thereto.

All metal surfaces not in close contact with other metal surfaces or with wooden floor or truss members, concrete, stone masonry, etc., shall be considered as exposed to deterioration by rusting and shall be thoroughly cleaned and painted the number of coats indicated in and made a part of the contract.

69.118. Number of Coats: Unless otherwise provided, metal after being cleaned to the satisfaction of the Engineer shall be painted at least two coats of paint.

69.119. Cleaning and Painting: The requirements and methods of procedure for maintenance cleaning and painting shall be the same as specified for shop and field painting.

Whenever roadway or sidewalk planking is laid too closely in contact with the metal to permit free access for proper cleaning and painting, the planks shall either be removed or shall be cut to provide at least a one inch clearance for that purpose. The removal or the cutting of planks shall be done as directed by the Engineer. All planks removed shall be satisfactorily replaced and if broken or otherwise injured to an extent rendering them unfit for use they shall be renewed at the expense of the Contractor.

MEASUREMENTS AND PAYMENTS

69.120. Basis of Payment: The contract price for fabrication and erection of structural steel shall include all labor, material, transportation and painting necessary for the proper completion of the work.

Payment will be made on a pound-price or a lump sum basis, as required by the terms of the contract. For the purpose of payment, such minor items as bearing plates, pedestals, etc., shall, unless otherwise provided, be considered as structural steel even though made of other materials.

Payment for all material used in full-size tests shall be made on a pound-price basis or pound-price contracts, and, unless otherwise provided, on a basis of actual cost plus ten (10) per cent for lump-sum contracts. The scrap value of all material tested to destruction shall be allowed as a credit upon such payments.

69.121. Weight Paid For: The payment for pound-price contracts shall be based on the weight of metal in the fabricated structure, including field rivets shipped. The weight of erection bolts, field paint, and all boxes, crates or other containers used for packing, together with sills, struts and rods used for supporting members during transportation, shall be excluded.

Weights paid for shall be shop-scale weights unless otherwise provided. If specified in the contract or permitted by the Engineer, computed weights as hereinafter provided may be made the basis of payment.

69.122. Variation in Weight: If the weight of any member is more than two (2) per cent less than the computed weight, it may be cause for rejection. This applies to both pound-price and lump-sum contracts.

If the total scale weight of any structure exceeds the computed weight by more than two (2) per cent, the weight in excess of two (2) per cent above the computed weight shall not be paid for.

69.123. Weighing of Members: Finished work shall be weighed in the presence of the Inspector, if practicable. The Contractor shall supply satisfactory scales and shall perform all work involved in handling and weighing the various parts.

69.124. Computed Weight: The weight of steel shall be assumed at four hundred and ninety (490) pounds per cubic foot. The weight of cast iron shall be assumed at four hundred and fifty (450) pounds per cubic foot.

The weights of rolled shapes, and of plates up to and including thirty-six (36) inches in width, shall be computed on the basis of their nominal weights and dimensions, as shown on the approved shop drawings, deducting for copes, cuts and open holes.
The weights of plates wider than thirty-six (36) inches shall be computed on the basis of their dimensions, as shown on the approved shop drawings, deducting for cuts and open holes. To this shall be added one-half of the allowed percentages of overrun in weight given in the Standard Specifications for Structural Steel for Bridges, Serial Designation A7-24, of the American Society for Testing Materials.

The weight of heads of shop driven rivets shall be included in the computed weight, assuming the weights to be as follows:

<table>
<thead>
<tr>
<th>Diameter of Rivet</th>
<th>Weight for 100 heads</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4 inch</td>
<td>4.0 pounds</td>
</tr>
<tr>
<td>5/8 inch</td>
<td>7.5 pounds</td>
</tr>
<tr>
<td>3/4 inch</td>
<td>12.5 pounds</td>
</tr>
<tr>
<td>7/8 inch</td>
<td>18.5 pounds</td>
</tr>
<tr>
<td>1 inch</td>
<td>27.0 pounds</td>
</tr>
</tbody>
</table>

The weight of casting shall be computed from the dimensions shown on the approved shop drawings, with an addition of ten (10) per cent for fillets and overrun.

To the total computed weight of metal may be added an allowance of four-tenths (0.4) of one per cent for shop paint.

Payment will be made under
1. Item No. 72, Structural Steel (per pound).
2. Item No. 73, Structural Steel Superstructure complete except floor (lump sum).
SECTION~70
TIMBER STRUCTURES

70.01. Description: All timber structures shall be built as indicated on the plans, conforming to the line grade and dimensions shown, and in accordance with the specifications for Piling, Concrete, Untreated Timber, Treated Timber, and other items which constitute the complete structure.

MATERIALS

70.02. Lumber and Timber: For the various structural purposes the following grades shall be used. Southern long leaf yellow pine timber shall be used unless otherwise specified.

(a) Truss members, floor beams, stringers and flooring shall be Dense Select Structural or Select Structural, as specified. Flooring, if untreated shall be thoroughly air seasoned or kiln dried.
(b) Caps, posts, sills and mud sills, and nailing strips shall be Select Structural or Common Structural, as specified.
(c) Guard timbers and retaining pieces; sash, cross and longitudinal bracing; and girts shall be Common Structural.
(d) Bulkheads shall be Common Structural or No. 1 Common, as specified.
(e) Rails, rail posts and truss housing shall be Grade D Select or No. 1 Common, as specified. Rails and rail posts, if untreated, shall be thoroughly air seasoned or kiln dried.
(f) Scupper blocks and cross bridging shall be No. 1 Common.
(g) Inside sheathing for truss housing shall be No. 1 or No. 2 Common, as specified.
(h) For temporary structures which are for use only during erection, members specified above to be of the Dense Select and Select Structural Grades may be of the Common Structural Grade. Members specified above to be of the Common Structural Grade may be of No. one (1) Common.

STRUCTURAL TIMBER LUMBER AND PILING

70.03. Species of Woods: The common and botanical names of the species of woods recognized in these specifications are defined as follows:

<table>
<thead>
<tr>
<th>Common Name</th>
<th>Botanical Name</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chestnut</td>
<td>Castanea dentata</td>
</tr>
<tr>
<td>Cypress, Southern</td>
<td>Taxodium distichum</td>
</tr>
<tr>
<td>Fir, Douglas</td>
<td>Pseudotsuga taxifolia (Coast type)</td>
</tr>
<tr>
<td>Gum, Black</td>
<td>Nyssa sylvatica</td>
</tr>
<tr>
<td>Oak, Red includes Red Oak</td>
<td>Quercus borealis and Quercus borealis manima</td>
</tr>
<tr>
<td>Black Oak</td>
<td>Quercus velutina</td>
</tr>
<tr>
<td>Southern Red Oak</td>
<td>Quercus rubra</td>
</tr>
<tr>
<td>Water Oak</td>
<td>Quercus nigra</td>
</tr>
<tr>
<td>Willow Oak</td>
<td>Quercus phellos</td>
</tr>
<tr>
<td>Scarlet Oak</td>
<td>Quercus cococinea</td>
</tr>
<tr>
<td>Pin Oak</td>
<td>Quercus palustris</td>
</tr>
<tr>
<td>Swamp Red Oak</td>
<td>Quercus rubra pagadadfolia</td>
</tr>
<tr>
<td>Blackjack Oak</td>
<td>Quercus marilandica</td>
</tr>
<tr>
<td>Laurel Oak</td>
<td>Quercus laurifolia</td>
</tr>
<tr>
<td>Oak, White includes White Oak</td>
<td>Quercus alba</td>
</tr>
<tr>
<td>Chestnut Oak</td>
<td>Quercus montana</td>
</tr>
<tr>
<td>Post Oak</td>
<td>Quercus stellata</td>
</tr>
</tbody>
</table>
Bur Oak.............................................................. Quercus macrocarpa
Overcup Oak.................................................. Quercus lyrate
Swamp Chestnut Oak........................................ Quercus prinus
Swamp White Oak............................................. Quercus bicolor
Live Oak........................................................ Quercus virginiana
Chinquapin Oak................................................ Quercus muehlenbergii
Pine, Southern Yellow includes
Loblolly Pine.................................................. Pinus taeda
Longleaf Pine.................................................. Pinus palustris
Pitch Pine........................................................ Pinus rigida
Pond Pine........................................................ Pinus serotina
Shortleaf Pine.................................................. Pinus echinata
Slash Pine...................................................... Pinus caribaea

70.04. Limitation of Use: Timbers of the following species shall not be used in exposed structures without preservative treatment:

The red oaks, black gum, and shortleaf, loblolly and pond pine.

LUMBER AND STRUCTURAL TIMBER

70.05. Heart Requirements: All timber to be used without preservative treatment shall show not less than the following amounts of heartwood:

Stringers, floorbeams and flooring: Eighty per cent (80%) of heart on any girth.

Caps, sills and posts: Seventy-five per cent (75%) of heart on each of the four sides measured across the side.

Bracing, struts, rails, etc.: Eighty per cent (80%) of heart on both sides measured across the side.

For timber which is to be pressure treated with creosote oil there shall be no heartwood requirement and the amount of sap wood shall not be limited.

70.06. Grading of Lumber and Timber: Yard lumber and structural timber shall be graded in accordance with grading rules, adopted by the regional associations of lumber manufacturers, which conform to the basic provisions of "American Lumber Standards."

Lumber ordered in multiple lengths shall be graded after having been cut to length.

70.07. Basic Grades of Lumber and Timber: The grades recognized by this specification are as follows:

70.08. Yard Lumber: Grade D Select—Allows any number of defects or blemishes which do not detract from a finish appearance, especially when painted.

   No. 1 Common—Sound and tight knotted stock.
   Size of defects and blemishes limited.
   May be considered watertight lumber.

   No. 2 Common—Allows large and coarse defects.
   May be considered grain-tight lumber.

70.09. Structural Timber: Dense select (Douglas Fir and Southern Yellow Pine only.)

   Select.

   Common—The structural grades are further divided, on the basis of use, size and defects, into the following subgrades:

   Joist and Plank
   Beam and Stringer
   Post and Timber
BASIC GRADING OF STRUCTURAL TIMBER

70.10. General Provisions:
(a) All grades shall contain only sound wood.
(b) The measurement of a knot shall be made on the section of the knot appearing on the surface under consideration.
(c) In Post and Timber grades and on the wide faces of Joist and Plank, the measure of a knot shall be on the mean or average diameter.
(d) On the narrow faces of Joist and Plank, and Beams and Stringers, the size of a knot shall be taken as its width between lines parallel to the edges of the timber.
(e) On the wide or vertical faces of Beams and Stringers, the smallest diameter of a knot shall be taken as its size.
(f) Knots on the edges of wide faces of Beams and Stringers are limited to the same size as on the adjacent narrow faces.
(g) Knots on narrow faces and edges of wide faces of Joist and Plank, and Beams and Stringers, may increase proportionately from the size allowed in the middle third to twice that size at the ends of the piece.
(h) The size of knots on the wide faces of Joist and Plank, and Beams and Stringers, may increase proportionately from the size allowed at the edge to that allowed at the center line.
(i) Cluster knots and knots in groups are not permitted.
(j) Knot holes and holes from other causes than knots shall be limited as provided for knots.
(k) Shake shall be measured on the ends of a piece, and its size shall be taken as its width between lines parallel to the wide faces of the piece. Checks and splits shall be limited as provided for shakes. No checks or combinations of checks with shakes which would reduce the strength to a greater extent than the allowable shake shall be permitted.
(l) No combination of wane and knots is permitted which would reduce the strength more than the maximum allowable knot.
(m) No pieces of exceptionally light weight shall be permitted in any grade, except that light weight pieces otherwise of Select Grade may be accepted in the Common Grade.

70.11. Selection for Close Grain: Douglas Fir and Southern Yellow Pine of the Select Grade shall be selected for close grain.

Douglas Fir or Southern Yellow Pine selected for close grain shall average on either one end or the other not less than six (6) nor more than twenty (20) annual rings per inch, measured over a three-inch portion of a radial line representative of the average growth on the cross section located as described below.

When such radial line is not representative, it shall be shifted sufficiently to present a fair average, but in boxed heart pieces the distance from the pith to the beginning of the three-inch portion of the line shall not be changed.

In case of disagreement two radial lines shall be closed, and the number of rings shall be the average of these lines.

Location of Radial Line

70.12. Douglas Fir: In side cut pieces the line shall be at a right angle to the annual rings and the center of the three (3) inch portion of the line shall be at the center of the end of the piece.

In boxed heart pieces the line shall run from the pith to the corner farthest from the pith. When the least dimension is six (6) inches or less the three (3) inch portion of the line shall begin at a distance of one (1) inch from the pith. When the least dimension is more than six (6) inches the three (3) inch
portion of the line shall begin at a distance from the pith equal of two (2) inches less than one-half (1/2)
the least dimension of the piece.

If a three (3) inch portion of the radial line cannot be obtained the measurement shall be made over
as much of the three (3) inch portion as is available.

70.13. Southern Yellow Pine: In boxed heart pieces the rate of growth shall be counted over the
third, fourth, and fifth inches from the pith along the radial line.

In cases where timbers do not contain the pith, and it is impossible to locate it with any degree of
accuracy, the same inspection shall be made over three inches on an approximate radial line beginning
at the edge nearest the pith in timbers over three (3) inches in thickness and on the second inch nearest
to the pith in timbers three (3) inches or less in thickness.

In material containing the pith but not a five (5) inch radial line, which is less than two (2) inches
by eight (8) inches in section or less than eight (8) inches in width, that does not show over sixteen (16)
square inches on the cross-section, the inspection shall apply to the second inch from the pith. In larger
material that does not show a five (5) inch radial line, the inspection shall apply to the three (3) inches
farthest from the pith.

70.14. Selection for Density: Douglas Fir and Southern Yellow Pine of the Dense Select Grade
shall be selected for density.

Douglas Fir or Southern Yellow Pine selected for density shall average on either one end or the other
not less than six annual rings per inch and in addition one-third, or more, summerwood, over the same
portion of a radial line as provided for selection for close grain. Coarse-grained material excluded by this
rule shall be accepted as dense if averaging one-half, or more, summerwood.

The contrast in color between summerwood and springwood shall be sharp and the summerwood
shall be dark in color, except in pieces having considerably above the minimum requirement for summer-
wood.

In case of disagreement two radial lines shall be chosen and the summerwood and number of rings shall
be the average of these lines.

70.15. Joist and Plank Grades: Nominal thicknesses: 2 inch to 4 inch.

Nominal widths: 4 inch and wider.
Standard widths, SIS or S2S: 3/8 inch off.
Extra Standard Thickness, 2 inch, SIS or S2S: 1/4 inch off.
Standard Widths, 2 inches to 7 inches, SIE or S2E: 3/8 inch off.
8 inches and wider, SIE or S2E: 1/4 inch off.
Standard Lengths: Multiples of two feet.

<table>
<thead>
<tr>
<th>Width of Face</th>
<th>Dense Select</th>
<th>Select</th>
<th>Common</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>On or near edge middle third of length</td>
<td>Center line of face</td>
<td>On or near edge middle third of length</td>
</tr>
<tr>
<td>4&quot;</td>
<td>3/4&quot;</td>
<td>1-1/4&quot;</td>
<td>1&quot;</td>
</tr>
<tr>
<td>6&quot;</td>
<td>1&quot;</td>
<td>2&quot;</td>
<td>1-1/2&quot;</td>
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<tr>
<td>8&quot;</td>
<td>1-3/8&quot;</td>
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<tr>
<td>16&quot;</td>
<td>2-1/2&quot;</td>
<td>4-5/8&quot;</td>
<td>3-3/8&quot;</td>
</tr>
</tbody>
</table>

KNOTS ON WIDE FACES
**KNOTS ON NARROW FACES OF BOXED HEART PIECES**  
**MIDDLE THIRD OF LENGTH**

<table>
<thead>
<tr>
<th>Thickness of piece</th>
<th>Size of Knot</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Select</td>
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<td>2&quot;</td>
<td>5/8&quot;</td>
</tr>
<tr>
<td>3&quot;</td>
<td>1&quot;</td>
</tr>
<tr>
<td>4&quot;</td>
<td>1-1/4&quot;</td>
</tr>
</tbody>
</table>

**SUM OF DIAMETERS OF KNOTS, IN CENTER HALF OF LENGTH ON ANY FACE, NOT TO EXCEED**

- **Dense Select**—Select  
  1½ times width of face.

- **Common**  
  Two times width of face.

**SHAKES AND CHECKS**

- **Dense Select**—Select  
  Green 1/4 width of end  
  Seasoned 1/3 width of end

- **Common**  
  4/10 width of end  
  4/9 width of end

**ANGLE OF GRAIN, CENTER HALF OF LENGTH**

- **Dense Select**—Select  
  1 in 12

- **Common**  
  1 in 10

**WANE**

- **Dense Select**—Select  
  1/8 thickness and/or width

- **Common**  
  1/4 thickness and/or width

70.16. **Beam and Stringer Grades:**  
Nominal thicknesses: 5 inch and thicker.  
Standard lengths: Multiples of two feet.  
Nominal widths: 8 inches and wider.  
S1S, S1E, S2S or S4S: 1/2 inch off each way.

**KNOTS**

<table>
<thead>
<tr>
<th>Width of Face</th>
<th>Narrow face and edge of wide face middle third of length</th>
<th>Center line of wide face</th>
<th>Narrow face and edge of wide face middle third of length</th>
<th>Center line of wide face</th>
</tr>
</thead>
<tbody>
<tr>
<td>5&quot;</td>
<td>1-1/4&quot;</td>
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</tr>
<tr>
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<td>3-5/8&quot;</td>
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<td>16&quot;</td>
<td>2-3/8&quot;</td>
<td>2-3/8&quot;</td>
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<td>20&quot;</td>
<td>3-7/8&quot;</td>
<td>5-7/8&quot;</td>
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<tr>
<td>22&quot;</td>
<td>4&quot;</td>
<td>5-7/8&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>24&quot;</td>
<td>4-1/4&quot;</td>
<td>6-1/2&quot;</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
SUM OF DIAMETERS OF KNOTS, IN CENTER HALF OF LENGTH ON ANY FACE, NOT TO EXCEED

Dense Select—Select
Width of Face

Common
1½ times width of face

SHAKES AND CHECKS

Dense Select—Select
Green 1/4 width of end
Seasoned 1/8 width of end

Common
4/10 width of end
4/9 width of end

ANGLE OF GRAIN, CENTER HALF OF LENGTH

Dense Select—Select
1 in 15

Common
1 in 10

WANE

Dense Select—Select
1/8 thickness and/or width

Common
7/16 thickness and/or width

70.17. Post and Timber Grades: (Note: The method of measuring knots in the Post and Timber Grades makes it impractical to assign bending stresses to these grades. Therefore square timbers to be used where strength in bending is a factor shall be graded as Beams and Stringers)

Nominal sizes: 6 inches x 6 inches and larger.
Standard lengths: Multiple of two feet.
S1S, S1E, S2S, or S4S: ½ inch off each way.

KNOTS

<table>
<thead>
<tr>
<th>Width</th>
<th>Dense Select and Select</th>
<th>Common</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Width of Face</td>
<td></td>
</tr>
<tr>
<td>6&quot;</td>
<td>1-1/2&quot;, 2&quot;, 2-1/2&quot;, 3&quot;</td>
<td>2-3/8&quot;, 3-1/8&quot;, 4&quot;</td>
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<tr>
<td>10&quot;</td>
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<tr>
<td>12&quot;</td>
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<td>22&quot;</td>
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<td></td>
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<tr>
<td>24&quot;</td>
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</tbody>
</table>

The sum of diameters of all knots within any six (6") of length shall not exceed twice the size of the maximum knot allowable; nor shall there be two of the maximum allowable knots in the same six (6") of length on any one face.

SHAKES AND CHECKS

Dense Select—Select
Green 4/10 width of end
Seasoned 1/2 width of end

Common
1/2 width of end
6/10 width of end

ANGLE OF GRAIN

Dense Select—Select
1 in 10

Common
1 in 8

38
WANE

Dense Select—Select
1/8 thickness and/or width

Common
1/4 thickness and/or width

70.18. **Hewn and Round Timber:** Hewn timbers used in place of sawed timbers shall conform in all respects to the grading rules for Structural Timber.

Round timbers used in place of sawed timbers shall be of a quality equal to that hereinafter specified for Timber Piles. Section 75.

The effective size of a round timber shall be considered the same as that of a square timber having sectional dimensions equal to those of the inscribed square of the round timber at the critical section.

Hewn and round timbers shall not be used except when specified or approved by the Engineer.

**TIMBER PRESERVATIVES**

70.19. **Preservative Oils:** The preservative oil used shall be as specified or directed by the Engineer and shall be one of the following, depending on the type of treatment.

The creosote and anthracene oils shall be distillates of coal-gas tar or coke-oven tar. The creosote-coal-tar solution shall be a coal-tar product of which at least eighty (80%) shall be a distillate of coal-gas tar or coke-oven tar, and the remainder shall be refined or filtered coal-gas tar or coke-oven tar.

<table>
<thead>
<tr>
<th>PRESSURE TREATMENT</th>
<th>SURFACE TREATMENT (Open Tank, Brush and Spray)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Creosote Oils</td>
<td>Creosote-Coal-Tar Solution</td>
</tr>
<tr>
<td>Gra. 1</td>
<td>Gra. 2</td>
</tr>
<tr>
<td>3%</td>
<td>3%</td>
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<tr>
<td>(1) It shall not contain water in excess of...</td>
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<tr>
<td>(2) It shall be fluid at 15°C. and crystal-free at 38°C...</td>
<td></td>
</tr>
<tr>
<td>(3) It shall not contain matter insoluble in benzol in excess of...</td>
<td></td>
</tr>
<tr>
<td>(4) The specific gravity at 38°/15.5°C. shall not be less than...</td>
<td></td>
</tr>
<tr>
<td>(5) The distillate based on water-free oil, shall be within the following limits:</td>
<td></td>
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<tr>
<td>Up to 210°C., not more than...</td>
<td></td>
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<tr>
<td>Up to 235°C., not more than...</td>
<td></td>
</tr>
<tr>
<td>Between 235°C., and 300°C., not more than...</td>
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<tr>
<td>Up to 355°C., not less than...</td>
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</tr>
<tr>
<td>(6) The float test of residue above 355°C., shall not exceed 50 seconds at 70°C., if the distillation residue above 355°C., exceeds...</td>
<td></td>
</tr>
<tr>
<td>(7) Coke residue of oil shall not exceed...</td>
<td></td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Anthracene Oil</th>
<th>Hvy. Creosote Oil</th>
</tr>
</thead>
<tbody>
<tr>
<td>1%</td>
<td>1%</td>
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<tr>
<td>Required</td>
<td>Required</td>
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<tr>
<td>0.5%</td>
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<td>1.13</td>
<td>1.06</td>
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39
70.20. **Zinc Chloride:** Zinc chloride shall be acid-free and shall not contain more than one-tenth (0.1%) iron. Fused or so solid zinc chloride shall contain at least ninety-four (94%) chloride of zinc. Concentrated zinc chloride shall contain at least fifty (50%) chloride of zinc.

70.21. **Sampling and Testing:** Preservative oils shall be sampled and tested in accordance with the “Methods of Sampling and Analyzing Creosote Oil” as provided in the Tentative Standard Methods of Sampling and Testing of the American Association of State Highway Officials (A. S. T. M. Standard Method, Serial Designation D38-18, slightly modified) in so far as this applies, except that the distillation tests of anthracene oil shall be made in accordance with the requirements of the standard specifications of the American Wood Preservers’ Association. Coke residue shall be determined in accordance with the Tentative Method of Test for Coke Residue of Creosote Oil, Serial Designation on D168-23T, of the American Society for Testing Materials.

Zinc chloride shall be tested in accordance with the Tentative Methods of Chemical Analysis of Zinc Chloride, Serial Designation D199-24T, of the American Society for Testing Materials.

70.22. **Structural Shapes:** Rods, plates and shapes shall be of structural steel or wrought-iron as specified, conforming to the requirements of Section 69. Eyebars shall conform to the requirements of Section 69, for structural steel eye-bars.

70.23. **Castings:** Castings shall be cast steel or gray-iron as specified, conforming to the requirements of Section 69, paragraphs 69.16 to 69.20, inclusive.

70.24. **Hardware:** Machine bolts, drift bolts and dowels may be either wrought-iron or medium steel. Washers may be cast O-gee or malleable castings, or may be cut from medium steel or wrought-iron plate, as specified.

Machine bolts shall have square heads and nuts unless otherwise specified. Nails shall be cut or round wire of standard form. Spikes shall be cut or wire spikes, or boat spikes, as specified.

Nails, spikes, bolts, dowels, washers and lag screws shall be black or galvanized, as specified.

70.25. **Paint:** All paint for timber structures shall conform to the requirements specified in Section 69 as regard to quality and shall conform in composition to the formulas in paragraphs 69.30, 69.46c or 69.46d. Unless specially provided paint provided in paragraph 69.46d shall be used on all timber.

**CONSTRUCTION METHODS**

70.26. **Storage of Material:** Lumber and timber on the site of the work shall be stored in piles. Untreated material shall be open-stacked at least twelve (12) inches above the ground surface, and piled to shed water and prevent warping. When required by the Engineer it shall be protected from the weather by suitable covering.

Creosoted timber and piling shall be close stacked, piled to prevent warping, and the tops of the stacks shall be covered with a two (2) inch layer of earth.

The ground underneath and in the vicinity of all material piles shall be cleared of weeds and rubbish.

70.27. **Workmanship:** Workmanship shall be first-class throughout. None but competent bridge carpenters shall be employed and all framing shall be true and exact. Nails and spikes shall be driven with just sufficient force to set the heads flush with the surface of the wood. Deep hammer marks in wood surfaces shall be considered evidence of poor workmanship and sufficient cause for the removal of the workman causing them. The workmanship on all metal parts shall conform to the requirements specified for Steel Structures, Section 69.

70.28. **Treated Timber: Handling.** Treated timber shall be carefully handled without sudden dropping, breaking of outer fibers, bruising or penetrating the surface with tools. It shall be handled with rope slings. Cant dogs, peaveys, hooks or pike-poles shall not be used.

**Framing and Boring:** All cutting, framing and boring of treated timber shall be done before treatment in so far as is practicable. In waters infested by marine borers, cutting and boring below high water shall be avoided.
Cuts and Abrasions: All cuts in treated piles or timbers, and all abrasions after having been carefully trimmed, shall be covered with two applications of a mixture of sixty (60) per cent creosote oil and forty (40) per cent roofing pitch or brush coated with at least two applications of hot creosote oil and covered with hot roofing pitch.

Bolt Holes: Before driving bolts, hot creosote oil shall be poured into all bolt holes in such a manner that the entire surface of the hole shall be thoroughly coated with the oil. Any unfilled holes, after being treated with creosote oil, shall be plugged with creosote plugs.

70.29. Untreated Timber: In structures of untreated timber the following surfaces shall be thoroughly coated with hot creosote oil before assembling: Ends, tops and all contact surfaces of sills, caps, floor beams and stringers; and all ends, joints, and contact surfaces of bracing and truss members. The back faces of bulkheads and all other timber which is to be in contact with earth shall be similarly treated.

Bolts passing through non-resinous wood shall preferably be galvanized.

70.30. Treatment of Pile Heads: After having been cut to receive the caps, and prior to placing the caps, pile heads shall be treated to prevent decay. The heads of creosoted piles shall be treated as follows:

The sawed surface shall be covered with three applications of a mixture of sixty (60) per cent creosote oil and forty (40) per cent roofing pitch or thoroughly brush coated with three applications of hot creosote oil and covered with hot roofing pitch. Upon this shall be placed, covering of medium-weight roofing felt, or canvas treated with tar and oil, or galvanized iron which shall be bent down over the sides of the pile to shed water, and firmly secured thereto with large headed roofing nails and trimmed to give a workmanlike appearance.

The heads of untreated piles shall be given one of the following treatments as may be specified or directed by the Engineer:

1. The sawed surface shall be thoroughly brush coated with two applications of hot creosote oil.
2. The sawed surface shall be heavily coated with red lead paint after which it shall be covered with cotton duck, of at least eight (8) ounce weight, which shall be folded down over the sides of the pile and firmly secured thereto with large-headed roofing nails. The edges of the duck shall be trimmed to give a workmanlike appearance. The duck shall then be waterproofed by being thoroughly saturated and coated with one or more applications of red lead paint.

70.31. Holes for Bolts, Dowels, Rods and Lag Screws: Holes for round drift bolts and dowels shall be bored with a bit one-sixteenth (1/16) inch less in diameter than the bolt or dowel to be used. The diameter of holes for square drift bolts or dowels shall be equal to the least dimension of the bolt or dowel.

Holes for machine bolts shall be bored with a bit of the same diameter as the bolt.

Holes for rods shall be bored with a bit one-sixteenth (1/16) inch greater in diameter than the rod.

Holes for lag screws shall be bored with a bit not larger than the body of the screw at the base of the thread.

70.32. Bolts and Washers: A washer, of the size and type specified, shall be used under all bolt heads and nuts which would otherwise come in contact with wood.

All bolts shall be effectually checked after the nuts have been finally tightened.

70.33. Countersinking: Countersinking shall be done wherever smooth faces are required. Recesses formed for countersinking shall be painted with hot creosote oil, and, after the bolt or screw is in place, shall be filled with hot pitch.

70.34. Framing: All lumber and timber shall be accurately cut and framed to a close fit in such manner that the joints will have even bearing over the entire contact surfaces. Mortises shall be true to size for their full depth and tenons shall make snug fit therein. No shimming will be permitted in making joints, nor will open joints be accepted.
70.35. **Pile Bents:** The piles shall be driven as accurately as possible in the correct location and to the vertical or batter lines indicated on the plans. In case a pile is driven out of line it shall be straightened without injury before it is cut off or braced. Piles damaged in driving or straightening, or piles driven below grade, shall be removed and replaced at the Contractor's expense. No shimming on tops of piles will be permitted.

The piles for any one bent shall be carefully selected as to size, to avoid undue bending or distortion of the sway bracing.

Cut-offs shall be accurately made to insure perfect bearing between the cap and piles of a bent.

70.36. **FRAMED BENTS**

**Mud Sills:** Untreated timber used for mud sills shall be of cedar, heart cypress, redwood or other durable timber. Mud sills shall be firmly and evenly bedded to solid bearing and tamped in place.

**Concrete Pedestals:** Concrete pedestals for the support of framed bents shall be carefully finished so that the sills or posts will take even bearing on them. Dowels or not less than three-fourths (¾) inch diameter and projecting at least six (6) inches above the tops of the pedestals, shall be set in them when they are cast, for anchoring the sills or posts.

**Sills:** Sills shall have true and even bearing on mud-sills, piles or pedestals. They shall be drift-bolted to mud-sills or piles with bolts of not less than three-fourths (¾) inch diameter and extending into the mud-sills or piles at least six (6) inches. When possible all earth shall be removed from contact with sills so that there will be free air circulation around them.

**Posts:** Posts shall be fastened to pedestals with dowels of not less than three-fourths (¾) inch diameter extending at least six (6) inches into the posts.

Posts shall be fastened to sills by one of the following methods, as indicated on the plans:

(a) By dowels of not less than three-fourths (¾) inch diameter extending at least six (6) inches into posts and sills.

(b) By drift bolts or not less than three-fourths (¾) inch diameter driven diagonally through the base of the post and extending at least nine (9) inches into the sill.

70.37. **Caps:** Timber caps shall be placed to secure an even and uniform bearing over the tops of the supporting posts or piles and to secure an even alignment of their ends. All caps shall be secured by drift bolts of not less than three-fourths (¾) inch diameter extending at least nine (9) inches into the posts or piles. The drift bolts shall be approximately in the center of the post or piles.

70.38. **Bracing:** The ends of bracing shall be bolted through the pile, post or cap, with a bolt of not less than five-eighths (5/8) inch diameter. Intermediate intersections shall be bolted, or spiked with wire or boat spikes, as indicated on the plans. In all cases spikes shall be used in addition to bolts.

70.39. **Stringers:** Stringers shall be sized at bearings and shall be placed in position so that knots near edges will be in the top portions of the stringers.

Outside stringers may have butt joints but interior stringers shall be lapped to take bearing over the full width of floor beam or cap at each end. The lapped ends of untreated stringers shall be separated at least one-half (½) inch for the circulation of air and shall be securely fastened by drift-bolting where specified. When stringers are two panels in length the joints shall be staggered.

Cross bridging between stringers shall be neatly and accurately framed and securely toe-nailed with at least two nails in each end.

70.40. **Wheel Guards and Railing:** Wheel guards, wheel guard blocks, joist blocks and railing shall be accurately framed in accordance with the plans and erected true to line and grade.

Unless otherwise specified, wheel guards and rails and rail posts shall be surfaced on four (4) sides (S4S.)

Wheel guards shall be laid in sections not less than twelve (12) feet long.
70.41. **Single Plank Floors:** Shall consist of a single thickness of plank supported by stringers or joists, the planks shall be laid heart side down, with one-quarter (¼) inch openings between them or seasoned material and with tight joints for unseasoned material. Each plank shall be spiked to each joist or nailing strip with not less than two spikes, the length of which shall be at least three (3) inches greater than the thickness of the plank. The ends of the planks shall be cut off on a straight line parallel to the center line of the roadway. The planks shall be carefully graded as to thickness and so laid that no two adjacent planks shall vary in thickness any more than one-sixteenth (1/16) inch.

Scupper blocks shall be securely spiked in place, wheel guards shall be laid true to line and grade and bolted through the scupper blocks and floor plank and, if required, through the outside joist or nailing piece.

70.42. **Double Plank Floors:** Double plank floors shall consist of two layers of plank supported on stringers or joists, the lower course of plank shall be pressure treated with creosote oil and shall be laid in the same manner as specified in paragraph 2.15.23 for single plank floors.

The top course of plank shall be laid diagonal or parallel to the center line of roadway, as may be specified and each plank shall be spiked to the lower course at intervals of not more than two (2) feet with two (2) spikes which shall extend into the lower course at least three (3) inches. Joist shall be staggered at least three (3) feet. If the planks are placed parallel to the center line of the roadway, special care shall be exercised to fasten their ends securely and at the ends of the bridge they shall be beveled.

Scupper blocks and wheel guards shall be placed as specified in Paragraph 2.15.23 for single plank floors.

70.43. **Laminated or Strip Floors:** Strip shall not be more than three (3) inches in thickness and shall be surfaced to a uniform thickness (S-I-S) and, when specified, to a uniform width (S-I-E). The strips shall be placed on edge and at right angles to the center line of the roadway. Each strip shall be spiked to the adjacent strip at intervals of two (2) feet, the spikes being staggered eight (8) inches in adjacent strips. The spikes shall be of sufficient length to pass through two (2) strips and at least half-way through the third. In addition the strips shall be toe-nailed to the stringers with 20d spikes, the nailing of successive strips, being staggered so that the spacing of spikes along each stringer shall not be less than six (6) inches. For strips three (3) inches in thickness spikes driven vertically through the strip and extending into the stringer not less than three (3) inches may be substituted for toe-nailing.

70.44. **Trusses:** Trusses, when completed, shall show no irregularities of line. Chords shall be straight and true from end to end in horizontal projection and in vertical projection shall show a smooth curve through panel points conforming to the correct camber. All bearing surfaces shall fit accurately. Uneven or rough cuts at the points of bearing shall be cause for rejection of the piece containing the defect.

70.45. **Truss Housing:** The carpentry on truss housing shall be equal in all respects to the best housework. The finished appearance of the housing is considered of primary importance and special care shall be taken to secure a high quality of workmanship and finish on this portion of the structure. Workmen wearing shoes with calks will not be permitted on the roof.

70.46. **Erection of Trusses:** Unless otherwise directed by the Engineer, the following sequence of operations shall be followed in the erection of trusses:

1. Build trusses.
2. Build floor.
3. Line up trusses and remove supports.
4. Remove falsework.
5. Correct alignment and camber if necessary.
6. Build housing (if required.)
7. Build handrail.
8. Paint.

70.47. **PRESERVATIVE TREATMENTS FOR TIMBER**

**SEASONING**

70.48. **Air Seasoning:** Materials to be treated preferably shall be air seasoned until the moisture remaining in the wood will not prevent the injection and proper distribution of the specified amount
of preservative. For air seasoning, the materials shall be stored as follows: Lumber shall be segregated according to size and each layer in the pile shall be separated by at least one-inch strips with an air space of one inch or more between each two pieces of lumber in any layer; for caps, stringers, posts or large timbers, at least two-inch strips shall be used to separate the layers. Alleys at least three (3) feet wide shall be left between rows of stacks and the material shall be at least twelve (12) inches off the ground on concrete or treated timber sills. Piles shall be stored in like manner, placing as nearly as practical only one length in a stack, using at least two (2) inch strips or saplings of equal size between each layer, and reversing all sticks in every other layer in order to keep the stack level. The space under and between the rows or stacks shall be kept free at all times of rotting wood, weeds or rubbish. The yard shall be so drained that no water can stand under the stacks or in their immediate vicinity.

70.49. Steam Seasoning for Southern Yellow Pine: Southern Yellow Pine may be steam seasoned until the moisture remaining in the wood will not prevent the injection and proper distribution of the specified amount of preservative. The material shall be steamed in the cylinder at not more than twenty (20) to twenty-five (25) pounds pressure per square inch for not more than eight (8) hours for sawed timber and not more than twenty (20) hours for piles, the maximum pressure being reached in not less than two (2) hours. The cylinder shall be provided with vents to relieve it of air and insure proper circulation of steam. After steaming is completed a minimum vacuum of twenty-two (22) inches shall be maintained for not less than fifteen (15) minutes, or until the wood is as dry and free from moisture as practicable. The cylinder shall be relieved continuously or frequently enough to prevent condensate from accumulating in sufficient quantity to reach the wood. Before the preservative is introduced the cylinder shall be drained of condensate.

70.50. Oil Seasoning for Douglas Fir: Douglas Fir may be seasoned by boiling in oil under a vacuum until the moisture remaining in the wood will not prevent the injection and proper distribution of the specified amount of preservative.

The material shall be boiled in creosote under a vacuum at temperatures which do not exceed two hundred and twenty (220) degrees F. for piling and two hundred (200) degrees F. for sawed timber and lumber. A minimum vacuum of twenty (20) inches shall be maintained during boiling. The seasoning period shall be maintained until condensation passing off from the timber is at the rate of approximately one-tenth (1/10) of a pound per cubic foot of timber per hour.

70.51. Preparation for Treatment: Each cylinder charge shall consist of pieces approximately equal in size and moisture and sapwood content, into which approximately equal quantities of preserving fluid can be injected. Pieces shall be so separated as to insure contact of steam and preservatives with all surfaces.

70.52. Plant Equipment: Treating plants shall be equipped with the thermometers and gauges necessary to indicate and record accurately the conditions at all stages of treatment, and all equipment shall be maintained in condition satisfactory to the purchaser. The apparatus and chemicals necessary for making the analyses and tests required by the purchaser shall also be provided by plant operators, and kept in condition for use at all times.

70.53. Penetration: The range of pressure, temperature, and time duration shall be controlled so as to result in a maximum penetration by the quantity of preservative injected. The vacuum requirements stipulated are in inches of mercury at sea-level, and necessary corrections shall be made for altitude.

In Southern Yellow Pine: The preservative shall permeate all of the sapwood and as much of the heartwood as practicable.

In Douglas Fir the minimum penetration for specified amount of creosote oil shall be as follows:

<table>
<thead>
<tr>
<th>Specified amount of creosote per cubic foot.</th>
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<tbody>
<tr>
<td>10#</td>
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<td>-----</td>
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<tr>
<td>Piling .........................................</td>
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<tr>
<td>Timber 12 inch x 12 inch and larger .......</td>
</tr>
</tbody>
</table>
For timbers less than 12 inch x 12 inch the required depth of penetration shall be determined by the formula:

\[ P = \frac{R \cdot Ps}{Rs} \]

where
- \( P \) = required penetration.
- \( Ps \) = specified penetration for 12 inch x 12 inch timbers.
- \( R \) = ratio of volume of the piece in question to its superficial area.
- \( Rs \) = ratio of volume of 12 inch x 12 inch timber to its superficial area.

The penetration of the preservative shall be based on black or dark oil and in no case will light discoloration of the wood due to treatment be taken into consideration in measuring the depth of penetration.

Tests for penetration shall be made by taking borings with an increment borer or a five-eighth (5/8) inch auger, all holes so bored to be plugged by the Contractor with tight fitting creosoted plugs.

As many penetration tests of lumber and piling shall be made as is considered necessary by the Inspector. In the case of piling, the holes shall be bored midway between the ends.

In the case of timber and lumber, every fourth stick of the charge may be bored.

70.54. Amount of Preservatives: The amount of preservative to be used shall be shown on the plans or specified and this amount shall be retained in the timber unless the oil has been injected to refusal. Unless otherwise specified the amount of preservative retained shall be as follows:

- **(a) Creosote or Creosote-Coal-Tar:**
  1. For piles and timber in general bridge construction:
     - Full-cell process, not less than twelve (12) lbs. of oil per cubic foot of timber, or empty-cell process, not less than eight (8) lbs. of oil per cubic foot of timber.
  2. For piles or timber in salt water subject to attack by Marine borers:
     - Full-cell process.

- **Southern Yellow Pine,** not less than twenty (20) pounds of oil per cubic foot of timber. **Douglas Fir,** not less than fifteen (15) pounds of oil per cubic foot of timber.

- **(b) Zinc Chloride:**
  For timber or lumber to be painted or not subject to water leaching: Not less than one-half (1/2) pound of dry salt per cubic foot of timber.

**PRESSURE TREATMENTS**

70.55. Pressure Treatments for Southern Yellow Pine. The following pressure processes shall be used for the treatment of Southern Yellow Pine.

- **(a) Full-cell Process.** (Oil Treatment.)

  Timber shall be subjected to a vacuum of sufficient intensity and duration to insure that the wood is as dry and free from air as practicable, and to permit a retention of the specified number of pounds of preservative per cubic foot of wood.

  The preservative shall be introduced between one hundred sixty-five degrees F. (165° F.), and two hundred degrees F., (200° F.), the cylinder filled without breaking the vacuum. The pressure shall then be raised to and maintained at a minimum of one hundred (100) lbs. per square inch or until the quantity of preservative required to insure the final retention stipulated is injected into the wood, or until the purchaser's representative is satisfied that the largest volumetric injection that is practicable has been obtained. The temperature of the preservative during the pressure period shall be not less than one hundred fifty degrees F., (150° F.), nor more than two hundred degrees F., (200° F.) and shall average at least one hundred eighty degrees F., (180° F.). After the pressure is completed the cylinder shall be
emptied speedily of preservative, and a vacuum of not less than twenty-two (22) inches promptly created and maintained until the wood can be removed from the cylinder free of dripping preservative.

(b) **Empty-Cell Process with Initial Air. (Oil Treatment):** Timber shall be subjected to air pressure of sufficient intensity and duration to provide, under a vacuum, the ejection of surplus preservative, and to insure a retention and proper distribution of the stipulated number of pounds of preservative per cubic foot of wood. The preservative shall be introduced between one hundred sixty-five degrees F., (165° F.), and two hundred degrees F., (200° F.), the cylinder pressure being maintained constant until the cylinder is filled with preservative.

The pressure shall then be raised to and maintained at a minimum of one hundred fifty (150) lbs. per square inch or until there is obtained the largest practicable volumetric injection that can be reduced to the stipulated retention by a quick high vacuum, or until the purchaser's representative is satisfied that the largest volumetric injection that is practicable has been obtained. The temperature of the preservative during the pressure period shall be not less than one hundred fifty degrees F., (150° F.), nor more than two hundred degrees F., (200° F.), and shall average at least one hundred eighty degrees F., (180° F.). After pressure is completed the cylinder shall be emptied speedily of preservative, and a vacuum promptly created and maintained until the wood can be removed from the cylinder free of dripping preservative.

(c) **Empty-Cell Process without Initial Air. (Oil Treatment):** The preservative between one hundred sixty-five degrees F., (165° F.) and two hundred degrees F., (200° F.), shall be introduced to the timber until the cylinder is filled. Pressure shall then be raised to and maintained at a minimum of one hundred (100) lbs. per square inch or until there is obtained the largest practicable volumetric injection that can be reduced to the stipulated retention by a quick high vacuum, or until the purchaser's representative is satisfied that the largest volumetric injection that is practicable has been obtained. The temperature of the preservative during the pressure period shall be of not less than one hundred fifty degrees F., (150° F.), nor more than two hundred degrees F., (200° F.), and shall average at least one hundred eighty degrees F., (180° F.). After pressure is completed the cylinder shall be emptied speedily of preservative and a vacuum of not less than twenty-two (22) inches promptly created and maintained for not less than thirty (30) minutes until the quantity of preservative injected is reduced to the required retention and the wood can be removed from the cylinder free of dripping preservative.

(d) **Zinc Chloride Treatment:** The treating solution which shall not have a strength exceeding five per cent (5%), and which shall be no stronger than necessary to obtain the required retention of preservative with the greatest volumetric absorption practicable, shall, be thoroughly mixed before use. Air-seasoned timber may be steamed in the cylinder for not less than one hour nor more than two hours, at a pressure of not more than twenty (20) lbs. per square inch. After steaming is completed a vacuum of at least twenty-two (22) inches shall be maintained for not less than fifteen (15) minutes or until the wood is as dry and free of air as practicable. If the vacuum is broken while the condensate is being drained from the cylinder a second vacuum as high as the first shall be created. The preservative shall be introduced without breaking the vacuum until the cylinder is filled. The pressure shall then be raised to and maintained at a minimum of one hundred (100) lbs. per square inch until the quantity of preservative required to insure the final retention stipulated is injected into the timber, or until less than five per cent (5%) of the total quantity required has been injected during the latter half of one hour throughout which the rate of injection has persistently decreased while the pressure has been held continuously at one hundred twenty-five (125) or more pounds per square inch.

The temperature of the preservative during the pressure period shall be not less than one hundred forty (140°) degrees F., nor more than two hundred (200°) degrees F and shall average at least one hundred fifty (150°) degrees F. After the pressure is completed, the cylinder shall be emptied speedily of preservative and a vacuum promptly created and maintained until the wood can be removed from the cylinder free of dripping preservative.

70.56. **Pressure Treatments for Douglas Fir:** The following pressure processes shall be used for the oil treatment of Douglas Fir:

**Heating with Oil:** Air-seasoned or kiln dried Douglas Fir will not be required to be boiled under a vacuum, but it may be desirable to hold the material in a creosote bath maintained at a temperature of one hundred eighty degrees (180°) to one hundred ninety degrees F., (190° F.), for a length of time which,
combined with the pressure period, is in the judgment of the operator necessary to secure the specified absorption.

(a) **Full-Cell Process:** Following the heating period, in the case of air-seasoned material, and the seasoning under vacuum period in the case of material artifically seasoned, the cylinder shall be filled with creosote and pressure applied as required to a maximum limit of one hundred seventy-five (175) pounds per square inch and maintained, taking into consideration the quantity of creosote absorbed during the bath, until the specified absorption of creosote has been obtained.

Temperature of the creosote during the pressure period shall be as high as possible, with a minimum limit of one hundred sixty degrees F. (160° F.), and a maximum limit of two hundred degrees F. (200°F.)

After pressure is completed, the cylinder shall be emptied of creosote and a vacuum of at least twenty (20) inches promptly created and maintained for a sufficient period of time to free the material of dripping creosote.

(b) **Empty-Cell Process with Initial Air:** Following the heating period, in the case of air-seasoned material, and the seasoning under vacuum period in the case of material artifically seasoned, the material shall be subjected to air pressure of an intensity and duration which, in the judgment of the operator, is sufficient to accomplish the final retention of creosote specified.

The preservative shall then be introduced, the air pressure being maintained constant, until the cylinder is completely filled.

Creosote shall then be pressed from the measuring tanks into the wood in a quantity sufficient, in the opinion of the operator, to leave the required retention at the completion of the process herein described. Maximum pressure shall in no case exceed two hundred (200) pounds per square inch. The temperature of the creosote during the pressure period shall be as high as possible, within a minimum limit of one hundred sixty degrees F (160° F.), and a maximum limit of two hundred degrees F. (200° F.)

After pressure is completed, the cylinder shall be quickly emptied of creosote and a vacuum of at least twenty (20) inches promptly created and maintained for such period of time as may be required to remove dripping creosote from the material.

**SURFACE TREATMENT FOR TIMBER**

70.57. **Open Tank Treatments:** Open tank treatment shall consist of a hot bath treatment or a hot and cold bath treatment as may be specified.

70.58. **Equipment:** All tanks used in the open tank process shall be of sufficient size to allow free circulation of the liquid around the largest amount of timber being treated in any operation.

Suitable liquid shall be maintained in the tanks to completely immerse the timber. When a number of pieces are being treated at each operation, each stick shall be separated from the others on all sides by square or round spaces not less than one-fourth (\(\frac{1}{4}\)) inch in least dimension. Suitable slings and handling devices shall be provided to make the material transfers necessary during the complete process without disturbing the stacked position of the pieces in the bundle.

For hot bath treatments at least one tank shall be supplied having suitable steam coils or other heating devices to keep the liquid at a uniform temperature throughout the tank of not less than two hundred forty degrees F. (240° F.), during the complete process.

For hot and cold bath treatments at least one hot tank shall be supplied as for the hot bath treatment and one cold tank having the same capacity as the hot tank. The cold tank shall be equipped with suitable cold water coils or water jackets so that the temperature of the liquid at the time of immersion of each batch of timber shall be no higher than the surrounding atmospheric temperature.

70.59. **Preparation of Material:** All timber to be treated shall be free from dirt, grease or other foreign matter which will in any way hinder the free penetration of the preservative. Framing shall be done before treatment. Round timber or timber with wane shall have the rough bark and inner bark removed as specified for wood piling in Division 3, Section 10.
70.60. **Time of Immersion:** The time of immersion as specified herein is for Southern Yellow Pine, Chestnut, Black Gum, Red Oak, Lodgepole Pine, Norway Pine and Pondosa Pine. The specified time of immersion shall be increased sixty-six and two-thirds (66 2/3) per cent for Southern Cypress Douglas Fir, and one hundred (100) per cent for White Oak and Eastern Spruce.

70.61. **Single or Hot Bath Treatment:** The timber shall be completely immersed in preservative in the hot tank, which shall be maintained at a temperature of one hundred ninety degrees F. (190° F.), for seasoned timber and two hundred thirty degrees F. (230° F.), for timber not seasoned. A tolerance of ten (10) degrees in either direction is permissible. For seasoned timber the immersion shall be for a period of not less than fifteen (15) minutes for two-inch timber, with an increase of five minutes in the immersion period for each additional inch in thickness. For timber other than seasoned the immersion period shall be doubled.

70.62. **Ordinary Hot and Cold Treatment:** The timber shall be completely immersed in preservative in the hot tank, which shall be maintained at a temperature of one hundred ninety degrees F., (190° F.), for seasoned timber and two hundred thirty degrees F. (230° F.), for timber not seasoned. A tolerance of ten (10) degrees in either direction is permissible. For seasoned timber the immersion shall be for a period of not less than fifteen (15) minutes for two-inch timber with an increase of five minutes in the immersion period for each additional inch in thickness. For timber other than seasoned the immersion period shall be doubled. At the end of this period the timber shall be removed from the hot tank and immediately immersed in the cold tank. At the time of transfer the preservative in the cold tank shall have a temperature as low as possible but in no case higher than the surrounding air temperature. The timber shall be completely immersed in the cold tank for a period one-half as long as the hot tank. Successive charges from the hot tank may be placed first in one cold tank and the next in a second cold tank in order to keep the cold tank temperature as low as possible at the time of immersion. Should the Contractor supply a cold tank capable of handling all material and with a cooling system which will keep the temperature at the time of all cold treatments as specified, only one cold tank may be required. Single cold tank equipment shall be subject to the approval of the Engineer.

70.63. **Heavy Hot and Cold Treatment:** The requirements for this treatment are the same as those specified above for the Ordinary Hot and Cold Treatment except that the time of immersion in the cold bath shall be the same as the time of immersion in the hot bath.

70.64. **Brush Treatment:** All timber to be given brush treatment shall be free from atmospheric moisture and in no case shall brush treatment be applied when the surface of the timber is wet. The surfaces to be treated shall be free from dirt, grease, or other foreign matter which will in any way hinder the maximum penetration of the preservative.

The preservative shall be heated in proper receptacles immediately adjacent to the point of application and shall be applied within the temperature range of one hundred seventy degrees (170° F.), to one hundred ninety degrees (190° F.), for seasoned wood and two hundred twenty degrees (220° F.), to two hundred forty degrees F. (240° F.), for unseasoned wood.

A minimum of two coats shall be applied to all surfaces to be treated except cut ends, joints and mortises which shall be given three coats. Each coat shall be allowed to penetrate before applying the next coat. All checks, bolt holes and cracks shall be run full of the preservative oil and an extra heavy treatment shall be given to knotty spots.

70.65. **Spray Treatment:** The condition of the timber prior to spray treatment shall conform to the requirements specified for brush treatment.

The temperature of the preservative shall be maintained at two hundred forty degrees F. (240° F.) The shortest length of hose practicable shall be used to prevent undue chilling between the spray tank and nozzle. Preservative shall be renewed frequently in the spray tank to prevent chilling. The spray shall be applied with a good pressure and only fine enough to prevent waste, until the preservative begins to run. Equipment employing air pressure which has a cooling effect on the hot preservative shall not be used.

Two liberal applications shall be made allowing sufficient time for the absorption of the first application before the second is made.
70.66. **Painting:** Untreated or salt treated wood rails and rail posts shall be painted with three (3) coats of white paint conforming to the requirements of Paragraph 69.49d.

Parts of the structure, other than rails and rail posts, which are to be painted shall be designated on the plans or in the supplemental specifications.

Metal parts, except hardware, shall be given one coat of shop paint and, after erection, two coats of field paint.

70.67. **Basis of Payment:** Payment for timber structures shall include the furnishing of materials, preservative treatment, equipment, tools and labor necessary for the erection and painting of the work in a satisfactory manner.

Lumber and timber shall be paid for at the contract price per thousand feet board measure (M. B. M.) for material remaining in the finished structure, which payment shall include the cost of all hardware. Computations of the amount of lumber and timber in the structure shall be based on nominal sizes and the shortest commercial length which could be used. No other allowance for waste will be made.

Metal parts, other than hardware, shall be paid for at the contract price per pound, the weight being computed in the same manner as specified for Steel Structures, Section 69.

Payment will be made under

Item No. 74, Bridge Timber (untreated) per M. B. M.
Item No. 75, Bridge Timber (treated) per M. B. M.
SECTION 71

PILING

(a) BEARING PILES

71.01. Description: Bearing piles shall consist of round or square piling of the kind and dimensions specified, placed at the locations and to the elevations indicated on the plans or as directed by the Engineer, and in conformity with the requirements and provisions of these Specifications.

MATERIAL

TIMBER PILES

71.02. General: Timber piles which will be below water level at all times, may be of any species of wood which will satisfactorily withstand driving.

In untreated piling for use in exposed work, the diameter of the heartwood shall be not less than (eight-tenths \(8/10\)) of the required diameter of the pile.

71.03. Quality: All wood piling shall be cut from sound and solid trees, preferably during the winter season. They shall contain no unsound knots. Sound knots will be permitted provided the diameter of the knot does not exceed four (4) inches or one-third \((1/3)\) of the diameter of the stick at the point where it occurs. Any defect or combination of defects which will impair the strength of the pile more than the maximum allowable knot shall not be permitted. The butts shall be sawed square and the tips shall be sawed square or tampered to a point not less than four (4) inches in diameter as directed by the Engineer.

Unless otherwise specified all piles shall be peeled by removing all of the rough bark and at least eighty \((80\%)\) of the inner bark. No strip of inner bark remaining on the stick shall be over three-quarters \((3/4)\) inch wide or over eight (8) inches long, and there shall be at least one (1) inch of clean wood surface between any two such strips. Not less than eighty \((80\%)\) of the surface on any circumference shall be clean wood.

Piles shall be cut above the ground swell and shall taper from butt to tip. A line drawn from the center of the tip to the center of the butt shall not fall outside of the center of the pile at any point more than one (1%) of the length of the pile. In short bends, the distance from the center of the pile to a line stretched from the center of the pile above the bend to the center of the pile below the bend shall not exceed four \((4\%)\) of the length of the bend or two and one-half \((2\ 1/2)\) inches. All knots shall be trimmed close to the body of the pile.

71.04. Dimension: Round piles shall have a minimum diameter at the tip, measured under the bark, as follows:

<table>
<thead>
<tr>
<th>Length of Pile</th>
<th>Tip Diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Less than 40 feet</td>
<td>8&quot;</td>
</tr>
<tr>
<td>40 to 60 feet</td>
<td>7&quot;</td>
</tr>
<tr>
<td>Over 60 feet</td>
<td>6&quot;</td>
</tr>
</tbody>
</table>

The minimum diameter of piles at a section four feet from the butt, measured under the bark, shall be as follows:

<table>
<thead>
<tr>
<th>Length of Pile</th>
<th>Diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sou. Yellow Pine—Sou. Cypress 20 feet and under</td>
<td>11&quot;</td>
</tr>
<tr>
<td>20 to 30 feet</td>
<td>12&quot;</td>
</tr>
<tr>
<td>30 to 40 feet</td>
<td>12&quot;</td>
</tr>
<tr>
<td>Over 40 feet</td>
<td>13&quot;</td>
</tr>
<tr>
<td>All other species</td>
<td>14&quot;</td>
</tr>
</tbody>
</table>

The diameter of the piles at the butt shall not exceed twenty (20) inches.
Square piles shall have the dimensions shown on the plans.

Cypress piles for untreated timber trestles shall be of red or black cypress and shall have heartwood with a minimum diameter at a section four feet from the butt of nine (9) inches for piles with a length of thirty feet and under and ten (10) inches for piles over thirty feet in length. In determining the heartwood diameter it shall be assumed that the taper is in the same ratio as the taper of the outside of the pile.

71.05. Timber Preservatives shall conform to the requirements specified under Timber Structures Section 70, Pamphlet “G.”

71.06. Materials for concrete piles shall conform to the material requirements specified in Section 7 “Concrete Structures” and Section 68 “Reinforcement in Structures” for the several items which constitute the structure.

CONSTRUCTION METHODS

71.07. Limitation of Use: Except for trestle work, timber piles shall be used only below permanent ground water level. Untreated timber piles shall not be used in water which is infested by marine borers.

In general, the penetration for any pile shall be not less than 10 feet in hard material and not less than one-third the length of the pile nor less than 20 feet in soft material.

For foundation work, no piling shall be used to penetrate a very soft upper stratum overlying a hard stratum unless the piles penetrate the hard material a sufficient distance to rigidly fix the ends.

71.08. Preparation for Driving: Piles shall not be driven until after the excavation is complete. Any material forced up between the piles shall be removed to correct elevation before masonry for the foundation is placed.

71.09. Caps: The heads of all concrete piles, and the heads of timber piles when the nature of the driving is such as to unduly injure them, shall be protected by caps of approved design, preferably leaving a rope or other suitable cushion next to the pile head and fitting into a casting which in turn supports a timber shock block. When the area of the head of any timber pile is greater than that of the face of the hammer, a suitable cap shall be provided to distribute the blow of the hammer throughout the cross section of the pile and thus avoid, as far as possible, the tendency to split or shatter the pile.

71.10. Collars: Collars or bands to protect timber piles against splitting and brooming shall be provided where necessary.

71.11. Pointing: Timber piles shall be pointed where soil conditions require it. When necessary, the piles shall be shod with metal shoes of a design satisfactory to the Engineer, the points of the piles being carefully shaped to secure an even and uniform bearing on the shoes.

71.12. Splicing Timber Piles: Full length piles shall always be used where practicable but if splices cannot be avoided an approved method of splicing shall be used. Piles shall not be spliced except by permission of the Engineer.

71.13. Method of Driving: Piles shall be driven with a gravity hammer, steam hammer, or a combination of water jets and hammer. Concrete piles preferably shall be driven by means of a combination of hammer and jets.

EQUIPMENT FOR DRIVING

71.14. Hammers for Timber Piles: Gravity hammers for driving timber piles shall weigh not less than two thousand (2000) pounds and the fall shall be so regulated as to avoid injury to the pile and in no case shall exceed twenty (20) feet.

71.15. Hammers for Concrete Piles: Concrete piles preferably shall be driven with steam or gravity hammers. Steam hammers for this purpose shall develop an energy per blow at each full stroke of the piston of not less than thirty-five hundred (3500) foot pounds per cubic yard of concrete in the
pile being driven. The total energy developed by the hammer shall be not less than twelve thousand (12000) foot pounds per blow.

Gravity hammers, when used, shall have a weight not less than that of the pile and the maximum drop shall not exceed eight (8) feet.

71.16. Leads: Pile driver leads shall be constructed in such manner as to afford freedom of movement to the hammer and they shall be held in position by guys or stiff braces to insure support to the pile during driving. Except where piles are driven through water, the leads preferably shall be of sufficient length so that the use of a follower will not be necessary.

71.17. Followers: The driving of piling with followers shall be avoided if practicably and shall be done only under written permission of the Engineer. When followers are used, one pile from every group of ten shall be a long pile driven without a follower, and shall be used as a test pile to determine the average bearing power of the group.

71.18. Water Jets: When water jets are used, the number of jets and the volume and pressure of water at the jet nozzles shall be sufficient to freely erode the material adjacent to the pile. The plant shall have sufficient capacity to deliver at all times at least one hundred (100) pounds per sq. in. pressure at two 3/4 in. jet nozzles. Before the desired penetration is reached the jets shall be withdrawn and the piles shall be driven with the hammer to secure the final penetration.

71.19. Allowable Variation in Driving: Piles shall be driven with a variation of not more than one-quarter (1/4) inch per foot from the vertical or from the batter line indicated.

DETERMINATION OF BEARING VALUES

71.20. Loading Tests: When required, the size and number of piles shall be determined by actual loading tests. In general, these tests shall consist of the application of a test load placed upon a suitable platform supported by the pile, together with suitable apparatus for accurately determining the superimposed weight and the settlement of the pile under each increment of load. The safe allowable load shall be considered as fifty (50) per cent of that load which, after forty-eight (48) hours application, causes a permanent settlement, measured at the top of the pile, of not more than one-fourth (1/4) inch. At least one pile for each group of one hundred piles shall be thus tested.

71.21. Timber Piles: In the absence of loading tests, the safe bearing values for timber piles shall be determined by the following formulas:

\[ P = \frac{2WH}{S+1.0} \] for gravity hammers

\[ P = \frac{2WH}{S+0.1} \] for single-acting steam hammers

\[ P = \frac{2H(W+Ap)}{S+0.1} \] for double-acting steam hammers

Where \( P \) = Safe bearing power in pounds.
\( W \) = Weight, in pounds, of striking parts of hammer.
\( H \) = Height of fall in feet.
\( A \) = Area of piston in square inches.
\( p \) = Steam pressure in pounds per square inch at the hammer.
\( S \) = The average penetration in inches per blow for the last 5 to 10 blows for gravity hammers and the last 10 to 20 blows for steam hammers.

The above formulas are applicable only when:

(a) The hammer has a free fall.
(b) The head of the pile is free from broomed or crushed wood fibre.
The penetration is at a reasonably quick and uniform rate. The bearing powers of timber piles, as determined by the foregoing formulas, shall be considered effective only when they are less than the crushing strengths of the piles. In general, piles shall be required to develop in bearing capacity of not less than fifteen (15) tons nor more than twenty-five (25) tons. However, the character of the soil penetrated, conditions of driving, distribution, sizes and lengths of the piles involved, and the computed load per pile shall be given due consideration in determining the reliability of driven piles.

In case water jets are used in connection with the driving, the bearing power shall be determined by the above formulas from the results of driving after the jets have been withdrawn, or a load test may be applied.

71.22. Concrete Piles: The bearing values for concrete piles shall be determined by means of the loading tests above specified. The formulas specified above for timber piling may be used as a rough approximation for precast concrete piles and they may also be applied to the driven core for cast-in-places piles.

71.23. Test Piles: When required, the Contractor shall drive test piles of a length and at the location designated by the Engineer. These piles shall be of greater length than the length assumed in the design in order to provide for any variation in soil conditions.

71.24. Order Lists for Piling: The Engineer shall furnish the Contractor with an itemized list showing the number and length of all piles and the Contractor shall furnish piles in accordance with such itemized list.

(b) TIMBER PILES

71.25. Storage and Handling: The method of storing and handling shall be such as to avoid injury to the piles. Special care shall be taken to avoid breaking the surface of treated piles and cantdogs, hooks or pikepoles shall not be used. Cuts or breaks in the surface of treated piling shall be given three brush coats of hot creosote oil of approved quality and hot creosote oil shall be poured into all bolt holes.

71.26. Elevation: The tops of all piling shall be sawed to a true plane as shown on the plans and at the elevation fixed by the Engineer. Piles which support timber caps or grillage work shall be sawed to the exact plane of the superimposed structure and shall exactly fit it. Broken, split or misplaced piles shall be drawn and properly replaced. Piles driven below the cut-off grade fixed by the Engineer shall be withdrawn and replaced by new and, if necessary, longer piles at the expense of the Contractor. All piles raised during the process of driving adjacent piles shall be driven down again if required by the Engineer.

(c) CONCRETE PILES

71.27. Manufacture of Precast Concrete Piles:

71.28. Size and Shape: Precast concrete piles shall be of approved size and shape. If a square section is employed, the corners shall be chamfered at least one inch. Piles preferably shall be cast with a driving point and for hard driving preferably shall be shod with a metal shoe of approved pattern. Piling may be either of uniform section or tapered. In general tapered piling shall not be used for trestle construction except for that portion of the pile which lies below the ground line; nor shall tapered piles be used in any location where the piles are to act as columns. In general, concrete piles shall have a cross sectional area, measured above the taper, of not less than one hundred and forty (140) square inches and when they are to be used in salt water they shall have a cross sectional area of not less than two hundred and twenty (220) square inches.

71.29. Class of Concrete: Class "D" concrete shall be used for precast concrete piles.

71.30. Reinforcement: Reinforcement for precast concrete piles shall consist of longitudinal bars in combination with lateral reinforcement in the form of hoops or spirals. The longitudinal reinforce-
ment shall be not less than one (1) per cent and preferably not less than one and one-half (1½) percent of the total cross-section of the pile. The reinforcement shall be placed at a clear distance from the face of the pile of not less than two (2) inches and when the piles are for use in salt water or alkali soils this clear distance shall be not less than three (3) inches. The driving point and also the top of the pile shall be protected against impact by means of special spiral winding or bands designed for this purpose. The reinforcing system preferably shall be of the unit type, rigidly wired or fastened at all inter-sections and held to true position in the forms by means of concrete blocks or other suitable device. When piles exceed fifty-five (55) feet in length, additional longitudinal reinforcement shall be added throughout the central one-third (1/3) of the length. Piling under retaining walls, arch footings, abutments, etc., shall be designed to withstand the lateral stresses induced.

71.31. **Form Work:** Forms for precast concrete piles shall conform to the general requirements for concrete form work as provided in Paragraph 67.36 under “Concrete Masonry.” Forms shall be accessible for tamping and consolidation of the concrete. Under good weather curing conditions side forms may be removed at any time not less than twenty-four (24) hours subsequent to placing concrete, but the entire pile shall remain supported for at least seven days and shall not be subjected to any handling stress until the concrete has set for at least twenty-one (21) days and for a longer period in cold weather, additional time to be determined by the Engineer.

71.32. **Casting:** Piling may be cast either in a vertical or horizontal position. Special care shall be exercised to puddle and tamp the concrete around the reinforcement and to avoid the formation of stone pockets.

During the placing of concrete the forms shall be vibrated by tapping with a hammer or wooden maul. Concrete shall be placed continuously in each pile and shall be carefully spaded, puddled and tamped, special care being exercised to avoid horizontal or diagonal cleavage planes, and to see that the reinforcement is properly embedded in the concrete.

71.33. **Finish:** As soon as the forms are removed, concrete piles shall be carefully gone over and pointed with 1:2 mortar, filling up all cavities or irregularities. Trestle piling exposed to view shall be finished above the ground line in accordance with the provisions governing the finishing of concrete columns. Foundation piling, that portion of trestle piling which will be below the ground surface, and piles for use in sea water or alkali soils shall not be finished except by pointing as above set forth.

71.34. **Curing:** Concrete piles shall be cured in accordance with the general provisions governing the curing of concrete Paragraph 67.33. As soon as the piles have set sufficiently to permit, they shall be removed from the forms and piled in a curing pile separated from each other by wood spacing blocks. No pile shall be driven until it has set for at least thirty (30) days and in cold weather for a longer period as determined by the Engineer. Concrete piles for use in sea water or alkali soils shall be cured for not less than sixty (60) days before being used.

71.35. **Storage and Handling of Precast Concrete Piles:** For precast concrete piles, the method of storing and handling shall be such as to eliminate the danger of fracture, by impact or undue bending stresses, in curing or transporting the piles from the molds and into the leads. In general, concrete piles shall be lifted by means of suitable bridle or sling attached to the pile at points not over twenty feet apart. In no case shall the method of handling be such as to induce stresses in the reinforcement in excess of twelve thousand (12,000) pounds per square inch, allowing one hundred percent of the calculated load for impact and shock effects.

In handling piles for use in sea water or alkali soils special care shall be exercised to avoid injury to the surface of the pile.

(d) **MANUFACTURE OF CAST-IN-PLACE CONCRETE PILES**

71.36. **Description:** Cast-in-place concrete piles shall be cast in strong metal shells which shall remain permanently in place.

71.37. **Metal Shells:** The metal shall be of a sufficient thickness and reinforced to such an extent that it will hold its original form and show no signs of distortion after the core has been withdrawn. The design of the shell shall be submitted to and approved by the Engineer before any driving is done.
71.38. Inspection of Shells: After the shell has been driven and the core withdrawn, the shell shall be inspected and approved before any concrete is placed. No payment will be made for any shell which has been improperly driven, is broken, or otherwise defective and, if necessary, any such shell shall be removed and replaced.

71.39. Class of Concrete: Class “D” concrete shall be used for cast-in-place concrete piles.

71.40. Reinforcement: Reinforcement for cast-in-place piles shall be of the unit type, rigidly fastened together and lowered into the shell before concrete is placed. No loose bars will be permitted. The reinforcement shall be secured in such a manner as to insure its proper location in the finished pile.

71.41. Placing Concrete: No concrete shall be placed until all driving within a radius of fifteen (15) feet has been completed, or until all the shells for any one bent have been completely driven. If this cannot be done, all driving within the above limits shall be discontinued until the concrete in the last pile cast has set at least seven (7) days.

Concrete shall be placed continuously in each pile, care being used to fill every part of the shell and to work concrete around the reinforcement without displacing it. No concrete shall be placed in shells containing an accumulation of water.

71.42. Extensions or “Build Ups”. Extensions, splices or “Build Ups” on concrete piles shall be avoided but when necessary they shall be made as follows:

After the driving is completed, the concrete at the end of the pile shall be cut away, leaving the reinforcing steel exposed for a length of forty diameters. The final cut of the concrete shall be perpendicular to the axis of the pile. Reinforcement similar to that used in the pile shall be securely fastened to the projecting steel and the necessary form work shall be placed, care being taken to prevent leakage along the pile. The concrete shall be of the same quality as that used originally in the pile. Just prior to placing concrete the top of the pile shall be thoroughly wetted and covered with a thin coating of neat cement, retempered mortar or other suitable bonding material. The forms shall remain in place not less than seven (7) days and shall then be carefully removed and the entire exposed surface of the pile finished as above specified.

MEASUREMENT AND PAYMENT

71.43. Payment for Timber Piles: Payment for timber piles shall include the cost of furnishing all materials, including collars, equipment, labor and other items necessary for driving and cutting off such piles as are required. It shall also include the placing, but not furnishing, of all permanent bracing and caps which may be required and the furnishing and placing of any temporary bracing necessary to hold the piles in alignment. Metal shoes, when required, shall be paid for as a separate item or as extra work.

The number of linear feet paid for shall be the actual number of feet remaining in the finished structure. In addition to this, payments based on the actual cost of materials, shall be allowed for that portion of any cutoff in excess of two (2) feet. No payment shall be made for two (2) feet of any cutoff. Cut-off lengths shall become the property of the state. No allowance will be made for false-work piling.

71.44. Payment for Concrete Piles:

(1) Cast-in-Place Piles: Payment for cast-in-place concrete piles shall include the cost of furnishing all materials, equipment, labor and other items necessary for driving and casting the piles. The number of linear feet paid for shall be the actual number of linear feet remaining in the finished structure.

(2) Precast Piles: Payment for precast concrete piles shall include the cost of furnishing all materials, equipment, labor and other items necessary for casting, curing and driving the piles as ordered. Payment for precast piles will be made on the basis of the actual number of linear feet ordered by the Engineer from which a deduction of one-half the contract price per linear foot will be made for the length of piling cut off. In case “build ups” are necessary the built up length will be paid for at the contract price per linear foot for piles in place but no allowance will be made for “build ups” which are made necessary by damage to the piles during driving.
71.45. **Payment for Piles Ordered and Not Driven:** Piles ordered and not driven shall be paid for on the basis of cost plus five (5) per cent and shall thereupon become the property of the state.

71.46. **Payment for Test Piles:** Test piles ordered by the Engineer shall be paid for as follows:

If the test indicates that piles should be used and if they are used, the test piles will be paid for as provided hereinabove for other piles. If, however, piling is not used in the structure, the test piles will be paid for as provided for extra work, due consideration being given to the cost of bringing the driver to the site and removing it from the work.

71.47. **Payment for Loading Tests:** Payment for loading tests shall include the cost of all materials, equipment and labor incidental to constructing the loading platform, procuring and placing the loading material, and removing and disposing of the platform and material to the satisfaction of the Engineer. Payment shall be made on the basis of the contract price for pile loading tests or, in the absence of such a price, shall be made on the basis of extra work.

Payment will be made under:
- Item No. 76, Timber Piling Treated (per linear foot).
- Item No. 77, Timber Piling Untreated (per linear foot).
- Item No. 78, Concrete Cast-in-Place Piles (per linear foot).
- Item No. 79, Concrete Precast Piles (per linear foot).
- Item No. 80, Loading Test (each).
SECTION 72

CONCRETE CULVERTS-RETAINING WALLS AND END WALLS

72.01. Description: All concrete culverts or structures twenty (20) feet or less (measured along the center line of road), all pipe culverts, end-walls, and retaining walls, shall be built as indicated on the plans, conforming to line, grade, dimensions, and design shown, and in accordance with the Specifications for "Concrete Structures" and "Pipe" of the several varieties, which are to constitute the complete structures.

MATERIALS

72.02. Materials: Materials used shall be those prescribed for the several items which constitute the structure.

CONSTRUCTION METHODS

72.03. General: The construction methods used shall be those prescribed for the several items which are to constitute the structure.

72.04. Foundations: All foundations shall be prepared as hereinbefore specified under “Excavation for Bridges, Culverts, and Retaining Walls” and they shall be inspected and approved by the Engineer previous to placing any concrete. All foundations shall be poured in the "Dry" except as provided in the Special Provisions or unless otherwise permitted by the Engineer in writing.

When footings can be placed in the dry without the use of cribs or cofferdams, backforms may be omitted at the discretion of the Engineer and the entire excavation filled with concrete to the required elevation of the top of the footing.

Whenever the natural foundation material is insufficient to safely support the structure, or whenever excessive erosion during the flood periods is to be expected, the Engineer may direct that piles be driven under the culvert. In this case the piling shall be proportioned to carry the entire load in accordance with the provisions and formulas governing the use of piling as set forth in these Specifications.

Timber grillage work may also be used to distribute the bearing area for culverts in very soft natural ground provided the location is such that no erosive action need be feared. Timber grillage of this character must be placed so as to lie below the lowest natural ground water level and shall be built as approved by the Engineer.

72.05. Construction Joints: Concrete in substructures shall be placed in such manner that all construction joints will be truly horizontal and, if possible, in such location as not to be exposed to view in the finished structure. Special care shall be taken to avoid construction joints through parallel wing-walls or any large surface which are to be treated architecturally.

72.06. Aprons and Curtain Walls: Aprons or curtain walls shall, in general, be carried down at both ends to the depth shown on the plans, but may be ordered by the Engineer to such additional depth as may appear necessary to prevent undermining.

If deemed necessary, in order to prevent scour, the Engineer may direct that the space between the wings be paved. In this event, the apron walls will extend in a straight line between the ends of wings, or at such location as may afford the best protection.

72.07. Drainage: Adequate drainage of fills around culverts shall be insured by the construction of weep holes, French drains, or underdrains as may be indicated on the plans or directed by the Engineer.

72.08. Class of Concrete: Unless otherwise specified, all reinforced concrete culverts shall be of Class "A" concrete. Headwalls shall be built of the class of concrete indicated on the plans.

72.09. Method of Placing Concrete: Each wing wall above top of footing shall be constructed as a monolith.

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When placing the bottom slab, suitable provision shall be made for bonding the sidewalls to the culvert base, preferably by means of longitudinal keys so constructed as to prevent, as far as possible, the percolation of water through the construction joint.

Before concrete is placed in the sidewalls, the bottom slab shall be thoroughly cleaned of all shavings, sticks, sawdust, or other extraneous material and the surface carefully chipped and roughened in order to insure a good bond as required by these Specifications.

72.10. Pipe Culverts: Pipe culverts under the roadbed shall be so placed that the minimum distance from finished grade of roadway to the top of pipe shall be as provided on the plans or ordered by the Engineer.

Construction methods applying to pipe culverts shall be as set forth under the several items covering the different kinds of pipe specified.

72.11. Headwalls: The ends of all pipe culverts shall be protected by concrete or masonry end walls unless otherwise ordered by the Engineer.

72.12. Method of Measurement: The quantities of the various items which constitute the completed and accepted structures will be measured for payment according to the plans and specifications for the several items. Only accepted work will be included and the dimensions used will be the neat dimensions shown on the plans or ordered in writing.

72.13. Basis of Payment: The measured quantities as provided above, will be paid for at the contract unit prices for the several items which prices shall be full compensation for furnishing, hauling and placing all material, all labor, equipment, tools and necessary incidentals. Such payment shall constitute full payment for the completed structure ready for use. Whenever the construction of the new structures involves the removal or demolition of an existing structure, such removal or demolition shall not be paid for except as “Excavation for Structures.”
SECTION 73

RUBBLE MASONRY

(a) CEMENT RUBBLE

73.01. Description: This item shall be composed of approved stones laid in mortar beds and shall be constructed in accordance with these Specifications and in conformity with the plans or as directed by the Engineer.

MATERIALS

73.02. Stone: One-man and derrick stone used in rubble or cyclopean concrete shall consist of tough, sound and durable rock. The stone shall be free from coatings, drys, seams, or flaws of any character. In general, the percentage of wear shall be not greater than six (6) per cent as determined by the "Abrasion Test for Broken Stone" provided in Standard Methods of Sampling and Testing of the American Association of State Highway Officials (A. S. T. M. Standard Method, Serial Designation D2-08, slightly modified).

Preferably, stone shall be angular in shape and shall have a rough surface such as will thoroughly bond with the surrounding mortar.

Mortar: Mortar shall be of Portland cement and sand mixed in the proportions of one (1) part of cement to three (3) parts of sand. At the option of the contractor, hydrated lime in amount not to exceed ten (10) per cent of the amount of cement used, may be added to the mortar. Materials for this mortar shall meet the requirements specified under Section 67 Concrete Structures.

CONSTRUCTION METHODS

73.03. Construction Methods: The manner of laying shall conform to the following requirements. The bottom or foundation courses shall be composed of selected large, flat stones and all courses shall be laid with bearing beds parallel to the natural bed of the material.

All hammering or tooling of the stone must be done before the stone is laid in the wall and no further dressing or tooling will be permitted after the stone is placed.

All stone shall be thoroughly saturated with water before being laid.

The mortar joints shall be full and the stone carefully settled in the mortar. No spalls will be allowed in the beds.

If during the course of construction the bond is broken between any stone and the surrounding mortar, the stone shall be removed and the mortar entirely cut out and replaced when the stone is relaid.

No stone less than six (6) inches in thickness and twelve (12) inches in width shall be used, except for filling the interior of the wall.

Stones shall decrease in thickness from bottom to top.

Stones must be carefully bonded and leveled and so selected that at least twenty (20) per cent of the stones are headers. Such headers shall be evenly distributed throughout the surface of the wall and preferably arranged to interlock. On walls 2"—0" or less in thickness, headers shall extend clear through.

Selected stone, roughly squared and pitched to line shall be used for angles and ends of walls. If required all corners or angles for exterior surfaces shall be finished with a chisel draft one and one-half (1 1/2) inches in width.

The minimum thickness of mortar between any two (2) stones shall be one-half (1/2) inch.

Vertical joints on the faces of walls shall be broken at least six (6) inches. In no case shall a vertical joint occur directly above or below a header.
The contractor shall construct weep holes where called for on the plans. No extra compensation will be rendered for the construction of weep holes, the cost of all labor material incident to the same being presumably included in the price bid for masonry.

Unless otherwise specified upon the plans, Copings, Bridge Seats, and Back-walls shall be constructed of Class “A” concrete, Section 67, paragraph 67.26, and shall conform to the requirements for concrete structures Section 67.

Concrete Copings shall be made in sections extending the full length of the wall, not less than eight (8) inches in thickness, and from five (5) to ten (10) feet long. The sections may be cast in place or pre-cast and set in place in full mortar beds.

73.04. (b) DRY RUBBLE

73.05. Description: This item shall be composed of approved stones laid without mortar, and shall be constructed in accordance with these specifications and in conformity with the plans or as directed by the Engineer.

73.06. Stone: Stone for this class of work may be of any material which is sound, durable, free from segregations, rifts or seams or any other defect operating to destroy its natural resistance to the weather.

CONSTRUCTION METHODS

73.07. Construction of Methods: All bottom or foundation courses shall be composed of selected, large, flat stone and all courses shall be laid with bearing beds parallel to the natural bed of the material. All hammering or tooling of the stone must be done before the stone is laid in the wall. All stone shall be carefully fitted in such manner as to reduce the maximum width of any open joint to not more than one (1) inch. No stone less than six (6) inches in thickness and twelve (12) inches in width shall be used except for filling the interior of the wall.

Stone shall decrease in thickness from bottom to top, and shall be carefully bonded and leveled and so selected that at least twenty (20) per cent of the stone are headers. Headers shall be evenly distributed throughout the surface of the wall and preferably arranged to interlock. Selected stone roughly squared and pitched to line shall be used for angles, or for the ends of walls. Vertical joints on all wall faces shall be broken at least four (4) inches.

73.08. Basis of Payment: These items will be paid for at the contract unit price per cubic yard for “Cement Rubble Masonry” or “Dry Rubble Masonry,” as the case may be, complete in place, which price shall be full compensation for all material equipment, tools, labor and incidentals necessary to complete the item in accordance with the plans and these specifications. In determining the volume of masonry structures the plan dimensions will be used and no allowance will be made for work done outside the neat lines indicated on the drawings.

Payment will be made under

Item No. 81, Cement Rubble Masonry (cubic yard).
Item No. 82, Dry Rubble Masonry (cubic yard).
SECTION 74

BRICK MASONRY

74.01. Description: Brick masonry shall consist of brick laid in cement mortar and shall include such construction with building brick or ornamental brick as may be specified. Brick pavements are not included under this designation.

MATERIALS.

74.02. Brick: Brick for masonry construction shall conform to the requirements of the Standard Specifications for Building Brick, Serial Designation C-21-20, of the American Society for Testing Materials, with subsequent amendments and additions thereto adopted by the Society. Unless otherwise specified, all bricks shall conform to the class requirements for "Hard Brick".

The bricks shall have a fine grained, uniform and dense structure, free from lumps of lime, laminations, cracks, checks, soluble salts or other defects which may in any way impair their strength, durability, appearance or usefulness for the purpose intended. Bricks shall emit a clear, metallic ring when struck with a hammer.

74.03. Mortar: Mortar shall be of Portland Cement and sand, mixed in the proportions of one part of cement to three parts of sand. At the option of the contractor hydrated lime in amount not to exceed 10 per cent of the amount of cement used, may be added to the mortar. Materials for this mortar shall meet the requirements specified in Section 67 for "Concrete Structures."

CONSTRUCTION METHODS

74.04. Laying: The brick shall be laid using the "shove-joint" method, so as to thoroughly bond them into the mortar. All brick shall be saturated with water before being laid, and headers and stretchers shall be so arranged as to thoroughly bond the mass. Unless otherwise specified brick work shall be of alternate headers and stretchers with alternate courses breaking joints, (Flemish Bond). Joints shall be finished properly as the work progresses, and shall be not less than one-quarter (1/4) of an inch nor more than one-half (1/2) of an inch in thickness. No spalls nor bats shall be used, except shaping around irregular openings or when unavoidable at corners. None but expert brick layers shall be employed on the work.

74.05. Basis of Payment: This item will be paid for at the contract unit price either by the cubic yard of "Brick Masonry", or by the thousand brick laid. (The method of payment will be set forth in the proposal), complete in place, which price shall be full compensation for all equipment, tools, labor, materials, scaffolding and other things necessary to complete the item in accordance with the plans and these specifications. Filling materials for the interior of the wall, when not of brick, and concrete or mortar copings, shall be paid for on the basis of the number of cubic yards actually placed.

Payment will be made under
Item No. 88 (a)—Brick Masonry (per cubic yard).
Item No. 83 (b)—Brick Masonry (per thousand brick laid).
SECTION 87

GENERAL FEATURES

TYPES AND CLASSIFICATION OF BRIDGES

87.01. (a) Types of Bridges:

The different types of bridges may be used within the following limits, due consideration being given to transportation and erection conditions in selecting the type to be used.

**Steel Structures:**
- Rolled beams for spans up to 45 feet
- Plate girders for spans up to 30 feet
- Riveted half-through trusses for spans from 45 to 100 feet
- Riveted trusses for spans above 90 feet
- Pin-connected trusses for spans above 150 feet

**Concrete and Stone Masonry Structures:**
- Slab spans, up to 24 feet
- Simple girder spans up to 18 to 60 feet
- Arches up to All span lengths

87.02. (b) Classification of Bridges:

The classification of bridges with reference to traffic shall be as follows:

**Class AA:** Bridges for specially heavy traffic units in locations where the passage of such loads is frequent.

**Class A:** Bridges for normally heavy traffic units and the occasional passage of specially heavy loads.

**Class B:** Bridges for light traffic units and the occasional passage of normally heavy loads. Class B bridges shall be considered as temporary or semi-temporary structures.

**Class D:** Bridges for electric railway traffic in addition to highway traffic. The latter may correspond to any one of the classes described above.

87.03. Clearance: Bridge construction for highway traffic only shall not encroach on the space indicated by the clearance diagram shown in Figure 1. On structures carrying electric railways the clearance shall be increased to meet the requirements of the case, but the minimum distance from center of track to structure shall be six (6) feet.

87.04. Width of Roadway: The minimum width of roadway shall be nine (9) feet for each line of traffic with a minimum width of twelve (12) feet for one line of traffic.

In structures on curves and carrying electric railways the clearance shall be increased where necessary to provide for curvature and super-elevation of rail.

87.05. Curbs: The projection of the curb, measured from that portion of the rail nearest the roadway, shall be not less than six (6) inches and preferably shall be not less than nine (9) inches. The curb height shall be not less than eight (8) inches above the adjacent finished roadway surface.

Concrete curbs shall be designed to resist a lateral force of not less than five hundred (500) pounds per linear foot of curb, applied at the top of the curb.

87.06. Railings: Substantial railings shall be provided along each side of the bridge for the protection of traffic. Preferably, the top of railing shall be not less than three (3) feet above the finished surface of the roadway adjacent to the curb and, when on a sidewalk, shall be not less than three (3) feet above the sidewalk floor.

In general, railings shall be of two classes, as follows:

1. Railings suitable for use on country bridges which are not subject to general pedestrian traffic.
2. Railings for the protection of pedestrians on bridges in cities and villages.

(a) **Metal Railings**: Metal railings shall be designed to resist a horizontal force of not less than one hundred (100) pounds per linear foot, applied at the top of the rail, and a vertical force of not less than one hundred (100) pounds per linear foot.

Metal railings of the first class may consist of not less than two lines of horizontal rails of approved section.

Metal railings of the second class shall consist of an upper and a lower horizontal rail connected by a suitable web. The clear distance between the top of curb or sidewalk and the lower rail shall not exceed six (6) inches.

In each connection of railing to the posts, truss members, etc., there shall be not less than two rivets or bolts each. Ample provision shall be made for movement due to temperature.

(b) **Concrete Railings**: Concrete railings shall be designated to resist a horizontal force of not less than one hundred and fifty (150) pounds per linear foot, applied at the top of the rail, and a vertical force of one hundred (100) pounds per linear foot.

Openings in concrete railings of the second class shall be proportioned with due regard to the safety of persons using the structure.

Provisions shall be made for the expansion and contraction of concrete railings at intervals consistent with the design.

87.07. **Drainage**: The transverse drainage of roadways shall be secured by means of a suitable crown in the roadway surface. Longitudinal drainage shall be secured by means of scuppers or drains of ample size, constructed in the gutters or curbs at suitable intervals. The details of floor drains shall be such as to prevent the discharge of drainage water against any portion of the structure. Overhanging details in concrete and timber floors preferably shall be provided with drip beads.
SECTION 88.

LOADS

88.01. **Loads:** Structures shall be proportioned for the following loads and forces:

(a) Dead Load.
(b) Live Load.
(c) Impact or dynamic effect of the live load.
(d) Lateral forces.
(e) Other forces, when these exist, as follows:
   - Longitudinal force; centrifugal force; force of stream current and drift; earth pressure; and the thermal forces.

Members shall be proportioned for that combination of loads and forces producing the maximum total stress, except as otherwise provided.

Upon the stress sheets for steel structures a diagram of the assumed live loads shall be shown and the stresses due to the various loads shall be shown separately. For structures of material other than steel, the assumed live loads shall be shown upon stress sheets or other drawings and, when required, the stresses due to the various loads shall be shown.

88.02. **Dead Load:** The dead load shall consist of the weight of the structure complete, including the weight of the roadway floor together with car tracks, pipes, conduits, cables, or other public utility services.

In the case of structures having concrete slab floors, an adequate allowance shall be made in the design dead load to provide for the weight of a suitable wearing surface. This allowance will depend upon the type of wearing surface contemplated; it shall be in addition to the weight of any monolithically placed concrete wearing surface; and shall be not less than fifteen (15) pounds per square foot of roadway.

The following weights are to be used in computing the dead load:

<table>
<thead>
<tr>
<th>Substance</th>
<th>Weight per cubic foot</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>490</td>
</tr>
<tr>
<td>Iron, cast</td>
<td>450</td>
</tr>
<tr>
<td>Bronze</td>
<td>524</td>
</tr>
<tr>
<td>Timber (treated or untreated)</td>
<td>60</td>
</tr>
<tr>
<td>Concrete</td>
<td>144</td>
</tr>
<tr>
<td>Loose sand and earth</td>
<td>100</td>
</tr>
<tr>
<td>Rammed sand or gravel and ballast</td>
<td>120</td>
</tr>
<tr>
<td>Macadam or gravel, rolled</td>
<td>140</td>
</tr>
<tr>
<td>Cinder filling</td>
<td>60</td>
</tr>
<tr>
<td>Pavement, other than wood block</td>
<td>150</td>
</tr>
<tr>
<td>Railway rails and fastenings</td>
<td>150 lbs. per ft. of track</td>
</tr>
</tbody>
</table>

88.03. **Live Load. Highway Live Loads:** The highway live load on the roadway portion of the bridge structure shall consist of trains of motor trucks, or equivalent loads, as hereinafter specified. Each loading is designated by the letter, H., followed by a numeral indicating the gross weight in tons of the heaviest truck involved in the loading.

88.04. **Traffic Lanes:** The truck trains or equivalent loads shall be assumed to occupy traffic lanes, each having a width of none (9) feet corresponding to the standard truck clearance width. Within the curb to curb width of roadway the traffic lanes shall be assumed to occupy any position, not involving overlapping of adjacent lanes, which will produce maximum stress. The extreme position of a lane with reference to the curb shall be that in which its outside clearance is in the vertical plane passing through the roadway edge of the curb.

88.05. **Standard Trucks:** The wheel spacing, weight distribution and clearance of the standard trucks used for design purposes shall be shown in Figure 2. As used in this specification, the weight of a truck indicates its gross loaded weight. Trucks in trains shall be spaced as shown in Figure 3.
88.06. **Standard Loadings:** The standard loadings shall be of three classes: namely, H20, H15, and H10, and may be either truck train loadings or equivalent loadings. Loadings H15 and H10 are 75% and 50%, respectively, of Loading H20.

(2) **Truck Train Loadings:** The truck train loading shall be as shown in Figure 3 and shall consist of one truck of the gross weight indicated by the loading classification, followed by or preceded by or both followed and preceded by a line of trucks of indefinite length, each of the following or preceding trucks having a gross weight of three-fourths of the gross weight indicated by the loading classification.

Trucks in adjacent lanes shall be considered as headed in the same direction.

(b) **Equivalent Loadings:** Equivalent loadings shall be used only for span lengths of sixty (60) feet or more. For span lengths of less than sixty (60) feet, the truck train loadings shall be used. The equivalent loading shall be as shown in Figure 4 and shall consist of a uniform load per linear foot of traffic lane combined with a single concentrated load so located longitudinally in the lane as to produce maximum stress. The concentrated load shall be considered as uniformly distributed across the lane on a line normal to the direction of the lane. For the computation of moments and shears different concentrated loads shall be used as indicated in Fig. 4.

88.07. **Selection of Loadings:** For the different classes of bridges the classes of loadings shall be as follows:

<table>
<thead>
<tr>
<th>Class of Bridge</th>
<th>Class of Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>AA</td>
<td>H20</td>
</tr>
<tr>
<td>A</td>
<td>H15</td>
</tr>
<tr>
<td>B</td>
<td>H10</td>
</tr>
</tbody>
</table>

88.08. **Application of Loads:** The loadings shall be applied in such manner as to produce maximum stress by either of the following two methods, due consideration being given to the reduced load intensities hereinafter specified for roadways wider than eighteen (18) feet.

1. Each traffic lane loading shall be considered as a unit and the number and position of the loaded lanes shall be such as will produce maximum stress.

2. The roadway shall be considered as uniformly loaded over its entire width with a load per foot of width equal to one-ninth (1/9) of the load of one traffic lane.

88.09. **Reduction in Load Intensity:** When the loaded width of roadway exceeds eighteen (18) feet the specified loads shall be reduced one per cent (1%) for each foot of loaded roadway width in excess of eighteen (18) feet with a maximum reduction of twenty-five per cent (25%) corresponding to a loaded roadway width of forty-three (43) feet. When the loads are lane loads the loaded width of roadway shall be the aggregate width of the lanes considered; when the loads are uniformly distributed over the entire width of roadway, the loaded width of roadway shall be the full width of roadway between curbs.

88.10. **Electric Railway Loads:** When highway bridges carry electric railway traffic, the railway loading shall be determined on the bases of the class of traffic which may be expected to operate. The possibility that the freight rolling stock of steam railroads may be operated shall be given consideration.

When not otherwise specified, the electric railway loading on each track shall be a train of two electric cars followed by, or preceded by, or both followed and preceded by, a uniform load. The cars shall be one of the classes shown in Figure 5. These cars are designated by numerals indicating the total loaded weight of each car. The uniform load per foot of track following or preceding electric cars shall be the uniform load corresponding to the class of highway loading specified six hundred forty (640) pounds per linear foot for H20 Loading. The portion of the roadway width assumed to be occupied by the railway loading shall have a width of ten (10) feet.

For freight car loading, the typical cars shown in Figure 6 may be assumed in the absence of more exact data.

The railway loading used shall be shown on the stress sheets.

Highway bridges carrying electric railway traffic shall be designed for the following loading conditions:
Width of each rear tire equals 1" per ton of total weight of loaded truck.

**STANDARD TRUCK**

Fig. 2

**TRUCK-TRAIN LOADING**

Fig. 3

**H-20 LOADING**
- Concentrated load \(18,000\) for Moment
- Uniform load \(640\) per linear foot of lane

**H-15 LOADING**
- Concentrated load \(13,500\) for Moment
- Uniform load \(480\) per linear foot of lane

**H-10 LOADING**
- Concentrated load \(9,000\) for Moment
- Uniform load \(320\) per linear foot of lane

**EQUIVALENT LOADING**

Fig. 4
### ELECTRIC RAILWAY LOADING

**Fig. 5**

<table>
<thead>
<tr>
<th>Ton Cars</th>
<th>Length</th>
<th>Uniform Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>80</td>
<td>58'</td>
<td>Uniform Load</td>
</tr>
<tr>
<td>60</td>
<td>53'</td>
<td>Uniform Load</td>
</tr>
<tr>
<td>40</td>
<td>46'</td>
<td>Uniform Load</td>
</tr>
<tr>
<td>30</td>
<td>44'</td>
<td>Uniform Load</td>
</tr>
<tr>
<td>20</td>
<td>40'</td>
<td>Uniform Load</td>
</tr>
</tbody>
</table>

### TYPICAL FREIGHT CARS

Total Loading Weight per car, including 10% overload

- 40 Ton Capacity: 128,000#
- 50 Ton Capacity: 152,000#
- 60 Ton Capacity: 176,000#
- 70 Ton Capacity: 210,000#
- 80 Ton Capacity: 224,000#

**Fig. 6**
1. The highway loads upon any portion of the roadway area including that portion occupied by the railway.

2. The electric railway loads on the car tracks and the highway loads on the remaining traffic lanes.

88.11. **Sidewalk Loading:** All sidewalk floors, stringers and their immediate supports shall be designed to support a live load of not less than one hundred (100) pounds per square foot of sidewalk area.

Girders or trusses of highway bridges also supporting sidewalks shall be designed for a sidewalk live load determined by the following formula:

\[
P = \left( \frac{3000}{L} \right) \left( \frac{(55 - W)}{50} \right)
\]

Where

- \( P \) = live load in pounds per square foot of sidewalk area, but not to exceed, one hundred (100) pounds per square foot.
- \( L \) = loaded length of sidewalk in feet.
- \( W \) = width of sidewalk in feet.

No impact increment shall be added to sidewalk loads.

Foot bridges shall be designed for a live load of one hundred (100) pounds per square foot without impact.

In calculating stresses in structures which support cantilevered sidewalks, the sidewalk shall be considered as fully loaded on one side of the structure only, when this condition produces maximum stress.

88.12. 4. **Impact:** Except in timber structures, all live load stresses, except those due to sidewalk loads and centrifugal, tractive and wind forces, shall be increased by an allowance for dynamic, vibratory and impact effects.

The amount of this allowance or increment is expressed as a fraction of the live load stress and for both electric railway and highway loadings is determined by the formula:

\[
I = \frac{50}{L + 125}
\]

Where \( I \) = impact fraction.

\( L \) = the length in feet of the portion of the span which is loaded to produce the maximum stress in the member.

88.13. 5. **Longitudinal Force:** Provision shall be made for the effect of a longitudinal force of ten (10%) per cent of the live load on the structure, applied four feet above the floor.

In viaduct towers the sections of members of longitudinal bracing shall be not less than those of members in corresponding panels of the transverse bracing.

88.14. **Lateral Forces:**

(a) The wind force on the structure shall be assumed as a moving horizontal load equal to thirty (30) pounds per square foot on one and one-half \((1\frac{1}{2})\) times the area of the structure as seen in elevation including the floor system and railing, and on one-half \((\frac{1}{2})\) the area of all trusses or girders in excess of two in the span.

(b) The lateral force due to the moving live load and the wind pressure against it, shall be considered as applied six (6) feet above the roadway and shall be as follows:

- Highway bridges, 200 pounds per linear foot.
- Highway bridges carrying electric railway traffic, 300 pounds per linear foot.

(c) The total assumed wind force shall be not less than three hundred (300) pounds per linear foot in the plane of the loaded chord and one hundred and fifty (150) pounds per linear foot, in the plane of the unloaded chord in truss spans, and not less than three hundred (300) pounds per linear foot of span in girder spans.
(d) In calculating the uplift, due to the above lateral forces, in the posts and anchorages of viaduct towers, highway viaducts shall be considered as loaded on the leeward traffic lane with a uniform load of four hundred (400) pounds per linear foot of lane and viaducts carrying electric railway traffic in addition to highway traffic shall be considered as loaded on the leeward track with a uniform load of eight hundred (800) pounds per linear foot of track.

(e) A wind pressure of fifty (50) pounds per square foot on the unloaded structure, applied as specified above in Paragraph (a), shall be used when it produces greater stresses than the combined wind and lateral forces of Paragraphs (a) and (b).

88.15. **Centrifugal Force:** Structures carrying electric railway traffic on a curve shall be designed to resist a lateral force equivalent to ten (10%) per cent of the moving railway loads, without impact; and this lateral force shall be considered as applied four (4) feet above the top of the rail.

88.16. **Forces of Stream Current and Drift:** All piers and other portions of structures which are subjected to the force of flowing water or drift shall be designed to resist the maximum stresses induced thereby.

88.17. **Pressure from Retained Material:** Structures designed to retain fills shall be proportioned to withstand pressures as given by Rankine's formula, provided, however, that no structure shall be designed for an equivalent fluid pressure of less than thirty (30) pounds per cubic foot. The above dead load pressure shall be increased to provide for any live load surcharge which may exist. All designs shall provide for the thorough drainage of backfilling material by means of weep holes with French drains, or other suitable means.

88.18. **Thermal Forces:** In fixed arched spans, provision shall be made for the stresses resulting from the following variations in temperature:

(a) **Metal Structures:** Moderate climate, from 0 degrees to +120 degrees Fahr.

The rise and fall in temperature shall be figured from an assumed mean temperature at time of erection.

(b) **Concrete Structures:** Moderate climate, 30 degrees F. rise. 40 degrees F. fall.
SECTION 89.

UNIT STRESSES

89.01. General: Except as otherwise provided herein, the several parts of a structure shall be so proportioned that the unit stresses will not exceed the following. Unless otherwise noted, all unit stresses are given in pounds per square inch.

89.02. STEEL STRUCTURES

Structural Grade and Rivet Steel:

(a) Tension:
   Axial tension, structural members, net section ........................................ 16,000
   Rivets in tension, where permitted .......................................................... 50% of single shear values
   Bolts, area at root of thread ......................................................................... 10,000

(b) Axial Compression:
   Axial compression, gross section ...................................................................
   \[\text{but not to exceed the value of } L/r = 40.\]
   \[L = \text{length of member, in inches.}\]
   \[r = \text{least radius of gyration, in inches.}\]

(c) Bending on Extreme Fiber:
   Rolled shapes, built sections and girders, net section .................................. 16,000
   Pins ............................................................................................................. 24,000
   Compression in flanges of beams and plate girders ...................................
   \[1 + \frac{1}{2000}\left(\frac{L}{b}\right)^2\]
   \[L = \text{length, in inches of the unsupported flange between lateral connections of knee braces.}\]
   \[b = \text{flange width, in inches.}\]

(d) Shear:
   Girder webs, gross section ........................................................................... 10,000
   Pins and shop driven rivets ................................................................ .......... 12,000
   Power driven field rivets and turned bolts .......................................................
   Hand driven rivets and unfinished bolts ........................................................ 8,000

(e) Bearing:
   Pins, steel parts in contact and shop driven rivets ...........................................
   Power driven field rivets and turned bolts .....................................................
   Hand driven rivets and unfinished bolts ....................................................... 16,000
   Expansion rollers, pounds per linear inch where d = diameter of roller in inches 
   .................................................................................................................. 600d

(f) Countersunk Rivets:
   In metal three-eighths (3/8) inch thick and over, half the depth of countersink shall be omitted calculating bearing area.
   In metal less than three-eighths (3/8) inch thick, countersunk rivets shall not be assumed to carry stress.

(g) Diagonal Tension:
   In webs of girders and rolled beams, at sections where maximum shear and bending occur simultaneously .............................................. 16,000
89.03. OTHER METALS

(a) Axial Tension:
   Wrought-Iron ........................................... 12,000

(b) Bending on Extreme Fiber:
   Cast Steel ............................................. 12,000
   Cast Iron ............................................. 3,000

(c) Shear:
   Cast Steel ............................................. 10,000
   Cast Iron ............................................. 3,000

(d) Bearing:
   Cast Steel ............................................. 14,000
   Cast Iron ............................................. 10,000
   Bronze sliding expansion bearings .................. 3,000

89.04. Bearing on Bridge Seats:
   Bearing on concrete masonry, limestone masonry and better ............... 500

CONCRETE STRUCTURES

89.05. Concrete.

89.06. (a) Direct Compression: Columns reinforced with longitudinal bars and separate lateral ties
   ........................................................................... 600-15L/D
   ........................................................................... 450
   Where L = unsupported length of column.
   D = least diameter of column.
   Piers and Pedestals .......................................... 450

(b) Compression Due to Bending:
   Beams and Slabs ........................................... 650
   Arch Rings, including temperature and rib shortening ....................... 1,000

(c) Tension ....................................................... Zero

(d) Shear (Diagonal Tension):
   Beams without shear reinforcement, Longitudinal bars not anchored 40
   Longitudinal bars anchored ................................... 60
   Beams with shear reinforcement .................................. 120

(e) Punching Shear ........................................... 120

89.07. Reinforcement.

(a) Tension:
   Beams and slabs ........................................... 16,000
   Arch rings, including temperature and rib shortening ....................... 20,000

(b) Compression: Fifteen (15) times stress is surrounding concrete.

(c) Bond:
   Bars not anchored .......................................... 80
   Bars adequately anchored by hooks or otherwise ................................ 120

Note: The above allowable unit stress values for concrete and for bond on reinforcement are based on an ultimate compressive strength value of twenty-two hundred (2200) pounds per square inch at the age of 28 days when tested in accordance with the Tentative Standard Method of Sampling and Testing of the American Association of State Highway Officials.
When the aggregate used and the laboratory and field control of concrete mixture are such that a uniformly higher ultimate compressive strength is insured, then the above unit stresses may be increased by a maximum of fifteen per cent (15%) for concrete having an ultimate strength of twenty-eight hundred (2800) pounds per square inch or more at the age of 28 days and proportionally for intermediate values of ultimate strength, except that the stress due to bending in arch ribs when the effects of temperature and rib shortening are included shall in no case exceed one thousand (1000) pounds per square inch. When economy may be obtained by the use of aggregates which will result in concrete having a lower ultimate strength than twenty-two hundred (2200) pounds per square inch at the age of 28 days, the above unit stresses shall be reduced in the proportion that the lower ultimate strength in pounds per square inch bears to 2200.

89.08. **Bearing Power of Soils:** For the design of foundations, the following unit bearing values may be assumed in the absence of definite information as to the actual bearing power of the foundation in question. In this tabulation it is intended to cover only broad basic groups of materials and to specify for these a maximum range in bearing power. These groups may be further subdivided to provide for special conditions.

<table>
<thead>
<tr>
<th>Material</th>
<th>Min.</th>
<th>Max.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alluvial soils</td>
<td>½</td>
<td>1</td>
</tr>
<tr>
<td>Clays</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>Sand, confined</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>Gravel</td>
<td>2</td>
<td>4</td>
</tr>
<tr>
<td>Cemented sand and gravel</td>
<td>5</td>
<td>10</td>
</tr>
<tr>
<td>Rock</td>
<td>5</td>
<td>10</td>
</tr>
</tbody>
</table>

89.09. **TIMBER STRUCTURES**

89.10. **Structural Grades of Timber:**

(a) The following unit stresses for structural grades of timber are for use with computed stresses which contain no allowance for live load impact:

<table>
<thead>
<tr>
<th>Species of Wood</th>
<th>Axial tension and bending in extreme fiber</th>
<th>Compression parallel to grain short columns Cs</th>
<th>Compression perpendicular to grain all grades Cp</th>
<th>Horizontal shear in beams</th>
<th>Ultimate modulus of elasticity all grades</th>
</tr>
</thead>
<tbody>
<tr>
<td>Select</td>
<td>Common</td>
<td>Select</td>
<td>Common</td>
<td>Select</td>
<td>Common</td>
</tr>
<tr>
<td>Cypress, Southern Red or Black</td>
<td>1200</td>
<td>900</td>
<td>1000</td>
<td>800</td>
<td>250</td>
</tr>
<tr>
<td>Gum, Black</td>
<td>900</td>
<td>750</td>
<td>750</td>
<td>625</td>
<td>200</td>
</tr>
<tr>
<td>Oak, Red and White</td>
<td>1200</td>
<td>1000</td>
<td>900</td>
<td>750</td>
<td>375</td>
</tr>
<tr>
<td>Pine, Southern Yellow</td>
<td>1400</td>
<td>1100</td>
<td>1000</td>
<td>800</td>
<td>225</td>
</tr>
<tr>
<td>Pine, Southern Yellow Dense Select</td>
<td>1600</td>
<td>1150</td>
<td>275</td>
<td></td>
<td>110</td>
</tr>
</tbody>
</table>
Values for Direct Shear parallel to Grain in details of joints may be taken greater than the values for Horizontal Shear in Beams.

(b) **Axial Compression in Timber Columns:**

\[ p = \frac{4}{3} \left( L \left( 1 - \frac{L}{40D} \right) \right) \]

The value of "p" shall not exceed the value of "Cs."
- \( p \) = Unit compressive stress in column.
- \( C_s \) = Unit stress for compression parallel to grain in short columns.
- \( L \) = Unsupported length of column.
- \( D \) = Least diameter of column.

(c) **Bearing on Inclined Surfaces:**

\[ p = C_p + (C_s - C_p) \frac{\tan^2 \theta}{8100} \]

\( p \) = Unit bearing stress on inclined surface.
- \( C_p \) = Unit stress for compression perpendicular to grain.
- \( C_s \) = Unit stress for compression parallel to grain in short columns.
- \( \theta \) = Angle in degrees between bearing surface and direction of fibers (or axis of piece.)

(d) **Horizontal Shear in Beams:**

Horizontal shear in beams shall be computed from the maximum shear occurring at a distance from the support equal to three (3) times the depth of the beam.
SECTION 90
DISTRIBUTION OF LOADS

90.01. **Through Earth Fills:** When the depth of fill is three (3) feet or more, concentrated loads shall be considered as uniformly distributed over a square, the sides of which are equal to one and three-fourths (1\(\frac{3}{4}\)) times the depth of fill. When such areas from several concentrations overlap, the total load shall be considered as uniformly distributed over the area defined by the outside limits of the individual areas, but the total width of distribution shall not exceed the total width of the supporting slab. For single spans the effect of live load may be neglected when the depth of fill is more than four (4) feet and exceeds the span length; for multiple spans it may be neglected when the depth of fill exceeds the distance between faces of end supports or abutments.

90.02. **In Concrete Slabs:** In calculating bending stresses due to wheel loads on concrete slabs, no distribution in the direction of the span of the slab shall be assumed. In the direction perpendicular to the span of the slab, the wheel load shall be considered as distributed uniformly over a width of slab which is known as the "effective width." In the following equations let

\[
\begin{align*}
S &= \text{span of slab in feet.} \\
W &= \text{width of wheel or tire in feet.} \\
X &= \text{distance in feet from the center of the nearest support to the center of wheel.} \\
E &= \text{effective width in feet for one wheel.}
\end{align*}
\]

**Case 1. Main Reinforcement Parallel to Direction of Traffic.**

\[E = 0.7 \times (S + W)\]

For this case the value of "E" shall not exceed seven (7) feet. When two wheels are so located on a transverse element of the slab that their effective widths overlap, the effective width for each wheel shall be \(\frac{1}{2} (E + a)\), where "a" is the distance between centers of wheels.

**Case 2. Main Reinforcement Perpendicular to Direction of Traffic.**

\[E = 0.7 \times (2x + W)\]

For this case the bending movement on a strip of slab one (1) foot in width shall be determined by placing the wheel loads in the position to produce maximum bending; determining the effective width for each wheel; and assuming the load delivered by each wheel to the one (1) foot strip to be the wheel load divided by its respective effective width. This design assumption does not provide for the effect of loads near unsupported edges. Therefore, at the ends of the bridge and at intermediate points where the continuity of the slab is broken, the edges of the slab shall be supported by diaphragms or other suitable means.

90.03. **In Flat Slabs Supported on Four Sides:** In the case of flat slabs supported along four edges and reinforced in both directions, the proportion of the load carried by the short span of the slab shall be assumed as given by the following equations:

**Load Uniformly Distributed**

\[p = \frac{b^4}{a^4 + b^4}\]

**Load Concentrated at Center**

\[p = \frac{b^4}{a^4 + b^4}\]

Where \(p = \) proportion of load carried by short span.

\(a = \) length of short span of slab.

\(b = \) length of long span of slab.

When the length of the slab exceeds one and one-half \((1\frac{1}{2})\) times its width, the entire load shall be assumed to be carried by the transverse reinforcement.

In placing the reinforcement in such slabs, consideration shall be given to the fact that the bending moment is greater near the center of the slab than near the edges. Also, in the design of the supporting beams, consideration shall be given to the fact that the loads delivered to the supporting beams are not uniformly distributed along the beams.
90.04. **In Longitudinal Beams or Stringers and in Transverse Floorbeams:**

90.05. **Shear:** In calculating end shears and end reaction in transverse floorbeams and longitudinal beams and stringers, no lateral or longitudinal distribution of the wheel load shall be assumed.

90.06. **Bending Movement in Longitudinal Beams or Stringers:** In calculating bending moments in longitudinal beams or stringers, no longitudinal distribution of the wheel loads shall be assumed. The lateral distribution shall be determined as follows:

(a) **Interior Stringers:** Interior stringers shall be proportioned for loads determined in accordance with the following table except that when the limiting stringer spacings are exceeded the stringer loads shall be determined by the reactions of the truck wheels, assuming the flooring between stringers to act as a simple beam.

<table>
<thead>
<tr>
<th>Kind of Floor</th>
<th>Floor designed for one truck</th>
<th>Floor designed for two trucks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fraction of a wheel load to</td>
<td>Limiting stringer spacing in</td>
</tr>
<tr>
<td></td>
<td>each stringer</td>
<td>Limiting stringer spacing in</td>
</tr>
<tr>
<td>Plank</td>
<td>S</td>
<td>4.0</td>
</tr>
<tr>
<td>Strip 4&quot; in thickness or wood block on 4&quot; plank sub-floor</td>
<td>S</td>
<td>4.5</td>
</tr>
<tr>
<td>Strip 6&quot; or more in thickness</td>
<td>S</td>
<td>5.0</td>
</tr>
<tr>
<td>Concrete</td>
<td>S</td>
<td>6.0</td>
</tr>
</tbody>
</table>

S = spacing of stringers in feet.

(b) **Outside Stringers:** The live load supported by outside stringers shall be the reaction of the truck wheels, assuming the flooring to act as a simple beam between stringers.

(c) **Total Capacity of Stringers:** The combined load capacity of the beams in a panel shall not be less than the total live and dead load in the panel.

90.07. **Bending Moment in Floorbeams:** In calculating bending moments in transverse floorbeams, no transverse distribution of the wheel loads shall be assumed.

**Distribution of Wheel Load on Floorbeams:** When longitudinal stringers are omitted and the floor is supported directly on the floorbeams, the latter shall be proportioned for a fraction of the wheel loads as indicated in the following table, except that when the limiting floorbeam spacing is exceeded the floorbeam loads shall be determined by the reactions of the truck wheels, assuming the flooring between floorbeams to act as a simple beam.
<table>
<thead>
<tr>
<th>Kind of Floor</th>
<th>Fraction of axle loads to each floor beam</th>
<th>Limiting floorbeam spacing in feet</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plank</td>
<td>S</td>
<td>4.0</td>
</tr>
<tr>
<td>Strip 4” in thickness or wood block on 4” plank sub-floor</td>
<td>S</td>
<td>4.5</td>
</tr>
<tr>
<td>Strip 6” or more in thickness</td>
<td>S</td>
<td>5.0</td>
</tr>
<tr>
<td>Concrete</td>
<td>S</td>
<td>6.0</td>
</tr>
</tbody>
</table>

90.08. Distribution of Electric Railway Wheel Loads: Electric railway wheel loads may be assumed to be uniformly distributed longitudinally over a length of three (3) feet. In case of ballasted floors, a lateral distribution of ten (10) feet for an axle load may be assumed.

90.09. Transmission of Dead Load of Fills to Culverts and Short Span Slabs: All culverts and short span slab structures carrying a superimposed earth fill shall be proportioned to carry the entire weight of all the filling material directly above the structure.
SECTION 91
SUBSTRUCTURES AND RETAINING WALLS

91.01. Piles:

(a) Use of Piling: In general, piling shall be used when footings cannot, at a reasonable expense, be founded on rock or other solid foundation material. In streams where erosion is possible, piling preferably shall be used (if possible to drive) as a protection against scour, even though the safe wearing resistance of the natural soil is sufficient to support the structure without piling.

(b) Design Loads: Preferably, structures shall be proportioned to limit the maximum design load on timber piles to eighteen (18) tons per pile. In no case shall they be designed to support more than twenty-two (22) tons per pile. The maximum design load on concrete piles may be assumed as from twenty-five (25) to thirty-five (35) tons per pile, depending on conditions.

Piles shall be designed to carry the entire superimposed load, no allowance being made for the supporting value of the material between the piles.

The supporting power of piles shall be determined by the application of test loads or by the use of formulas as specified in Section 75.

(c) Spacing: Footing areas shall be so proportioned that pile spacing shall be not less than two feet six inches (2'-6"), center to center. The distance from the side of any pile to the nearest edge of the footing shall be not less than nine (9) inches.

(d) Batter Piles: When it is necessary to use piles under arch abutments, batter piles shall be used.

(e) Concrete Piles: Precast concrete piles shall be of approved size and shape. If a square section is employed, the corners shall be chamfered at least one (1) inch. Piles preferably shall be cast with a driving point and for hard driving preferably shall be shod with a metal shoe of approved pattern. Piling may be either of uniform section or tapered. In general, tapered piling shall not be used for trestle construction except for that portion of the pile which lies below the ground line; nor shall tapered piles be used in any location where the piles are to act as columns. In general, concrete piles shall have a cross sectional area, measured above the taper, of not less than one hundred and forty (140) square inches and when they are to be used in salt water they shall have a cross sectional area of not less than two hundred and twenty (220) square inches.

Reinforcement for precast concrete piling shall consist of longitudinal bars in combination with lateral reinforcement in the form of hoops or spirals. The longitudinal reinforcement shall be not less than one per cent and preferably not less than one and one-half per cent of the total cross-section of the pile. The reinforcement shall be placed at a clear distance from the face of the pile of not less than two (2) inches and when the piles are for use in salt water or alkali soils this clear distance shall be not less than three (3) inches. The driving point and also the top of the pile shall be protected against impact by means of special spiral winding or bands designed for this purpose. The reinforcing system preferably shall be of the "unit" type rigidly wired or fastened at all intersections. When piles exceed fifty-five (55) feet in length, additional longitudinal reinforcement shall be added throughout the central one-third (1/3) of the length. Piling under retaining walls, arch footings, abutments, etc., shall be designed to withstand the lateral stresses induced.

91.02. Footings.

(a) Depth: The depths of footings shall be determined with respect to the character of the foundation materials and the possibility of undermining. Except where solid rock is encountered or in other special cases, the footings of all structures, other than culverts, which are exposed to the erosive action of stream currents preferably shall be founded at a depth of not less than four (4) feet below the permanent bed of the stream. Stream piers and arch abutments preferably shall be founded at a depth of not less than six (6) feet below stream bed. The above preferred minimum depths shall be increased as conditions may require.

Footings not exposed to the action of stream currents shall be founded on a firm foundation and at a depth below frost.
Footings for culverts shall be carried to an elevation sufficient to secure a firm foundation, or a heavy reinforced floor shall be used to distribute the pressure over the entire horizontal area of the structure. In any location liable to erosion, apron or cut-off walls shall be used at both ends of the culvert and, where necessary, the entire floor area between the wing walls shall be paved. Baffle walls or struts across the unpaved bottom of a culvert barrel shall not be used where the stream bed is subject to erosion. When conditions require, culvert footings shall be reinforced longitudinally.

(b) Anchorage: Footings on solid rock, unless they are restrained by an overburden of resistant material shall be effectively anchored by means of anchor bolts, dowels, keys or other suitable means.

(c) Distribution of Pressure: All footings shall be designed to keep the maximum soil pressures within safe bearing values. In order to prevent unequal settlement, footings shall be designed to keep the pressure as nearly uniform as practicable. In footings having unequal pressures and requiring piling, the spacing of the piles shall be such as to secure as nearly equal loads on each pile as may be practicable.

(d) Spread Footings: Spread footings which act as cantilevers may be decreased in thickness from the junction of the footing slab with column or wall toward the edge of the footing, provided sufficient section is maintained at all points to provide the necessary resistance to diagonal tension and bending stresses. This decrease in section may be accomplished by sloping the upper surface of the footing or by means of vertical steps. Stepped footings shall be cast monolithically.

Except in small structures, no footing shall have a thickness at the edge of less than two (2) feet. When piles are used, the footing shall have an edge thickness of not less than eighteen (18) inches above the tops of the piles.

(e) Internal Stresses in Spread Footings: Spread footings shall be considered as under the action of downward forces, due to the superimposed loads, resisted by an upward pressure exerted by the foundation material and distributed over the area of the footings as determined by the eccentricity of the resultant of the downward forces. Where piles are used under footings, the upward reaction of the foundation shall be considered as a series of concentrated loads applied at the pile centers, each pile being assumed to carry its computed proportion of the total footing load.

Footings shall be designed for bending stresses, for diagonal tension stresses, and for punching shear around the periphery of the column or pier shaft. The critical section for bending shall be taken at the face of the column, wall or pier shaft. Bending need not be considered unless the projection of the footing is more than two-thirds (2/3) the depth.

When a single spread footing supports a column, pier or wall, this footing shall be assumed to act as a cantilever. When two or more piers or columns are placed upon a common footing, the footing slab shall be designed for the actual conditions of continuity and restraint.

(f) Reinforcement: Footing slabs shall be reinforced for bending stresses and, where necessary, for diagonal tension. All bars shall be effectively anchored to develop in bond the computed stress in the bar.

The reinforcement for square footings shall consist of two or more bands of bars. The reinforcement necessary to resist the bending moment in each direction in the footing shall be determined as for a reinforced concrete beam; the effective depth of the footing shall be the depth from the top to the plane of the reinforcement. The required reinforcement shall be spaced uniformly across the footing, unless the footing width is greater than the side of the column or pedestal plus twice the effective depth of the footing, in which case the width over which the reinforcement is spread may equal the width of the column or pedestal plus twice the effective depth of the footing plus one-half (½) the remaining width of the footing. In order that no considerable area of the footing shall remain unreinforced, additional bars shall be placed outside of the width specified, but such bars shall not be considered as effective in resisting the calculated bending moment. For the extra bars a spacing double that used for the reinforcement within the effective belt may be used. When reinforcement is used in more than one direction the allowable unit bond stresses shall be reduced as follows:

For two-way reinforcement ............................................ 25%
For each additional direction ...................................... 10%

(g) Transfer of Stress from Vertical Reinforcement: The stresses in the vertical reinforcement of columns or walls shall be transferred to the footings by extending the reinforcement into them a suffi-
cient distance to develop the strength of the bars in bond, or by means of dowels anchored in the footings and overlapped or fastened to the vertical bars in such manner as to develop their strength. If the dimensions of the footings are not sufficient to permit the use of straight bars, the bars may be hooked or otherwise mechanically anchored in the footings.

91.03. **Abutments.**

(a) **General** Abutments shall be designed to withstand earth pressure, the weight of abutment and superstructure, live load over any portion of the superstructure or approach fill, wind forces, and tractive force when the latter exists. The design shall be investigated for any combination of these forces which may produce the most severe condition of loading.

Abutments shall be designed to be safe against overturning about the toe of the footing, against sliding on the footing base and against crushing of foundation material or overloading of piles at the point of maximum pressure.

In computing stresses in abutments, the weight of filling material directly over an inclined or stepped rear face, or over a reinforced concrete spread footing extending back from the face wall, may be considered as part of the effective weight of the abutment. In the case of a spread footing, the rear projection shall be designed as a cantilever supported at the abutment stem and loaded with the full weight of the superimposed material.

The cross section of stone masonry or plain concrete abutments shall be proportioned to avoid the introduction of tensile stress in the material.

(b) **Reinforcement for Temperature:** Except in gravity abutments, not less than one-eighth (0.125) square inch of horizontal reinforcement per foot of height shall be provided near exposed surfaces not otherwise reinforced, to resist the formation of temperature and shrinkage cracks.

(c) **Wing Walls:** Wing walls shall be of sufficient length to retain the roadway embankment to the required extent and to furnish protection against erosion. For ordinary materials, in the absence of accurate data, the slope of the fill shall be assumed as one and one-half (1½) horizontal to one (1) vertical and wing lengths computed on this basis.

(d) **Drainage:** The filling material behind abutments shall be effectively drained by weep holes with French drains, placed at suitable intervals.

91.04. **Retaining Walls.**

(a) **General:** Retaining walls shall be designed to withstand earth pressure, including any live load surcharge, and the weight of the wall, in accordance with the general principles specified above for abutments.

Stone masonry and plain concrete walls shall be of the gravity type. Reinforced concrete walls may be of either the cantilever, counterforted, buttressed, or cellular type.

(b) **Base or Footing Slabs:** The rear projection or heel of base slabs shall be designed to support the entire weight of the superimposed material.

The base slabs of cantilever walls shall be designed as cantilevers supported by the wall.

The base slabs of counterforted and buttressed walls shall be designed as fixed or continuous beams of spans equal to the distance between counterforts or buttresses.

(c) **Vertical Walls:** The vertical stems of cantilever walls shall be designed as cantilevers supported at the base.

The vertical or face walls of counterforted and buttressed walls shall be designed as fixed or continuous beams. The face walls shall be securely anchored to the supporting counterforts or buttresses by means of adequate reinforcement.

(d) **Counterforts and Buttresses:** Counterforts shall be designed as T-beams. Buttresses shall be designed as rectangular beams. In connection with the main tension reinforcement of counterforts there shall be a system of horizontal and vertical bars or stirrups to effectively anchor the face wall and base slab. These stirrups shall be anchored as near the outside faces of the face walls, and as near the bottom of the base slab, as practicable.
(e) **Reinforcement for Temperature**: Except in gravity walls, not less than one-eighth (0.125) square inch of horizontal reinforcement per foot of height shall be provided near exposed surfaces not otherwise reinforced, to resist the formation of temperature and shrinkage cracks.

(f) **Expansion Joints**: Expansion joints shall be provided at intervals not exceeding thirty (30) feet for gravity walls and fifty (50) feet for reinforced walls.

(g) **Drainage**: The filling material behind all retaining walls shall be effectively drained by weeps holes with French drains, placed at suitable intervals. In counterforted walls there shall be at least one drain for each pocket formed by the counterforts.

91.05. **Piers**:

(a) **General**: Piers shall be designed to withstand the dead and live loads superimposed thereon; wind pressure acting on the pier and superstructure; the forces due to stream current, and drift; and tractive forces at the fixed ends of spans.

Where necessary, piers shall be protected against abrasion by facing them with granite, vitrified brick, timber or other suitable material within the limits of damage by debris.

(b) **Pier Nose**: In streams carrying drift, the pier nose shall be designed as a cutwater. When a steel angle or other metal nosing is used it shall be effectively secured to the masonry by means of suitable anchor bolts having countersunk heads.

91.06. **Tubular Steel Piers**:

(a) **Use**: Preferably, tubular steel piers shall not be used and they shall never be used in locations where they will be subjected to lateral earth pressure. In special cases their use may be permitted, in which cases the following requirements shall apply.

(b) **Depth**: The general requirements governing the depths of foundations as above set forth shall govern in the case of tubular steel piers except that steel tubes resting upon gravel foundation without piling shall in no case be carried to a depth less than eight (8) feet below the permanent bed of the stream and to such additional depth as may be necessary to eliminate all danger of undermining.

(c) **Piling**: Piles used in connection with tubular steel piers shall extend into the concrete filling a sufficient distance to thoroughly brace the tubes. In general, these piles shall extend not less than six (6) to eight (8) feet above the bottom of the concrete.

(d) **Dimensions of Shell**: The minimum thickness of the metal in the shells of tubular piers shall be five-sixteenths (5/16) inch. This thickness shall be increased where necessary to secure strength and rigidity for placing the shell. In all cases the pier shall be designed for safe pile or soil bearing values as specified herein, but when the diameter required by these values is greater than that required for the superstructure bearing, the diameter may be reduced at any splice point. The minimum diameter of steel cylinders used for piers shall be forty-two (42) inches.

(e) **Splices and Joints**: All horizontal joints shall be riveted butt joints. Vertical joints may be lapped if the corners of the plates are properly scarfed. When field splicing is necessary the lower section of the tube shall extend at least two (2) feet above the water line when in position.

(f) **Bracing**: Adequate bracing connecting the tubes of cylinder piers shall be provided. In general, this bracing shall consist of a steel or concrete girder diaphragm effectively secured to the tubes. The depth of this diaphragm shall be as great as conditions will permit.
SECTION 92

STRUCTURAL STEEL DESIGN

92.01. Spacing of Trusses and Girders: Main trusses and girders shall be spaced a sufficient distance apart center to center, to be secure against overturning by the assumed lateral and other forces.

92.02. Depth Ratios. Trusses preferably shall have a depth not less than one-tenth (1/10) the span, plate girders a depth not less than one-twelfth (1/12) the span, and rolled beams a depth of not less than one-twentieth (1/20) the span. If less depths than these are used, the sections shall be increased so that the maximum deflection will not be greater than if these limiting ratios had not been exceeded.

92.03. Dimensions for Stress Calculation:

(a) Effective Span: For the calculation of stresses, span lengths shall be assumed as follows:
   - Beams and girders, distance between centers of bearings.
   - Trusses, distance between centers of end pins or of bearings.
   - Floorbeams, distance between centers of trusses or girders.
   - Stringers, distance between centers of floorbeams.

(b) Effective Depth: For the calculation of stresses, effective depths shall be assumed as follows:
   - Riveted trusses, distance between centers of gravity of the chords.
   - Pin-connected trusses, distance between centers of chord pins.
   - Plate girders, distance between centers of gravity of the flanges but not to exceed the distance back to back of flange angles.

92.04. Reversal of Stress: Members subject to reversal of stress during the passage of live load shall be proportioned as follows: Determine the tensile and the compressive stresses and increase each by fifty percent (50%) of the smaller; then proportion the member so that it will be capable of resisting each increased stress. The connections shall be proportioned for the sum of the actual stresses.

No pin-connected member shall be subjected to reversal of stress.

When the live load and the dead load stresses are of opposite sign, only seventy per cent (70%) of the dead load stresses shall be considered as effective in counteracting the live load stress.

92.05. Combined Stresses:

(a) Axial and Bending: Members subject to both axial and bending stresses shall be proportioned so that the combined fibre stresses will not exceed the allowable axial stress. Members continuous over panel points shall be proportioned for the live and dead load bending moments computed for a simple beam having a span equal to one (1) panel length.

(b) Stresses due to Lateral and Longitudinal Forces and Temperature:

In proportioning the various parts of the structure, provision shall be made for the following stress combinations:

<table>
<thead>
<tr>
<th>Group A</th>
<th>Dead Load</th>
<th>Live Load</th>
<th>Impact</th>
<th>Centrifugal Force</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group B</td>
<td>Lateral Force</td>
<td>Longitudinal Force</td>
<td>Temperature</td>
<td></td>
</tr>
</tbody>
</table>

Members subject to the stresses of Group A in combination with the stresses of Group B, either direct or flexural or both, shall be designed for any of the following combinations, at Unit Stresses 25% greater than those specified, but the resulting sections shall be not less than would be required if the stresses of Group A were considered alone.

1. The combined stresses of Group B in combinations with dead load only.
2. The combined stresses of Group A in combination with 50% of the combined stresses of Group B.
3. The combined stresses of Group A in combination with temperature only.

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92.06. **Secondary Stresses:** Members and their details shall be proportioned to reduce secondary stresses to a minimum. In simple trusses without subdivided panels the secondary stresses due to deformation in any member whose width measured in the plane of flexure is less than one-tenth (1/10) of its length need not be considered. When this ratio is exceeded, or where subdivided panels are used, the secondary stresses shall be computed.

In members designed for secondary stresses in combination with other stresses the specified allowable unit stresses may be increased 30% but the sections shall be not less than required for primary stresses.

92.07. **Allowance for Overload.** For the calculation of stress reversal or counter stresses, the specified live loads, either uniform or concentrated, shall be increased one hundred percent (100%) and for this loading condition the specified unit stresses shall be increased not more than fifty percent (50%). The resulting sections shall be not less than would have been required had the allowance for overload not been considered.

92.08. **Compression Flanges of Beams and Girders:** The gross area of the compression flanges of beams and plate girders shall be not less than the gross area of the tension flanges.

The laterally unsupported length of the compression flanges of beams and girders shall not exceed forty (40) times the flange width. When the unsupported length of flange exceeds twelve (12) times the flange width, the compressive stress in pounds per square inch shall not exceed

\[
\frac{L}{19000 - 250} \times b \quad \text{(Maximum value, 16000 lbs.)}
\]

where

- \(L\) = length, in inches, of unsupported flange, between lateral connections or knee braces.
- \(b\) = flange width in inches.

92.09. **Proportioning Rolled Beams:** Rolled beams shall be proportioned by the moments of inertia of their sections. Proper allowance shall be made for any reduction in strength due to rivet holes in the tension flange or for any reduction in allowable stress due to the length of unsupported compression flange.

92.10. **Limiting Lengths of Members:**

(a) **Compression Members:** The ratio of unsupported length to the least radius of gyration shall not exceed one hundred and twenty (120) for main compression and stiffening members nor one hundred and forty (140) for laterals and sway bracing. In proportioning the top chords of low trusses the unsupported length shall be assumed as the length between the rigid verticals.

(b) **Tension Members:** For main riveted tension members the ratio of length to least radius of gyration shall not exceed two hundred (200).

92.11. **Effective Bearing Area:** The effective bearing area of a pin, bolt, or a rivet shall be its nominal diameter multiplied by the thickness of the metal on which it bears.

92.12. **Effective Diameter of Rivets:** In proportioning rivets, the nominal diameter of the rivet shall be used.

92.13. **Size of Rivets:** Rivets shall be of the size specified but generally shall be three-quarters (3/4) inch or seven-eights (7/8) inch in diameter. Five-eights (5/8) inch rivets shall not be used in members carrying calculated stress except in two and one-half (21/2) inch legs of angles and in flanges of six (6) inch and seven (7) inch beams and channels.

The diameter of rivets in angles carrying calculated stress shall not exceed one-fourth (1/4) of the width of the leg in which they are driven. In angles whose size is not so determined five-eights (5/8) inch rivets may be used in two (2) inch legs, three-quarters (3/4) inch rivets in two and one-half (21/2) inch legs and seven-eights (7/8) inch rivets in three (3) inch legs.

In no case, except in handrails, shall structural shapes be used which do not admit the use of five-eights (5/8) inch diameter rivets.
92.14. **Pitch of Rivets:** The minimum allowable distance between centers of rivets shall be three (3) times the diameter of the rivet but preferably shall be not less than the following:

- For 7/8 inch diameter rivets—3 inches.
- For 3/4 inch diameter rivets—2 1/2 inches.
- For 5/8 inch diameter rivets—2 1/4 inches.

The maximum allowable pitch in the line of stress shall not exceed six (6) inches or sixteen (16) times the thickness of the thinnest outside plate or angle connected, except in angles having two gage lines with rivets staggered where the pitch in each line may be twice the above with a maximum of ten (10) inches.

In webs of members composed of two or more plates in contact, the rivets shall be spaced not more than ten (10) inches between centers in gage and pitch, provided such rivets serve no other purpose than to hold the plates in close contact. Tension members composed of two angles in contact shall be stitch riveted using a pitch not greater than twelve (12) inches.

92.15. **Pitch in Ends of Compression Members:** In the ends of built compression members the pitch of rivets connecting the component parts of the member shall not exceed four (4) times the diameter of the rivet for a length equal to one and one-half (1 1/2) times the maximum width of member. Beyond this point the rivet pitch shall be gradually increased for a length equal to one and one-half (1 1/2) times the maximum width of the member until the maximum spacing is reached. In angles having two (2) lines of staggered rivets in one leg, the pitch on each line may be twice that specified above but not greater than that allowed for the body of the member.

92.16. **Edge Distance of Rivets:** The minimum distance from the center of any rivet to a sheared edge shall be:

- For 7/8 inch diameter rivets—1 1/2 inches.
- For 3/4 inch diameter rivets—1 1/4 inches.
- For 5/8 inch diameter rivets—1 1/8 inches.

The minimum distance from rolled or planed edges, except flanges of beams and channels, shall be:

- For 7/8 inch diameter rivets—1 1/4 inches.
- For 3/4 inch diameter rivets—1 1/8 inches.
- For 5/8 inch diameter rivets—1 inch.

The maximum distance from any edge shall be eight (8) times the thickness of the thinnest outside plate, but shall not exceed five (5) inches.

92.17. **Long Rivets:** Long rivets subjected to calculated stress and having a grip in excess of four and one-half (4 1/2) diameters shall be increased in number at least one (1%) percent for each additional one-sixteenth (1/16) inch of grip. If the grip exceeds six (6) times the diameter of the rivet, specially designed rivets shall be used.

92.18. **Rivets in Tension:** Rivets in direct tension shall, in general, not be used. However, where so used their value shall be one-half (1/2) that permitted for rivets in shear. Countersunk rivets shall not be used in tension.

92.19. **Parts Accessible.** The accessibility of all parts of a structure for inspection, cleaning and painting shall be insured by the proper proportioning of members and the design of their details.

92.20. **Open Sections and Pockets:** Closed sections shall in general be avoided. Pockets or depressions which will retain water shall be avoided as far as possible and those which are unavoidable shall be provided with effective drain holes or shall be effectively filled with waterproof material.

Details shall be arranged so that the retention of dirt, leaves or other foreign matter will be reduced to a minimum. Wherever angles are used, either singly or in pairs, they preferably shall be placed with the vertical legs extending downward.

92.21. **Symmetrical Sections:** Main members shall be proportioned so that their neutral axes shall be as nearly as practicable in the center of the section.

In general, the gravity axes of main truss and other important members, meeting to form a joint, shall intersect in a common point so as to avoid eccentricity of stress. In case of unavoidable eccentric-
city the members affected thereby shall be proportioned and the connection details designed to resist the stresses produced.

92.22. Effective Area of Angles in Tension: The effective area of single angles in tension shall be assumed as the net area of the connected leg plus fifty percent (50%) of the area of the unconnected leg.

The effective area of a double angle tension member shall be assumed as eighty (80) percent of the net area of the member unless the end details and connections are such that the individual angles are held against bending in both directions, in which case the full net area may be used. When the angles connect to separate gusset plates, as in the case of a double-webbed truss, the gusset plates shall be stiffened by diaphragms in the line of the connected angles or by tie plates extending to the ends of the angles if they are to be considered as offering such resistance to bending that the full net area can be used. When the angles are connected back to back on opposite sides of a single gusset plate the support may be assumed to be sufficient to allow the use of the full net section.

Lug angles shall not be considered as effective in transmitting stress.

92.23. Strength of Connections: Unless otherwise provided all connections shall be proportioned to develop not less than the full strength of the members connected.

No connections, except for lacing bars and handrails, shall contain less than three (3) rivets.

92.24. Splices: Continuous compression members in riveted structures, such as chords and trestle posts, shall have milled ends and full contact bearing at the splices.

All splices, whether in tension or compression, shall be proportioned to develop the full strength of the members spliced and no allowance shall be made for milled ends of compression members.

Splices shall be located as close to panel points as possible and, in general, shall be on that side of the panel point which is subjected to the smaller stress.

The arrangement of the plates, angles or other splice elements shall be such as to make proper provision for the stresses in the component parts of the members spliced.

92.25. Indirect Splices: In all splice plates not in direct contact with the parts they connect, the number of rivets on each side of the joint shall be in excess of the number which would otherwise be required for a contact splice to the extent of two extra transverse lines for each intervening plate.

92.26. Fillers: Where indirect splices involve rivets carrying stress and passing through fillers, the fillers shall be extended beyond the splicing material and the extension secured by additional rivets sufficient in number to develop the section of the filler.

When the filler is less than one-quarter (¼) inch thick the splicing material shall also be extended.

92.27. Gusset Plates: Gusset or connecting plates shall be used for connecting all main members, except in pin-connected structures. In proportioning and detailing these plates the rivets connecting each member shall be located, as nearly as practicable, symmetrically with the axis of the member. However, the full development of the elements of the member shall be given due consideration. The gusset plates shall be of ample thickness to resist shear, direct stress and flexure acting on the weakest or critical section of maximum stress.

Re-entrant cuts shall be avoided as far as possible.

92.28. Minimum Thickness of Metal. The minimum thickness of structural steel shall be five-sixteenth (5/16) inch except for fillers and railings. However, gusset plates shall not be less than three-eights (3/8) inch in thickness.

Metal subject to marked corrosive influence shall be increased in thickness, or specially protected against corrosion.

Cast steel shall not be less than one (1) inch and cast iron not less than one and one-quarter (1¼) inch thick, except for filler blocks.

92.29. Compression Members: In built compression members the metal shall be concentrated as much as possible in the webs and flanges, so that the center of gravity of the section may be as near the center lines of the member as practicable.
92.30. **Plates in Compression**: Cover plates of built-up compression members and cover plates on the compression flanges of plate girders shall have a minimum thickness of one-fortieth (1/40), and the web plates of compression members a minimum thickness of one-thirtieth (1/30), of the transverse distance between the lines of rivets connecting them to the flanges. However, failing to meet this requirement, the width of plate between the connecting lines of rivets in excess of forty (40) times the thickness for cover plates and thirty (30) times the thickness for web plates, shall not be considered as effective in resisting stress.

92.31. **Outstanding Flanges**: Outstanding compression flanges of girders and main compression members shall have a minimum thickness of one-twelfth (1/12) of the width of outstanding flange. For lateral bracing and other secondary members this minimum thickness may be one-fourteenth (1/14) of the width of the outstanding flange.

92.32. **Tie Plates**: The open sides of compression members shall be provided with lacing bars and shall have tie plates as near each end as practicable and at intermediate points where the lacing is interrupted. Compression members composed of two angles and a cover plate shall have, on their open sides, ties composed of short lengths of channel section with the flanges riveted to the vertical legs of the angles.

Tension members composed of shapes shall have their separate segments connected by tie plates or by tie plates and lacing bars.

The thickness of the tie plates shall be not less than one-fiftieth (1/50) of the distance between the connecting lines of rivets. Tie plates shall be connected by not less than three (3) rivets on each side and in members having lacing bars the last rivet in the tie plate shall preferably also pass through the end of the adjacent bar.

For main compression members, the end tie plates shall have a length not less than one and one-half (1½) times the perpendicular distance between the lines of rivets connecting them to the member, and the intermediate tie plates a length not less than that distance. For main tension members the end tie plates shall have the length above specified for end tie plates on main compression members and the length of the intermediate tie plates shall be not less than three-quarters (¾) the length specified for intermediate tie plates on compression members. In tension members whose elements are connected by tie plates only, the distance center to center of plates shall not exceed three (3) feet.

For lateral struts and other secondary members, the length of end and intermediate tie plates shall be not less than three-quarters (¾) the perpendicular distance between the lines of rivets connecting them to the member.

92.33. **Lacing Bars**: The lacing of compression members shall be proportioned to resist shearing stresses normal to the member not less than those calculated by the formulas:

\[
R = \frac{4I}{C \times L} (16000 - p),
\]

\[
R = \frac{0.4pI}{C \times L}
\]

In which

- \( R \) = normal shearing stress in pounds.
- \( I \) = moment of inertia of section about an axis perpendicular to the plane of the latticing.
- \( C \) = distance from neutral axis to extreme fiber, in inches.
- \( p \) = average compressive unit stress in the member; \( p = \frac{P}{A} \)
- \( L \) = length of member, in inches.

The greater of the values given by these two formulas shall be used.

If the lacing of a horizontal or inclined compression member is in a vertical plane, the shear in the lacing caused by the weight of the member shall be added to the shear calculated by the formulas above.

The shear shall be considered as divided equally among all shear resisting elements in parallel planes.
whether made up of continuous plates or of lattice. The size of the bar shall be determined by the column formula Paragraph 89.02 in which “L” shall be taken as the distance between the connections to the main sections.

The minimum width of lacing bars shall be:

- For 7/8 inch diameter rivets—2 1/2 inches.
- For 3/4 inch diameter rivets—2 1/4 inches.
- For 5/8 inch diameter rivets—2 inches.

Lacing bars having two (2) rivets in each end shall be used for flanges five (5) inches or more in width. The minimum thickness of bars shall be one-fortieth (1/40) of the distance between end rivets in the case of single lacing and one-sixtieth (1/60) of this distance for double lacing, but not less than five-sixteenth (5/16) inch.

Double lacing, riveted at the intersections, shall be used when the perpendicular distance between rivet lines exceeds fifteen (15) inches.

The inclination of single lacing shall generally be about sixty (60) degrees and for double lacing it shall be about forty-five (45) degrees to the axis of the member. Lacing bars of compression members shall be so spaced that the L/r of the portion of the flange included between lacing bar connections will be not greater than forty (40), and not greater than two-thirds (2/3) of the L/r of the member.

Shapes of equivalent strength may be used instead of flats.

92.34. **Net Section at Pin Holes:** In pin-connected riveted tension members, the net section across the pin hole shall be not less than 140 per cent, and the net section back of the pin hole not less than 100 per cent of the net section of the body of the member.

92.35. **Net Section of Riveted Tension Members:** In calculating the required area of riveted tension members, net sections shall be used in all cases and, in deducting rivet holes, they shall be taken as one-eighth (1/8) inch larger than the nominal diameter of the rivet.

The net section shall be the least area which can be obtained by deducting from the gross sectional area, the area of holes cut by any straight or zigzag section across the member, counting the full area of the first hole and a fractional part of each succeeding hole, the fractional part being determined by the formula:

\[ X = 1 - \frac{S^2}{4gh} \]

Where \( X \) = fraction of rivet hole to be deducted.

\( S \) = stagger or longitudinal spacing of rivet with respect to rivet on last gage line.

\( g \) = distance between gage lines, or transverse spacing.

\( h \) = diameter of rivet holes, or nominal diameter of rivet plus 1/8 inch.

92.36. **Location of Pins:** Pins shall be located, with respect to the neutral axes of the members, so as to reduce to a minimum secondary stresses due to bending.

92.37. **Pin Plates:** Pin plates shall be of sufficient thickness to provide the required bearing area upon the pin; they shall be as wide as the dimensions of the member will allow; and their length, measured from pin center to end, shall be at least equal to their width. Pin plates shall contain sufficient rivets to distribute their due proportion of the pin pressure to the full cross section of the members; only the rivets located within two lines drawn from the pin center toward the body of the member and inclined at forty-five (45) degrees to the axis of the member shall be considered effective for this purpose. In the case of members composed of web plates and flange angles (with or without a cover plate) there shall be at least one outside pin plate covering the vertical legs of the flange angles.

At the end of compression members at least one pair of pin plates shall extend not less than six (6) inches beyond the near edge of the tie plate.

All pin-connected compression members shall be provided with hinge plates having a minimum thickness of three-eights (3/8) inch.
92.38. **Forked Ends:** Forked ends on compression members will be permitted only when unavoidable. When used, a sufficient number of pin plates shall be provided to give each jaw the full strength of the compression member. At least one (1) pair of these plates shall extend to the far edge of the tie plates, and the others not less than six (6) inches beyond the near edge of the tie plates.

92.39. **Pins and Pin Nuts:** Pins shall be proportioned for the maximum shears and bending moments produced by the stresses in the members connected. If there are eye-bars among the parts connected, the diameter of the pins shall be not less than three-fourths (3/4) of the width of the widest bar attached.

Pins shall be of sufficient length to secure a full bearing of all parts connected upon the turned body of the pin. They shall be secured in position by hexagonal chambered nuts or by hexagonal solid nuts with washers. Where the pins are bored, through rods with cap washers may be used. In general, malleable castings conforming to the requirements of paragraph 69.21 and 69.22 shall be used for pin nuts. Pin nuts shall be secured by cotters in the screw ends.

92.40. **Bolts:** Unless specifically authorized, bolted connections will not be permitted. Bolts, when used, shall be unfinished or turned as specified and shall meet the requirements of paragraph 69.63.

Bolts in tension shall have double nuts.

92.41. **Upset Ends.** Bars and rods with screw ends shall be upset to provide a cross sectional area at the root of the thread which shall exceed the net section of the body of the member by at least fifteen (15) percent.

92.42. **Sleeve Nuts:** Sleeve nuts shall not be used.

92.43. **Expansion:** Provision for expansion and contraction, to the extent of one eighth inch for each ten feet of span, shall be made for all bridges. Expansion ends shall be firmly secured against lifting or lateral movement.

92.44. **Expansion Bearings:** Spans of less than seventy (70) feet may be arranged to slide upon metal plates with smooth surfaces. Spans of seventy (70) feet and over shall be provided with rollers or rockers, or with the special sliding bearings described below. Neither rollers nor rockers shall be used for expansion bearings at the top of trestle posts.

92.45. **Fixed Bearings:** Fixed bearings shall be firmly anchored.

92.46. **Hinged or Pin Bearings:** Spans of seventy feet and over shall have hinged or pin bearings at both ends. The pedestals or shoes shall be so designed that all loads will act through the end pins which will be located directly over the geometrical center of the bearing.

92.47. **Rollers:** Expansion rollers shall not be less than six (6) inches in diameter for span lengths of one hundred (100) feet or less and this minimum shall be increased not less than one (1) inch for each additional one hundred (100) feet of span, and proportionately for intermediate lengths. They shall be connected together by substantial side bars and shall be effectually guided so as to prevent lateral movement, skewing or creeping. The rollers and bearing plates shall be protected from dirt and water as far as possible and the construction shall be such that water will not be retained and that the roller nests may be inspected and cleaned with the minimum difficulty.

92.48. **Rockers:** Pin bearing expansion rockers shall be of cast steel or cast iron.

92.49. **Special Sliding Expansion Bearings:** Sliding plates for the expansion bearings of spans of seventy feet and over shall be of Class A bronze conforming to the requirements of paragraph 69.23 and 69.24. These plates shall be chamfered at the ends and shall be held securely in position, usually by being inset into the metal of the pedestals and sole plates. Provision shall be made against any accumulation of dirt which will obstruct their free movement.

92.50. **Pedestals and Shoes:** Pedestals and shoes shall be designed to secure rigidity and stability and to distribute the reaction uniformly over the entire bearing area. Preferably, they shall be made of cast steel or structural steel. The bottom bearing widths shall not exceed the top bearing widths by more than twice the depth of pedestal and, when involving pin bearings, this depth shall be measured from the center of pin.
Where built pedestals and shoes are used the web-plates and the angles connecting them to the base plates shall be not less than five eights (5/8) inch thick. If the size of the pedestal permits, the webs shall be rigidly connected transversely. The minimum thickness of the metal in cast steel pedestals shall be one (1) inch.

92.51. **Inclined Bearings:** For spans on an inclined grade without pin or hinged bearings, the sole plates shall be beveled so that the sub-structure bridge seats will be level.

92.52. **Anchor Bolts:** Trusses, girders and I-beams shall be securely anchored to their substructures. Anchor bolts shall be roughened by being screw-threaded or swedged to secure a satisfactory grip upon the material used to embed them in the holes.

The following are the minimum requirements for each bearing:

For I-beam spans the outer beams shall be anchored at each end with two bolts one inch in diameter, set ten inches in the masonry.

For girder and truss spans

<table>
<thead>
<tr>
<th>Spans</th>
<th>Bolts</th>
<th>Diameter</th>
<th>Set in Masonry</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fifty (50') feet or less</td>
<td>2</td>
<td>1 inch</td>
<td>10 inches</td>
</tr>
<tr>
<td>51 to 100 feet</td>
<td>2</td>
<td>1 3/4 inch</td>
<td>1'-0&quot;</td>
</tr>
<tr>
<td>101 to 150 feet</td>
<td>2</td>
<td>1 3/4 inch</td>
<td>1'-3&quot;</td>
</tr>
<tr>
<td>151 feet and over</td>
<td>4</td>
<td>1 3/4 inch</td>
<td>1'-6&quot;</td>
</tr>
</tbody>
</table>

Anchor bolts subject to tension, as in the column base of trestle bents and towers, shall be designed to engage a mass of masonry which will secure a resistance equal to one and one-half (1 1/2) times the calculated uplift.

92.53. **FLOOR SYSTEM**

92.54. **Floorbeams:** Floorbeams preferably shall be at right angles to the trusses of main girders and shall be rigidly connected thereto. In general, floorbeam connections shall be located above the bottom chord and, in riveted work, the bottom chord lateral system shall engage both the bottom chord and the floorbeams. Floorbeam connections to pin connected trusses preferably shall be above the bottom chord pin but, if located below, the vertical posts shall be extended below the pins to secure rigid connections to the floorbeams.

92.55. **End Floorbeams:** Except in skew bridges end floorbeams shall be provided in all truss and girder spans. End floorbeams preferably shall be designed to permit the use of jacks for the future lifting of the superstructure, under which condition the specified unit stresses shall not be exceeded by more than fifty (50) per cent.

End floorbeams shall be arranged to permit future painting of the sides of the beams adjacent to the abutment backwalls.

92.56. **Stringers:** Steel stringers preferably shall be riveted between the floorbeams, with end connections to the floorbeam webs.

92.57. **End Struts:** When end floorbeams are not used the end panel stringers shall be secured in correct locations by end struts securely connected to the stringers and to the main trusses or girders. The end panel lateral bracing shall be rigidly attached to the main trusses or girders and shall also be attached to the end struts. Adequate provision shall be made for the expansion movement of stringers.

92.58. **End Connections for Floorbeams and Stringers:** The end connection angles of floorbeams and stringers shall be not less than three-eights (3/8) inch in thickness. When milled ends are required, the thickness of connection angles shall be one-sixteenth (1/16) inch greater than for connection angles not required to be milled. Except in cases of special end floorbeam details, end connections for floorbeams and stringers shall be made with two angles at each end. Bracket or shelf angles which may be used to furnish support during erection shall not be considered in determining the number of rivets required to transmit end shears.

End connection angles shall develop the full depth of the webs by having a length as great as the flanges will permit.
In the preparation of end connection details, special care shall be exercised to provide ample clearance for the driving of field connection rivets.

Where timber stringers frame into floor beams, shelf angles with stiffeners shall be provided to carry the whole reaction. Shelf angles shall be not less than seven-sixteenths (\(\frac{7}{16}\)) inch thick.

Any type of floorbeam hanger which will permit the rotation or the longitudinal motion of the floorbeam, shall not be used.

92.59. Expansion Joints: To provide for expansion and contraction movement, suitable floor expansion joints shall be provided at the expansion ends of all spans and at other points where they may be required.

Apron plates, when used, shall be designed to properly bridge the joint and to prevent, as far as possible, the deposit of roadway debris upon the bridge seats.

92.60. Sidewalk Brackets: Sidewalk brackets shall be connected directly to the top and bottom flanges of floor beams.

92.61. BRACING

92.62. Design of Bracing: Lateral, longitudinal and transverse bracing shall be composed of angles or other shapes offering resistance to deformation when subjected to compression stress, and shall have riveted connections.

In general, bracing shall consist of a double system of diagonal tension members with transverse compression members. The diagonals in each system shall be proportioned to carry the total lateral stress in tension, the transverse struts (or floorbeams) acting as compression members for both systems.

All intersections of lateral and sway bracing shall be riveted to add rigidity and prevent deformation.

92.63. Minimum Size of Angles: The smallest angle used in bracing shall be three by two and one-half (3x2\(\frac{1}{2}\)) inches. There shall not be less than three (3) rivets in each end connection of the angles.

92.64. Lateral Bracing: Bottom lateral bracing shall be provided in all bridges except I-beam spans, from which it may be omitted. Bottom laterals preferably shall be supported by rigid hangers at the intersections.

Top lateral bracing shall be provided in deck spans and in through spans having sufficient head room. Lateral bracing for compression chords shall preferably consist of either two or four angle latticed sections; and so designed as to effectively engage both flanges of the chords.

Lateral bracing shall have concentric connections to chords at end joints, and preferably throughout. The connections between the lateral bracing and the chords shall be designed to avoid, as far as possible, any bending stress in the truss members.

92.65. Portal and Sway Bracing: Through truss spans shall have portal bracing, preferably of the two plane or box type, rigidly connected to the end post and top chord flanges, and constructed as deep as the minimum clearance will allow. When a single plane portal is used it preferably shall be located in the central transverse plane of the end posts, with diaphragms between the webs of the posts to provide for a proper distribution of the portal stresses. The portal bracing shall be designed to take the full end reaction of the top chord lateral system and the end posts shall be designed to transfer this reaction to the truss bearings.

Deck truss spans shall have adequate sway bracing at the ends and at all intermediate panel points. This bracing shall occupy the full depth of the trusses below the floor system. The bracing shall be proportioned to transfer the end reaction of the top lateral system to the substructure.

Through truss spans shall have sway bracing at each intermediate panel point if the height of the trusses is such as to permit a depth of five (5) feet or more for the bracing. When the height of the trusses will not permit of such depth the top lateral struts shall be provided with knee braces. Top lateral struts shall be at least as deep as the top chord. Sway bracing shall be of ample strength to transfer one-half (\(\frac{1}{2}\)) of the wind pressure to the leeward truss.

92.66. Cross Frames: Deck plate girder spans shall be provided with cross frames at each end proportioned to resist all lateral forces, and shall have intermediate cross frames at intervals not exceed-
ing twenty (20) feet. These frames shall be connected to the outstanding legs of the stiffener angles and to the girder flanges.

92.67. **Low Truss Spans**: The vertical truss members and the floorbeam connections of low truss spans shall be proportioned to resist a lateral force, applied at the top chord panel points of the truss, determined by the following equations:

\[ R = 150 (A + P) \]

Where \( R \) = lateral force in pounds.

\( A \) = area of cross section of chord in square inches.

\( P \) = panel in length in feet.

This rigidity may be secured in part by extending one or both of the floorbeam connection angles upward along the inside of the post. Preferably outrigger brackets attached to the vertical posts on the outside of the trusses shall not be used.

92.68. **Through Girder Spans**: Through plate girder spans shall be stiffened against lateral deformations by means of gusset plates, or knee braces with solid webs, attached to the stiffener angles and floorbeams. If the unsupported length of the inclined edge of the gusset plate exceeds sixty (60) times its thickness, the gusset plate shall have stiffener angles riveted along its edge.

These braces generally shall extend to the clearance line and preferably shall be spaced not farther apart than fifteen (15) feet.

92.69. **Bracing of Long Columns**: The bracing of long columns shall be designed to fix the column in both the lateral and the longitudinal directions, at or near the same point.

**PLATE GIRDERS**

92.71. **Proportioning**: Plate girders shall be proportioned either by assuming the flanges to be concentrated at their centers of gravity or by the moment of inertia of the net section. In the former case one-eight of the gross area of the web is available as net flange area but the effective depth shall not be assumed to be greater than the distance back to back of flange angles. For girders having unusual cross sections the moment of inertia method shall be used.

92.72. **Flange Sections**: The gross section of the compression flange shall not be less than the gross section of the tension flange. The compression flange preferably shall be stayed against lateral deflection at intervals not exceeding twelve (12) times its width.

The flange angles shall form as large a portion of the gross area of the flange as practicable.

When flange plates are used, at least one plate on the top flange shall extend the full length of the girder except where flange is to be covered with concrete. Any additional flange plates shall be of such length as to allow two rows of rivets to be placed at each end of the plate beyond its theoretical end, and there shall be a sufficient number of rivets at the ends of each plate to develop its full stress value before the theoretical end of the next outside plate is reached.

Flange cover plates shall be equal in thickness, or shall diminish in thickness from the flange angle outward. No plate shall have a thickness greater than that of the flange angles.

92.73. **Web Plates**: Web plates shall be proportioned for both the vertical and horizontal shearing stresses. The thickness of web plates, except those to be cased in concrete, shall be not less than \( 1/20 \sqrt{D} \) in which \( D \) is the distance in inches between flanges.

92.74. **Flange Rivets**: The number of rivets connecting the flange angles to the web plates shall be sufficient to develop the increment of flange stress transmitted to the flange angles, combined with any load that is applied directly to the flange. For electric railways, one wheel load, when applied directly to the flange, shall be assumed to be distributed uniformly over a length of three (3) feet.

92.75. **Flange Splices**: Splices in flange parts shall not be used except by special permission of the Engineer. Two parts shall not be spliced at the same cross section and, if practicable, splices shall be located at points where there is an excess of section. The net section of the splices shall exceed by ten (10) per cent, the net section of the parts spliced. Flange angle splices shall consist of two (2) angles, one on each side. Splice angles shall be fitted to secure close contact with the material spliced.
92.76. **Web Splices:** Web plates shall be symmetrically spliced by plates on each side. The splice shall be equal in strength to the web in both shear and moment. There shall be at least two (2) rows of rivets on each side of the joint. The splice plates for shear shall be of the full depth of the girder between flanges.

92.77. **End Stiffeners:** Plate girders shall have stiffener angles over end bearings, the outstanding legs of which shall be as wide as the flange angles will allow and shall fit tightly against them. These end stiffeners shall be proportioned for bearing on the outstanding legs of the flange angles, no allowance being made for the legs fitted to the fillets of the flange angles. End stiffeners shall be arranged to transmit the total end reaction and to distribute it over the bearings. They shall not be crimped and there shall be a sufficient number of rivets in their connection to the web.

92.78. **Intermediate Stiffeners.** Intermediate stiffener angles shall be riveted in pairs to the web of the girder. The outstanding leg of each angle shall have a width of not more than sixteen (16) times its thickness and not less than two (2) inches plus one thirtieth (1/30) of the depth of the girder.

Intermediate stiffeners shall be spaced at intervals not exceeding:
- (a) 6 feet;
- (b) The depth of the web;
- (c) The distance given by the formula,

\[ d = \frac{12000 - S}{40} \]

Where \( d \) = distance between rivet lines of stiffeners, in inches.

\[ t = \text{Thickness of web, in inches}. \]

\[ s = \text{Web shear, in pounds per square inch, at the point considered}. \]

When the depth of the web between the flange angles or side plates is less than sixty (60) times the web thickness, intermediate stiffeners may be omitted.

Intermediate stiffener angles shall be placed at points of concentrated loading and shall be designed to transmit the reactions to the girder web. Such stiffeners shall not be crimped.

92.79. **Ends of Through Girders:** The upper corners of through plate girders, where exposed, shall be neatly rounded to a radius consistent with the size of the flange angles and the vertical height of the girder above the roadway. The first flange plate or a plate of the same width will be bent around the curve and continued to the bottom of the girder. In a bridge consisting of two or more spans only the corners on the extreme ends need be rounded, unless the spans have girders of varying heights, in which case the higher girders shall have their top flanges neatly cut down at the ends to meet the top corners of the girders in the adjacent spans.

92.80. **End Bearings:** End bearings of girders on masonry shall be raised above the bridge seat by metal pedestals or plates a height of at least six (6) inches.

92.81. **Sole and Masonry Plates:** Sole and masonry plates shall have a thickness not less than three-quarters (3/4) of an inch and not less than the thickness of the flange angles plus one-eighth (1/8) of an inch. Preferably they shall not be longer than eighteen (18) inches.

92.82. **Camber:** In general, camber will not be required in plate girders except for long spans or special conditions. When used, it shall be sufficient in amount to meet the requirements of the Engineer.

92.83. **TRUSSES**

92.84. **Main Features:** Preference will be given to trusses with single intersecting web members or other forms of trusses possessing the least ambiguity in computed stresses and the greatest elements of service ability. Adjustable members in any part of the structure preferably shall be avoided. Members shall be symmetrical about the central planes of trusses and all parts shall be so designed that they can be inspected, cleaned and painted.

Through riveted and pin-connected spans will generally have inclined end posts. Low truss spans shall be of the riveted type. In low truss spans, laterally unsupported hip joints or “flying hips” shall be avoided.
92.85. **Top Chords and End Posts:** Top chords and end posts of low and through truss spans shall be made usually of two side segments with one cover plate and with tie plates and lacing on the open side. In chords of light section, tie plates and lacing may be used in place of a cover plate.

Top chords of deck trusses subjected to direct loading shall be designed for the cross bending occasioned by the dead, live and impact loads of the floor system, in addition to the direct chord stresses, and all top chord splices shall be proportioned for those stresses and any shearing stresses they may receive.

Where the shape of the truss permits, compression chords shall be built continuous, with splices located as near the panel points as possible and preferably on the side subjected to the smaller stress.

The top chord sections of low truss spans shall be so proportioned that the radius of gyration about the vertical axis of the member shall be at least one and one-half \((1 \frac{1}{2})\) times the radius of gyration about the horizontal axis.

92.86. **Bottom Chords:** The bottom chords of riveted trusses generally shall be spliced near panel points and on the side farthest away from the center of the span.

Bottom chords composed of angles preferably shall be constructed with the vertical legs of the angles extending downward.

92.87. **Working Lines and Gravity Axes:** For compression members of unsymmetrical sections, such as chord sections formed of side segments and a cover plate, the working line shall coincide as nearly as practicable with the gravity axis of the section except that eccentricity may be introduced to counteract dead load bending. For symmetrical sections the working line shall coincide with the gravity axis. For two-angle bottom chord or diagonal members the working line may be taken as the gage line nearest the back of the angle.

92.88. **Camber:** In long spans and those designed to carry electric railways, the length of the truss members shall be such that the camber will equal the deflection produced by the dead load plus full live load without impact. Trusses shall in general be given a camber by increasing the length of the top chords an amount in each panel length equal to three-sixteenth \((\frac{3}{16})\) inch for each ten \((10)\) feet of their horizontal projection.

92.89. **Rigid Members in Pin-Connected Trusses:** Pin connected trusses shall have stiff riveted members in the first two main panels of the bottom chords at each end of the span, and all web members performing the function of suspenders shall be of stiff riveted construction.

92.90. **Counters and Adjustable Members:** If web members are subject to reversal of stress, their end connections shall be riveted. Rigid counters are preferred. Adjustable counters, when used, shall have open turn-buckles and in the design of these members an allowance of ten thousand \((10,000)\) pounds shall be made for initial stress. Only one set of diagonals in any panel shall be adjustable. Sleeve nuts and loop bars shall not be used.

92.91. **Eye-Bars:** Eye-Bar heads shall have a cross sectional area through the center of the pin hole exceeding that of the body of the bar by at least thirty-five \((35)\) percent. The net section adjacent to the head shall be not less than that of the main body of the bar. The thickness of the bar shall be not less than one-eighth \((\frac{1}{8})\) of the width and not less than one-half \((\frac{1}{2})\) inch and not greater than two \((2)\) inches. The form of the head shall be submitted to the Engineer for approval before the bars are made. The diameter of the pin shall be not less than three-fourths \((\frac{3}{4})\) of the width of the widest bar connected.

92.92. **Packing Eye-Bars:** The eye-bars of a set shall be packed symmetrically about the central plane of the truss and as nearly parallel as practicable, but in no case shall the inclination of any bar to the plane of the truss exceed one-sixteenth \((\frac{1}{16})\) inch per foot. Bars shall be packed as closely as practicable and held against lateral movement, but they shall be arranged so that adjacent bars in the same panel will be separated by at least one-half \((\frac{1}{2})\) inch.

All intersecting diagonal bars not far enough apart to clear each other at all times shall be well clamped together at intersections.

Steel filling rings shall be provided, when required, to prevent lateral movements of eye-bars or other members connected upon pins.
92.93. **Diaphragms:** Diaphragms shall be provided in the trusses at the end connections of all floorbeams. In general, such diaphragms shall extend down to the bottom flange of the floorbeam and for at least two (2) rivet spaces above the top flange.

The gusset plates engaging the pedestal pin at the end of a truss shall be rigidly connected by a diaphragm which shall, in general, take direct bearing on the pin. Similarly, the pedestal webs shall, where practicable, be connected by a diaphragm which shall, in general, take bearing on the pin.

A diaphragm shall be provided between gusset plates engaging main members whenever the end tie plate is located at a distance of four (4) feet or more from the point of intersection of the members. In general, the web of this diaphragm shall be located in the place of the latticed flange.

92.94. **Sole and Masonry Plates:** Sole and masonry plates supporting trusses and columns shall each have a thickness of not less than three quarter (3/4) inch. The bottom chords of trusses shall be raised above the bridge seat at least six (6) inches by the use of metal plates or pedestals.

92.95. **VIADUCTS.**

92.96. **Type:** Viaducts shall consist usually of alternate tower spans and free spans of plate girders or riveted trusses supported on trestle towers. However, in viaducts having a column height less than thirty-five (35) feet, trestle bents may alternate with the towers.

In viaducts requiring freedom of waterway and in structures having a less total column height than twenty (20) feet, the number of intermediate trestle bents may be increased over that specified above but, in general, shall not exceed four (4) in number. Ample rigidity shall be secured in the attachment of the superimposed spans to the column caps of the bents.

92.97. **Bents and Towers:** Each trestle bent shall be composed preferably of two main supporting columns. Towers shall be composed of two (2) bents rigidly braced and strutted both longitudinally and transversely.

92.98. **Single Bents:** Single bents shall have hinged ends, or else shall be designed to resist bending.

92.99. **Batter:** Bents preferably shall have a sufficient spread at base to prevent uplift under the assumed lateral loadings. In general, the width of the bent at its base shall not be less than one-third (1/3) of its height.

92.100. **Depth of Girders:** The depths of plate girders in viaducts preferable shall be uniform.

92.101. **Girder Connections:** Girders of tower spans shall be fastened at each end to the tops of the columns or to the cross girders. Preferably there shall be a line of girders resting directly over the columns. One end of the girders between towers shall be riveted to the support, and there shall be an effective expansion bearing at the other end. No bracing or sway frame shall be common to abutting spans.

If girders are not supported directly on the columns, provision shall be made for the transmission of the longitudinal forces to the tower bracing.

92.102. **Bracing:** Towers shall be thoroughly braced, both transversely and longitudinally, with stiff members having riveted connections. Longitudinal and transverse struts shall be placed at caps and bases and at all intermediate panel points. All bracing connections shall be made by gusset plates. The sections of members of longitudinal bracing in each panel shall not be less than those of the members in corresponding panels of the transverse bracing.

Column splices generally shall be located close to and above the panel points of the bracing.

Horizontal diagonal bracing shall be provided at the tops and bases of towers and at least at all intermediate panel points of the lateral bracing where the tower columns are spliced.

Provision shall be made in column bearings for expansion of the tower bracing. The struts at the base of towers shall be strong enough to slide the movable shoes with the structure unloaded. The coefficient of friction shall be taken at twenty-five hundredths (0.25).
92.103. **Sole and Masonry Plates:** Sole and masonry plates shall each be not less than three-quarter (\( \frac{3}{4} \)) inch thick.

92.104. **Anchorage:** Viaduct bents preferably shall have a sufficient spread at the base to prevent tension in any windward leg. When this is impracticable, the column anchorages shall be designed to safely resist not less than one and one-half (1\( \frac{1}{2} \)) times the calculated uplift.

92.105. **Approval of Plans:** The construction plans shall consist of shop detail, erection and other working plans showing details, dimensions, sizes of material and other information and data necessary for the complete fabrication and erection of the metal work. Approval of the construction plans shall be secured before fabrication of steel work is commenced.

The Contractor shall furnish the Engineer with such blueprint copies of the plans as may be required for approval and construction purposes and upon completion of the work the original plans if required shall be supplied to the Engineer. No deviation from the approved plans will be permitted without the written order of the Engineer.
SECTION 93

CONCRETE DESIGN

93.01. General Assumptions: The design of reinforced concrete members under these specifications shall be based on the following assumptions:

(a) Calculations are made with reference to unit working stresses and safe loads, as elsewhere specified herein, rather than with reference to ultimate strength and ultimate loads.

(b) A plane section before bending remains plane after bending.

(c) The modulus of elasticity of concrete in compression is constant within the limits of working stresses; the distribution of compressive stress in flexure is therefore rectilinear.

(d) The value of the modulus of elasticity of concrete shall be assumed as one-fifteenth (1/15) that of steel in computations of strength; and shall be assumed as one-eighth (1/8) that of steel in computing the deflection of reinforced concrete beams which are free to move longitudinally at the supports.

(e) Concrete shall be assumed as offering no tensile resistance.

(f) The bond between concrete and metal reinforcement is assumed to remain unbroken throughout the range of working stresses. Under compression the two materials are therefore stressed in proportion to their moduli of elasticity.

(g) Initial stress in the reinforcement due to contraction or expansion of the concrete is neglected, except in the design of reinforced concrete columns.

93.02. STANDARD NOTATION

Rectangular Beams:

\[ f_t = \text{tensile unit stress in longitudinal reinforcement.} \]

\[ f_c = \text{compressive unit stress in extreme fiber of concrete.} \]

\[ E_s = \text{modulus of elasticity of steel.} \]

\[ E_c = \text{modulus of elasticity of concrete.} \]

\[ n = \frac{E_s}{E_c} \]

\[ M = \text{bending moment, or moment of resistance in general.} \]

\[ A_s = \text{effective cross-sectional area of tension reinforcement.} \]

\[ b = \text{width of beam.} \]

\[ d = \text{effective depth, or depth from compression surface of beam to center of tension reinforcement.} \]

\[ k = \text{ratio of depth of neutral axis to effective depth, } d. \]

\[ j = \text{ratio of lever arm of resisting couple to depth, } d. \]

\[ jd = d - z = \text{arm of resisting couple.} \]

\[ p = \text{ratio of effective area of tension reinforcement to effective area of concrete in beam} = \frac{A_s}{bd} \]

T-Beams:

\[ b = \text{width of flange.} \]

\[ b' = \text{width of stem.} \]

\[ t = \text{thickness of flange.} \]

Beams Reinforced for Compression:

\[ A' = \text{area of compressive steel.} \]

\[ p' = \text{ratio of effective area of compression reinforcement to effective area of concrete in beam} = \frac{A'}{bd} \]
\( f' \) = compressive unit stress in longitudinal reinforcement.
\( C \) = total compressive stress in concrete.
\( C' \) = total compressive stress in steel.
\( d' \) = depth from compression surface of beam to center of compression reinforcement.
\( z \) = depth from compression surface of beam to resultant of compressive stresses.

**Shear, Bond and Web Reinforcement:**

\( V \) = total shear.
\( V' \) = external shear on any section after deducting that carried by the concrete.
\( v \) = shearing unit stress.
\( u \) = bond stress per unit of area of surface of bar.
\( o \) = perimeter of bar.
\( \Sigma o \) = sum of perimeters of bars in one set.
\( a \) = spacing of web reinforcement bars measured perpendicular to their direction.
\( s \) = spacing of web reinforcement bars, measured at the netural axis and in the direction of the longitudinal axis of the beam.
\( A_v \) = total area of web reinforcement in tension within a distance, \( a \), or the total area of all bars bent up in any one plane.
\( C_\theta \) = angle between web bars and longitudinal bars.
\( f_v \) = tensil unit stress in web reinforcement.

**Columns:**

\( A \) = total net area.
\( A_c \) = net area of concrete in the column (total column area minus steel area).
\( A_r \) = effective cross-sectional area of longitudinal reinforcement.
\( P \) = total safe load.

**DESIGN FORMULAS**

**Flexure of Rectangular Reinforced Concrete Beams and Slabs**

Computations of flexure in rectangular reinforced concrete beams and slabs shall be based on the following formulas:

(a) **Reinforced for Tension Only:** (See Figure 7)

Position of neutral axis,
\[
k = V2pn + \frac{(pn)^2}{-pn}
\]

Arm of resisting couple,
\[
j = 1 - \frac{k}{3}
\]

Compressive unit stress in extreme fiber of concrete,
\[
f_c = \frac{2M}{jkbd^2} = \frac{2pf_v}{k}
\]

Tensil unit stress in longitudinal reinforcement,
\[
f_s = \frac{M}{A_{0jd}} = \frac{M}{pjbd^2}
\]

Steel ratio for balanced reinforcement,
\[
p = \frac{1}{f_s} \left( \frac{f_s}{f_c} \right) \left( \frac{f_c}{nf_c + 1} \right)
\]

Note:—For approximate computations, the following assumptions may be made:
\[ j = \frac{7}{8} \]
\[ k = \frac{3}{8} \]
\[ A_s = \frac{M}{14000d} \]
\[ f_c = \frac{6M}{bd^2} \]

(b) **Reinforced for both Tension and Compression:** (See Figure 8)

Position of neutral axis
\[ k = \sqrt{2n \left( p + p' \frac{d'}{d} \right) + n^2 (p + p')} - n (p + p') \]

Position of resultant compression,
\[ z = \frac{1}{3k \cdot d + 2p'nd' \left( k - \frac{d'}{d} \right)} \]
\[ k + 2p'n \left( k - \frac{d'}{d} \right) \]

Arm of resisting couple,
\[ jd = d - z \]

Compressive unit stress in extreme fiber of concrete,
\[ f_c = \frac{6M}{bd^2 \left[ 3k - k^2 + \frac{6p'n}{k} \left( k - \frac{d'}{d} \right) \left( 1 - \frac{d'}{d} \right) \right]} \]

Tensile Stress in longitudinal reinforcement,
\[ f_s = \frac{M}{pjd^2} = nf_c \left( \frac{1-k}{k} \right) \]

Compressive stress in longitudinal reinforcement,
\[ f'_s = nf_c \left( \frac{k-d'}{d} \right) \]

**Flexure of Reinforced Concrete T-Beams:** (See Figure 9):

 Computations of flexure in reinforced concrete T-beams shall be based on the following formulas:

(a) **Neutral Axis in the Flange:**
Use the formulas for rectangular beams and slabs.

(b) **Neutral Axis Below the Flange:**
The following formulas neglect the compression in the stem.
Position of the neutral axis,
\[ kd = \frac{2ndA_s + bt^3}{2nA_s + 2bt} \]

Position of resultant compression,
\[ z = \left( \frac{3kd - 2t}{2kd - t} \right) \frac{t}{3} \]

Arm of resisting couple,
\[ jd = d - z \]
Compressive unit stress in extreme fiber of concrete,

\[ f_t = \frac{Mkd}{bt \left( kd - \frac{1}{2}t \right) jd} = f_n \left( \frac{k}{1-k} \right) \]

Tensile unit stress in longitudinal reinforcement,

\[ f_s = \frac{M}{A_{s}jd} \]

(For approximate results, the formulas for rectangular beams may be used),

The following formulas take into account the compression in the stem; they are recommended where the flange is small compared with the stem:

Position of neutral axis,

\[ kd = \sqrt{\frac{2ndA_s + (b-b')t}{b'} + \left( \frac{nA_s + (b-b')t}{b'} \right)^2} - \frac{nA_s + (b-b')t}{b'} \]

Position of resultant compression,

\[ z = \frac{(kd-t)^2 (t+1/3 (kd-t) b')}{t (2kd-t) b + (kd-t) b'} \]

Arm of resisting couple,

\[ jd = d - z \]

Compressive unit stress in extreme fiber of concrete,

\[ f_t = \frac{2Mkd}{(2kd-t) bt + (kd-t) b'} \]

Tensile unit stress in longitudinal reinforcement,

\[ f_s = \frac{M}{A_{s}jd} \]

**Shear, Bond and Web Reinforcement:**

Diagonal tension and shear in reinforced concrete beams shall be calculated by the following formulas:

Shearing unit stress,

\[ v = \frac{V}{bdj} \]

Stress in vertical web reinforcement,

\[ f_v = \frac{V's}{A_{s}jd} \]

When a series of web bars or bent-up longitudinal bars is used, the web reinforcement shall be designed in accordance with the formula:

\[ A_v = \frac{V'a}{f_{s,jd}} = \frac{V's \sin \theta}{f_{s,jd}} \]
When the web reinforcement consists of bars bent up in a single plane so as to reinforce all sections of the beam which require it, the bent up bars shall be designed in accordance with the formula:

\[ A' = \frac{A'}{f_v \sin \alpha} \]

The bond between concrete and reinforcement bars in reinforced concrete beams and slabs shall be computed by the formula:

\[ u = \frac{V}{j\delta \theta} \]

(For approximate results "j", in the above formulas, may be taken as 7/8.)

As regards shear and bond stress for tensile steel, the above formulas apply also to beams reinforced for compression.

**Columns with Lateral Ties:**

The safe axial load on columns reinforced with longitudinal bars and separate lateral ties or hoops shall be determined by the formula:

\[ P = f_c (A_1 + nA_e) = f_c A (1 + (n-1) p) \]

Compressive unit stress in concrete:

\[ f_c = \frac{P}{A (1 + (n-1) p)} \]

Compressive unit stress in longitudinal reinforcement,

\[ f_a = n f_c \]

**93.04. Effective Span Lengths:** The effective span lengths of freely supported beams and slabs shall be the distance between centers of the supports, but shall not exceed the clear span plus the depth of beam or slab. The span length for continuous or restrained beams built monolithically with supports shall be the clear distance between faces of supports. Where brackets having a width not less than the width of the beam, and making an angle of forty-five \((45°)\) degrees or more with the axis of a restrained beam, are built monolithically with the beam and support, the span shall be measured from the section where the combined depth of the beam and bracket is at least one-half \((1/2)\) more than the depth of the beam. Maximum negative moments are to be considered as existing at the ends of the span, as above defined. No portion of a bracket shall be considered as adding to the effective depth of the beam.

**93.05. Moments in Floor Slab:** Concrete floor slabs built continuously over supporting beams or joists shall be designed for eighty percent \((80%)\) of the maximum live load bending moment of a simply supported slab of the same span.

**93.06. Expansion Joints:** Provision for end expansion shall be made in all concrete slabs or girder bridges having a clear span length in excess of forty feet \((40')\). When multiple span construction is used, of spans forty feet \((40')\) or less in length expansion joints shall be provided at intervals of not more than eighty \((80')\) feet. When no expansion joints are used, the superstructure being cast integrally with the abutments, the reinforcement in the slab or girders shall be increased to provide for the thermal forces induced by a temperature drop of forty \((40)\) degrees Fahrenheit.

In concrete floors or metal structures, expansion joints shall be provided at both fixed and expansion ends of the span.

**93.07. T-BEAMS.**

**Effective Flange Width:** In beam and slab construction, effective and adequate bond and shear resistance shall be provided at the junction of the beam and slab. The slab may then be considered an integral part of the beam but its assumed effective width as a T-beam flange shall not exceed the following:

(a) One-fourth of the span length of the beam.
(b) The distance center to center of beam.
(c) Six times the width of the beam.
(d) Eight times the least thickness of the slab plus the width of the girder stem.
Shear: The flange of the slab shall not be considered as effective in computing the shear and diagonal tension resistance of T-beams.

Isolated Beams: Isolated beams in which the T-form is used only for the purpose of providing additional compression area, shall have a flange thickness of not less than one-half ($\frac{1}{2}$) the width of the web, and a total flange width of not more than four (4) times the web thickness.

Diaphragms: For T-beams spans over forty (40) feet in length diaphragms or spreaders shall be placed between the beams at the middle or third points.

REINFORCEMENT

Spacing: The clear distance between reinforcing bars preferably shall not be less than three (3) inches and, in slabs, not more than one and one-half ($1\frac{1}{2}$) times the thickness of the slabs.

The minimum covering, measured from the surface of the concrete to the face of any reinforcing bar, shall be not less than two (2) inches except in slabs where the minimum covering shall be one (1) inch. In the footings of abutments and retaining walls and in piers the minimum covering shall be three (3) inches. In work exposed to the action of sea water the minimum covering shall be four (4) inches except in precast concrete piles where a minimum of three (3) inches shall be used.

Splicing: Tensile reinforcement shall not be spliced at points of maximum stress. When reinforcement is spliced, the splice bars shall lap sufficiently to develop the full strength in bond.

Anchorage: Anchorage of longitudinal reinforcement may be provided by extending the bars a sufficient distance beyond the theoretical point of termination to develop their full strength in bond. Anchorage may also be provided by bending the end of the bar through one-hundred-eighty (180) degrees to a diameter not less than six (6) times the diameter of the bar, the total length of the hook being not less than sixteen (16) diameters of the bar.

Reinforcement for negative moment shall be thoroughly anchored at or across the support, or shall extend into the span a sufficient distance to develop by bond the tensile stresses.

Maximum Sizes: The maximum size of bar reinforcement shall be one and one-fourth ($1\frac{1}{4}$) inches square or equivalent, unless the particular conditions warrant the adoption of special reinforcement design. When structural steel shapes are used for reinforcement, no section having a surfaced area per foot of length of more than one-hundred-fifty (150) square inches shall be used as a reinforcing member unless mechanical bond is provided by means of lugs, bars or other details which will effectively bond the member to the surrounding concrete mass.

93.09. Design of Web Reinforcement: When the allowable unit shearing stress for concrete is exceeded, web reinforcement shall be provided by one of the following methods:

(a) Longitudinal bars bent up in series or in a single plane.
(b) Vertical stirrups.
(c) Combination of bent-up bars and vertical stirrups.

When any of the above methods of reinforcement are used, the concrete may be assumed to carry external vertical shear not to exceed forty (40) pounds per square inch, the remainder of the shear being carried by the web reinforcement.

The webs of T-beams shall be reinforced with vertical stirrups in all cases.

Bent-Up Bars: Bent-up bars used as web reinforcement may be bent at any angle between twenty (20) and forty-five (45) degrees with the longitudinal reinforcement. The radius of bend shall not be less than four (4) diameters of the bar.

The spacing of bent-up bars shall be measured perpendicular to their direction and in a plane parallel to the longitudinal axis of the beam. This spacing shall not exceed three-fourths ($\frac{3}{4}$) of the effective depth of beam. The first bar from the support shall cross the neutral axis of the beam at a distance from the face of the support, measured along the axis of the beam, not greater than one-fourth ($\frac{1}{4}$) of the effective depth.
Vertical Stirrups: The spacing of vertical stirrups shall not exceed three-fourths ($3/4$) of the effective depth of the beam. The first stirrup shall be placed at a distance from the face of the support not greater than one-fourth ($1/4$) of the effective depth of the beam. Stirrups shall surround three sides of the tensile reinforcement.

Anchorage: Web reinforcement shall be securely anchored in the compression portion of the beam, which may be considered as developing bond for a vertical distance equal to four-tenths ($4/10$) the effective depth of the beam. Stirrups and bent-up bars shall be securely anchored to the tensile reinforcement.

93.10. Columns: The ratio of the unsupported length of a column to its least diameter or dimension shall not exceed four (4) for unreinforced and fifteen (15) for reinforced sections. The least diameter or dimension in no case shall be less than fifteen (15) inches. For reinforced concrete viaduct construction or for “pedestal” or “buried” abutments, the least dimension shall be not less than twenty-four (24) inches.

The reinforcement of columns shall consist of at least four (4) longitudinal bars tied together with lateral ties or hoops enclosing the longitudinal reinforcement. The longitudinal reinforcement shall be of bars not less than one (1) inch in diameter and shall have a total cross-sectional area of not less than seven-tenths (0.7% per cent of the total cross-sectional area of the column. Reinforcement in excess of two (2%) per cent of the cross-sectional area of the column shall not be considered in computing compressive stresses. The lateral ties or hoops shall be not less than one-fourth ($1/4$) inch in diameter and space not farther apart than twelve (12) inches.

Bending: When columns are subjected to bending stresses due to eccentric loads, monolithic construction or lateral forces, they shall be so proportioned that the combined direct and bending stresses shall not exceed the allowable unit compressive stresses herein specified.

Columns placed in earth fills, as in the case of “pedestal” or “buried” abutments, shall be designed to withstand the earth pressure from the rear, disregarding the effect of the fill in front.

Column Struts: Longitudinal or lateral struts used to brace columns shall be proportioned to support a uniform load equal to at least twice the dead load of the strut.

93.11. CONCRETE ARCHES

Shape of Arch Ring: Arch rings shall be selected as to shape in such a manner that the axis of the ring shall conform, as nearly as practicable to the equilibrium polygon for full dead load or, if desired, to the equilibrium polygon for full dead plus one-half ($1/2$) live load over the full span.

Spandrel Walls: When the spandrel walls of filled spandrel arches exceeds five (5) feet in height above the intrados they shall be designed as vertical slabs supported by transverse diaphragm walls or deep counterforts. Vertical cantilever walls over five (5) feet in height, or counterforts having a back slope of less than forty-five (45) degrees with the vertical, shall not be used, on account of the excessive and indeterminate stresses set up in the arch ring by torsion.

Expansion Joints: Vertical expansion joints shall be placed in the spandrel walls of arches to provide for the movement due to the temperature change and arch deflection. These joints shall be placed at the ends of spans and at intermediate points, generally not more than fifty (50) feet apart.

Reinforcement: Arch rings in reinforced concrete construction shall be reinforced with a complete double line of longitudinal reinforcement consisting of an intradosal system and an extradosal system connected by a series of stirrups or tie-rods.

For barrel arches, a system of transverse reinforcement, thoroughly anchored to the longitudinal reinforcement, shall be used in both intrados and extrados. The transverse reinforcement shall be proportioned to resist the bending stresses, due to any overturning action of the spandrel wall.

For rib arches, hoops or tie bars shall be used in connection with the longitudinal rib reinforcement, as in the case of reinforced concrete columns due to any overturning action of the spandrel wall.
Waterproofing: Preferably, the top of the arch ring and the interior faces of the spandrel walls of all filled spandrel arches shall be waterproofed with a membrane waterproofing constructed in accordance with the requirements specified in Section 84.

Drainage of Spandrel Fill: The fills of filled spandrel arches shall be effectively drained by a system of tile drains or French drains laid along the intersections of the spandrel walls and arch ring and discharging through suitable outlets in the piers and abutments. The location and detail of the drainage outlets shall be such as to eliminate, as far as possible, the discoloration by drainage water of the exposed masonry faces.

93.12. Viaduct Bents and Towers: When concrete columns are used in viaduct construction, bents and towers shall be effectively braced by means of longitudinal and transverse struts. For heights greater than forty (40) feet, both longitudinal and transverse cross or diagonal bracing preferably shall be used and the footings for the columns forming a single bent shall be thoroughly tied together.
SETION 94

DESIGN OF TIMBER STRUCTURES

94.01. **Bolts:** Bolts of diameters not exceeding one (1) inch preferably shall be spaced not closer than six (6) inches center to center, not less than six (6) inches from the center of the bolt to the end of any timber, and not less than two and one-half (2 1/2) inches from the center of the bolt to the other side of any timber. These distances preferably shall be increased for bolts larger than one inch (1") in diameter. Inclined bolts through timber preferably shall be provided with beveled cast washers to eliminate the cutting of inclined daps in the timber.

94.02. **Washers:** A washer shall be used under all bolt heads and nuts which would otherwise come in contact with wood. Washers may be cast of plate and shall be designed to prevent excessive crushing of the wood when the bolts are tightened. For bolts in an important location, such as joints and splices, and for rods, the washers shall be designated to develop the bolt or rod in tension at the unit bearing stresses specified for compression perpendicular to the grain of timber.

A standard circular washer shall be used under the heads of all lag screws.

94.03. **Hardware for Sea-Coast Structures:** The hardware used in structures on the sea-coast preferably shall be galvanized.

94.04. **Columns and Posts:** No column shall have an unsupported length greater than thirty (30) times its least dimension.

The strength of built-up columns composed of two or more sticks bolted together, either with or without packing blocks, shall be considered as equal to the combined strength of the single sticks, each considered as an independent column.

94.05. **Pile and Framed Bents:**

**Pile Bents:** Pile bents generally shall not exceed forty (40) feet in height. Pile bents over ten (10) feet high shall be sway-braced transversely with diagonal braces on each side of the bent, and shall be adequately braced longitudinally. In general, pile bents shall contain not less than four bents each and the outside piles preferably shall be battered. The pile shall be designed for safe bearing and for column action.

**Framed Bents:** Framed bents may be supported on piles, concrete pedestals or mud sills. All bents shall be sway-braced transversely and adequate provision shall be made for longitudinal bracing. In general, framed bents shall contain not less than four (4) posts each and the outside post of the bent shall be battered. The posts shall be designed as columns.

**Sills and Mud Sills:** Mud sills, and all sills which are to be located in close proximity to the ground surface, preferably shall be given a preservative treatment. When possible, the design shall be such as to insure that sills will be located clear of all earth so that there may be a free circulation of air around them. Sills shall be fastened to mud sills or piles with drift bolts of not less than three-fourths (3/4) inch diameter and extending into the mud sills or piles at least six (6) inches. Sills shall be fastened to pedestals with dowels of not less than three-fourths (3/4) inch diameter, set in the pedestals and extending into the sills at least six (6) inches.

Posts shall be fastened to sills by dowels of not less than three-fourths (3/4) inch diameter extending at least six (6) inches into posts and sills, or by drift bolts of not less than three-fourths (3/4) inches diameter driven diagonally through the base of the posts and extending at least nine (9) inches into the sill. Posts shall be fastened to pedestals with dowels of not less than three-fourths (3/4) inch diameter and extending into the posts at least six (6) inches.

**Caps:** Timber caps shall be not less in size than ten (10) by ten (10) inches. They shall be fastened with drift bolts of not less than three-fourths (3/4) inch diameter, extending at least nine (9) inches into the piles or posts.

**Bracing:** Single story bracing shall not exceed twenty (20) feet in height. The minimum size of transverse sway braces shall be three (3) by eight (8) inches. All bracing shall be bolted through the piles, posts or caps at the ends; at intermediate intersections it may be bolted or spiked. In all cases, spikes shall be provided in addition to bolts. The bolts used shall be of not less than five-eighths (5/8) inch diameter.
Pile Bent Abutments: Pile bent abutments shall be adequately braced or anchored to resist earth pressure. Bulkhead plank shall be not less than three (3) inches thick and preferably shall be treated. It shall be fastened to the piles with spikes, the length of which shall be at least three (3) inches greater than the thickness of the plank.

94.06. TRUSSES

Joints and Splices: Joints shall be detailed to shed water to the maximum degree practicable. Joints and splices shall be designed to develop the computed stresses in the members connected and preferably to develop the full strength of those members. Posts or struts bearing against the sides of timber members preferably shall be provided with metal end bearings. Joints involving end bearing or inclined surfaces shall be avoided, preference being given to square-cut ends of timber bearings against blocks.

In end-shoe plates and tension-splice plates the bearing faces of lugs or tables shall have a smooth even surface. If rolled plates or bars are used for tables, they shall be milled or cold sawed on the bearing edges. The bolts holding the lugs or tables in the notches in the timber shall be placed as near to the lugs or tables as possible. No metal lug or table shall have a bearing face less than five-eights (5/8) inch thick. In details of end-shoes employing lugs of tables set in the lower chord, the spacing of such lugs or tables shall be arranged so that no lug or table occurs directly under the end of the end post. The end joint between lower chord and end post shall provide definite lines of action and shall be a simple joint depending for its strength upon type of detail. When inclined bolts are used to connect the main members of an end-joint, such bolts shall be at an angle of not more than sixty (60) degrees with the center line of lower chord. Holes in timbers for inclined bolts in details employing end-shoe plates shall be one-fourth (1/4) inch larger than the nominal diameter of the bolt.

No daps in chords for butt blocks shall be less than three-fourths (3/4) inch deep.

Tension splices shall be of such a type that the effects of cross shrinkage of the timber will be a minimum. Neither steel table fish plates nor shear pin splices shall be used on timbers over eight (8) inches thick, since the cross shrinkage of the timber will allow the plates or pads to separate. The shear-pin joint shall be used only with fully seasoned timber and gas-pipe shall not be used for shear pins.

Floorbeams: Floorbeams shall be sized at bearing points. In floorbeams composed of two or more timbers, the timbers shall be separated by at least two (2) inches for air circulation. Floorbeams shall be connected to the main truss members by means of rods or structural shapes.

Hangers: Hangers generally shall be rods having upset ends with a suitably designed washer or bearing plate at each end. Upset ends shall conform to the requirements specified for Structural Steel Design, Division 4, Paragraph 4.6.00.

Eyebars and Counters: The requirements specified for Structural Steel Design, Section 991 for counters, eyebars and eyebar packing shall apply to such members when used in timber trusses.

Bracing: Timber trusses shall be provided with a rigid system of laterals in the plane of the loaded chord. When the details will permit, this lateral bracing shall be securely fastened to all longitudinal stringers. Lateral bracing, preferably rigid, in the plane of the unloaded chord, and rigid portal and sway bracing shall be provided in all trusses having sufficient head room. Outrigger brackets connected to extensions of the floorbeams shall be used for bracing through trusses having head room insufficient for a top lateral system.

Camber: Camber, in addition to that required to provide for dead load and shrinkage, shall be provided in timber trusses in sufficient amount to give the structure a good appearance.

94.07. FLOORS AND RAILING

Stringers: Stringers shall be of sufficient length to take bearing over the full width of caps or floorbeams, except outside stringers which may have butt joints. Preferably they shall be of two panel lengths placed with staggered joints. The lapped ends of untreated stringers shall be separated at least one-half (1/2) inch for air circulation. Stringers shall be secured to caps or floorbeams.

Bridging: Stringers shall be braced by cross bridging in each panel. The bridging shall be not less in size than two (2) inches by four (4) inches.
**Nailing Strips:** When timber floors are supported by steel joists, the joists shall be provided with nailing strips which shall be bolted either to the top flanges or the webs.

When nailing strips are bolted to the flanges they shall be used on all joists. They shall be not less than four (4) inches deep and shall be wider than the supporting flange. They shall be secured with five-eighths (5/8) inch bolts through the flanges, spaced not more than four (4) feet apart and not more than eighteen (18) inches from the ends of the strips.

Nailing strips bolted to the webs shall be not farther apart than five (5) feet and shall be not less than four (4) inches thick to provide a spiking face of sufficient width. They shall be held clear of the flanges by blocks between the web and strip, and bolted through the web with five-eights (5/8) inch bolts spaced not more than four (4) feet apart and not more than eighteen (18) inches from the ends of the strips.

**Flooring:** Roadway floor plank shall have a nominal thickness of not less than three (3) inches. Sidewalk floor plank shall have a nominal thickness of not less than two (2) inches.

The minimum size of material used for laminated or strip floors shall be two (2) inches by four (4) inches.

**Retaining Pieces:** Retaining pieces, where required, shall be not less than six (6) inches in width. In general they shall be secured in place by five-eighths (5/8) inch bolts at three (3) foot intervals and spiked at one (1) foot intervals.

**Wheel Guards:** Wheel guards having a cross section of not less than four (4) inches by six (6) inches shall be provided on each side of the roadway. The guard timbers shall be in lengths of not less than twelve (12) feet. They shall be secured with five-eights (5/8) inch bolts at the ends and at intermediate points not more than four (4) feet apart.

In strip floors or cambered floors, not provided with retaining pieces, the wheel guards shall be placed directly on the flooring with scupper holes at suitable intervals. In other floors the wheel guards shall be supported by scupper blocks not less than four (4) inches thick and one (1) foot long, held in place by spikes and a bolt through the wheel guard and flooring, and spaced not more than five (5) feet apart.

**Cambered Floors:** In strip floors or floors crowned for drainage, the ends of the flooring shall be securely held down by the retaining pieces or wheel guards. In this case the bolts through the retaining pieces or wheel guards shall pass through the flooring and through the outside stringer or spiking piece.

**Drainage:** Adequate provision shall be made for the proper drainage of timber floors.

**Railings:** Wood railings shall consist of not less than two horizontal lines of rails. Rails shall have a cross section not less than two (2) inches by six (6) inches.

Rail posts shall have a cross section not less than four (4) inches by six (6) inches and shall be spaced not more than eight (8) feet apart.

Preferably, rails shall be surfaced four sides (S4S) and painted white.

**Fire Stops:** To check the spread of fire lengthwise of the structure, timber floors or trestles of any considerably length preferably shall be provided with fire stops.

In timber floors these fire stops should be provided at intervals not over seventy-five (75) feet apart. They may consist of diaphragms of wood or fire resistant material at least as thick as the flooring, located over caps or floor beams and completely filling the openings between the joists.

In timber trestle bridges, in addition to the fire stops in the floor, fire curtains should be provided at intervals of one hundred (100) feet or more. These curtains may consist of plank or asbestos-covered metal spiked to the bents. They should extend downward from the bottom of the joists at least five (5) feet and horizontally at least to the ends of the caps. A fire stop between the joists should be located over each curtain.
APPENDIX - B

STEEL CANTILEVER
WITH A SUSPENDED SPAN

By MR. S. E. BRASWELL

Also CONCRETE ARCH by
ELASTIC THEORY

By MR. S. E. BRASWELL
ECONOMIC ANALYSIS AND TYPE SELECTION FOR A HIGHWAY BRIDGE OVER TALLULAH GORGE

A thesis
Submitted for the Degree of
MASTER OF SCIENCE IN CIVIL ENGINEERING
by
Samuel E. Braswell
B. S. in C. E. 1925
Georgia School of Technology

Approved by
Professor of Civil Engineering.
June 1930
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Three-Hinged Steel Arch and Concrete Arch by
Cochrane's Method. Thesis by Mr. G. T. Barkin--- Appendix A
Specifications ------------------------------- Appendix B
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This thesis has been prepared to give the design of a steel arched cantilever highway bridge, an open spandrel reinforced concrete arch highway bridge and a three-hinged steel arch highway bridge, for the spanning of Tallulah River gorge, at Tallulah Falls, Georgia and to make a comparison between the three bridges to determine which bridge would be the most economical one to build at that particular crossing.

When it was decided to prepare a thesis on the economic design of a highway bridge, it was desired first to learn as much about the different types of bridges considered as possible, then to put the design and data in such form that it would be of use to others.

In the beginning it was thought that more information would be gained and that the subject matter of the thesis would be accepted with more confidence by the writer and by others if the design and obtaining of quantities were done according to accepted methods employed by all designers throughout the country. That is, that all designs and quantities were checked by some one other than the designer. With this in view, the writer and Mr. G. T. Parkin, another candidate for a master's degree, chose the same subject for their thesis, and checked one another on their design and quantities.

After all calculations were made and checked for design and quantities, an equal division was attempted, by
Mr. Parkin taking the three hinged steel arch, and the open spandrel reinforced concrete arch, designed by Cochrane's Method, and the writer took the steel arched cantilever and the open spandrel reinforced concrete arch designed by the Elastic Theory as modified by Urquhart and O'Rourke. With this division made each worked independently of the other in writing his thesis with the exception of the occasional asking of opinions.

It has been attempted to give proper credit in the body of the thesis for all data and detail obtained from other books. For theory and current, practice, the following books were used: "Movable and Long Span Bridges," Vol. 3. Hool and Kinne; "Roofs and Bridges, Part 4, Merriman and Jacobey," "Modern Framed Structures," Part 1, 2, and 3 Johnson, Bryan and Turneaure; "Notes on the Design of Concrete Structures" and "Engineering Economics," Prof. F. C. Snow, Georgia School of Technology, "Design of Concrete Structures," Urquhart and O'Rourke; "Concrete Engineers Hand Book," Hool and Johnson; "Economics of Highway Bridge Types," McCullough; "Analysis of one Section of a Reinforced Concrete Arch," G. T. Parkin.

The writer is greatly indebted and wishes to express his appreciation to Profs. F. C. Snow and J. M. Smith for their many helpful suggestions and criticisms and their untiring efforts in recommending and securing reference books and data for use in this thesis, and for their assistance
in the planning of this thesis.

The writer is under obligation to the following men and organizations for their cooperation in the furnishing of data and information; Mr. C. N. Crocker, Bridge Department, State Highway Department of Georgia; Mr. R. M. Mason, Blue Print Department, State Highway Department of Georgia; Mr. P. A. Blackwell, Chief Engineer Virginia Bridge and Iron Co.; The American Institute of Steel Construction; and the American Bridge Co.

Special mention must be made and proper credit given to Mr. G. T. Parkin for his spirit of cooperation, and industriousness in checking the writer's work and in planning his thesis so that the desired end could be obtained and for his many helpful suggestions and criticisms.

The body of the thesis was divided so that each type of bridge was presented separate and complete within its self. This was done so that any one interested in only one of the particular types could turn to that one immediately.

A complete copy of Mr. Parkin's thesis has been included in the appendix of this thesis. This was done to give source of information and data about the three hinged steel arched bridge which was used in the comparison of the three types, and because the writer desired to have under one cover for future reference the design and details of the three hinged steel arch and the design and details of the
concrete arch by Cochrane's method.

Samuel Eric Braswell

Atlanta, Georgia

June 1930.
PART I

STEEL ARCHED CANTILEVER

HIGHWAY BRIDGE
STEEL ARCHED CANTILEVER BRIDGE

A steel arched cantilever bridge was one of the types of highway bridges selected for investigation for the spanning of Tallulah Gorge at Tallulah Falls, Ga.

Tallulah Falls, Ga. is on a branch line of the Southern R. R. and is approximately twenty miles north of Cornelia, in the mountaineous section of Northeast Georgia. It is a section of beautiful mountaineous scenery and is popular with tourists in the summer.

At present the only highway structure across the gorge for a distance of forty miles is a narrow one lane bridge, built on top of a gravity type concrete dam of the Georgia Power Co. The narrow bridge is insufficient for the traffic demands made upon it and the top of the dam is beginning to show cracks from the vibration of traffic traveling over the bridge.

The proposed site is 60 ft. downstream from the present bridge on top of the dam, and enables the securing of a more favorable approach alignment. The north and south approach, of the present bridge, each have two short radius curves ending at the bridge. The proposed bridge site will eliminate one of these curves and increase the radius of the other.

The arched cantilever was selected as one type for consideration, because of its potential aesthetical qualities, possible economy of material, and because it
could be erected by the cantilever method, eliminating false-work. The finished grade of the proposed bridge at the center of the span is 150 ft. from the bottom of the gorge. The depth and width of the gorge would call for the use of considerable false work to erect a simple truss.

As can be seen from the elevation of the bridge print #1, the bridge is 402 ft. long. It consists of a suspended span of 78 ft. 0 in., two cantilever arms of 72 ft. 0 in. and two anchor arms of 90 ft. 0 in. To enable a comparison to be made between the different types of bridges, which might have different lengths, a project length of 420 ft. was taken which included the bridge and a part of the approach embankment.

The rise of the main span, which is 222 ft. 0 in. c. to c. of piers, was taken as 30 ft. 0 in. This gave a ratio of rise to span of approximately 1/7 which came within the accepted economical rise ratio of from 1/6 to 1/10.

The upper chord was given a parabolic camber of 1 ft. 6 in. in 201 ft. The cross beams and stringers were detailed so as to reflect this camber in the finished grade of the floor slab of the bridge.

The bridge was designed according to Standard Specifications of the State Highway Dept. of Georgia, dated 1928 (see appendix). H-15 loading was used which in truck-train loading is one 15-ton truck in a train of 11½ ton trucks, and is used for span lengths of less than
60 ft. In H-15 equivalent loading, which is used for span lengths of 60 ft. or more, a uniform load of 480 lb. per linear foot of lane with one concentrated load of 13500 lb. for moment or 19500 lb. for shear is used.

In designing the suspended span under vertical loads, the stresses were determined by the use of influence lines, see "Modern Framed Structures", Johnson, Bryan, Turneaure, Part 1, Chapter V. The suspended span did not vary from ordinary design except in member L-5 L-10 which was a connecting member between the cantilever arm and the suspended span. It carried no stress, so it was proportioned to harmonize with the other members of the truss and cantilever arm.

Stresses in the members of the cantilever arms, due to vertical loads, were found by Maxwell diagrams and did not call for any variation of common practice, except in determining the stress in U-5 L-5. This member was common to the cantilever arm and anchor arm, so it was necessary to consider it in that light rather than as a member of the cantilever arm, or as a member of the anchor arm. The stress in U-5 L-5 was determined by the summation of vertical components at the panel point L-5.

Stresses in the anchor arms were determined by influence lines with the exception of U-5 L-5 as explained above.

The lateral forces considered were the wind forces
on the structure and the lateral force due to the moving live load and the wind pressure against it. In considering the effect of lateral forces, the suspended span and anchor arms were treated as simple trusses while the cantilever arms were treated as a truss cantilevered out from a support.

When the cantilever arm is acted upon by wind the upper lateral system, or bracing of the upper chord, can only transmit forces to the pier by virtue of its stiffness as a cantilever. A portion of these stresses is doubtless transmitted in this way from the central portion of the span back to the upper panel points above the pier, then down the vertical cross frame to the pier. By far the greater portion of these forces is transmitted vertically at each panel point to the lower lateral system by virtue of the cross frame or vertical sway bracing at such panel point.

The upper lateral system, therefore, may be considered to transmit a certain percentage of the total wind loadings active against its panel points, but for the sake of rigidity, should doubtless be designed to withstand the entire wind pressure.

The two preceding paragraphs were taken from "Movable & Long Span Steel Bridges", Hool & Kinne, Section 8, page 388. For a complete discussion of this method of treating wind stresses see the book referred to above. The path through which the stresses due to the wind pass is called path "A" in Hool & Kinne.
While the upper lateral system was designed to transmit the total wind loading to the pier for the sake of rigidity, it was known that only a percentage of the wind loading actually acted through the upper lateral system. The remaining percentage of wind loading was transmitted from the upper panel points to the lower panel points by the cross frames, and from the lower panel points to the piers by means of the lower chord.

Due to the fact that the percentage of the wind loading traveling through path "A" to the piers, and the percentage of the wind loading that is transmitted through the cross frames to the lower panel points and then to the piers is not known, it is deemed advisable for safety and rigidity to consider the full wind loading to travel through each path. By far the greater percentage will be transmitted through the cross frames but the additional rigidity obtained by the increased size of the upper lateral system warrants the assumption that the wind forces travel through both paths.

A 15-ton crane was decided upon for erection purposes. In using a 15-ton crane the bridge would not be subjected during erection to its designed live load capacity. The dead loads during erection would be much less than the dead loads designed for so it was obvious that erection stresses would not equal, much less exceed the design stresses under H-15 loading.

In an arched cantilever bridge the economical ratios
of span lengths will vary from the economical ratios of span lengths in a truss cantilever bridge. However, the general proportioning of the total lengths of bridge will hold for the two different types. In general, the suspended span should be longer than the cantilever arm, this tends to reduce the moment at the piers, and the anchor arm should be long to decrease or avoid anchorages.

It was found necessary to use a relative short suspended span of 78 ft. 0 in. to avoid undesirable depth in the center of the bridge. If the depth at the center of the bridge is increased, the length of all columns from the center of the bridge to the piers are increased because the lower chord is parabolic. The short suspended span, and the location of the piers being fixed by the ground contour, resulted in a cantilever arm of 72 ft. 0 in. which was longer than the ideal length would have been, and an anchor arm of 90 ft. 0 in. which was shorter than the ideal length would have been. However, the variation from the ideal lengths were thought to be offset by economy in the pier heights and the using of the anchorages as abutments.

In the floor system a 7 in. concrete slab reinforced top and bottom was used with a 2 in. Bituminous Mat as a wearing surface. A thin slab reinforced top and bottom was used to reduce the dead load. The curb was 9 in. deep and 10 in. wide. The slab was supported by stringers spaced 4 ft. 0 in. c to c, which framed in with the cross beams at panel points.
The railing as designed of small angles had a weight of 45 lb. per foot of railing. This seemed rather light, but was deemed to have sufficient strength for safety. Throughout the structure every expedient was used to decrease the dead load, because it was believed that it would prove economical in the long run.

DESIGN
SUSPENDED SPAN

Dead Loads.

D. L. per foot of truss for slab and paving 1125 lb.
" " " " curb 167 "

\[ U_0, 13', U_{10}, 13, U_6, 13, U_2 \]

\[ U_9, 8.4, 13, 8.4, 13, 8.4 \]

\[ L_{10}, 13, 15.5, 8.4, 15.5, 8.4, 15.5 \]

\[ L_{11}, 13, 15.5, 8.4, 15.5, 8.4, 15.5 \]

\[ L_{12}, 13, 15.5, 8.4, 15.5, 8.4, 15.5 \]

\[ 6 @ 13' = 78' - 0'' \]
Wt. of truss per lin. ft. of truss
(Estimated from "Steel Highway Bridges"

Total dead load per lin. ft. of truss. 1742 lb.

Live loads.

For live loads the H-15 equivalent loading was used
which is:

Equivalent uniform live load per ft. of truss 480 lb.
plus a concentrated live load of 13500 lb. for B. M. or
19500 lb. for shear.

Impact.

For impact see Ga. State Highway Dept. Standard
Specifications in appendix.

Determination of stresses.

Influence lines were used entirely in determining
dead load and live load stresses in the truss members of the
suspended span. The wind bracing was designed in the usual
way for a simple truss.

Sample calculations for the plotting of influence
lines and their use in determining stresses in the suspended
span are given below using U-11 L-11 as a typical example.
For a detailed discussion of influence lines see, "Modern
Framed Structures", Johnson, Bryan, Turnesure, Part 1
Chapter V.
Pass section a-a through U-10 U-11, U-11 L-11 and L-11 L-12.

Calculation for influence line for U-11 L-11:

L-11 L-12 intersects upper chord at pt. (A) 279 ft. from R-1.
Place unit load at U'-9. R-1 = 0.
Moment about (A) = 0. Stress in U-11 L-11 = 0.
Place unit load at U-10, R-1 = 2/3.
Moment about (A), 2/3 X 297 - 271 U-11 L-11 = 0.

U-11 L-11 = \( \pm 0.73 \)

Stress in U-11 L-11 due to unit load placed at U-11 = \( \pm 0.73 \)
Place unit load at U-10, R-1 = 5/6
Moment about (A), 5/6 X 297 - 271 U-11 L-11 - 284 X 1 = 0

U-11 L-11 = \( -0.133 \)

Stress in U-11 L-11 due to unit load placed at U-10 = \( -0.133 \)
(Note: When unit load is placed to left of section a-a it gives a moment in the equation for equilibrium).

Place unit load at U-9, R-1 = 1
Moment about (.) = 0
Scales; Vert. 1" = 1/8
Horiz. 1" = 10'

Comp. area = 63.2 X .73 / 2 = 23.07
Ten. area = 14.8 X .133 / 2 = 0.984

Obtaining stresses from influence lines:

Compression,

D. L. = D. L. per lin. ft. X area = 1742 X 23.07 = 40200 lb.
U. L. L. = U.L.L. " " " = 480 X 23.07 = 11100 "
C. L. L. = C.L.L. X ordinate = 13500 X .73 = 9850 "
Impact = 26.6% (U.L.L. ≠ C.L.L.) = .266 (11100 ≠ 9850 )
Total ----------------------------------------------- 66730 "

Tension,

D. L. = D.L. per lin. ft. X area = 1742 X .984 = 1710 lb.
U. L. L. = U.L.L. " " " = 480 X .984 = 470 "
C. L. L. = C.L.L. X ordinate = 13500 X .133 = 1790 "
Impact = 35.7% (U.L.L. ≠ C.L.L.)
= .357 (11100 ≠ 9850 )
Total ----------------------------------------------- 4776 "
Design stress = total compression - dead load tension

\[= 66730 \text{ lb.} - 1710 \text{ lb.} = 65020 \text{ lb.}\]

For stresses and design of members in suspended span see, "Suspended Truss Table," Print No. 2.

CANTILEVER ARM

Dead Loads.
Concentrated load from suspended truss, taken from designed weight of truss.

Load from one truss:

Steel, \(71830 / 2\) = 35915 lb.

Slab and curb = 99500 "

Total from one suspended truss ------- 135415 "

Total concentrated at U-9 = 135415 / 2 = 67707 "

\[= 1710 \text{ lb.} = 65020 \text{ lb.}\]
## SUSPENDED TRUSS TABLE

<table>
<thead>
<tr>
<th>Member</th>
<th>Des Stress Kips</th>
<th>Prop of Section A</th>
<th>Unsurf Length inches</th>
<th>Section Used</th>
<th>Unit Stress Allowable #</th>
<th>Unit Stress Final #</th>
<th>Length in Feet</th>
<th>Wt. of Mem without Lacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>U₀ - U₀</td>
<td>122</td>
<td>11.72</td>
<td>3.66</td>
<td>156</td>
<td>426 2-10&quot;E @ 20&quot;</td>
<td>14100</td>
<td>13</td>
<td>520</td>
</tr>
<tr>
<td>U₀ - U₄</td>
<td>224</td>
<td>17.60</td>
<td>3.42</td>
<td>156</td>
<td>456 2-10&quot;E @ 30&quot;</td>
<td>13900</td>
<td>13</td>
<td>180</td>
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<tr>
<td>U₁ - U₂</td>
<td>265</td>
<td>20.54</td>
<td>3.34</td>
<td>156</td>
<td>466 2-10&quot;E @ 3.5&quot;</td>
<td>13800</td>
<td>13</td>
<td>910</td>
</tr>
<tr>
<td>L₉ - L₁₀</td>
<td>124</td>
<td>11.72</td>
<td>3.71</td>
<td>156</td>
<td>2-10&quot;E @ 20&quot;</td>
<td>16000</td>
<td>13</td>
<td>520</td>
</tr>
<tr>
<td>L₄ - L₁₁</td>
<td>217</td>
<td>17.60</td>
<td>4.56</td>
<td>156</td>
<td>2-10&quot;E @ 20&quot;</td>
<td>16000</td>
<td>13</td>
<td>520</td>
</tr>
<tr>
<td>L₁₁ - L₁₂</td>
<td>217</td>
<td>14.61</td>
<td>4.56</td>
<td>156</td>
<td>2-10&quot;E @ 30&quot;</td>
<td>16000</td>
<td>13</td>
<td>780</td>
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<tr>
<td>U₉ - L₁₀</td>
<td>152</td>
<td>12.35</td>
<td>11.11</td>
<td>194</td>
<td>1-10&quot;C.B @ 42&quot;</td>
<td>16000</td>
<td>13</td>
<td>680</td>
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<tr>
<td>U₀ - L₁₆</td>
<td>126</td>
<td>9.11</td>
<td>7.87</td>
<td>186</td>
<td>1-10&quot;C.B @ 31&quot;</td>
<td>16000</td>
<td>13</td>
<td>480</td>
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<tr>
<td>U₁ - L₁₂</td>
<td>63</td>
<td>9.11</td>
<td>7.87</td>
<td>183</td>
<td>1-10&quot;C.B @ 31&quot;</td>
<td>16000</td>
<td>13</td>
<td>474</td>
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<td>U₉ - L₉</td>
<td></td>
<td></td>
<td></td>
<td>140</td>
<td>1-10&quot;C.B @ 31&quot;</td>
<td>12550</td>
<td>11.64</td>
<td>298</td>
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<tr>
<td>U₀ - L₁₀</td>
<td>102</td>
<td>9.11</td>
<td></td>
<td>115</td>
<td>608 1-10&quot;C.B @ 31&quot;</td>
<td>11150</td>
<td>9.62</td>
<td>298</td>
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<tr>
<td>U₁ - L₆</td>
<td>65</td>
<td>10.1</td>
<td></td>
<td>101</td>
<td>1-10&quot;C.B @ 31&quot;</td>
<td>8400</td>
<td>8.40</td>
<td>261</td>
</tr>
</tbody>
</table>

### Lower lateral system

- 12.3"x3"x116@6"

<table>
<thead>
<tr>
<th>Truss</th>
<th>26580</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lower Lateral</td>
<td>1464</td>
</tr>
<tr>
<td>Stringers</td>
<td>154.50</td>
</tr>
<tr>
<td>Cross Beams</td>
<td>8510</td>
</tr>
<tr>
<td>Railings</td>
<td>5460</td>
</tr>
<tr>
<td>Sub Total</td>
<td>57464</td>
</tr>
</tbody>
</table>

25% For Details: 14,366

Total Wt. Suspended Span: 71,830
The following table gives approximate panel point loads for weight of steel in cantilever arm, determined from the design of a steel arch previously made.

<table>
<thead>
<tr>
<th>Panel</th>
<th>Panel Point Loads for Cantilever Arm.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Panel</td>
</tr>
<tr>
<td></td>
<td>Truss D. L.</td>
</tr>
<tr>
<td></td>
<td>Floor Beams</td>
</tr>
<tr>
<td></td>
<td>Stringers</td>
</tr>
<tr>
<td></td>
<td>Floor Slab</td>
</tr>
<tr>
<td></td>
<td>Total</td>
</tr>
<tr>
<td></td>
<td>Total upper panel</td>
</tr>
<tr>
<td></td>
<td>Total lower panel</td>
</tr>
<tr>
<td></td>
<td>* 67707 is additional load from suspended span concentrated at U-9.</td>
</tr>
</tbody>
</table>

Notes:
- 25% of dead load considered concentrated at lower panel pt.
- 75% " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " " 

For determination of dead load stresses see "Dead Load Stress Diagram-Cantilever Arm." Print No. 3.

As a check on the Dead Load Diagram the horizontal component at U-5 necessary for equilibrium was calculated algebraically and then determined from the Dead Load Diagram. They were found to vary by only 0.185%. 
Live Loads.

For live load H-15 equivalent live loading was used. The following loading gave maximum stresses in the chord members:

Panel point U-9.

<table>
<thead>
<tr>
<th>Loading</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniform live load from suspended span</td>
<td>18.72 kips</td>
</tr>
<tr>
<td>Concentrated load</td>
<td>13.50 &quot;</td>
</tr>
<tr>
<td>Uniform load from half panel</td>
<td>4.32 &quot;</td>
</tr>
<tr>
<td>Total</td>
<td>30.54 &quot;</td>
</tr>
</tbody>
</table>

For all other panel points 8.64 "

For impact see S. H. D. specifications in appendix.

Using the above panel point loads a stress diagram was drawn see, "Live Load Stress Diagram-Cantilever Arm, Webb System-Chord System." Prints No. 4 and 5.

Stresses in the web members were proportional to the shear in the panels. To obtain maximum shear in each panel the concentrated load of 13.5 kips was placed at the panel point where it would produce maximum stress in the adjoining panel.

Wind Loads.

380 lb. per ft. on loaded chord (upper)

180 " " " " unloaded " (lower)

Load per lin. ft. on upper or loaded chord is composed of 200 lb. per lin. ft. on moving load / 30 lb. per sq. ft. on 1.5 X exposed area of upper chord. (Exposed area of upper chord and exposed area of lower chord assumed same as
1. Stress Diagram - Cantilever Arm
   Upper and Lower Chord

   Notes:
   + = Compression
   - = Tension
   All forces & stresses in Kips

   Loads of 14, 14, 13, 12 = 141,480 = 864 Kips

   Load at C2 = 1812 = Load from 3rd span
   1350 = Tensile load
   542 = Half panel uniform live load
   3664 = Total load in Kips.
Notes:
+ = Compression
- = Tension
F = Dead Load concentrated successively of upper panel points.
U = Uniform Live Load
All stresses in Ksi.

G.A. SCHOOL OF TECHNOLOGY
LL. STRESS DIAGRAM
STEEL ARCHED CANTILEVER
HIGHWAY BRIDGE
JUNE 1930
S.E. Brownell
Truss Scale 1"=6'-0"-Diagram Scale 1"=10000'
exposed area of upper and lower chord of steel arch previously designed, which was 4 sq. ft.)

Load per ft. on upper chord = \( 200 \times (30 \times 1.5 \times 4) = 380 \) lb.

Load per ft. on lower chord = \( 30 \times 1.5 \times 4 = 180 \) "

Panel load for upper chord = \( 18 \times 380 = 6840 \) "

Windward load = \( 6840 / 2 = 3.420 \) kips

Leeeward " " " = 3.420 "

It was found that the determination of wind stresses and the design of bracing members amounted to about half of all the design work. This was caused by assuming the wind forces finally reaching the piers through two paths and by not being able to find a text or any reference which covered all phases of wind stresses in an arched cantilever.

The following is a sample calculation of the wind stresses in a cross frame due to the overturning effect of the wind about the lower panel point.

\[
\begin{align*}
W &= 3.42 \\
V_8' &= W' = 3.42 \\
15.85 &= 23.4 \\
18.63 &= L_8
\end{align*}
\]

Stress in L-8 L'8 = \( 6840 \times 23.4 / 17.375 = 9.25 \) "

Stress in U-8 L'8 = \( 6840 \times 23.4 / 17.375 = 24.42 \) kips
Stress in U-8 L-8 = \frac{(6840 \times 15.779)}{18.63} = 7.720 \text{ kips}

(p'8 is the vertical load applied at lower panel point due to overturning effect of wind. These values used in, "Wind Stress Diagram-Cantilever Arm." Print No. 6).

Wind Stresses for Upper Chord Bracing.

\[
\begin{align*}
\text{Panel load} & = 380 \times 18 = 6840 \text{ kips} \\
W' & = \frac{W}{2} = \frac{6840}{2} = 3420 \\
\text{Load from suspended span} & = 380 \times 39 = 14820 \text{ kips} \\
F & = \frac{F'}{2} = \frac{14820}{2} = 7410 \text{ kips} \\
\text{Length of U-5 U-6} & = 24.1 \\
\text{Sec. } \theta & = \frac{24.1}{16} = 1.5 \\
\text{Members assumed in tension.} \\
\text{Stress in U'-8 U-9} & = 18.24 \times 1.5 = 27.4 \text{ kips}
\end{align*}
\]
Notes:
+ = Compression
- = Tension
All forces and stresses in Kips
forces are total vertical wind loads
on lower chord due to cross frames
+ load from lower laterals.

GA SCHOOL OF TECHNOLOGY
WIND STRESS DIAGRAM
STEEL ARCHED CANTILEVER
HIGHWAY BRIDGE
JUNE 1930
S.E.Broadwell
Truss Scale: 1" = 6' - Diagram Scale: 1/4" = 100 Kips
12/28/41/48
Stress in $U' - 7$ $U - 8 = 25.08 \times 1.5 = -37.6$ kips

" " $U' - 6$ $U - 7 = 31.92 \times 1.5 = -47.8"$

" " $U' - 5$ $U - 6 = 33.76 \times 1.5 = -59.1"$

Summation $V = 0$

Stress in $U' - 9$ $U - 9 = \neq 9.12$ kips

" " $U' - 8$ $U - 8 = 21.66"$

" " $U' - 7$ $U - 7 = 28.50"$

" " $U' - 6$ $U - 6 = 36.34"$

" " $U' - 5$ $U - 5 = 42.18"$

Stress In Upper Chord Due To Wind.

Stress in $U - 8$ $U - 9 = .00$ kips

" " $U - 7$ $U - 8 = 18 \times 18.24 / 16 = -20.50"$

" " $U - 6$ $U - 7 = 18.24 \times (36 \neq 68.40)18 / 16 = -48.80"$

" " $U - 5$ $U - 6 = 18.24 \times (54 \neq 13.68) \times 27 / 16 = -84.70"$

Wind Stresses In Lower Laterals.

L-11 L-12 -------------------------------18.00 ft.

L-10 L-11 -------------------------------18.05 ft.

L-9 L-10 -------------------------------18.10 ft.

L-8 L-9 -------------------------------18.48 ft.

L-7 L-8 -------------------------------18.90 ft.

L-6 L-7 -------------------------------19.43 ft.

L-5 L-6 -------------------------------20.10 ft.

Direct wind stress in lower chord members.

L-8 L-9 --- 19.52 \times 18 / 18.63 ---------------- = -19.90 kips

L-7 L-8 --- 19.52 (36 \neq 9.8)18 / 19.58 = -45.00"
L-6 L-7 --- 19.52 (54 / 19.6) 27 / 20.79 = - 76.40 kips
L-5 L-6 --- 19.52 (72 / 29.4) 36 / 22.26 = -109.00 "

Vertical Panel Point Wind Reactions On Lower Chord Caused By
Lower Lateral System.

\[ p''6 = \text{shear at L-6 X h}_6 / c_6 \text{ (Typical formula).} \]

\[ p''9 = 19.520 (4.208 / 17.94) \] 4.575 kips
\[ p''8 = 29.320 (5.735 / 18.62) \] 9.100 "
\[ p''7 = 39.120 (7.363 / 19.56) \] 14.730 "
\[ p''6 = 48.920 (8.941 / 20.79) \] 21.300 "

Total Vertical Wind Loads on Lower Chord Due to Cross Frames
Plus That From Lower Laterals.

\[ P_x = p'_x \neq p''x \]

\[ p_6 \] 11.230 \neq 21.800 30.030 kips
\[ p_7 \] 9.360 \neq 14.730 24.090 "
\[ p_8 \] 7.720 \neq 9.100 16.820 "
\[ p_9 \] 12.960 \neq 4.575 17.535 "

Length Of Diagonals.

\[ L-8 L'-9 = \sqrt{\frac{c_8-c_9}{2}} \frac{2}{(L-8 L-9)^2} \]

\[ L-8 L'-9 = \sqrt{18.29} \neq (18.48) \] 26.00 ft.
\[ L-7 L'-8 = \sqrt{19.11} \neq (18.90) \] 26.90 "
\[ L-6 L'-7 = \sqrt{20.19} \neq (19.43) \] 28.00 "
\[ L-5 L'-6 = \sqrt{21.53} \neq 9(20.10) \] 29.40 "
<table>
<thead>
<tr>
<th>Member</th>
<th>Length</th>
<th>c</th>
<th>Sec. Θ = L/c</th>
<th>Shear in panel</th>
<th>Stress, kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>L-8 L'-9</td>
<td>26.0</td>
<td>17.94</td>
<td>1.45</td>
<td>19.52</td>
<td>28.35</td>
</tr>
<tr>
<td>L-7 L'-8</td>
<td>26.9</td>
<td>18.63</td>
<td>1.44</td>
<td>29.32</td>
<td>42.25</td>
</tr>
<tr>
<td>L-6 L'-7</td>
<td>28.0</td>
<td>19.58</td>
<td>1.43</td>
<td>39.12</td>
<td>56.00</td>
</tr>
<tr>
<td>L-5 L'-6</td>
<td>28.4</td>
<td>20.79</td>
<td>1.42</td>
<td>48.92</td>
<td>69.40</td>
</tr>
</tbody>
</table>

Lower panel point wind load = 180 lb. X 18 = 2.56 kips or 1.28 kips on each side.

Load transferred from upper panel point = 3.42 kips on each side. Therefore, 1.28 + 3.42 = 4.7 kips on each side of bridge at lower panel point.

Total lateral load from suspended truss = 14.82 kips or 7.41 kips each side.

Wind loads on lower chord together with those transferred from upper chord.

\[
\begin{align*}
\text{Stress in L-9 L'-9} & \quad \text{---------} \quad \neq 9.76 \text{ kips} \\
\text{"} \quad \text{L-8 L'-8} & \quad \text{---------} \quad \neq 24.42 \text{ "} \\
\text{"} \quad \text{L-7 L'-7} & \quad \text{---------} \quad \neq 34.22 \text{ "} \\
\text{"} \quad \text{L-6 L'-6} & \quad \text{---------} \quad \neq 44.02 \text{ "} \\
\text{"} \quad \text{L-5 L'-5} & \quad \text{---------} \quad \neq 53.82 \text{ "}
\end{align*}
\]
For stresses and design of cantilever arm see, "Stress and Design Table-Cantilever Arm." Print No. 7.

ANCHOR ARM.

For determination of stresses in the truss members of the anchor arm see, "Influence Lines - Anchor Arm." Print No. 8.

Wind bracing members were designed as for a simple truss.

For the combination of stresses to determine design stress see specifications in appendix.

For stresses and design in anchor arm see, "Stress and Design Table - Anchor Arm." Print No. 9.

FOUNDATION.

Foundation conditions were ideal. Solid rock is encountered throughout the gorge a few inches below an overburden of decomposed and weathered rock.

PIERS, ABUTMENTS AND ANCHORAGE.

See, "Details of Piers and Anchorages." Print No. 10.

From the influence line for the anchorage, the total reaction at the anchorage for one truss was found to be 76,800 lb. for critical loading conditions. For both trusses the reaction would be 153,600 lb. Using a factor of safety of 2, the weight of the counter balancing anchorage would have to be $2 \times 153,600 = 307,200$ lb.

The volume of concrete required in one anchorage would be $307,200 / 150 \times 27 = \text{cu. yds}$. For quantities for piers and abutments see, "Details of Piers and Anchorages."
## STRESS TABLE

<table>
<thead>
<tr>
<th>Section Consists of</th>
<th>Prop of Section</th>
<th>Length</th>
<th>Allowable Stress</th>
<th>Final Stress</th>
<th>Total Wt</th>
<th>Total Wt. for Total</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<td></td>
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<td></td>
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</tbody>
</table>

## DESIGN TABLE

<table>
<thead>
<tr>
<th>Final Stress</th>
<th>Wt. For Mem.</th>
<th>Total Wt.</th>
<th>Total Wt. for Total</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Note for allowable stress in compression following formula used:

\[ \sigma = \frac{15,000 \times 133.33}{918} \]

Total, 3,550 lb. per arm = 7,100 lb. total.
Quantities for Piers:
275 Cu Yds, C1'B' Conc
7750 Lbs Reinf. Steel
250 Cu Yds, S.P. Exc.

Quantities for Abutments:
152 Cu Yds C1'B' Conc
5400 Lbs Reinf. Steel
400 Lbs Struct. Steel
145 Cu Yds, S.P. Exc.

Left Pier: h = 28.5, b = 4.85, a = 4.2
Right Pier: h = 30.5, b = 6.54, a = 5.35

Georgia School of Technology
Details of Abutments and Piers
Steel Arched Cantilever Highway Bridge
June 1930, Scale 1" = 10' 5C Broswell
Unit load at \( U_0 \), R-L (Reaction at left)
- \( U_2 \) = \( \frac{1}{2} \times \frac{72}{30} \) = 0.4
- \( U_9 \) = \( \frac{72}{30} \) = 2.4
- \( U_5 \) = 0.0
- \( U_0 \) = 1.0

Net Area = \( \frac{1}{2} \times 90 \times 1 \) = 45
- \( \frac{1}{2} \times 150 \times 0.8 \) = 60

Net Area = 15

U.D.L. R-L one truss = \( 15 \times 2000 \) = 30000#

U.L.L. \( = 60 \times 480 \) = 28800#

C.L.L. \( = 0.8 \times 13500 \) = 10800#

I \( = 0.182 \times 39600 \) = 7200#

Total \( \approx 76800# \)

**Influence Line for Anchorage**

Scales: Horiz. 1" = 50', Vert. 1" = 3#
In order to make a comparison between a steel arched cantilever bridge and other types of bridges that might be selected for investigation, a length of project was taken which would include all possible lengths of bridges that could be economically considered.

Station 24 + 87 was taken as the beginning of the project, and station 29 + 07 was taken as the end of the project. The length of the project was 420 ft. 0 in. The difference between the length of project and the length of bridge was 18 ft. 0 in. which was the length of the approach embankment falling within the project. This approach embankment amounted to 580 cu. yds. and was included in the summary of quantities.

SUMMARY OF QUANTITIES

Class "A" Concrete

Floor slab and curb --------------------- 206 cu. yds.

Reinforcing Steel

Floor slab and curb --------------------- 20530 lbs.

Piers ------------------------------- 7750 "

Abutments ---------------------------- 5400 "

Total 33680 "

Print No. 10.

APPROACH FILL
### Class "D" Concrete

- Piers: 275 cu. yds.
- Abutments: 152 cu. yds.
- Total: 427 cu. yds.

### Solid Rock Excavation

- Piers: 250 cu. yds.
- Abutments: 400 cu. yds.
- Total: 650 cu. yds.

### Common and Borrow Excavation

- Total: 580 cu. yds.

### Structural Steel

- Suspended span, cantilever arm and anchor arm: 517772 lb.
- Eye-bars and shapes in anchorates: 400 lb.
- Shoes and pins: 14000 lb.
- Total: 522172 lb.

### 2 in. Bituminous Mat

- Total: 893 cu. yds.

### Estimate of Cost

<table>
<thead>
<tr>
<th>Item</th>
<th>Quantity</th>
<th>Unit Price</th>
<th>Total Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>206 cu. yds. Cl. &quot;D&quot; Conc.</td>
<td>22.00</td>
<td>$</td>
<td>4532.00</td>
</tr>
<tr>
<td>427 &quot; &quot; &quot; &quot;B&quot; &quot;</td>
<td>20.00</td>
<td>$</td>
<td>8640.00</td>
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<tr>
<td>33680 lb. rein. steel</td>
<td>0.06</td>
<td>$</td>
<td>2020.80</td>
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<tr>
<td>395 cu. yds. solid rock exc.</td>
<td>5.00</td>
<td>$</td>
<td>1975.00</td>
</tr>
<tr>
<td>580 &quot; &quot; com. and bor. &quot;</td>
<td>0.35</td>
<td>$</td>
<td>203.00</td>
</tr>
<tr>
<td>522172 lbs. struct. steel</td>
<td>0.0775</td>
<td>$</td>
<td>40468.33</td>
</tr>
<tr>
<td>893 sq. yds. bit. mat.</td>
<td>1.50</td>
<td>$</td>
<td>1339.50</td>
</tr>
</tbody>
</table>

**Sub-total:** $59078.63

**10% Engineering and Contingencies:** $5907.86

**Total:** $64986.49
The advantages of a cantilever bridge are: economy of material, savings in widths of piers, which may carry one support instead of two as in the case of a series of simple spans, ease of erection and saving of falsework, statically determinate, in this case, which simplifies design, and the potential aesthetical qualities of the type.

In this design all the advantages as stated above were factors in the selection of the type for comparison but the two main factors were; ease of erection and saving in false work, and the potential aesthetical qualities of the type.

The fact that the bridge was to span a deep gorge precluded the investigation of any type of steel bridge that necessitated the use of false work in erection. The ability to use the dead weight of the abutments as an anchorage was another economical erection factor.

The profile was not ideal in that it did not allow the economical ratio of anchor arm to cantilever arm, however, it is believed that the profile approached the ideal as near as any gorge crossing could be expected to. The profile was adaptable to the aesthetical treatment of the bridge which was highly desired, due to the setting of the bridge in the region of beautiful natural scenery where Nature had carved out deep gorges with symmetrical lines and raised mountains which tower over the gorges with majestic grandeur. The bridge was to be in keeping with its setting and every attempt was made to harmonize it with the brand scale of surrounding
Nature by giving it graceful curves and symmetry of outline, and by proportioning the members so as to give it massive appearance.

Economy and aesthetics were considered throughout the design, and it is believed that at no time was one materially sacrificed for the other. Detailing and maintenance were also given consideration. Abutting members were designed to simplify detailing, and all members and parts of members were kept above the minimum required thickness and made accessible for painting and possible future replacement.
PART II

REINFORCED CONCRETE ARCH

HIGHWAY BRIDGE
GEORGIA SCHOOL OF TECHNOLOGY
REINFORCED CONCRETE ARCH
HIGHWAY BRIDGE
TALLULAH GORGE
TALLULAH FALLS GA
June 1936
SE Braswell
REINFORCED CONCRETE BRIDGE

A reinforced concrete arch bridge was one of the types of highway bridges selected for investigation for the spanning of Tallulah Gorge at Tallulah Falls, Ga.

The concrete arch was selected as one type for consideration because it could be developed aesthetically and could be considered as of permanent construction.

The concrete arch was designed by the Elastic Theory as modified by Urquhart and O'Rourke.

Print No. 1. is an elevation of the concrete arch at the bridge site. Print No. 17. as a half section near crown.

Order of Procedure.

1. The shape of the arch ring was determined by Cochrane's formula. Print No. 2.

2. The dead loads from superstructure and columns were determined from designs previously made. The dead load weight of the arch ring was determined from shape of arch ring as determined by Cochrane's formula. For dead loads see Print No. 3.

3. Arch ring thickness. Print No. 4.

4. The shape of the arch ring was drawn and the line of pressure determined by force polygons. Print No. 5.

5. The position of a new axis was located so that summation \( y \ ds/I = 0 \). Print No. 6.

6. Determination of Ho, Vo, Mo. Print No. 7.

7. Determination of moments and thrusts at crown.
Determination of moments and thrusts at springing

Print No. 9.

9. Influence diagrams for thrusts. Print No. 11.
10. Thrusts at crown and springing under critical condition. Print No. 12.
12. Determination of stresses at crown and springing.

Print No. 14.

13. Investigation for shear. Print No. 15.
14. Unit shear. Print No. 16.
### SHAPE OF ARCH RING BY COCHRANE'S FORMULA

\[
y = \frac{8r^2}{6+5r} \left( 3c^2 + 10c^4r \right)
\]

<table>
<thead>
<tr>
<th>X</th>
<th>(X \div \frac{8}{6}) = c</th>
<th>X^2</th>
<th>3c^2</th>
<th>10c^4r</th>
<th>3c^2 + 10c^4r</th>
<th>\frac{10c^4r}{3c^2 + 10c^4r}</th>
</tr>
</thead>
<tbody>
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<td>4.25</td>
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<td>0.88849</td>
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</table>
### Dead Loads On Spondrel Columns

<table>
<thead>
<tr>
<th>Component</th>
<th>Load (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Paving &amp; slab</td>
<td>3000#</td>
</tr>
<tr>
<td>Railing</td>
<td>400#</td>
</tr>
<tr>
<td>Curb</td>
<td>562#</td>
</tr>
<tr>
<td>4&quot; Fillets</td>
<td>38#</td>
</tr>
<tr>
<td>6&quot; Fillets</td>
<td>38#</td>
</tr>
<tr>
<td>Cross beams</td>
<td>795#</td>
</tr>
<tr>
<td>Cantilever brackets</td>
<td>124#</td>
</tr>
<tr>
<td>Inter girder</td>
<td>223#</td>
</tr>
<tr>
<td>&quot; &quot; brackets</td>
<td>24#</td>
</tr>
<tr>
<td>Outside &quot;</td>
<td>406#</td>
</tr>
<tr>
<td>Total, constant for all columns</td>
<td>5605#</td>
</tr>
</tbody>
</table>

### Spandrel Wall

- On point No 1: 6595#
- Per ft. of bridge = 6595 ÷ 2 = 3298#

Columns are 18" x 21" at top. Batter 1/4" in 12".
4150# = total wt. of one col. from curved girder.

### Dead Loads

<table>
<thead>
<tr>
<th>Location</th>
<th>Sup. Struct. Col. Loads</th>
<th>Arch Ring</th>
<th>Tie Walls</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>56066</td>
<td>34100</td>
<td>0</td>
<td>90200</td>
</tr>
<tr>
<td>B</td>
<td>56000</td>
<td>35200</td>
<td>0</td>
<td>92000</td>
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<td>C</td>
<td>51800</td>
<td>35300</td>
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<tr>
<td>F</td>
<td>42100</td>
<td>48700</td>
<td>6700</td>
<td>140000</td>
</tr>
</tbody>
</table>

**Live load = 480# per ft. of arch.**
Notation:

- $S =$ length of half arch axis
- $X =$ any point on arch axis
- $S_x =$ dist. along axis to point $X$
- $t_x =$ thickness of arch ring at pt. $X$
- $t_0 =$ " crown = 2.667 ft.
- $u =$ ratio of thickness at any point to thickness at crown.

Interpolating from Table № 2 Cochrane’s Method $S$ for rise ratio of $0.3228 = 0.62318 \cdot l$

$l =$ horiz. distance from springing to springing

$S = 0.62318 \cdot 210 = 130.87$

<table>
<thead>
<tr>
<th>$u = \frac{S_x}{S}$</th>
<th>$S_x = u \cdot S$</th>
<th>$u = \frac{t_x}{t_0}$</th>
<th>$t_x = u \cdot t_0$</th>
<th>$\frac{t_x}{2}$</th>
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<td>0.000</td>
<td>1.000</td>
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<td>1.009</td>
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</table>
Note: For method of determination of line of pressure see "Concrete Engineers Handbook", Keel and Johnson, page 658. Arch axis is from Cochrane's formula, see Print 110.
| Location of Axis Such That \( y \frac{As}{I} = 0 \) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| \( a \) | \( \frac{a}{2} \) | \( I_c \) | \( (\frac{a}{2} - d)^2 \) | \( 1AI_4 \) | \( I \) | \( As \) | \( yc \) | \( \frac{yc}{As/I} \) | \( As/I \) | \( y \) | \( x \) |
| 2.667 | 13 | 1.333 | 7.93 | 1.166 | 2.27 | 10.20 | 4.25 | 0.00 | 0.00 | 0.417 | 14.98 | 4.25 |
| 2.700 | 12 | 1.350 | 8.19 | 1.210 | 2.36 | 10.55 | 8.60 | 0.72 | 0.581 | 14.26 | 12.75 |
| 2.700 | 10 | 1.350 | 8.19 | 1.210 | 2.36 | 10.55 | 8.70 | 4.35 | 3.585 | 10.63 | 29.75 |
| 2.717 | 9 | 1.358 | 8.36 | 1.230 | 2.39 | 10.75 | 9.00 | 7.35 | 6.150 | 7.63 | 38.25 |
| 2.730 | 8 | 1.365 | 8.46 | 1.250 | 2.43 | 10.89 | 9.30 | 11.22 | 9.600 | 8.37 | 40.75 |
| 2.810 | 7 | 1.405 | 8.80 | 1.320 | 2.57 | 11.37 | 9.70 | 15.92 | 13.600 | 8.55 | 46.75 |
| 3.000 | 6 | 1.500 | 11.21 | 1.560 | 3.03 | 14.24 | 10.40 | 21.77 | 15.900 | 7.30 | 55.25 |
| 3.200 | 5 | 1.600 | 13.61 | 1.820 | 9.54 | 17.15 | 10.80 | 28.56 | 18.000 | 6.30 | 63.75 |
| 3.700 | 4 | 1.850 | 21.07 | 2.560 | 4.98 | 26.05 | 11.80 | 36.80 | 16.720 | 4.54 | 72.25 |
| 4.230 | 3 | 2.145 | 32.84 | 3.570 | 6.35 | 39.19 | 12.70 | 46.24 | 14.750 | 3.19 | 89.25 |
| 5.420 | 2 | 2.710 | 66.28 | 6.050 | 11.75 | 78.03 | 14.00 | 57.87 | 10.270 | 1.79 | 97.75 |
| 6.67 | 1 | 3.935 | 123.44 | 9.540 | 18.50 | 141.94 | 19.00 | 68.00 | 6.250 | 0.92 | 105.00 |
| 117.250 | | | | | | | | | | | 7.823 |

\[
Y = \frac{\varepsilon \frac{yc}{I} \frac{As}{I}}{\varepsilon} = \frac{117.214}{7.823} = 14.98 = \text{dist. of new } X \text{ axis below crown.}
\]
### Table III

<table>
<thead>
<tr>
<th>( m_1 )</th>
<th>( m_2 )</th>
<th>( m_3 )</th>
<th>( m_4 )</th>
<th>( m_5 )</th>
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</tr>
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<td>7</td>
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<td>97</td>
<td>98</td>
<td>99</td>
<td>100</td>
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</tbody>
</table>

**Notes:**
- The table contains values for various parameters. Each row represents a different set of values, with columns likely representing different variables or conditions.
- The exact interpretation of each column and row requires additional context that is not provided in the image.
### Table IV: Crown

**Determination of Thrusts and Moments at Crown**

*Note: M, V, and Hs from Table III*

<table>
<thead>
<tr>
<th>Section</th>
<th>Hs*</th>
<th>Hx</th>
<th>Vx</th>
<th>Mx</th>
<th>M</th>
<th>N</th>
<th>Dead Load</th>
<th>Live Load for Max Compression</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>upper fibre</td>
</tr>
<tr>
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<td>- .315</td>
<td>+ .004</td>
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<td>0</td>
<td>0</td>
<td>.116</td>
<td>- .025</td>
</tr>
<tr>
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<td>- .947</td>
<td>+ .027</td>
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<td>0</td>
<td>0</td>
<td>.794</td>
<td>-1.153</td>
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<td>0</td>
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<tr>
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<td>- .739</td>
<td>+ .173</td>
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<td>-1.189</td>
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<tr>
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<td>-10.530</td>
<td>+ .293</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>11.555</td>
<td>+1.025</td>
</tr>
<tr>
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<td>-.786</td>
<td>-11.774</td>
<td>+ .426</td>
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<td>0</td>
<td>0</td>
<td>18.661</td>
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<td>-11.774</td>
<td>+ .426</td>
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<td>0</td>
<td>18.661</td>
<td>+6.859</td>
</tr>
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<td>-10.530</td>
<td>- .293</td>
<td>0</td>
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<td>0</td>
<td>11.555</td>
<td>+1.025</td>
</tr>
<tr>
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<td>-.532</td>
<td>- .796</td>
<td>- .173</td>
<td>0</td>
<td>0</td>
<td>0</td>
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</tr>
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<th>N (kN)</th>
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<th>N Due to dead load</th>
<th>LL Thrust for max. comp. at springing (Upper Fibre - Lower Fibre)</th>
<th>N (kN)</th>
<th>N Due to dead load</th>
<th>LL Thrust for max. comp. at springing (Upper Fibre - Lower Fibre)</th>
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<td>-3500</td>
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<td>-17338</td>
<td>-0.9435 -1.9372</td>
<td>-0.30</td>
<td>-1941</td>
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<tr>
<td>D</td>
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<td>109700</td>
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<td>90700</td>
<td>-48252</td>
<td>-0.9830 -89158</td>
<td>-0.53</td>
<td>-8021</td>
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<tr>
<td>B</td>
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<td>92000</td>
<td>-64676</td>
<td>-0.9734 -89552</td>
<td>-0.05</td>
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<td>-70899</td>
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<td>-0.0167 -2.2938</td>
<td>-0.025</td>
<td>-77825</td>
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</tbody>
</table>

**Note:** It is evident that with any given unsymmetrical loading the thrusts of the two springings will be different, and unit loads of say F and E will produce the same thrust of R'2 that F and E will produce of R-1. Then for instance, the thrust of the springing on the left side is to be 4.38 for live loads on D through C. Now the thrust for this condition at the right springing may be found by leading points C through D and determining the thrust at the left again.
### Stresses Due To Thrust

*Note: Stresses are in lbs. per sq. ft.*

<table>
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<tr>
<th></th>
<th>Crown</th>
<th>Springing</th>
<th>Average</th>
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<tr>
<td>D.L.</td>
<td>-31097</td>
<td>-23055</td>
<td>-27077</td>
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<tr>
<td>Springing</td>
<td>-3085</td>
<td>-1178</td>
<td>-2388</td>
</tr>
<tr>
<td>Lower fibre</td>
<td>Not needed on account no neg. moment.</td>
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<td></td>
</tr>
<tr>
<td>Temp</td>
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</tr>
<tr>
<td>Fall 40°</td>
<td>+259</td>
<td>+61</td>
<td>+160</td>
</tr>
<tr>
<td>Rise 40°</td>
<td>-259</td>
<td>-61</td>
<td>-160</td>
</tr>
</tbody>
</table>

\[ H_0 = \frac{wt^2E}{2E_2y^2} \frac{288000000}{2 (18.14.9)} = \pm 4000 \]

*See Urquhart & O'RourK P. 367*
### Sprunging

\[ \cos \theta = 0.5368 \times 4000 = 2147.11 \text{ N} \cdot A = 2147.35.28 \pm 61 \]

#### Max Comp. in Upper Fibre, Sprunging

<table>
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<th>M</th>
<th>Cg</th>
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<td>+1083413</td>
<td>-27077</td>
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<tr>
<td>L.L.</td>
<td>+41984</td>
<td>+666412</td>
<td>-2388</td>
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<tr>
<td>A.S.</td>
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<td>-80900</td>
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<tr>
<td>Total</td>
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<td>+1493225</td>
<td>-29390</td>
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</table>

#### Max Comp. in Upper Fibre, Crown

<table>
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<tr>
<td>A.S.</td>
<td>+1880</td>
<td>+30900</td>
<td>+75</td>
</tr>
<tr>
<td>Total</td>
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<td>+509198</td>
<td>-29390</td>
</tr>
</tbody>
</table>

### UNIT STRESSES - SPRUNGING

\[ M = 1665205 \text{ N} \cdot \text{lbs} \]
\[ \rho = 0.00417 \]

\[ \bar{d} = 0.01445 \]

\[ \bar{h} = \frac{1665205 \times 12}{60 \times (856211) \times 0.1445} = 455 \text{ ft/ln} \]

#### Impact Considered

\[ M = 1665205 + 173000 = 1838205 \text{ N} \cdot \text{lbs} \]

\[ M = 5755665 \text{ N} \cdot \text{lbs} \]

\[ \rho = 0.0104 \]

\[ \bar{d} = 0.0988 \text{ call it 0.1} \]

### UNIT STRESSES - CROWN

\[ M = 5755665 \text{ N} \cdot \text{lbs} \]
\[ \rho = 0.00417 \]

\[ \bar{d} = 0.01445 \]

\[ \bar{h} = \frac{5755665 \times 12}{60 \times (858972) \times 0.1445} = 455 \text{ ft/ln} \]

#### Impact Considered

\[ M = 5755665 + 45100 = 620666 \text{ N} \cdot \text{lbs} \]

\[ M = 508972 + 7000 = 515972 \]
INVESTIGATION FOR SHEAR

Section taken slightly to left of point considered

\[ \cos \theta = 0.644 \]
\[ \sin \theta = 0.165 \]

Unit load at:

<table>
<thead>
<tr>
<th>Section</th>
<th>( V_f )</th>
</tr>
</thead>
<tbody>
<tr>
<td>F</td>
<td>( 0.025 \times 0.765 + 0.004 \times 6.44 = 0.644 )</td>
</tr>
<tr>
<td>E</td>
<td>( 0.130 \times 0.765 + 0.027 \times 6.44 = 0.644 )</td>
</tr>
<tr>
<td>D</td>
<td>( 0.318 \times 0.765 + 0.082 \times 6.44 = 0.644 )</td>
</tr>
<tr>
<td>C</td>
<td>( 0.532 \times 0.765 + 0.173 \times 6.44 = 0.644 )</td>
</tr>
<tr>
<td>B</td>
<td>( 0.703 \times 0.765 + 0.293 \times 6.44 = 0.644 )</td>
</tr>
<tr>
<td>A</td>
<td>( 0.786 \times 0.765 + 0.426 \times 6.44 = 0.644 )</td>
</tr>
<tr>
<td>A'</td>
<td>( 0.786 \times 0.765 - 0.426 \times 6.44 = 0.644 )</td>
</tr>
<tr>
<td>B'</td>
<td>( 0.703 \times 0.765 - 0.293 \times 6.44 = 0.644 )</td>
</tr>
<tr>
<td>C'</td>
<td>( 0.532 \times 0.765 - 0.173 \times 6.44 = 0.644 )</td>
</tr>
<tr>
<td>D'</td>
<td>( 0.318 \times 0.765 - 0.082 \times 6.44 = 0.644 )</td>
</tr>
<tr>
<td>E'</td>
<td>( 0.130 \times 0.765 - 0.027 \times 6.44 = 0.644 )</td>
</tr>
<tr>
<td>F'</td>
<td>( 0.025 \times 0.765 - 0.004 \times 6.44 = 0.644 )</td>
</tr>
</tbody>
</table>

\[ \text{Values are: } -0.622, -0.527, -0.348, -1.126, +0.077, +0.232, +3.28, +3.350, +2.96, +1.190, +0.083, +0.016 \]
**Unit Shear**

8160 #LL at each column; plus placing 19500# concentrated LL at critical point according to specifications

<table>
<thead>
<tr>
<th>Section</th>
<th>Vf</th>
<th>Dead Load</th>
<th>Vf Dead Load +</th>
<th>Vf Live Load +</th>
<th>Vf Live Load -</th>
</tr>
</thead>
<tbody>
<tr>
<td>F</td>
<td>-.622</td>
<td>14,0000</td>
<td>-8,7000</td>
<td>17200</td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>-.527</td>
<td>136,450</td>
<td>-7,1600</td>
<td>4300</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>-.348</td>
<td>109,700</td>
<td>-3,8000</td>
<td>2840</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>-.126</td>
<td>90,700</td>
<td>-1,1400</td>
<td>1030</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>+.077</td>
<td>92,000</td>
<td>+7,080</td>
<td>629</td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>+.232</td>
<td>90,200</td>
<td>+20,900</td>
<td>1890</td>
<td></td>
</tr>
<tr>
<td>A'</td>
<td>+.328</td>
<td>90,200</td>
<td>+29,600</td>
<td>2680</td>
<td></td>
</tr>
<tr>
<td>B'</td>
<td>+.350</td>
<td>92,000</td>
<td>+32,200</td>
<td>2860</td>
<td></td>
</tr>
<tr>
<td>C'</td>
<td>+.296</td>
<td>90,700</td>
<td>+26,900</td>
<td>2420</td>
<td></td>
</tr>
<tr>
<td>D'</td>
<td>+.190</td>
<td>109,700</td>
<td>+20,700</td>
<td>1550</td>
<td></td>
</tr>
<tr>
<td>E'</td>
<td>+.083</td>
<td>136,450</td>
<td>+11,300</td>
<td>677</td>
<td></td>
</tr>
<tr>
<td>F'</td>
<td>+.016</td>
<td>140,000</td>
<td>+22,40</td>
<td>130</td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>-.57080</td>
<td></td>
<td>12836</td>
<td>25370</td>
<td></td>
</tr>
</tbody>
</table>

Temperature shear = H₀ sinθ = ±4000 x 0.765 = ±3060

Omit arch shortening for shear

Total shear = -57080 -25370 -3060 = -85510

Area = 5 x 4.8 = 24.0 sq. ft

**Unit shear** = \( \frac{85510}{24 x 144} = 24.8 \) # per sq. in
HALF SECTION NEAR CROWN
Scale 1/2" = 1'-0"
Summary of Quantities:

<table>
<thead>
<tr>
<th>Description</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforcing steel</td>
<td>192580 lbs.</td>
</tr>
<tr>
<td>Class &quot;A&quot; concrete</td>
<td>1359 cu. yds.</td>
</tr>
<tr>
<td>Class &quot;B&quot;</td>
<td>141.4 &quot; &quot;</td>
</tr>
<tr>
<td>Rock excavation</td>
<td>207.6 &quot; &quot;</td>
</tr>
<tr>
<td>Embankment</td>
<td>400 &quot; &quot;</td>
</tr>
<tr>
<td>Concrete pavement</td>
<td>923 sq. yds.</td>
</tr>
<tr>
<td>Handrail</td>
<td>846 lin. ft.</td>
</tr>
</tbody>
</table>
## Estimate of Cost

<table>
<thead>
<tr>
<th>Item Description</th>
<th>Rate</th>
<th>Quantity</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>192580 lb. reinf. steel</td>
<td>.06</td>
<td>$11554.80</td>
<td></td>
</tr>
<tr>
<td>1395 cu. yds. cl. A conc.</td>
<td>35.00</td>
<td>47565.00</td>
<td></td>
</tr>
<tr>
<td>141.4 &quot; &quot; &quot; B &quot;</td>
<td>20.00</td>
<td>2828.00</td>
<td></td>
</tr>
<tr>
<td>207.6 &quot; &quot; rock exc.</td>
<td>5.00</td>
<td>1038.00</td>
<td></td>
</tr>
<tr>
<td>400 &quot; &quot; embankment</td>
<td>.35</td>
<td>140.00</td>
<td></td>
</tr>
<tr>
<td>923 sq. yds. concrete pavement</td>
<td>2.00</td>
<td>1846.00</td>
<td></td>
</tr>
<tr>
<td>846 lin. ft. handrail</td>
<td>2.50</td>
<td>2115.00</td>
<td></td>
</tr>
<tr>
<td><strong>Sub-total</strong></td>
<td></td>
<td>67086.80</td>
<td></td>
</tr>
<tr>
<td>10 % Eng. &amp; Cont'g.</td>
<td></td>
<td>$6708.68</td>
<td></td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td>$73795.48</td>
<td></td>
</tr>
</tbody>
</table>

(27)
PART III

ECONOMICAL ANALYSIS AND
TYPE SELECTION
ECONOMICAL ANALYSIS AND TYPE SELECTION.

As stated by Mr. C. B. McCullough in his "Economics of Highway Bridge Types", the field of highway bridge engineering as a whole may be conveniently considered as composed of six distinct major phases of activity, namely:

Bridge location.
Preliminary investigational work and stream study.
Economic analysis and type selection.
Detailed design.
Construction.
Maintenance and operation.

It is believed that it can be safely said without fear of contradiction that economic analysis and type selection is the most important of the six phases as stated above, and that in the past it has received less consideration than any of the other phases. This is borne out by the fact that the writer knows of but one book published on the subject while there are thousands published on design and detail.

The possibilities of savings by an accurate analysis and the selection of the right type of structure by bridge engineers throughout the United States alone, are enormous and would run into many millions of dollars a year.

The past few years have witnessed some activity in the investigation of more than one type of bridge for a proposed crossing, particularly in the field of long span bridges. An example of the above is found in Devil's Island Bridge now
under construction for the Port Authorities of New York. From the nature of the crossing one would have thought that a suspension bridge would have been the only type that would prove economical. However, when a comparison between a three-hinged steel arch and a suspension bridge was made it was found that the arch could be built at a savings of half a million dollars.

There are many factors which enter into the making of an economic analysis of a bridge and the selection of the proper type. These factors would necessarily vary with the section of the country and the location of the bridge in that section of the country, because; materials, labor, construction seasons and aesthetics all have weights and values that vary in different proportions for different locations and sections.

It is possible to build a structure of concrete more economically in the heart of a steel producing region or, vice versa, a steel structure more economically near a cement mill, because of factors other than material entering into the equation.

The general factors controlling type selection are:

Stream behavior.

Requirements of navigation.

Traffic considerations.

Architectural features and scenic considerations.

Conditions of available funds.

Of the above factors, architectural features and scenic considerations will be found to be the hardest one to evaluate because, as would be suspected, it is almost entirely
dependant upon personal opinions. Steel has its advocates, as well as concrete, while one person places more value upon aesthetics than another. The other factors may be determined by the information at hand, by a close study of the requirements, and by the collection and tabulation of data over a long period of time.

The fundamentals of economic analysis are:

Source of funds for highway bridge improvements.
Capital costs.
Maintenance and renewal costs.
Insurance costs.
   A. Fire insurance
   B. Flood insurance
Operation costs.
   A. Character of the roadway surface.
   B. Narrow roadways.
   C. Horizontal alignment.
   D. Grade line treatment.
Rental value.
Salvage value.

It will not be attempted to go into the detail of the above fundamentals, only a brief discussion will be given.

Funds for the construction of highway bridges are in general derived from two principal sources: From direct tax such as property tax, gasoline tax, or license tag tax; or by direct borrowing as in the issuing of bonds.

If it is considered that a bridge of permanent con-
struction upon which there will be no maintenance charges is 
built for C dollars it will represent no loss of wealth then, 
as C dollars in money has been exchanged for C dollars in  
bridge construction, However, if C dollars had been loaned out 
at r interest (r being the annual interest rate) an amount of 
RC dollars would have been received. Therefore, if the amount 
C dollars is expended on a bridge instead of being loaned out  
at r interest an amount of RC dollars will be lost through the 
loss of interest each year. Or, in other words, the bridge will  
cost RC dollars each year. C is the capital or first cost.  
Annual expense ------------------------------- $rC

It was assumed above that the bridge was of permanent  
construction. However, that is not possible and it will be  
necessary to repair the bridge at intervals and to eventually  
replace the bridge when it wears out. Therefore, it will be  
necessary to spend say M dollars a year on maintenance and it  
will be necessary to deposit a sufficient amount say R dol-

lars a year at compound interest which will provide at the end  
of the life of the bridge an amount equal to the original cost  
of C dollars to replace the bridge with a new one.  
The total annual cost for maintenance, capital, and  
renewal is represented by R = rC / M / R.  
It may be desired to insure the bridge against loss  
or damage by fire or floods, if so there will be an annual  
charge of I dollars for the insurance premium.  
Operation costs may be divided into two main classes :  
The cost of operation a bridge such as a swing bridge or a
lift bridge, and the cost to traffic operating over the bridge. In regards to the last class of operation costs it can be understood that it would cost more to operate a vehicle over a bridge with a rough floor than it would to operate a vehicle over one with a smooth floor.

The operation of the bridge will be designated by 0' and the cost to the traffic will be designated by 0.

A bridge can be considered as having an earning capacity in several ways, namely: traffic service, development of scenic resources, enhancement of abutting property values, advertising to the community and to the state, and other attendant gains. The last four ways are hard to give a definite concrete value although there is no doubt that they exist.

A beautiful bridge over a river in a high class residential neighborhood, where both the river banks and the nearby surroundings are appropriately landscaped, will serve to enhance the value of the surrounding property; while an ugly, unsightly bridge would serve to depreciate the value of the surrounding property. However, the weight of enhancement or depreciation could hardly be determined absolutely, and any approximation would be affected by personal opinion and would therefore vary.

The earning capacity of a bridge may be considered as a rental value or as a fixed rental charge of p percent of the first cost.

A bridge at the end of its economic life may have a salvage value. The piers and abutments or part of them may
be used in the new construction, or the material in the bridge may have a scrap value. In some instances a bridge could be thought of as having a negative scrap value. One type of bridge might have a scrap value over and above the cost of demolition while another type at the same bridge site might be worthless after being demolished and might require an expenditure for the removal of the debris.

Let $S$ represent the salvage or scrap value of the bridge. In this case the term $R$ representing the renewal cost should be multiplied by the factor $(C - S) / C$ since the sinking fund need only accumulate the additional cost $(C - S)$ of a replacement.

The discussion of the general factors controlling type selection and the fundamentals of economic analyses along with the method of evaluating them has been taken from "Economics of Highway Bridge Types", McCullough. The discussion has not been taken word for word but this method and gist has been followed closely. The writer did not feel capable of developing a method of his own. No originality is claimed in the discussion which has been given except in the suggestion that a bridge might have a negative scrap value.

The following notation and equations used in the determining of the total annual cost of the three types of bridges that will now be considered have been taken from "Economics of Highway Bridge Types", McCullough.

\[ T.A.C. \ (Total \ Annual \ Cost) = rC \neq M \neq R \left( \frac{C - S}{C} \right) - pC. \]

The above equation considers scrap value and rental
value but does not consider operation of the bridge or traffic operation.

\[ R = \frac{C}{(1 + r)^n - 1} \]

r ----------- annual interest rate.
C ----------- capital account or initial cost of bridge
M ----------- average annual maintenance cost.
R ----------- The amount which must be deposited at the end of each year to accumulate with compound interest or \( r\% \) per annum an amount equal to \( C \) dollars in \( n \) years.

\[ R = \frac{C}{(1 - r)^n} \]

S = salvage value in dollars.
P = percent of first cost used as a rental value.

It will now be attempted to make a type selection for the proposed highway bridge over Tallulah Gorge.

After an investigation of the bridge site and all pertinent facts, all probable types of bridges were considered from which three types were selected for final analysis. The three types will henceforth be referred to as Type A, Type B, and Type C.

Type A.

Three-hinged Steel Arch.

<table>
<thead>
<tr>
<th>Estimate of cost.</th>
<th>@</th>
<th>$</th>
</tr>
</thead>
<tbody>
<tr>
<td>686145 lb. struct. steel</td>
<td>.0775</td>
<td>50851.24</td>
</tr>
<tr>
<td>46714 &quot; reinf. &quot;</td>
<td>.06</td>
<td>2802.84</td>
</tr>
<tr>
<td>224.5 cu. yd. cl. D conc.</td>
<td>22.00</td>
<td>4939.00</td>
</tr>
<tr>
<td>123.7 &quot; &quot; &quot; A &quot;</td>
<td>21.00</td>
<td>2597.70</td>
</tr>
<tr>
<td>124.2 &quot; &quot; &quot; B &quot;</td>
<td>20.00</td>
<td>2484.00</td>
</tr>
<tr>
<td>303.4 &quot; &quot; rock exc.</td>
<td>5.00</td>
<td>1517.00</td>
</tr>
<tr>
<td>Item</td>
<td>Quantity</td>
<td>Rate</td>
</tr>
<tr>
<td>-------------------------------------------</td>
<td>----------</td>
<td>-------</td>
</tr>
<tr>
<td>200 cu. yd. embankment</td>
<td>0.35</td>
<td>70.00</td>
</tr>
<tr>
<td>923 sq. yd. bituminous pavement</td>
<td>1.50</td>
<td>1384.50</td>
</tr>
<tr>
<td>530 lin. ft. wire cable</td>
<td>0.25</td>
<td>132.50</td>
</tr>
<tr>
<td><strong>Sub-total</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10% Eng. &amp; Contg.</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Amount of steel construction</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Other construction listed as concrete</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Type B.**

Steel Arched Cantilever.

<table>
<thead>
<tr>
<th>Item</th>
<th>Quantity</th>
<th>Rate</th>
<th>Amount</th>
</tr>
</thead>
<tbody>
<tr>
<td>522172 lb. struct. steel</td>
<td>0.0775</td>
<td>40468.33</td>
<td></td>
</tr>
<tr>
<td>35680 &quot; reinf. &quot;</td>
<td>0.06</td>
<td>2020.80</td>
<td></td>
</tr>
<tr>
<td>206 cu. yd. cl. D conc.</td>
<td>22.00</td>
<td>4532.00</td>
<td></td>
</tr>
<tr>
<td>427 &quot; &quot; &quot; B &quot;</td>
<td>20.00</td>
<td>8640.00</td>
<td></td>
</tr>
<tr>
<td>395 &quot; &quot; rock exc.</td>
<td>5.00</td>
<td>1975.00</td>
<td></td>
</tr>
<tr>
<td>580 &quot; &quot; com. &amp; bor.</td>
<td>0.35</td>
<td>203.00</td>
<td></td>
</tr>
<tr>
<td>833 sq. &quot; bituminous mat</td>
<td>1.50</td>
<td>1339.50</td>
<td></td>
</tr>
<tr>
<td><strong>Sub-total</strong></td>
<td></td>
<td></td>
<td>59078.53</td>
</tr>
<tr>
<td>10% Eng. &amp; Contg.</td>
<td></td>
<td></td>
<td>5907.85</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td>64986.49</td>
</tr>
<tr>
<td><strong>Amount of steel construction</strong></td>
<td></td>
<td></td>
<td>44515.16</td>
</tr>
<tr>
<td><strong>Other construction listed as concrete</strong></td>
<td></td>
<td></td>
<td>20471.33</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td>64986.49</td>
</tr>
</tbody>
</table>
## Type C.

**Reinforced Concrete Arch.**

Estimate of cost:

<table>
<thead>
<tr>
<th>Item</th>
<th>Quantity</th>
<th>Unit Cost</th>
<th>Total Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>192580 lb. reinf. steel</td>
<td>@ .06</td>
<td>$11554.80</td>
<td></td>
</tr>
<tr>
<td>1369 cu. yd. cl. A Conc</td>
<td>@ 35.00</td>
<td>47565.00</td>
<td></td>
</tr>
<tr>
<td>141.4 &quot; &quot; &quot; B &quot;</td>
<td>@ 20.00</td>
<td>2828.00</td>
<td></td>
</tr>
<tr>
<td>207.6 &quot; &quot; rock exc.</td>
<td>@ 5.00</td>
<td>1038.00</td>
<td></td>
</tr>
<tr>
<td>400 &quot; &quot; embankment</td>
<td>@ .35</td>
<td>140.00</td>
<td></td>
</tr>
<tr>
<td>923 sq. yds. concrete pavement</td>
<td>@ 2.00</td>
<td>1846.00</td>
<td></td>
</tr>
<tr>
<td>846 lin. ft. handrail</td>
<td>@ 2.50</td>
<td>2115.00</td>
<td></td>
</tr>
</tbody>
</table>

Sub-total ----------------------------------- $70836.80

10% Eng. & Contg.                         

Total -------------------------------------- $73795.48

All construction considered as concrete   $73795.48

---

**Recapitulation of first costs.**

**Type A.**

<table>
<thead>
<tr>
<th>Item</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel construction</td>
<td>$55936.36</td>
</tr>
<tr>
<td>Conc.</td>
<td>$17520.30</td>
</tr>
<tr>
<td>Total</td>
<td>$73456.66</td>
</tr>
</tbody>
</table>

**Type B.**

<table>
<thead>
<tr>
<th>Item</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Construction</td>
<td>$44515.16</td>
</tr>
<tr>
<td>Conc.</td>
<td>$20471.33</td>
</tr>
<tr>
<td>Total</td>
<td>$64986.49</td>
</tr>
</tbody>
</table>
Type C.

All Concrete Construction $73995.48

With the first cost determined, by applying the prevalent interest rates and assuming the rental values, a constant K can be obtained from Fig. 11 of Chapter V "Economics of Highway Bridge Types", McCullough, which when multiplied by the first cost will give the total annual cost.

The following rates and percents are used in obtaining K from Fig. 11.

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>R</td>
<td>rate of compound interest</td>
<td>3.5%</td>
</tr>
<tr>
<td>r</td>
<td>&quot; annual &quot;</td>
<td>6.0%</td>
</tr>
<tr>
<td>p</td>
<td>rental value of Type C</td>
<td>2.0%</td>
</tr>
<tr>
<td>p</td>
<td>&quot; &quot; A and E</td>
<td>1.5%</td>
</tr>
</tbody>
</table>

To obtain the value of K for 1.5% it will be necessary to interpolate between the values p = 1.0% and p = 2.0%

All the types were fireproof so no insurance rate was considered. Neither were operating expenses considered because these were considered equal for all three types.

From Fig. 11 the following values of K are obtained.

<table>
<thead>
<tr>
<th>Type</th>
<th>Value of K</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Construction</td>
<td>K = 6.1%</td>
</tr>
<tr>
<td>Steel</td>
<td>K = 7.7%</td>
</tr>
<tr>
<td>Conc. in steel const.</td>
<td>K = 6.6%</td>
</tr>
<tr>
<td>Type</td>
<td>Cost.</td>
</tr>
<tr>
<td>------</td>
<td>---------</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel const</td>
<td>863536.36</td>
</tr>
<tr>
<td>Conc.</td>
<td>17520.30</td>
</tr>
<tr>
<td>Type B</td>
<td></td>
</tr>
<tr>
<td>Steel</td>
<td>44515.16</td>
</tr>
<tr>
<td>Conc.</td>
<td>20471.33</td>
</tr>
<tr>
<td>Type C</td>
<td></td>
</tr>
<tr>
<td>Conc.</td>
<td>73795.48</td>
</tr>
</tbody>
</table>

From the above table it can readily be seen that Type A would be eliminated from any further consideration and that the choice would lie between Type B and Type C.

There is another factor that should be considered before making a type selection, and that is the scrap value of the two types. Type C is of concrete construction throughout and would have no scrap value. Type B is of steel superstructure with concrete piers and abutments. It is thought that the abutments and piers might be utilized in designing any placement structure, more so since they can be considered of mass construction and therefore their economic life can be considered as almost indefinite. And the superstructure, which is composed entirely of shapes instead of built up sections, should have a comparatively high scrap value. The shapes could be reclaimed and used in smaller structures. Another factor which would tend to increase its scrap value would be its close proximity to a railroad.

After investigations made by Mr. C. T. Parkin, it has been decided that $3000.00 will be a fair scrap value to
allow for Type B.

Taking the economical service period of Type B from Fig. 7 McCullough as 50 years and dividing it into $3000.00 we get $60.00 which is the amount the total annual cost may be reduced. Applying this correction to Type B we have:

<table>
<thead>
<tr>
<th>Type</th>
<th>Total Annual Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>$4718.00</td>
</tr>
<tr>
<td>C</td>
<td>$4501.52</td>
</tr>
<tr>
<td>Difference</td>
<td>$217.26</td>
</tr>
</tbody>
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Throughout this economical analysis the writer has tried to give proper consideration and values to all items without being prejudiced toward any one type of construction and it is believed that all factors have been dealt with that would enter into type selection.

In concluding the writer would recommend that Type C, The Reinforced Concrete Arch be the type selected. His reason for this selection is because the analysis shows that the total annual cost between Reinforced Concrete Arch and the Steel Arched Cantilever amounts to $217.26 in favor of the Concrete Arch, and that it is believed the Concrete Arch will lend its self to aesthetical development more so than the Steel Arched Cantilever.
APPENDIX - C

INFORMATION CONCERNING
WEIGHTS OF DETAILS AND UNIT
COSTS OF STRUCTURAL STEEL
Virginia Bridge & Iron Co.

Steel Structures

Bridges, Buildings, Etc.

General Offices: Roanoke, Va.
M.C. Drawers P.O.
File No. 676

Mr. C. T. Parke,
State Hwy. Dept. of Ga.,
Bridge Dept.,
Macon, Ga.

My dear Mr. Parke:

I have your letter of October 16 in which you advise that you are making a design for a 260' three-hinged arch with 2-50' deck truss approach spans and also a 492' cantilever arch as an alternate for the same crossing.

You ask what percentage to add to the weight of the members to give you an approximate weight of the details for these two bridges. The 25 to 45% that you give is close enough for preliminary figures depending upon, of course, the make up of the members. If the members are of H-sections, of course the lower figure will apply and if built-up latticed sections are used the higher figure will apply and these amounts will generally average about 33-1/3%. Percentage of the rivet heads alone will run about 2%. After using these approximate percentages for details and design is made, I suggest that you lay out the gusset plates determined by the number of rivets required for the various members and then calculate the weight of these details and make such changes in your dead load stresses as are necessary.

Percentages are very dangerous things and the above method is the one we always use because it does not take long and the degree of accuracy is very close to the actual shipping weight of the structure.

There will probably be for such small structures as you are contemplating very little difference between the weights of the arch and the cantilever arch design than for simple structures of the same span. Both of these structures are not really in the field of the arch or the cantilever and unless there is some erection condition that I do not know of, I do not see the necessity of using an arch span or the cantilever span unless it is for aesthetic reasons. I note that you are using a 20' roadway, which is in accordance with present standard highway practice allowing for 2-9' lanes of traffic. However, the time is rapidly approaching, due to the constant increase in size of automobiles, trucks, busses and moving vans, when a 9' lane of traffic will be a name only. I do not believe that any modern bridge should be built for less than 2-12' traffic lanes.
I notice that you are battering the arch span. With the depth given and the width of the bridge given, this battering is hardly necessary for stability and will add a great deal to the cost of fabrication. The same is true for the cantilever design.

I notice that all of the above is to be subject to a thesis entitled, "Economic Design of a Proposed Highway Bridge over Tallulah Gorge", and as the remarks I have made are in the interest of economy and intended as constructive criticism only, I am hoping that they will be received in the same spirit and trust that the information that I have given you will be a help to you.

If there is anything that I have omitted and you need, please ask for it and I will be glad to supply you if I can.

Yours very truly,

[Signature]

Chief Engineer.

VIRGINIA BRIDGE & IRON CO.

Date 10-21-29
November 4, 1929

My. E. C. Durkin,
My. E. H. Braswell,
State Highway Dept.,
Charleston, S.C.

Gentlemen:

I have your letter of October 30, giving additional reasons for the type you selected for the Tabbish Gorge Bridge. Under the circumstances, your types are justified and I believe that the cantilever type design is preferable.

You ask me for an approximate bid price. I would say that the steel work only for either of these bridges could be made approximately $115 to $150 a ton, say $135. This may seem rather high but the erection will be very expensive, possibly $100 a ton, including two coats of field paint and a small margin for contingencies.

Please understand that this is not a bid and in no way is the Virginia Bridge & Iron Co. to be held responsible for this but you can use it for comparative purposes in your thesis.

If there is anything else or any other information that we can give you, do not hesitate to call upon us,

Yours very truly,

Chief Engineer.
Mr. G. T. Jarkin,
State Highway Dept. of Georgia,
Bridge Department,
East Point, Ga.

Dear Sir:

Your letter of October 16th addressed to American Bridge Company has been referred to the writer for attention.

It is not possible to give a percentage of the details referred to the main sections of bridge work that will be correct in a variety of cases. These percentages vary largely with the magnitude of the work, the design and the manner in which the details are developed. In some very large and heavy work the percentage may be as small as twenty and on the other hand I have estimated work where it ran as high as fifty.

Your question concerning the percentage of rivet heads requires practically the same answer for here again it depends very largely on the same practice. For ordinary work this percentage is likely to vary from four to six but where many rolled shapes of modern design are used the percentage tends to reduce rapidly.

Very truly yours,

O.E. Hovey
Assistant Chief Engineer
December 26, 1920

Mr. J.J. Partin,
State Highway Department of Georgia,
Savannah, Ga.,

Dear Sir:

I have been delayed in answering your inquiry regarding the percentage of detail to the main member sections, due to the fact that I have been out of the city a great deal of the time, and have not had an opportunity to look up the question which you raise.

In going over my notes made several years ago, I find the following notation which may be of interest to you.

Add the following percentages to the weight of main members to cover the weight of details:

- Roof Trusses, ordinary: - - - - - - - - - - - - - - - - - 20%
- Roof Trusses, very light: - - - - - - - - - - - - - - - - - - 20%
- Highway Bridges, simple span, eye beam: - - - - - - - - - - - 20% to 30%
- Highway Bridges, draw span, eye beam: - - - - - - - - - - - 35% to 40%
- Highway Bridges, simple span, eye bar: - - - - - - - - - - - 25% to 30%
- Highway Bridges, simple span, riveted: - - - - - - - - - - - 15% to 20%
- Highway Bridges, draw span, riveted: - - - - - - - - - - - 35% to 40%
- Highway Bridges, single span, riveted: - - - - - - - - - - - 35% to 40%
- Highway Bridges, single span, riveted: - - - - - - - - - - - 35% to 40%

These percentages are added to what is the weight of the members themselves and a similar percentage should be added to the weight of the integral system.

I note from your letter that you contemplate using solid rolled sections for the truss members, and for this reason the percentages to be added will be somewhat lower than those given, which contemplate the use of built up sections.

Yours very truly,

[Signature]

STEEL insures strength and security