Model for Permit Loads for Barnes Slough Bridge

by

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Continuous Span, Simple Span

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Abstract

ALDOT has an eleven-span flat slab concrete bridge over Barnes Slough and Jenkins Creek on the northbound side of US Highway 82/231 that was built in 1915 for which there are no construction drawings or other available data. This bridge is referred to as “Barnes Slough Bridge”. The goals of this research are to define the capacity of Barnes Slough Bridge, rate the bridge, and provide a permit model that can be modeled in AASHTOWare software. ALDOT can then use this model to provide permits for non-standard trucks to travel over this bridge. These goals were achieved in four steps. First, the design methods from early 1900s were reviewed with an aim to identify the methods used to design this bridge. Second, field measurements and tests were performed to confirm the accuracy of assumptions made based on the literature review. Third, the data collected from field measurements were analyzed to finalize the characteristics and parameters that would be used to define the capacity of the Barnes Slough Bridge. Fourth, this information was used to build a permit model of the Barnes Slough Bridge in AASHTOWare. The permit model is a model of one effective width of slab.

This research defined the cross sectional capacity of the Barnes Slough Bridge and provided load ratings. However, the performance of the structure through its life and during the live load tests suggests that the structure has significant capacity and perhaps more than the expected capacity shown in analyses. Hence, choosing an effective width larger than the value defined by AASHTO was the most appropriate parameter that could incorporate additional

capacity into an AASHTOWare model while keeping the measured parameters the same as those measured. These findings suggest that the load ratings and the permit model given in this research underestimates the true capacity of this bridge. Future work by others will provide finite element based ratings of this structure to define the actual effective width. Upon completion of that work ALDOT will then be able to choose a final effective width for the permit model.

Acknowledgements

First of all I would like to thank Alabama Department of Transportation and Auburn University Highway Research Center for this opportunity and their financial support of this research. I would also like to thank my advisory committee consisting of Dr. J. Michael Stallings, Dr. Robert Barnes, and Dr. Nowak for continuous guidance, support, and the knowledge they have shared beyond the scope of this project and have taught me valuable life lessons.

Next, I would like to thank my partner Charles Stephen. If it was not because of him I could not have taken advantage of this opportunity in my life. He has inspired me, encouraged me to focus on my goal, and helped me move forward throughout the years in every aspect. Lastly, but not least, I would like to thank my dear parents who have always believed in me, supported me, trusted me, and have been patient from across the globe.

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1 Introduction

1.1 Motivation

ALDOT has an eleven-span flat slab concrete bridge over Barnes Slough and Jenkins Creek on the northbound side of US Highway 82/231 at milepost 162.56 (Figure 1-1) for which there are no construction drawings or other details that can be used to perform a load rating of the structure. This bridge is approximately one mile south of Taylor Road on the south side of Montgomery. ALDOT's "Bridge Card" for the structure indicates that the bridge was widened by approximately 4 ft in 1930, and the visual inspection of the bridge indicates that width has been added to the east side of the bridge twice and to the west side once. The sequence and time of these additions are unknown.

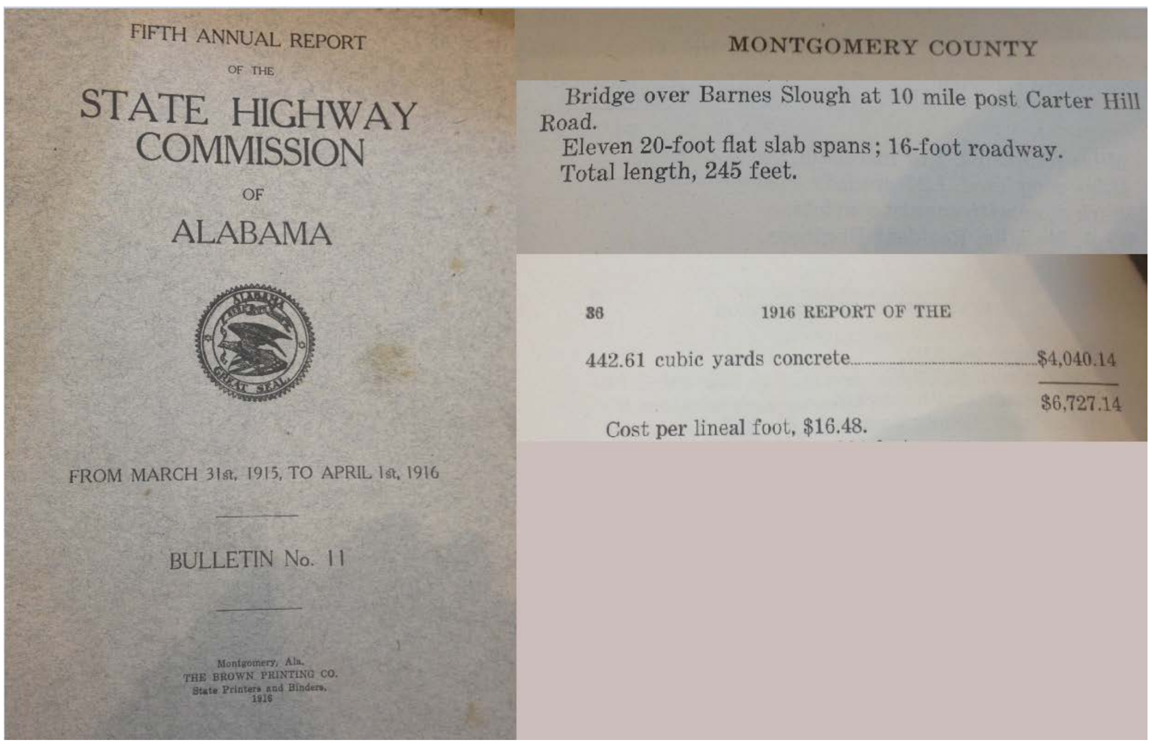
Figure 1-1: View of East Side of Barnes Slough Bridge

Figure 1-2: Bottom View of the Barnes Slough Bridge Looking East

Also the existence of some cracks on the sides of the slab near the supports were indicative of shrinkage or temperature cracking in the concrete, but there were no significant signs of flexural or shear cracking, nor evidence of anchorage or bond failure. Under two footings beneath the first and second support on the east side 6 in. scour has occurred. Currently the bridge carries unrestricted traffic. This is allowed by AASHTO's *The Manual for Bridge Evaluation* (2011) in cases where a reinforced concrete bridge of unknown details has carried unrestricted traffic without developing signs of distress. But, because the structural details of the bridge are unknown, ALDOT cannot perform an analysis to justify issuing a permit to any overweight, non-standard trucks. So, overweight, non-standard trucks are detoured around this bridge. ALDOT would like to have the ability to consider requests for permits on this heavily travelled route.

1.2 Historical Research

A search of historical documents was performed. The focus was to pinpoint an era when the bridge was built in order to achieve an understanding of the methods used to design the structure. By using the Auburn University library resources and the Alabama Department of Archives and History in Montgomery, the construction year was established as 1915 from a report of the state of Alabama Highway Commission (State Highway Commission of Alabama 1916).


FIFTH ANNUAL REPORT
OF THE
STATE HIGHWAY COMMISSION
OF
ALABAMA

FROM MARCH 31st, 1915, TO APRIL 1st, 1916

BULLETIN No. 11

Montgomery, Ala.
THE BROWN PRINTING CO.
State Printers and Binders.
1916

MONTGOMERY COUNTY

Bridge over Barnes Slough at 10 mile post Carter Hill Road.
Eleven 20-foot flat slab spans; 16-foot roadway.
Total length, 245 feet.

36 1916 REPORT OF THE

442.61 cubic yards concrete........ $4,040.14

$6,727.14

Cost per lineal foot, $16.48.

Figure 1-3: Records of Expenditure for Construction of Barnes Slough Bridge
(State Highway Commission of Alabama 1916)

State Highway Commission of Alabama (1916) has the itemized list of infrastructure built between March 31, 1915 and April 1, 1916. A description of the Barnes Slough Bridge is shown on a photo of part of page 35 and 36 of that publication in Figure 1-3. These pages describe a bridge over Barnes Slough at the 10 mile post of Carter Hill Road. The description of the bridge as eleven 20-foot flat slab spans matches the original details of the existing bridge before it was widened. The location referred to as Barnes Slough is now referred to as Jenkins Creek, which is the location of the bridge that is the subject of this research. Through comparing the description of the location of the bridge on an old map of Montgomery County to a current map of Montgomery County it was confirmed that these are the same bridge.

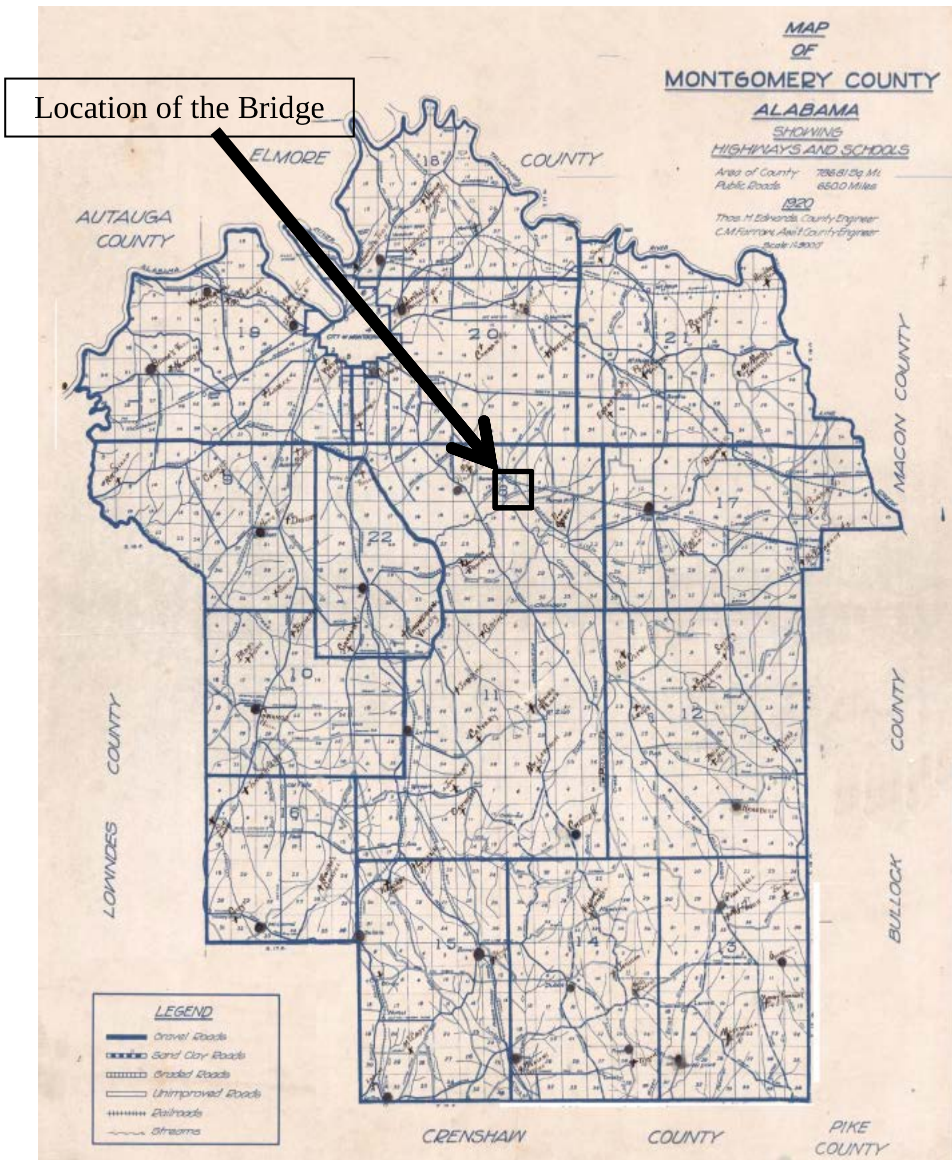


Figure 1-4: Map of Montgomery County, Alabama by Thomas H. Edwards, 1920 (http://alabamamaps.usa.edu)

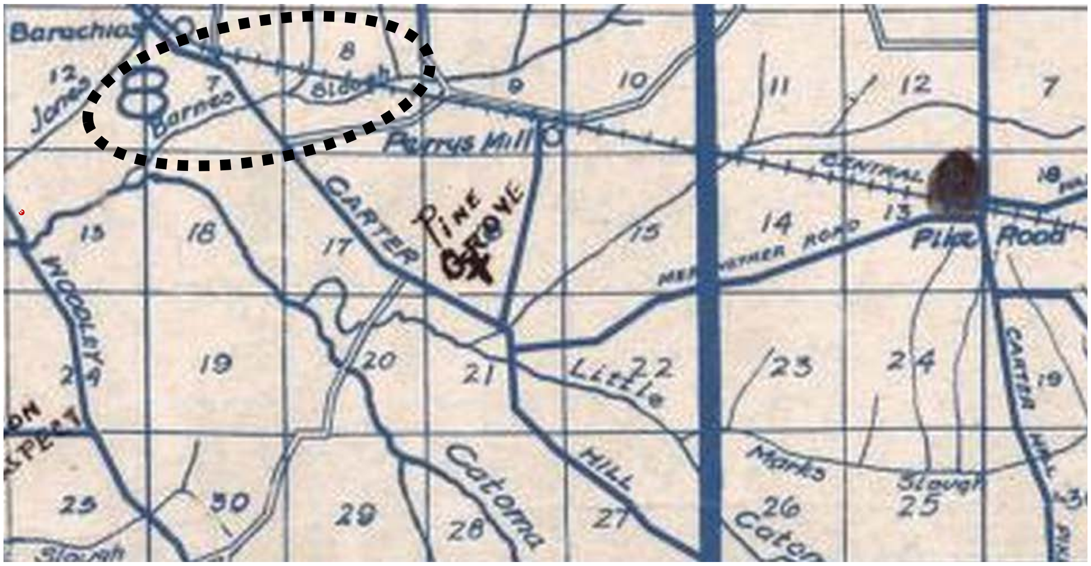


Figure 1-5: Map of Montgomery County, Alabama by Thomas H. Edwards, 1920. Range 19 E, Township 15 N, Section 7
(http://alabamamaps.ua.edu)

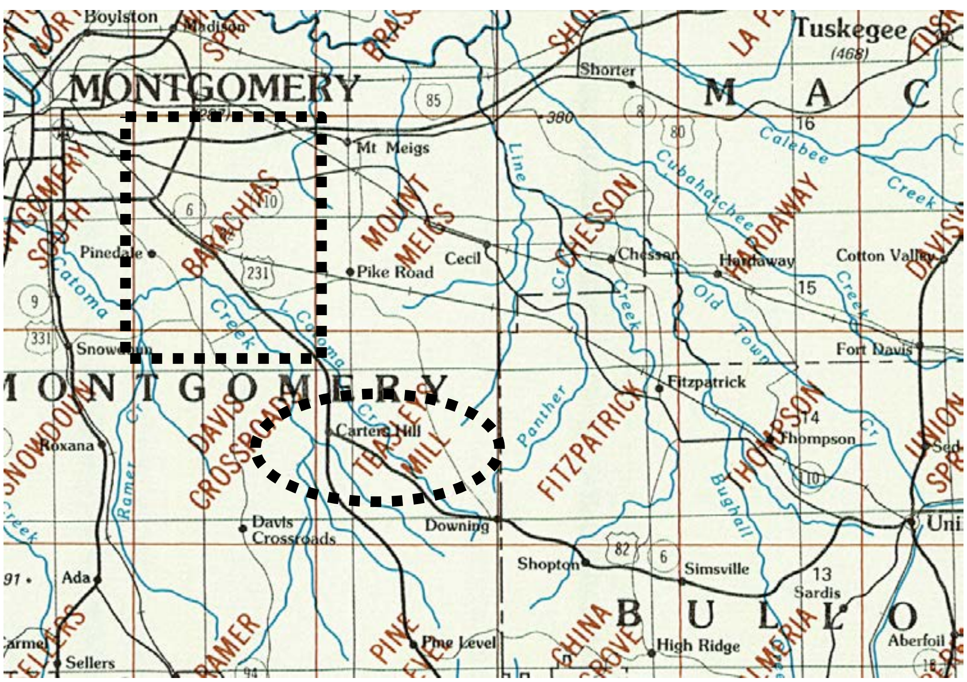


Figure 1-6: Current map of Montgomery County. Range 19 E, Township 15 N, Section 7 in 1993
(Alabama Index to topographic and other MAP COVERAGE. Scale: 1:24000. Denver Colorado. United States Geological survey, 1993)

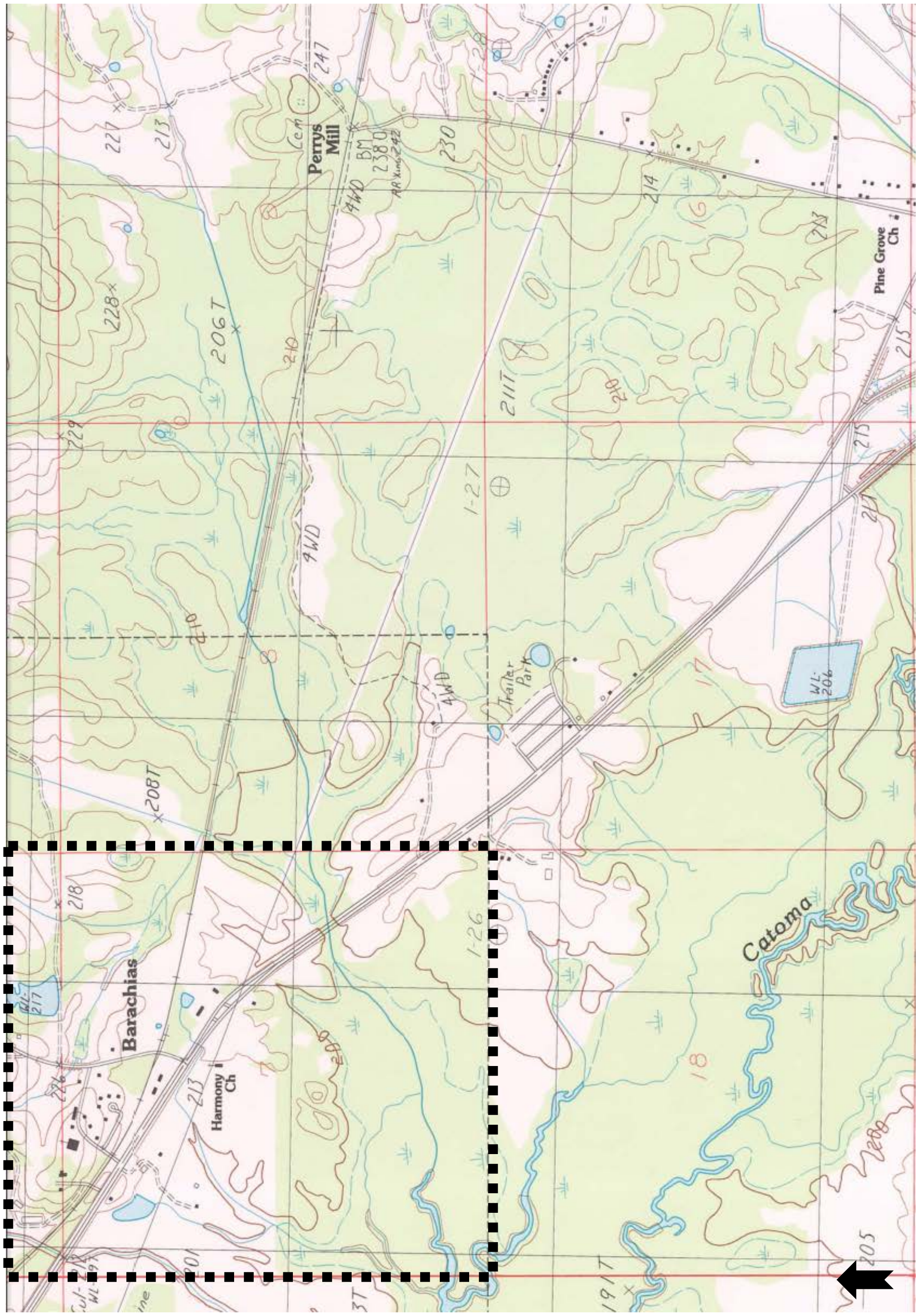


Figure 1-7: Current map of Montgomery County. Range 19 E, Township 15 N, Section 7 in 1999
(BARACHIAS Quadrangle ALABAMA-Montgomery CO. Scale: 1:24000. Denver Colorado. United States Geological survey, 1999)

A map of Montgomery County in Alabama from 1920 (Figure1-4) was found using the following link:

http://alabamamaps.ua.edu/historicalmaps/counties/montgomery/montgomery.html

Figure 1-4 shows the full map from 1920. Figure 1-5 is a portion of the map from Figure 1-4, and it shows Barnes Slough in section 7 of range 19 E, Township 15 N. A visual comparison can be made between the modern maps (Figure 1-6 and 1-7) and the old map (Figure 1-5) to confirm that the bridge on US Highway 82/231, which is the subject of this project, is the bridge over Barnes Slough reported by the State Highway Commission of Alabama (1916).

Although details of the bridge widening projects are not available, confirmation that the original construction of the bridge was in 1915 narrowed the research focus. Any bridge or found documents from that time frame became a useful reference for understanding the engineers' design process in 1915.

1.3 Project Objectives and Scope

The objective of this research is to provide ALDOT with a structural model of the eleven-span concrete flat slab bridge that can be used for analyses required for issuing permits to non-standard trucks. This objective is being accomplished by ALDOT Research Project 930-889 completing the tasks listed in Table 1-1. This thesis addresses tasks 1,2,5,6, and 7 of Project 930-889

Table 1-1: Tasks of ALDOT Research Project 930-889

Task	**Activity**
1	Evaluation of Standard Reinforced Concrete Slabs
2	Baseline Structural Model
3	Field Test
4	Advanced Structural Analysis of Baseline Model
5	Final Structural Model
6	Study of ALDOT Permit Process
7	Development of the Final Report

Chapter 2 of this thesis includes a literature review of the concepts and methods used to design flat slabs in the early 1900s in addition to defining the expected values for parameters such as material properties, and resistance. Chapter 3 illustrates applications of these concepts for bridges from the 1920's to calculate the reinforcement in the slab. Also the concept of bridge rating is introduced. ALDOT uses AASHTOWare for the purpose of rating. In Chapter 3 a brief explanation of the software is provided. In Chapter 4 these concepts are applied to estimate the amount of reinforcement needed in Barnes Slough Bridge. After the initial calculations and estimating the amount of steel and capacity of the bridge, some field tests were done. These tests included the ground penetration radar, Schmidt hammer tests, core tests, and nondestructive live load testing to understand the behavior of the structure and establish the material properties and cross sectional properties. Lastly, a series of models were built in AASHTOWare (*AASHTOWare*. Computer software.*Http://www.aashtoware.org/Pages/default.aspx*. Vers. 2014. AASHTO, n.d. Web.) to rate the current structure. In Chapter 5, a finalized model of the bridge is provided. This model can be used for the purpose of issuing permits for non-standard trucks. Chapter 6 provides conclusions and recommendations.

2 Literature Review

2.1 Introduction

To provide ALDOT with a structural model of the Barnes Slough Bridge, which is needed for rating of the bridge and issuing permits for non-standard trucks, a primary focus of this research was to determine the amount of reinforcement in the slab. One strategy was to use equipment and field tests that detected the rebar diameter, clear cover, spacing, and the concrete strength. These test results were used to directly calculate the cross-section capacity. Another strategy was to understand how the engineers in the early 1900s designed a reinforced concrete slab for bridges.

Various sources were studied to confirm a series of methods and material properties that were used during these time periods to calculate cross sectional capacity of the different segments of the slabs from the early 1900s until 1960s. These studies include elastic cross section analysis, material properties, loadings of dead load and live loads and truck load, whether the concept of a multi-presence factor existed, impact, structural analysis methodology (how they loaded the structure to cause the maximum effect on the span), effective width of the slab, and the relationship between the cross-sectional components that resulted in a certain capacity. In Chapter 4, the amount of reinforcement from each time frame is listed to compare these results against the amount of reinforcement measured in field tests to gain an understanding of when these newer portions were added. Lastly, the shear and reinforcement development length requirements for slabs were reviewed to perform these checks for the slab. Below the concepts learned are divided by the year starting with the earliest years followed by the shear and development length requirements. The complete explanation of the process is documented in the early 1900s section, and the following years only explain the methods and concepts for that time frame that were different than the early 1900s. A summary of the process of each method is included in the Appendices for ease of understanding the differences between each decade.

2.2 Early 1900s Concepts

2.2.1 Elastic Cross Section Analysis

Engineers in early 1900s used an elastic analysis of a cracked section for reinforced concrete. The maximum concrete compression and tension in the reinforcement were limited to allowable values. In this section the relationship between different components of a reinforced concrete cross section and how to calculate the resisting moment according to these components are explained. The following discussion is similar to one presented by Kirkham (1932) and AASHO (1931).

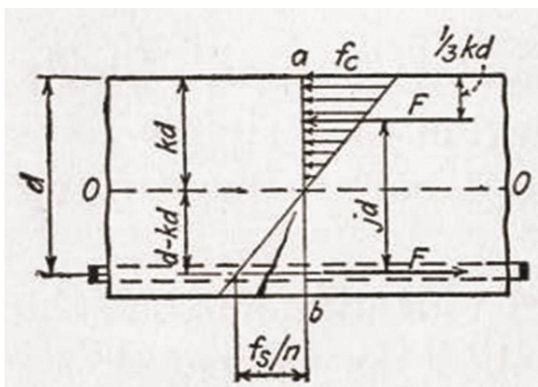


Figure 2-1: Stress Diagram for Reinforced Concrete Beam (Kirkham, 1932)

Figure 2-1 shows a generalized stress diagram of a reinforced concrete beam, in which the concrete below the neutral axis is cracked, and reinforcing bars take all the tension due to cross section bending. The stress diagram is defined in the same figure. The total force in the reinforcement and also for the total force in the concrete, Equation 1 defines the relationship below:

$$F = \frac{M}{jd} \qquad \text{(Eq -1)}$$

Where

F = Resultant compressive force in the concrete and also resultant tension in the reinforcement
M = Represents the bending moment on the beam
jd = Distance from the center of the reinforcement to the resultant compressive force in the concrete

For the total force in the concrete we also have:

$$F = \frac{1}{2}\frac{f_c}{kdb} \qquad \text{(Eq -2)}$$

Where

f_c = Compressive stress on the concrete at the top of the beam for positive moment
kd = Distance from the extreme compression fiber to the neutral axis
b = The width of flexure compression zone

From this last equation we obtain:

$$f_c = \frac{2F}{kdb} \qquad \text{(Eq -3)}$$

or:

$$f_c = \frac{2M}{jkbd^2} \qquad \text{(Eq -4)}$$

for the maximum stress in the concrete. All the variables are defined previously.

From similar triangles (shown in Figure 2-1) we have:

$$\frac{f_c}{\frac{f_s}{n}} = \frac{kd}{d - kd} = \frac{k}{1 - k} \qquad \text{(Eq -5)}$$

Where

f_s = Tensile stress in the reinforcement
d = Distance from the extreme fiber to the center of the reinforcement

From equilibrium of the resultant compression force and tension force:

$$\frac{1}{2} f_c bkd - nA_s \left(\frac{f_s}{n}\right) = 0 \quad \text{(Eq -6)}$$

Combining Eq-5 and Eq-6 results in:

$$k = \sqrt{2\rho n + (\rho n)^2} - \rho n \quad \text{(Eq -7)}$$

and,

$$\rho = \frac{A_s}{bd} \quad \text{(Eq -8)}$$

Where

n = Ratio of the modulus of elasticity of steel to that of concrete
ρ = Ratio of the area of flexural tension steel to the area of effective cross section

As seen in Figure 2-1, $jd = d - \frac{1}{3}kd$ and by dividing by d, we get:

$$\mathrm{j} = 1 - \frac{1}{3}\mathrm{k} \quad \text{(Eq -9)}$$

A_s is the area of steel required, calculated using the following equation:

$$\mathrm{A_s} = \frac{\mathrm{F}}{\mathrm{f_s}} \quad \text{(Eq -10)}$$

or:

$$\mathrm{A_s} = \frac{\mathrm{M}}{\mathrm{f_s} jd} \quad \text{(Eq -11)}$$

2.2.2 Material Properties:

The expectation was that the material strength in early 1900s was lower than the values used today, therefore, there was a need to find common material strengths used in that era. One of these findings was that the engineers of early the 1900s, like today, categorized the material strength depending on the type of the construction. Another finding was that they used allowable stress values for design. Although not one source categorized the material properties of steel and concrete used in design, the allowable stress values for steel and concrete were established through their use in various sources from the early 1900s period. Figure 2-2 and 2-3 show examples of notes found in two different sources. These values are listed in Table 2-1 below: (Hool and Whitney, 1921; S.E. Slocum, 1914)

Table 2-1: List of Material Properties found in various sources

Parameters	**Values**
Modulus of Elasticity of Steel, E_S (ksi)	30,000
Modulus of Elasticity of Concrete, E_C (ksi)	1200 - 2500
Allowable Steel Tensile Stress, f_s (ksi)	16000
Allowable Concrete Compressive Stress, f_c (ksi)	0.650

1. AVERAGE VALUES OF PHYSICAL CONSTANTS

Material	Ultimate Tensile Strength lb./in.²	Ultimate Compressive Strength lb./in.²	Ultimate Shearing Strength lb./in.²	Ultimate Flexural Strength (Modulus of Rupture) lb./in.²	Elastic Limit lb./in.²	Unit Elongation at Elastic Limit inch	Young's Modulus of Elasticity E lb./in.²	Modulus of Shear (Modulus of Rigidity) lb./in.²	Weight lb./ft.³	Coefficient of Linear Expansion 1° F.
Hard steel	100,000	120,000	80,000	110,000	60,000	0.0012	30,000,000	12,000,000	490	.0000074
Structural steel	60,000	60,000	50,000	60,000	35,000	0.0012	30,000,000	12,000,000	490	.0000061
Wrought iron	50,000	50,000	40,000	50,000	25,000	0.0010	25,000,000	10,000,000	480	.0000068
Cast-iron tension	20,000	. . . .	20,000	35,000	6,000	0.0004	15,000,000	6,000,000	450	.0000063
" compression	. . . .	90,000	. . . .	. . . .	20,000	. . . .	. . . .	. . . .	. . . .	. . . .
Brass, drawn	43,000	11,000	. . . .	. . . .	. . . .	. . . .	14,500,000	5,000,000	530	. . . .
" cast	24,000	. . . .	. . . .	. . . .	. . . .	. . . .	9,500,000	. . . .	520	. . . .
Copper, drawn	32,000	. . . .	. . . .	. . . .	. . . .	. . . .	15,000,000	6,000,000	550	.0000094
" cast	22,000	. . . .	. . . .	. . . .	. . . .	. . . .	12,000,000	. . . .	. . . .	. . . .
Timber, with grain	10,000	8,000	600	9,000	3,000	0.0020	1,500,000	. . . .	40	.0000028
" across grain	. . . .	. . . .	3,000	. . . .	. . . .	. . . .	. . . .	400,000	. . . .	. . . .
Concrete	300	3,000	1,000	700	1,000	. . . .	2,000,000	. . . .	150	.0000055
Stone	. . . .	6,000	1,500	2,000	2,000	. . . .	6,000,000	1,800,000	160	.0000050
Brick	. . . .	3,000	1,000	800	1,000	. . . .	2,000,000	. . . .	125	.0000050

Figure 2-2: The Modulus of Elasticity of Concrete (S.E. Slocum, 1914)

FLAT SLABS

FLAT SLAB FLOORS
NEW YORK CITY BUILDING CODE
INTERIOR SQUARE PANELS

CAP CONSTRUCTION

f_c = 650 for positive moment
f_c = 750 for negative moment
f_s = 16,000
n = 15

Figure 2-3: Concrete Designer's Manual (Hool and Whitney, 1921)

The modulus of elasticity of concrete was confirmed to be between 1200 ksi and 2500 ksi, and modulus of elasticity of steel was 30,000 ksi. Hence, two common values of 15 and 12 were used as modular ratio, n, which corresponded to higher and lower values of modulus of elasticity. Additionally the use of a modular ratio of 15 was confirmed in a few other sources (Turneaure and Maurer, 1911; Trusted Steel Company, 1910).

2.2.3 Loadings:

The engineers in early 1900s had a very similar understanding of dead load and live load as today. The dead load consists of the weight of the structure, including the floor, floor covering, sidewalks, railings, and any fixed loads due to car tracks, pipe lines, conduits, etc. In a case of concrete slab floors, an allowance was made in the design dead load to provide for the weight of the wearing surface. This allowance depended on the type of wearing surface, and it was considered less than 15 pounds per square foot of roadway. The weight of the pavement if not wood plank, was 150 pounds per cubic foot. The maximum live load depended upon the locality of the structure. The live load varied from interurban cars, streets cars, heavy trucks, and dense crowds of people in near cities and towns, to light trucks, slow-moving traction engines (Figure 2-4), and droves of livestock in outlying country districts.

As for highway live loads, the standard truck shown in Figure 2-4 can be considered as the unit of loading. These trucks are known as H20, H15, and H10. The numerals following the

"H" in each case indicate the weight of the truck in tons. Figure 2-3 shows the spacing of axles and tires for a truck. In addition to truck loading, equivalent loading was used, which is also known as lane-loading. Equivalent loading consists of uniform load per linear foot of traffic lane combined with a single concentrated load so placed on the span as to produce maximum load effect. According to AASHO (1931) the equivalent loading was not used for spans less than 60 ft. Figure 2-5 shows the distribution of truck weight over four wheels, the axle spacing, and the lateral position of a truck on a roadway lane. Figure 2-7 shows the equivalent loading configurations.

Figure 2-4: Traction Engine from Early 1900s
(http://www.cheffins.co.uk/assets/news/358_2-m.jpg)

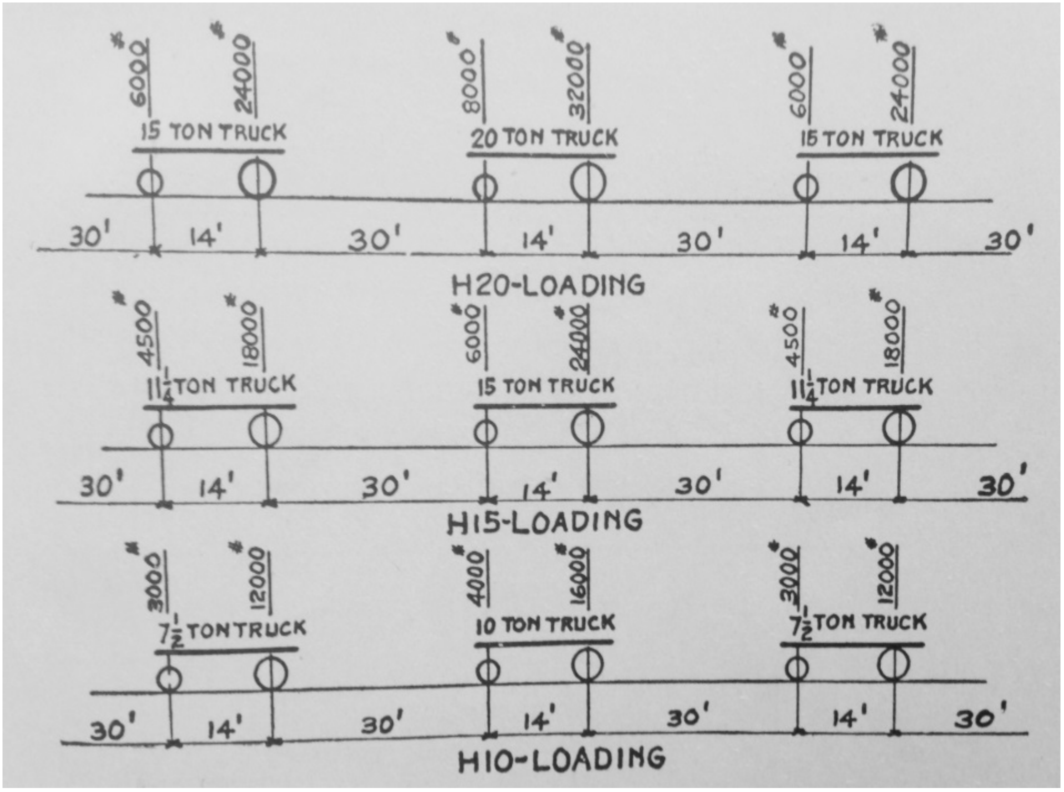


Figure 2-5: H20, H15, H10 Truck loading (AASHO, 1931)

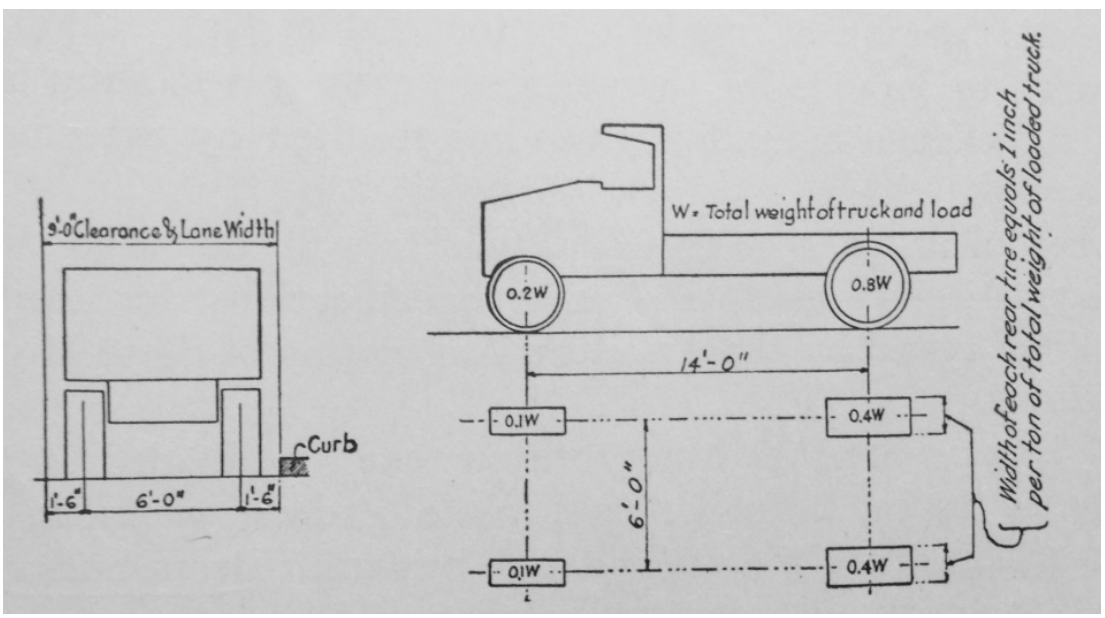


Figure 2-6: Spacing of the axles (AASHO, 1931)

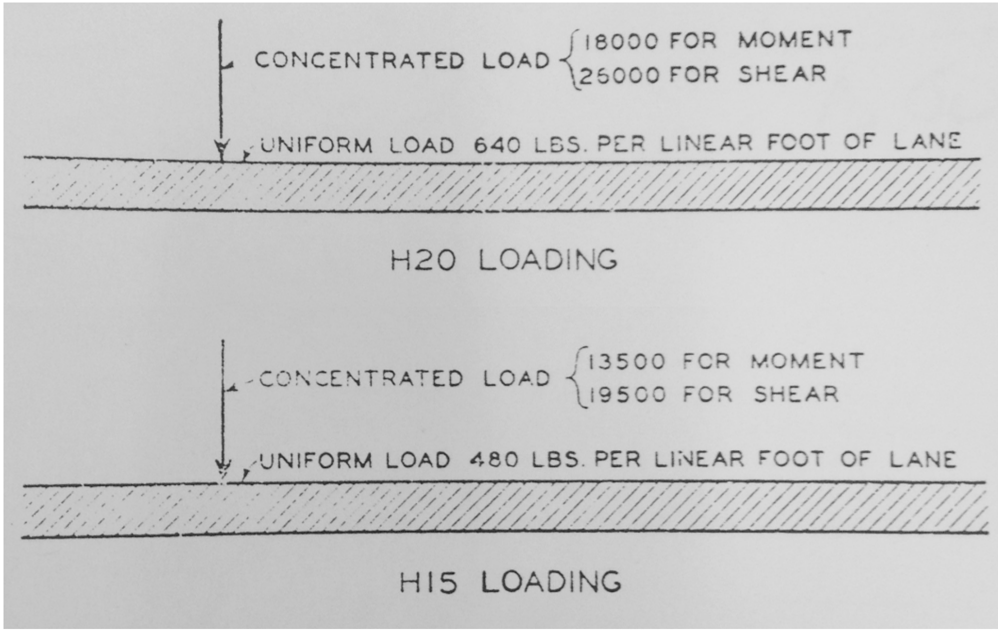


Figure 2-7: Equivalent Loading Configuration (AASHO, 1931)

2.2.4 Impact:

The relationship below gives the value for coefficient of impact, C_I, which would apply to live load from AASHO (1931).

$$C_I = \frac{50}{L + 125} \qquad \text{(Eq -12)}$$

Where

L = Length in ft of the portion of the span loaded to produce maximum live load effect in the member considered. (AASHO, 1931)

2.2.5 Reduction in Load Intensity

According to AASHO (1931), if the loaded width of the roadway exceeded two lanes, width of 18 ft, the specified loads were reduced one percent for each foot of loaded roadway width in excess of 18 ft with a maximum reduction of 25 percent, corresponding to a loaded roadway width of 43 ft. When the loads were lane loads, the loaded width of the roadway was the aggregate width of the lanes considered; if the load lane were distributed over the entire width of the roadway, the loaded width of the roadway was the full width of roadway between curbs.

2.2.6 Number of Traffic Lane:

The number of traffic lanes was an important unknown since it impacted the effective width used in designing the slab. This number was decided based on the American Highway Engineers Handbook (Blanchard, 1919) to be two lanes. Equation 13 was used to define the number of traffic lanes.

$$\boldsymbol{Y} = \frac{\mathrm{x}}{6} + 16 \geq 12 \text{ feet} \qquad \text{(Eq -13)}$$

Where

Y = roadway width in feet
X = number of vehicles traveling simultaneously

Also AASHO (1931) recommended a width of 10 ft for a lane of traffic and in case of a 16 ft roadway, the roadway would be designed for two traffic lanes. In cases where the roadway width was less than the desired 18 ft, and more than the minimum width of 16 ft, the effective width was calculated using the concept of overlap. The concept of overlap is explained in Section 2.2.7.

2.2.7 Effective Width in Concrete Slab:

In 1931 the concept of effective slab width was used to calculate the required resistance for which the bridge should be designed. For roadways with two traffic lanes, the roadway had two zones. The inner zone, which carried a maximum moment due to the two trucks' wheels passing each other at the middle of the road, and outer zones that were designed to carry the lesser amount of live load moment due to one truck's wheel line. The method of calculating the effective widths were confirmed to be the same in two sources (Kirkham, 1932; AASHO, 1931).

In calculating bending moment due to wheel loads on concrete slabs, no distribution in the direction of the span of the slab was assumed. In the direction perpendicular to span of the slab, the wheel load was considered as distributed uniformly over an effective width of slab. This effective width was obtained from the following formula:

$$\boldsymbol{E_O} = 0.7S + W \leq 7 \text{ feet} \qquad \text{(Eq -14)}$$

Where

$E_{I \text{ or } O}$ = Effective width in ft for one wheel in the inner zone or the outer zone as defined in the subscript.
S = Length in ft of the portion of the span loaded to produce maximum live load effect in the member considered
W = Width of the wheel or tire in ft

For cases when two wheels are located on a transverse element of the slab and when the roadway width is less than the recommended width of 18 ft, the concept of overlapped effective width was considered. The overlap meant that there was missing width in the middle of the roadway due to lane widths less than the recommended length; thus the effective width was overlapped. Equation 15 below shows how the effective width is calculated in a case of an overlap:

$$\boldsymbol{E_I} = \frac{1}{2}\,(E_O + C_r) \qquad \text{(Eq -15)}$$

Where

C_r = The distance between centers of wheels of two adjacent trucks

2.3 1935 Design Methods

According to AASHO (1935) all of the concepts explained in section 2.1 applied to this era. Allowable stress design was the method of design; however, the loading used for design and the effective width formula was defined differently. Below these concepts are explained.

2.3.1 Loadings

In 1935, different loadings applied to different classes of bridges. For any given class, the loading was applied to produce the maximum effect on the member considered. The loadings for class AA and class A were H20 and H15 respectively. The truck loading was used for spans less than 60 ft (Figure 2-4), and truck lane loadings were used for spans greater than 60 ft (Figure 2-6).

2.3.2 Effective Width

In 1935, the effective width formula was defined using the equation below:

$$\boldsymbol{E} = 0.6\,S + 2W \tag{Eq -16}$$

Where

E = effective width of slab in ft for one wheel Load.
S = length in ft of the portion of the span loaded to produce maximum live load effect in the member considered
W = width of the tire with a maximum value if 1.25 ft.

2.4 1936 Design Methods

All concepts defined in Section 2.2 applied to this era with the exceptions discussed below related to impact and loading in combination with effective widths as defined by GA DOT (1936).

2.4.1 Impact

The impact coefficient was defined in the *Standard Specification for Construction of Roads and Bridges for Georgia State Highway Department* (1936). The main formula was the same as the one in Equation 12; however, it had an exception for spans less than 45 ft. For such small spans, the impact coefficient was 0.3. (GA DOT, 1936)

2.4.2 Effective Width and Loading

An alternative method was described by GA DOT (1936). This method was for a case of a single load at the center of the span. Equation 17 was used to define the effective width for the outer zone:

$$E_O = 0.6S + 2W \tag{Eq -17}$$

Where

$E_{I\text{ or }O}$ = Effective width in ft for one wheel in the inner zone or the outer zone as defined in the subscript.
S = Center to center span of slab in ft
W = Width of tire with a maximum value of 1.25 ft.

And in case of loadings on parallel elements of a slab, the maximum bending moment shall be calculated as shown in Section 2.2 and increased by the following percentage shown below:

$$\frac{B}{S} = 0 \;\; 100\ \% \tag{Eq -18-1}$$

$$\frac{B}{S} = 0.1 \;\; 60\ \% \tag{Eq -18-2}$$

$$\frac{B}{S} = 0.4 \;\; 30\ \% \tag{Eq -18-3}$$

$$\frac{B}{S} = 1.0 \;\; 10\ \% \tag{Eq -18-4}$$

$$\frac{B}{S} = 1.4 \;\; 0\ \% \tag{Eq -18-5}$$

Where

B = Distance between the parallel loaded elements

In cases the ratio of B/S is in between the above values, intermediate values of B/S were obtained by interpolating the values above.

GA DOT (1936) defined the inner zone width as shown below:

$$E_I = \frac{1}{2}\left(E_O + C_r \right) \tag{Eq -19}$$

Where

Cr = Distance between the center of exterior wheels of two adjacent trucks. This value was defined as 3 feet.

2.5 1937 Design Methods

The design methodology used in 1937 was the allowable stress design. There was no different concept reported from this time. However, a document from this year by Hool (1937) showed that in the early 1900s the engineers believed that because the modulus of elasticity of a material is the ratio of stress to deformation, it followed that, for equal deformations, the stresses in the steel and concrete were as their moduli of elasticity, thus:

$$\frac{f_s}{f_c} = \frac{E_s}{E_C} = n \qquad \text{(Eq -20)}$$

And,

$$f_s = nf_c \qquad \text{(Eq -21)}$$

2.6 1941 Design Methods

The general method of design was still allowable stress design at this time. Some of the differences from the concepts introduced in the early 1900s were the steel allowable stress, the loadings, and the effective width. Below these concepts are explained.

2.6.1 Material Properties:

The allowable steel stress changed from 16,000 psi to 18,000 psi in tension for flexural members. (AASHO, 1941)

2.6.2 Loadings:

In computing the maximum load effect due to either truck loading or lane loading, each 10 foot traffic lane or a single standard truck was considered as a unit. The loading configuration was so that it would produce the maximum load effect on a member. H-S lane-loading was used for spans larger than 40 ft and H-S truck loading was considered for spans less than 40 ft. For H loading, either lane-loading or truck loading was used for design depending on which one caused the maximum effect. The lane-loading and truck loading configurations are shown in Figures 2-8, 2-9, 2-10, and 2-11.

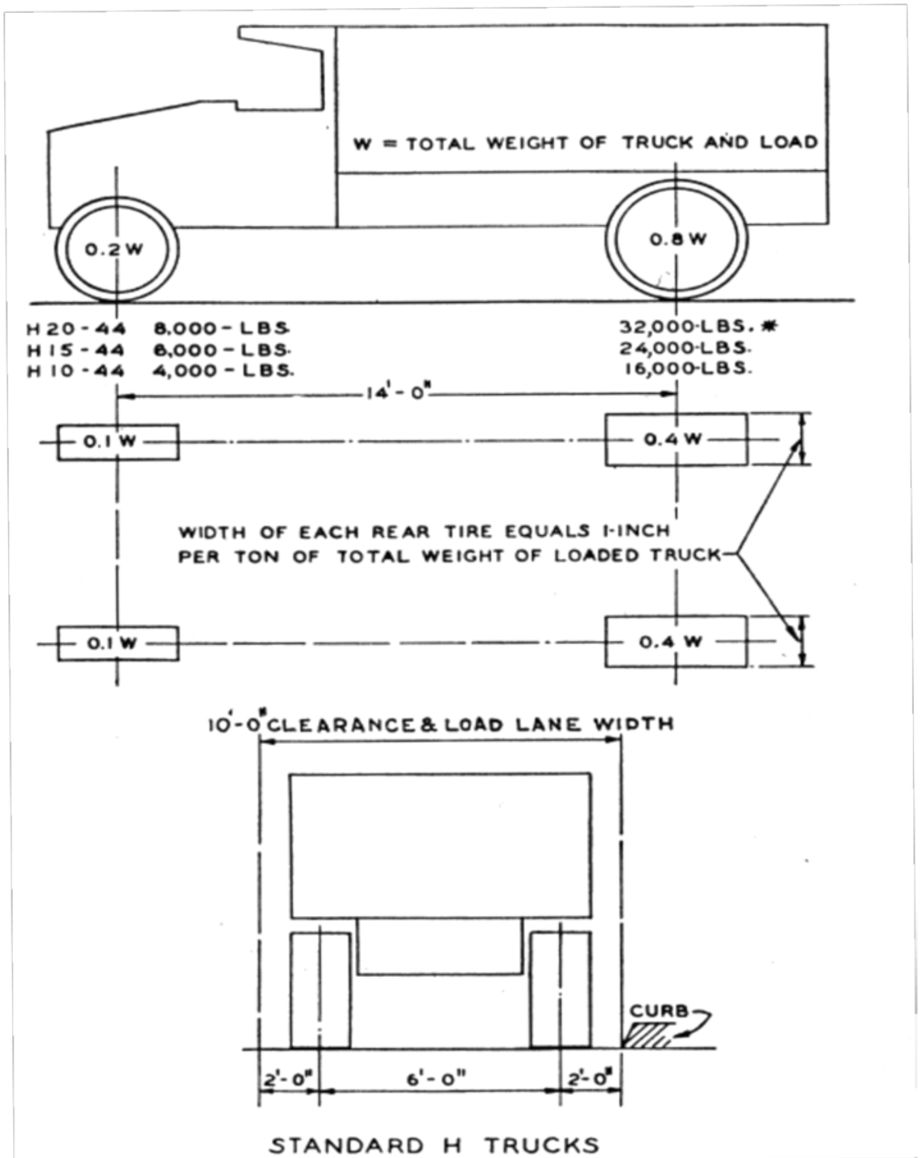


Figure 2-8: Standard H Truck Loading Configuration (AASHO, 1941)

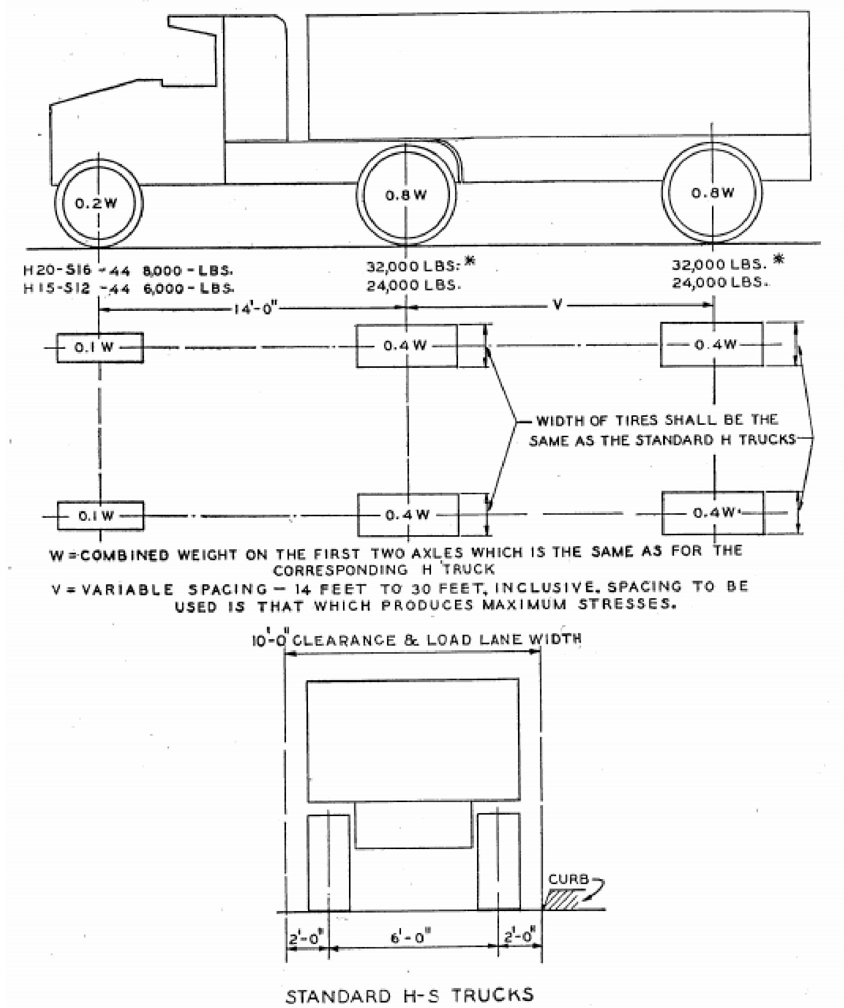


Figure 2-9: Standard H-S Truck Loading Configuration (AASHO, 1941)

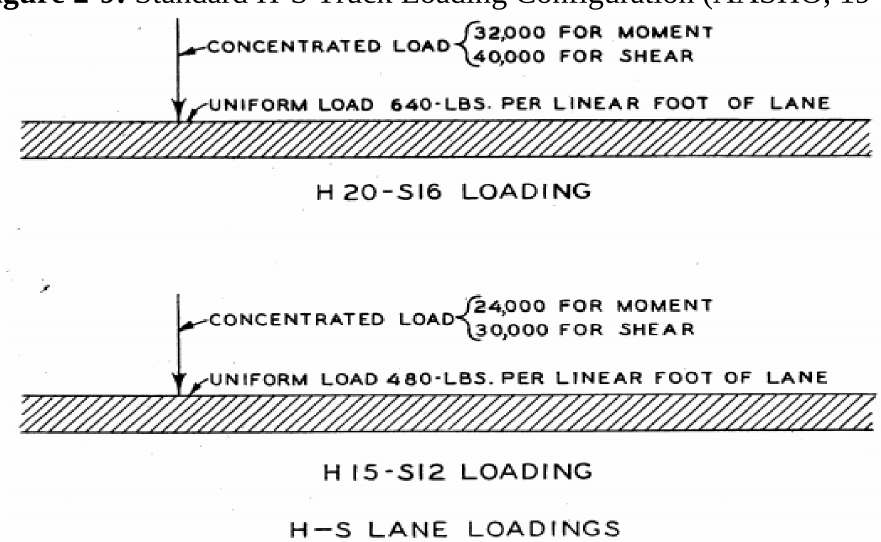


Figure 2-10: Standard H-S Lane-loading Configuration (AASHO, 1941)

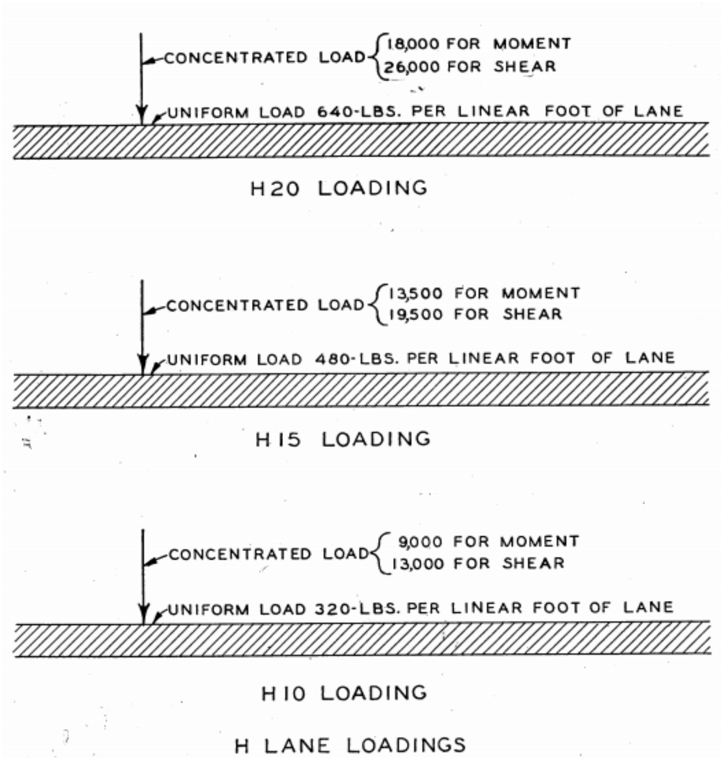


Figure 2-11: Standard H Lane-loading Configuration (AASHO, 1941)

2.6.3 Load Intensity Reduction

The maximum load effects were produced in any member by loading any number of traffic lanes simultaneously, the following percentages (Table 2-2) of the resultant live load was used in lieu of improbable coincident maximum loadings.

Table 2-2: Reduction in Load Intensity that Corresponds to the Number of Traffic Lanes

Number of Lanes	**Percentage**
One or two lanes	100
Three lanes	90
Four lanes or more	75

2.6.4 Effective Width:

According to AASHO (1941) for H loading and spans over 12 ft the effective width for reinforcement parallel to traffic was calculated using two cases (a) and (b) below; whichever provided the more conservative results was applied in design. Below these formulas are explained:

(a) Wheel Load

$$E = \frac{10N + Wr}{4N} \qquad \text{(Eq -22)}$$

(b) Lane-loading

$$\text{Momnet due to Uniform Load} = \frac{NQ}{0.5W + 5N} \text{per square foot slab} \qquad \text{(Eq -23)}$$

$$\text{Momnet due to Concentrated load} = \frac{NP'}{0.5Wr + 5N} \text{ per foot width of slab} \qquad \text{(Eq -24)}$$

Where

E = Width of slab over which a wheel load is distributed
N = Maximum number of lanes of traffic permissible on bridge
Wr = Width of roadway between curbs on bridge
Q = Uniform lane load per linear foot of lane
W = Width of graded roadway across culverts
S = For simple spans the span length shall be the distance center to center of supports but not to exceed clear span plus thickness of slab.
P = Load on one wheel
P' = Concentrated lane load per lane

In Case b, the values obtained from uniform load and concentrated load were added, and the final value would be the total live load moment in kips per foot of width. For main reinforcement parallel to traffic designed for H-S loading, for spans more than 12 ft and up to and including 40 ft truck loading was used. Lane-loading was used for lengths over 40 ft.

2.7 1949 Design Methods

In this era, the allowable stress design method was used to design a reinforced concrete slab. The differences in defining some of the concepts between early 1900s and 1949 were in the material properties, loadings, and effective width calculations. Below these concepts are reviewed.

2.7.1 Material Properties

In this era, the reinforcing steel had the same properties as that specified in 1941. The value of allowable steel tensile stress was 18,000 psi. (AASHO 1949) Also by this time the modular ratio was calculated independent of deformations using only the following relationship (Jensen, 1943):

$$n = \frac{E_S}{E_C} \quad \text{(Eq -25)}$$

In 1960, Pauw (1960) also used this relationship in his article: S*tatic modulus of elasticity of concrete as affected by density.*

2.7.2 Loadings

According to AASHO (1949) the type of loading used for design was the kind that causes the maximum load effect on the member considered, whether it was for continuous spans or simple spans. At this time there were no restrictions due to the length of the spans directly; however, an appendix was provided for simple spans with lengths varying form 1 ft to 300 ft with their corresponding moment and shear values for design. Figure 2-12 shows the H and HS lane-loading configuration which were different than the previous years. Figures 2-13 through 2-16 show the maximum moment and shear values listed for all span lengths.

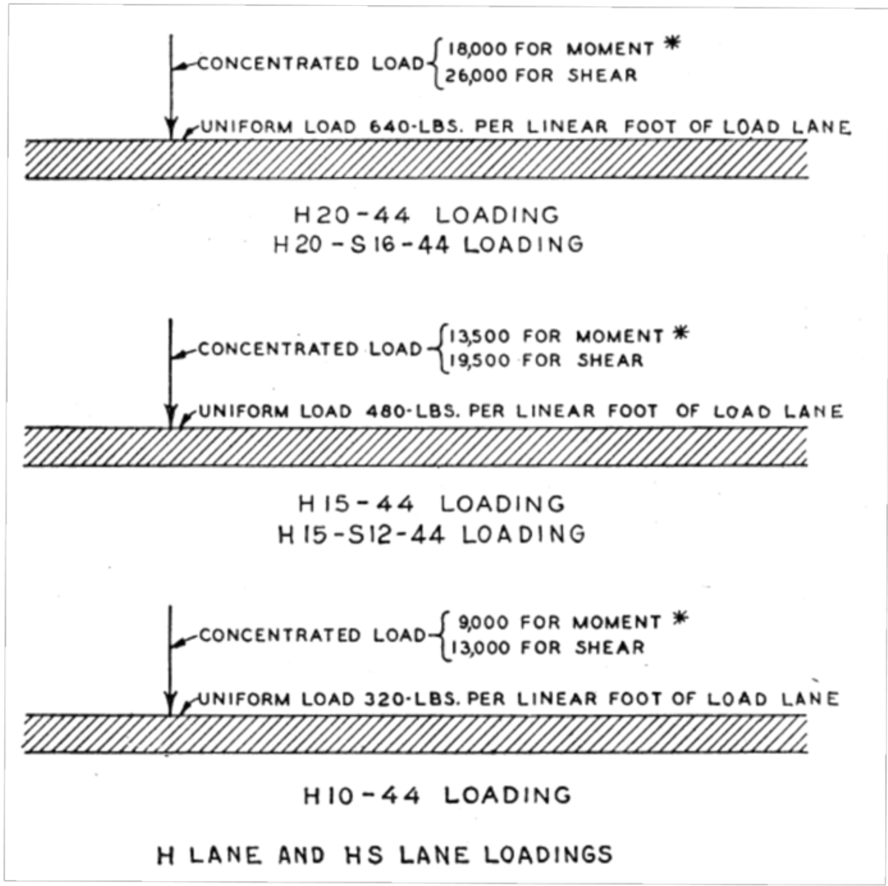


Figure 2-12: H and HS Lane-loading Configuration (AASHO, 1949)

Loading—H 15-44

Table of Maximum Moments, Shears and Reactions—Simple Spans, One Lane

Spans in feet; moments in thousands of foot-pounds; shears and reactions in thousands of pounds.

These values are subject to specification reduction for loading of multiple lanes.

Impact not included.

Span	Moment	End shear and end reaction (a)	Span	Moment	End shear and end reaction (a)
1	6.0 (b)	24.0 (b)	42	274.4 (b)	29.6
2	12.0 (b)	24.0 (b)	44	289.3 (b)	30.1
3	18.0 (b)	24.0 (b)	46	304.3 (b)	30.5
4	24.0 (b)	24.0 (b)	48	319.2 (b)	31.0
5	30.0 (b)	24.0 (b)	50	334.2 (b)	31.5
6	36.0 (b)	24.0 (b)	52	349.1 (b)	32.0
7	42.0 (b)	24.0 (b)	54	364.1 (b)	32.5
8	48.0 (b)	24.0 (b)	56	379.1 (b)	32.9
9	54.0 (b)	24.0 (b)	58	397.6	33.4
10	60.0 (b)	24.0 (b)	60	418.5	33.9
11	66.0 (b)	24.0 (b)	62	439.9	34.4
12	72.0 (b)	24.0 (b)	64	461.8	34.9
13	78.0 (b)	24.0 (b)	66	484.1	35.3
14	84.0 (b)	24.0 (b)	68	506.9	35.8
15	90.0 (b)	24.4 (b)	70	530.3	36.3
16	96.0 (b)	24.8 (b)	75	590.6	37.5
17	102.0 (b)	25.1 (b)	80	654.0	38.7
18	108.0 (b)	25.3 (b)	85	720.4	39.9
19	114.0 (b)	25.6 (b)	90	789.8	41.1
20	120.0 (b)	25.8 (b)	95	862.1	42.3
21	126.0 (b)	26.0 (b)	100	937.5	43.5
22	132.0 (b)	26.2 (b)	110	1,097.3	45.9
23	138.0 (b)	26.3 (b)	120	1,269.0	48.3
24	144.0 (b)	26.5 (b)	130	1,452.8	50.7
25	150.0 (b)	26.6 (b)	140	1,648.5	53.1
26	156.0 (b)	26.8 (b)	150	1,856.3	55.5
27	162.7 (b)	26.9 (b)	160	2,076.0	57.9
28	170.1 (b)	27.0 (b)	170	2,307.8	60.3
29	177.5 (b)	27.1 (b)	180	2,551.5	62.7
30	185.0 (b)	27.2 (b)	190	2,807.3	65.1
31	192.4 (b)	27.3 (b)	200	3,075.0	67.5
32	199.8 (b)	27.4 (b)	220	3,646.5	72.3
33	207.3 (b)	27.5 (b)	240	4,266.0	77.1
34	214.7 (b)	27.7	260	4,933.5	81.9
35	222.2 (b)	27.9	280	5,649.0	86.7
36	229.6 (b)	28.1	300	6,412.5	91.5
37	237.1 (b)	28.4			
38	244.5 (b)	28.6			
39	252.0 (b)	28.9			
40	259.5 (b)	29.1			

(a) Concentrated load is considered placed at the support. Loads used are those stipulated for shear.

(b) Maximum value determined by Standard Truck Loading.

Otherwise the Standard Lane Loading governs.

Figure 2-13: List of Maximum Moment and Shear Values for H 15-44 for Simple Spans (AASHO, 1949)

Loading—H 20-44

Table of Maximum Moments, Shears and Reactions—Simple Spans, One Lane

Spans in feet; moments in thousands of foot-pounds; shears and reactions in thousands of pounds.

These values are subject to specification reduction for loading of multiple lanes.

Impact not included.

Span	Moment	End shear and end reaction (a)
1	8.0 (b)	32.0 (b)
2	16.0 (b)	32.0 (b)
3	24.0 (b)	32.0 (b)
4	32.0 (b)	32.0 (b)
5	40.0 (b)	32.0 (b)
6	48.0 (b)	32.0 (b)
7	56.0 (b)	32.0 (b)
8	64.0 (b)	32.0 (b)
9	72.0 (b)	32.0 (b)
10	80.0 (b)	32.0 (b)
11	88.0 (b)	32.0 (b)
12	96.0 (b)	32.0 (b)
13	104.0 (b)	32.0 (b)
14	112.0 (b)	32.0 (b)
15	120.0 (b)	32.5 (b)
16	128.0 (b)	33.0 (b)
17	136.0 (b)	33.4 (b)
18	144.0 (b)	33.8 (b)
19	152.0 (b)	34.1 (b)
20	160.0 (b)	34.4 (b)
21	168.0 (b)	34.7 (b)
22	176.0 (b)	34.9 (b)
23	184.0 (b)	35.1 (b)
24	192.0 (b)	35.3 (b)
25	200.0 (b)	35.5 (b)
26	208.0 (b)	35.7 (b)
27	216.9 (b)	35.9 (b)
28	226.8 (b)	36.0 (b)
29	236.7 (b)	36.1 (b)
30	246.6 (b)	36.3 (b)
31	256.5 (b)	36.4 (b)
32	266.5 (b)	36.5 (b)
33	276.4 (b)	36.6 (b)
34	286.3 (b)	36.9
35	296.2 (b)	37.2
36	306.2 (b)	37.5
37	316.1 (b)	37.8
38	326.1 (b)	38.2
39	336.0 (b)	38.5
40	346.0 (b)	38.8
42	365.9 (b)	39.4
44	385.8 (b)	40.1
46	405.7 (b)	40.7
48	425.6 (b)	41.4
50	445.6 (b)	42.0
52	465.5 (b)	42.6
54	485.5 (b)	43.3
56	505.4 (b)	43.9
58	530.1	44.6
60	558.0	45.2
62	586.5	45.8
64	615.7	46.5
66	645.5	47.1
68	675.9	47.8
70	707.0	48.4
75	787.5	50.0
80	872.0	51.6
85	960.5	53.2
90	1,053.0	54.8
95	1,149.5	56.4
100	1,250.0	58.0
110	1,463.0	61.2
120	1,692.0	64.4
130	1,937.0	67.6
140	2,198.0	70.8
150	2,475.0	74.0
160	2,768.0	77.2
170	3,077.0	80.4
180	3,402.0	83.6
190	3,743.0	86.8
200	4,100.0	90.0
220	4,862.0	96.4
240	5,688.0	102.8
260	6,578.0	109.2
280	7,532.0	115.6
300	8,550.0	122.0

(a) Concentrated load is considered placed at the support. Loads used are those stipulated for shear.

(b) Maximum value determined by Standard Truck Loading.

Otherwise the Standard Lane Loading governs.

Figure 2-14: List of Maximum Moment and Shear Values for H 20-44 for Simple Spans **(AASHO, 1949)**

Loading—H 15-S 12-44

Table of Maximum Moments, Shears and Reactions—Simple Spans, One Lane

Spans in feet; moments in thousands of foot-pounds; shears and reactions in thousands of pounds.

These values are subject to specification reduction for loading of multiple lanes.

Impact not included.

Span	Moment	End shear and end reaction (a)	Span	Moment	End shear and end reaction (a)
1	6.0 (b)	24.0 (b)	42	364.0 (b)	42.0 (b)
2	12.0 (b)	24.0 (b)	44	390.7 (b)	42.5 (b)
3	18.0 (b)	24.0 (b)	46	417.4 (b)	43.0 (b)
4	24.0 (b)	24.0 (b)	48	444.1 (b)	43.5 (b)
5	30.0 (b)	24.0 (b)	50	470.9 (b)	43.9 (b)
6	36.0 (b)	24.0 (b)	52	497.7 (b)	44.3 (b)
7	42.0 (b)	24.0 (b)	54	524.5 (b)	44.7 (b)
8	48.0 (b)	24.0 (b)	56	551.3 (b)	45.0 (b)
9	54.0 (b)	24.0 (b)	58	578.1 (b)	45.3 (b)
10	60.0 (b)	24.0 (b)	60	604.9 (b)	45.6 (b)
11	66.0 (b)	24.0 (b)	62	631.8 (b)	45.9 (b)
12	72.0 (b)	24.0 (b)	64	658.6 (b)	46.1 (b)
13	78.0 (b)	24.0 (b)	66	685.5 (b)	46.4 (b)
14	84.0 (b)	24.0 (b)	68	712.3 (b)	46.6 (b)
15	90.0 (b)	25.6 (b)	70	739.2 (b)	46.8 (b)
16	96.0 (b)	27.0 (b)	75	806.3 (b)	47.3 (b)
17	102.0 (b)	28.2 (b)	80	873.7 (b)	47.7 (b)
18	108.0 (b)	29.3 (b)	85	941.0 (b)	48.1 (b)
19	114.0 (b)	30.3 (b)	90	1,008.3 (b)	48.4 (b)
20	120.0 (b)	31.2 (b)	95	1,074.9 (b)	48.7 (b)
21	126.0 (b)	32.0 (b)	100	1,143.0 (b)	49.0 (b)
22	132.0 (b)	32.7 (b)	110	1,277.7 (b)	49.4 (b)
23	138.0 (b)	33.4 (b)	120	1,412.5 (b)	49.8 (b)
24	144.5 (b)	34.0 (b)	130	1,547.3 (b)	50.7
25	155.5 (b)	34.6 (b)	140	1,682.1 (b)	53.1
26	166.6 (b)	35.1 (b)	150	1,856.3	55.5
27	177.8 (b)	35.6 (b)	160	2,076.0	57.9
28	189.0 (b)	36.0 (b)	170	2,307.8	60.3
29	200.3 (b)	36.6 (b)	180	2,551.5	62.7
30	211.6 (b)	37.2 (b)	190	2,807.3	65.1
31	223.0 (b)	37.7 (b)	200	3,075.0	67.5
32	234.4 (b)	38.3 (b)	220	3,646.5	72.3
33	245.8 (b)	38.7 (b)	240	4,266.0	77.1
34	257.7 (b)	39.2 (b)	260	4,933.5	81.9
35	270.9 (b)	39.6 (b)	280	5,649.0	86.7
36	284.2 (b)	40.0 (b)	300	6,412.5	91.5
37	297.5 (b)	40.4 (b)			
38	310.7 (b)	40.7 (b)			
39	324.0 (b)	41.1 (b)			
40	337.4 (b)	41.4 (b)			

(a) Concentrated load is considered placed at the support. Loads used are those stipulated for shear.

(b) Maximum value determined by Standard Truck Loading (one H-S truck). Otherwise the Standard Lane Loading governs.

Figure 2-15: List of Maximum Moment and Shear Values for H 15-S 12-44 for Simple Spans (AASHO, 1949)

Loading—H 20-S 16-44

Table of Maximum Moments, Shears and Reactions—Simple Spans, One Lane

Spans in feet; moments in thousands of foot-pounds; shears and reactions in thousands of pounds.

These values are subject to specification reduction for loading of multiple lanes.

Impact not included.

Span	Moment	End shear and end reaction (a)	Span	Moment	End shear and end reaction (a)
1	8.0 (b)	32.0 (b)	42	485.3 (b)	56.0 (b)
2	16.0 (b)	32.0 (b)	44	520.9 (b)	56.7 (b)
3	24.0 (b)	32.0 (b)	46	556.5 (b)	57.3 (b)
4	32.0 (b)	32.0 (b)	48	592.1 (b)	58.0 (b)
5	40.0 (b)	32.0 (b)	50	627.9 (b)	58.5 (b)
6	48.0 (b)	32.0 (b)	52	663.6 (b)	59.1 (b)
7	56.0 (b)	32.0 (b)	54	699.3 (b)	59.6 (b)
8	64.0 (b)	32.0 (b)	56	735.1 (b)	60.0 (b)
9	72.0 (b)	32.0 (b)	58	770.8 (b)	60.4 (b)
10	80.0 (b)	32.0 (b)	60	806.5 (b)	60.8 (b)
11	88.0 (b)	32.0 (b)	62	842.4 (b)	61.2 (b)
12	96.0 (b)	32.0 (b)	64	878.1 (b)	61.5 (b)
13	104.0 (b)	32.0 (b)	66	914.0 (b)	61.9 (b)
14	112.0 (b)	32.0 (b)	68	949.7 (b)	62.1 (b)
15	120.0 (b)	34.1 (b)	70	985.6 (b)	62.4 (b)
16	128.0 (b)	36.0 (b)	75	1,075.1 (b)	63.1 (b)
17	136.0 (b)	37.7 (b)	80	1,164.9 (b)	63.6 (b)
18	144.0 (b)	39.1 (b)	85	1,254.7 (b)	64.1 (b)
19	152.0 (b)	40.4 (b)	90	1,344.4 (b)	64.5 (b)
20	160.0 (b)	41.6 (b)	95	1,433.2 (b)	64.9 (b)
21	168.0 (b)	42.7 (b)	100	1,524.0 (b)	65.3 (b)
22	176.0 (b)	43.6 (b)	110	1,703.6 (b)	65.9 (b)
23	184.0 (b)	44.5 (b)	120	1,883.3 (b)	66.4 (b)
24	192.7 (b)	45.3 (b)	130	2,063.1 (b)	67.6
25	207.4 (b)	46.1 (b)	140	2,242.8 (b)	70.8
26	222.2 (b)	46.8 (b)	150	2,475.1	74.0
27	237.0 (b)	47.4 (b)	160	2,768.0	77.2
28	252.0 (b)	48.0 (b)	170	3,077.1	80.4
29	267.0 (b)	48.8 (b)	180	3,402.0	83.6
30	282.1 (b)	49.6 (b)	190	3,743.1	86.8
31	297.3 (b)	50.3 (b)	200	4,100.0	90.0
32	312.5 (b)	51.0 (b)	220	4,862.0	96.4
33	327.8 (b)	51.6 (b)	240	5,688.0	102.8
34	343.5 (b)	52.2 (b)	260	6,578.0	109.2
35	361.2 (b)	52.8 (b)	280	7,532.0	115.6
36	378.9 (b)	53.3 (b)	300	8,550.0	122.0
37	396.6 (b)	53.8 (b)			
38	414.3 (b)	54.3 (b)			
39	432.1 (b)	54.8 (b)			
40	449.8 (b)	55.2 (b)			

(a) Concentrated load is considered placed at the support. Loads used are those stipulated for shear.

(b) Maximum value determined by Standard Truck Loading (one H-S truck). Otherwise the Standard Lane Loading governs.

Figure 2-16: List of Maximum Moment and Shear Values for H 20-S 16-44 for Simple Spans (AASHO, 1949)

2.7.3 Load Intensity Reduction

In 1949, the load intensity reduction followed the same concepts developed in 1941.

2.7.4 Number of Traffic Lanes

The criterion in defining the number of traffic lanes was different. The lane-loading or standard truck was assumed to occupy a width of 10 ft. These loads were placed in design traffic lanes having a width of:

$$W_n = \frac{W_c}{N} \tag{Eq -26}$$

Where:

W_n = Width of design traffic lane
W_C =Roadway width between curbs exclusive of median strip
N = number of design traffic lanes as shown in the following table

Table 2-3 shows the number traffic lanes for increasing increments of roadway width.

Table 2-3: Number of Traffic Lanes for Different Roadway Widths

W_C (ft)	No. of Design Lanes
20 to 30 inc.	2
Over 30 to 42 inc.	3
Over 42 to 54 inc.	4
Over 54 to 66 inc.	5
Over 66 to 78 inc.	6
Over 78 to 90 inc.	7
Over 90 to 102 inc.	8
Over 102 to 114 inc.	9
Over 114 to 126 inc.	10

2.7.5 Effective Width

The effective width defined at this time was different than how engineers defined in early 1900s. According to AASHO (1949) the effective width was the same as that defined in 1941 in Section 2.4.3.

2.8 1957 Design Methods

In 1957 allowable stress design was used for design, and the all the concepts used in defining the cross section capacity were the same as in the early 1900s. However, the material properties, loadings, load intensity reduction, number of traffic lanes, and effective width were defined differently.

2.8.1 Material properties

The allowable tensile stress value for steel at this time was considered to be 18,000 psi. (AASHO, 1957)

2.8.2 Loadings

The loading configurations defined in AASHO (1957) were the same as those previously described in AASHO (1949). At this time there were also a series of tables used to define maximum moment and shear values for simple spans varying from 1 ft to 300 ft. These values are the same as those shown in Figure 2-10 through 2-13.

2.8.3 Load Intensity Reduction

In 1957, the load intensity reduction followed the same concepts used in 1941. (AASHO, 1957)

2.8.4 Number of Traffic Lanes

The criterion for defining the number of traffic lanes was different than how it was defined in early 1900s. The concepts explained from 1949 are the basis for defining the number of traffic lane in 1957. (AASHO, 1957)

2.8.5 Effective Width

The effective width in 1957 was defined similarly to the effective width defined by AASHO in 1941.

2.9 1961 Design Methods

The design method considered at this time was also allowable stress design. Similarly to other time frames, some parameters were defined differently than the early 1900s. Of these different parameters are allowable steel stress, loadings, number of traffic lanes, and effective width. These concepts are explained below.

2.9.1 Material Properties

In this era, the reinforcing steel had the same properties as that specified in 1941. The value of allowable tensile stress was 18,000 psi. (AASHO, 1961)

2.9.2 Loadings

The loadings at this time were the same as those used in 1949. (AASHO, 1961)

2.9.3 Number of Traffic Lanes

The number of traffic lanes on a roadway were defined the same as in 1949. (AASHO, 1961)

2.9.4 Effective Width

The effective was defined differently during this time. The formula used for effective width is the same as what is used in AASHTO (17^{th} ed., 2002). The effective slab width is a width which carries one wheel-line of loading. The formula to define the effective width is presented below.

$$E = 4 + 0.06S \leq 7 \text{ feet} \qquad \text{(Eq -27)}$$

Where:

E = Effective width with a maximum value of 7 ft. Lane loads are distributed over a width of 2E.
S = For simple spans the span length shall be the distance center to center of supports but not to exceed clear span plus thickness of slab.

For simple spans, the maximum live load moment per foot of width of slab, without impact, is closely approximated for two cases of H20 and H15 by the following formulas:

a) H20-S16 Loading:

For spans up to and including 50 ft:

$$LLM = 900S \text{ (foot-pound)} \quad \text{(Eq -28)}$$

For spans 50 ft to 100 ft:

$$LLM = 1000(1.30\,S - 20) \text{ (foot- pound)} \quad \text{(Eq -29)}$$

b) H15-S12 Loading:

Use ¾ of the values obtained from the Eq-28 and Eq-29 for H20-S16 loading.

$$LLM = 0.75(900\,S\,) \quad \text{(Eq -30)}$$

Moment in continuous spans was obtained by suitable analysis using the truck or appropriate lane-loading.

2.10 Current Design and Analysis Methods

2.10.1 AASHTO 17th Ed. Method

Today the resistance of reinforced concrete slabs is calculated using ultimate strength design principle. The maximum moment due to live load for the load factor rating (LFR) method is calculated according to the AASHTO (17^{th} ed., 2002). AASHTO (17^{th} ed., 2002) follows the process used since 1961 to calculate moment and the effective width as described by Equations 27 through Equation 30.

2.10.2 AASHTO LRFD Method

One last check was to consider the LRFD method in AASHTO (LRFD, 2014) and compare the results with other methods in an aim for better understanding of how the engineers in early 1900s calculated the capacity of a slab. AASHTO (LRFD, 2014) is based on limit state design philosophy. Only the calculation of effective width in accordance to AASHTO (LRFD, 2014) was of an interest in this research since the other concepts such as loading criterion do not apply to the older methods reviewed so far. Below the concept of effective width is explained.

$$E_{I-1} = 84.0 + 1.44\sqrt{\boldsymbol{L_1}} W_1 \leq E_{I-2}\ \frac{12.0\,W}{N} \qquad \text{(Eq -31)}$$

Where

W_1 = Modified edge to edge W equal or lesser than 60 ft for multiple lane, and 30 ft for one lane
W_p = Physical edge to edge W of bridge in feet
N = Number of design lanes as specified in Article 3.6.1.1.1. in AASHTO(LRFD, 2014)
L_{max} = 60 ft

All values in inches.

2.11 Shear and Development of Reinforcement

The shear strength and the development length in the slab were checked in accordance with AASHTO (17th ed., 2002). Below these requirements are explained.

2.11.1 Shear Check

The shear capacity of the slab is defined according to AASHTO (17th ed). In a case where there are no stirrups in the slab, the required factored shear capacity of the cross section is defined as:

$$\emptyset V_c = \emptyset 2\sqrt{f'_c}\, b_w d \qquad \text{(Eq -32)}$$

Where:

$\emptyset$ = Resistance factor for Shear, 0.75
V_c = Concrete shear strength
f'_c = Concrete compressive strength in psi
b_w = Cross section width
d = Effective depth

2.11.2 Development Length

According to AAHTO (17th ed., 2002) section 8.24.1. through 8.24.2.3. the following checks should be satisfied for the development length of simply supported spans. Section 8.24.1 applied to all reinforcement in the slab, and section 8.24.2 applied to the development length of positive moment reinforcement. Firstly, the general requirements state that:

> <u>8.24.2.1</u> The calculated tension or compression in the reinforcement at each section shall be developed on each side of that section by embedment length, hook or mechanical device, or a combination thereof. Hooks may be used in developing bars in tension only.

> <u>8.24.1.2</u> Critical sections for development of reinforcement in flexural member are at points of maximum stress and at points within the span where adjacent reinforcement terminates or is bent. The provisions of Article 8.24.2.3 must also be satisfied.

> <u>8.24.1.2.1</u> Reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance equal to the effective depth of the member, 15 bar diameters, or 1/20 of the clear span, whichever is greater, except at supports of simple spans and the free ends of cantilevers.

> <u>8.24.1.2.2</u> Continuing reinforcement shall have an embedment length not less than the development length l_d beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure

> <u>8.24.1.3</u> Tension reinforcement may be developed by bending across the web in which it lies or by making it continuous with the reinforcement on the opposite face of the member.

> <u>8.24.1.4</u> Flexural reinforcement within the portion of the member used to calculate the shear strength shall not be terminated in a tension zone unless one of the following conditions is satisfied:

8.24.1.4.1 Shear at the cutoff point does not exceed two-thirds of the permitted, including the shear strength of reinforcement provided.

8.24.1.4.2 Stirrups are in excess of that required for shear is provided along each terminated bar over a distance from the termination point equal to three-fourths the effective depth of the member. The excess stirrup area, A_v, shall not be less than $60b_ws/f_y$. Spacing, s, shall not exceed $d/(8\beta_b)$ where β_b is the ratio of the area of reinforcement cut off to the total area of tension reinforcement at the section.

8.24.1.4.3 For No. 11 bars and smaller, the continuing bars provide double the area required for flexure at the cutoff point and the shear does not exceed three-fourths that permitted.

Below the requirements from Section 8.24.2 of AASHTO (17^{th} ed., 2002) are listed. These checks applied to positive moment reinforcement in the slabs.

8.24.2.1 At least one-third of the positive moment reinforcement in simple members and one-fourth the positive moment reinforcement in continuous member shall extend along the same face of the member into the support.

8.24.2.3 At simple supports and at point of inflection, positive moment tension reinforcement shall be limited to a diameter such that l_d computed for f_y by Article 8.25 satisfies Equation (8-65), need not be satisfied for reinforcement terminating beyond center line of simple supports by a standard hook, or a mechanical anchorage at least equivalent to a standard hook.

$$l_d \leq \frac{M}{V} + l_a \qquad \text{(Eq -33)}$$

$$M = A_s f_y \left(d - \frac{a}{2}\right) \qquad \text{(Eq -34)}$$

$$a = \frac{A_s f_y}{0.85(f'_c b)} \qquad \text{(Eq -35)}$$

Where:

M = The computed moment capacity assuming all positive moment tension reinforcement at the section to be fully stressed.
V = Maximum shear force at the section.
l_a = Embedment length. l_a at the support shall be the embedment length beyond the center of the support.
b = The width of flexure compression zone

AASHTO (17th ed., 2002) Section 8.25.1 gives the following equations for the tension development length for bars No. 11 and smaller:

$$l_d = \frac{0.04\, A_b f_y}{\sqrt{f'c}} \qquad \text{(Eq -36)}$$

Where:

A_b = Area of the bar

Other parameters are defined above.

3 Bridge Design from 1920s and the Modern Rating Process

3.1 Introduction

In this chapter, the studies of bridges from the 1920s are presented as case studies. Design methods from 1910 until 1930 are referred to in thesis as the contemporary design, as these design concepts did not belong to one year or decade, and they are from the time period of the original design of the Barnes Slough Bridge. One case study is of standard simple span flat slab bridges from 1922. The geometry and reinforcement in the slabs is documented in a series of drawings provided by ALDOT. The other case study is a two-span continuous bridge from 1924 for which there are drawings. These case studies provided great examples of the outcome of the engineers' design in that era. After discussing these cases, the principles of modern rating of these structures are introduced using AASHTO LFD in addition to presenting the AASHTOWare software. This software is currently used by departments of transportation throughout the nation to rate bridges.

3.2 1922 Simple Span Bridges

A standard drawing provided by SHDA (1922) has tabulated values for the amount of reinforcement in simply supported spans with span lengths varying from 6 ft to 20 ft in 2 ft increments, and roadways widths of 16 ft, 18 ft, and 20 ft. The title block and the general notes from this drawing are shown in Figures 3-1 and 3-5 and Tables 3-1 through 3-2. With the information that was provided in the drawings the research focused on understanding how the amount of reinforcement required in the slab was calculated.

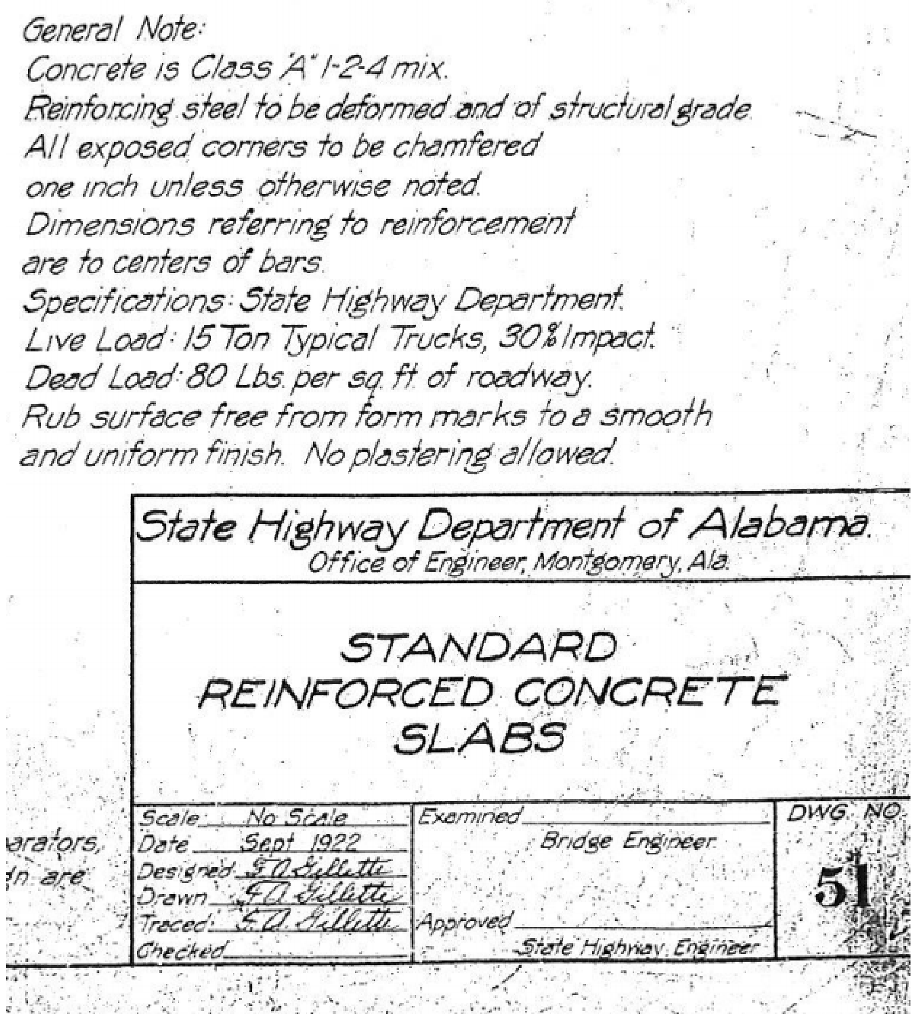


Figure 3-1: Title Block and General Notes for Standard Drawings of Simple Span Bridges (SHDA, 1922)

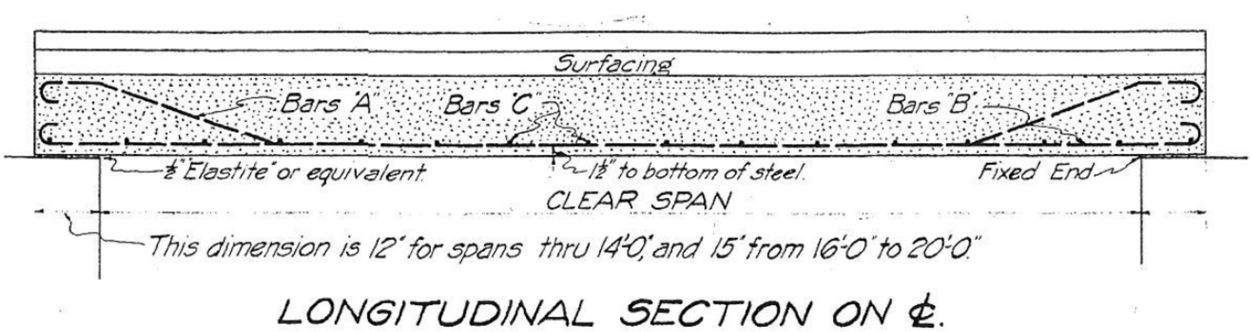


Figure 3-2: Details of Bars A, B, and C Configuration in a Section Cut for SHDA (1922)

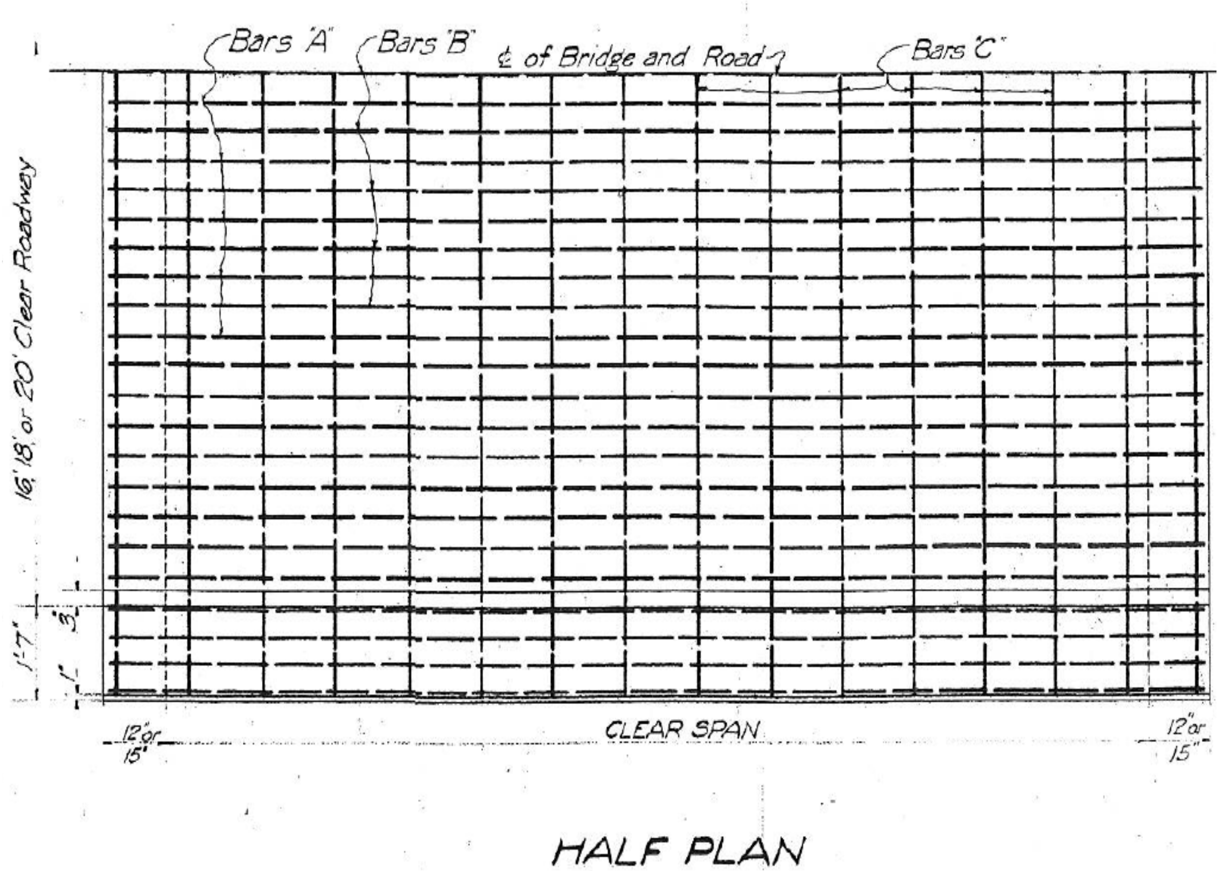


Figure 3-3: Bars A, B, and C Configuration in a Plan View for SHDA (1922)

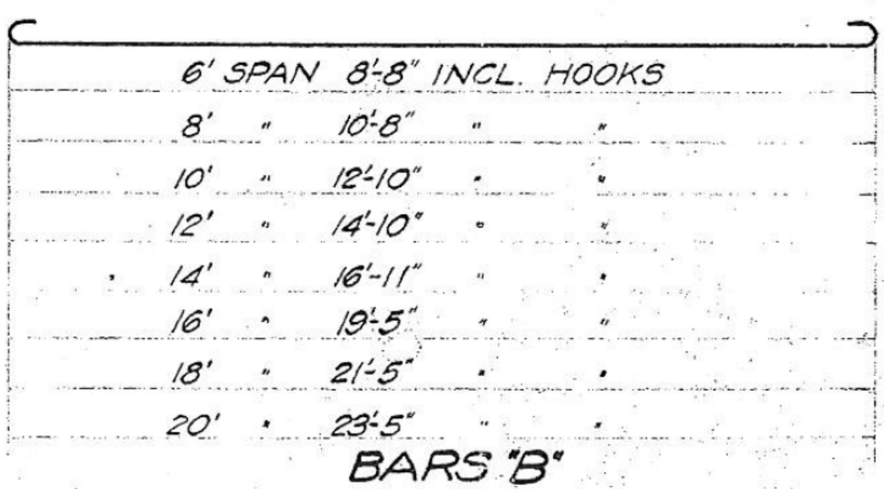


Figure 3-4: Bars "B" Dimension for Different Span Lengths for SHDA (1922)

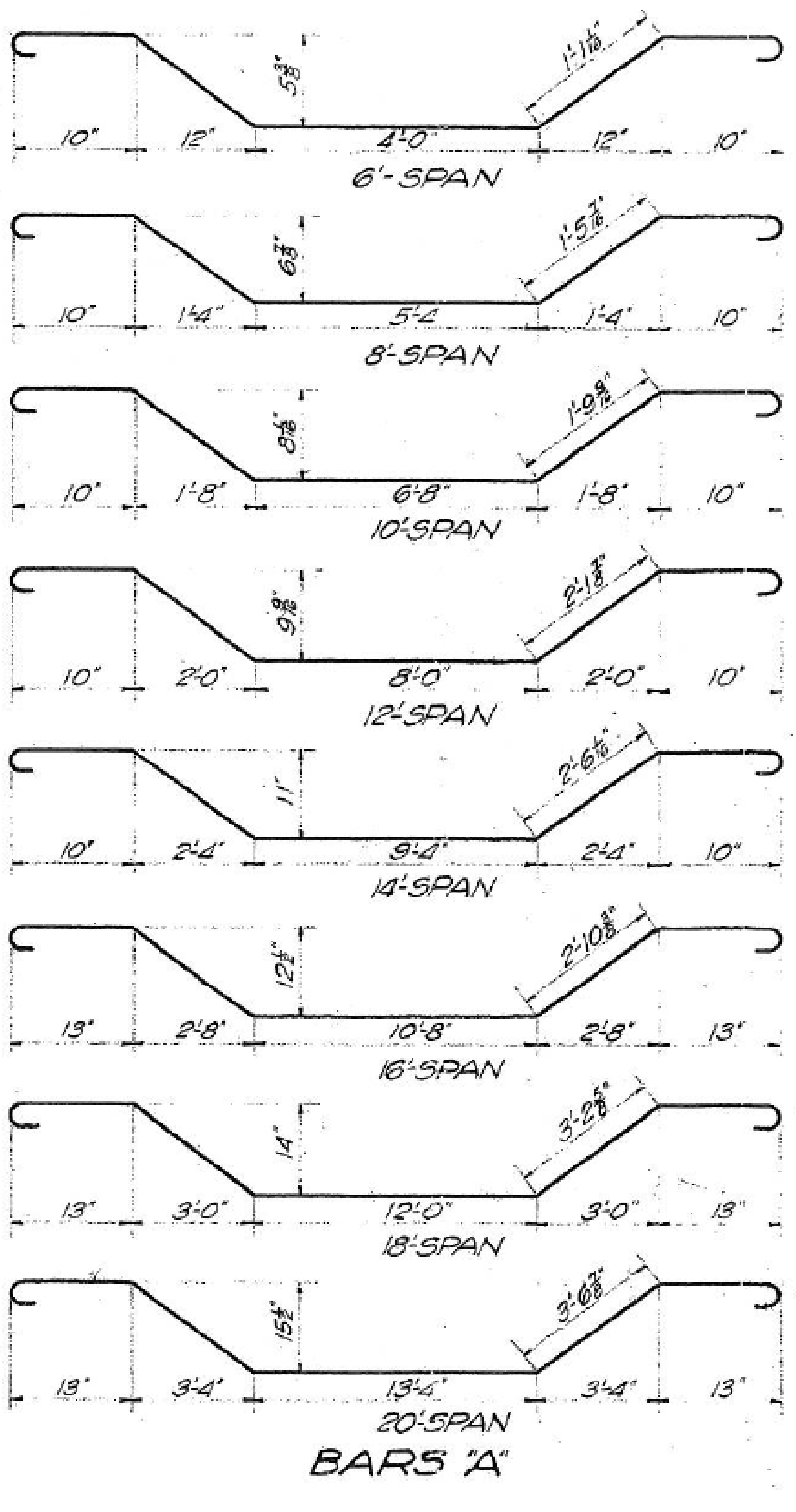


Figure 3-5: Bars "A" Dimensions for Different Span Length for SHDA (1922)

Table 3-1:1922 Standards for Size and Spacing of the Bars for Different Lengths of Spans (SHDA, 1922)

DESIGN DATA						
CLEAR SPAN	DIMENSIONS		BARS "A" & "B"		BARS "C"	
	"H" DEPTH OF SLAB	TO C.G. OF STEEL	SIZE	C. TO C.	SIZE	C. TO C.
6	8 5/8"	7 3/4"	3/4" φ	7"	1/2" □	18"
8	10 1/8"	9 1/4"	3/4" φ	6"	1/2" □	18"
10	11 7/16"	10 1/2"	7/8" φ	7"	1/2" □	18"
12	12 15/16"	12"	7/8" φ	6"	1/2" □	18"
14	14 1/2"	13 1/2"	1" φ	7 1/2"	1/2" □	18"
16	16"	15"	1" φ	6 1/2"	1/2" □	18"
18	17 1/2"	16 1/2"	1" φ	6"	1/2" □	18"
20	19"	18"	1" φ	5 1/2"	1/2" □	18"

Table 3-2: Number of Bars, Length, and Types of Bars for Different Lengths and Roadway Widths for SHDA (1922)

STEEL ESTIMATE, I-SLAB									
CLEAR SPAN	16'-ROADWAY			18'-ROADWAY			20'-ROADWAY		
	BARS	LENGTH	NO. REQUIRED	BARS	LENGTH	NO. REQUIRED	BARS	LENGTH	NO. REQUIRED
6	A	8'-10"	16	A	8'-10"	18	A	8'-10"	20
	B	8'-8"	17	B	8'-8"	19	B	8'-8"	21
	C	18'-9"	6	C	20'-9"	6	C	22'-9"	6
8	A	10'-10"	19	A	10'-10"	21	A	10'-10"	23
	B	10'-8"	20	B	10'-8"	22	B	10'-8"	24
	C	18'-9"	8	C	20'-9"	8	C	22'-9"	8
10	A	13'-3"	16	A	13'-3"	18	A	13'-3"	20
	B	12'-10"	17	B	12'-10"	19	B	12'-10"	21
	C	18'-9"	9	C	20'-9"	9	C	22'-9"	9
12	A	15'-3"	19	A	15'-3"	21	A	15'-3"	23
	B	14'-10"	20	B	14'-10"	22	B	14'-10"	24
	C	18'-9"	10	C	20'-9"	10	C	22'-9"	10
14	A	18'-5"	15	A	18'-5"	17	A	18'-5"	19
	B	16'-11"	16	B	16'-11	18	B	16'-11	20
	C	18'-9"	12	C	20'-9"	12	C	22'-9"	12
16	A	20'-2"	17	A	20'-2"	19	A	20'-2"	21
	B	19'-5	18	B	19'-5"	20	B	19'-5"	22
	C	18'-9"	13	C	20'-9"	13	C	22'-9"	13
18	A	22'-2"	19	A	22'-2"	21	A	22'-2"	23
	B	21'-5"	20	B	21'-5"	22	B	21'-5"	24
	C	18'-9"	15	C	20'-9"	15	C	22'-9"	15
20	A	24'-3"	20	A	24'-3"	22	A	24'-3"	24
	B	23'-5"	21	B	23'-5"	23	B	23'-5"	25
	C	18'-9"	16	C	20'-9"	16	C	22'-9"	16

3.2.1 Area of Tension Reinforcement Required

By applying the concepts that were introduced in Chapter 2 the required amount of reinforcement was calculated. The parameters used here to calculate the amount of reinforcement required are listed in Table 3-3. Figure 3-6 shows the design truck configuration on the span. The methods used to calculate the amount of reinforcement in the slab were based on the earliest methods learned from the contemporary time. These methods were defined by AASHO (1931) and Kirkham (1932). According to Kirkham (1932), although the calculations were done both for the inner zone and the outer zones, in practice the one that controlled the design was applied across the whole cross section. The uniform distribution of the reinforcement in the slab (Figure 3-3) confirmed that this concept was applied. The concept of inner zone explained by Kikham (1932) is the same as effective width used by other sources. In all cases the amount of reinforcement calculated for the inner zone controlled the design. For example calculations, refer to Appendix A.

Table 3-3: Summary of Parameters Used for the Analysis of 1922 Standard Simple Spans

Parameters	Value
Allowable Concrete Compressive Stress, fc	650 psi
Allowable Steel Tensile Stress, fs	16,000 psi
Modular Ratio	15
Reinforced Concrete Unit Weight	150 pcf
Live Load	H 15 Truck
Impact Allowance (Contemporary)	30%
Impact Allowance (OFOR Loading)	30%
Superimposed Dead Load (Contemporary)	80 psf
Superimposed Dead Load (OFOR Loading)	80 psf
Slab Thickness, H	19 in.
Depth to Tension Reinforcement, d	18 in.
Effective Width of Slab (Contemporary)	5 ft
Effective Width of Slab (OFOR Loading)	4 ft
Span Length – Clear span length	20 ft

In addition to studying the contemporary methods to calculate the amount of reinforcement in the slab, another method of loading was considered. This different loading was used to evaluate the effect of roadway width on the design. In this method the roadway was loaded by two trucks, 4 wheel-lines, and this load effect plus impact was divided by the roadway width to calculate the moment per foot of width. In this configuration the effective width is one-fourth of the roadway. This configuration is referred to as OFOR (One-fourth of the Roadway). Below the results for both methods are presented in Table 3-4.

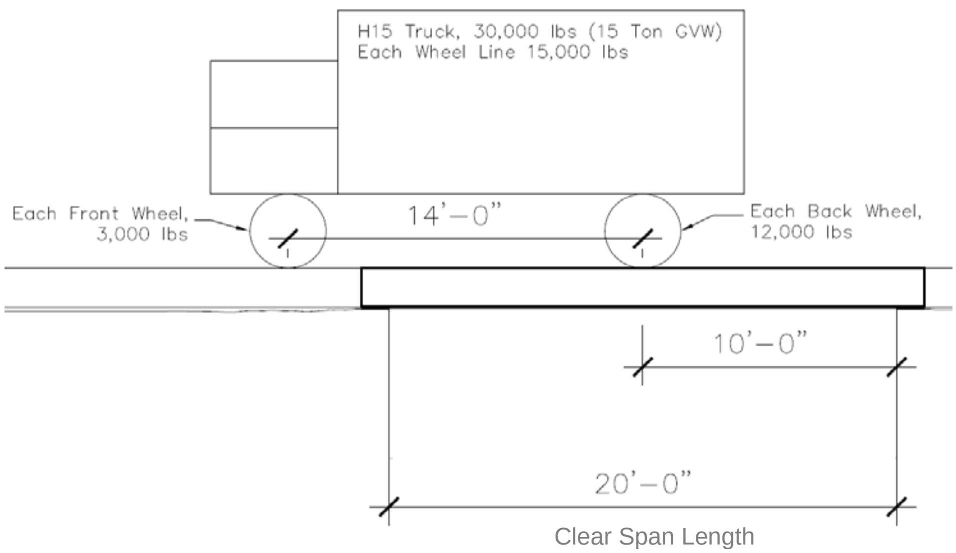


Figure 3-6: Diagram of 20-ft Simply Supported Span with One Wheel-Line of an H15 Truck Positioned To Cause Maximum Moment

A comparison made between the results from these two methods and the tabulated values from SHDA (1922) showed that the studies generated results that were similar to the amount of reinforcement listed in the drawings, but the results from these three methods did not follow the same pattern. These patterns increased and decreased differently for different span lengths and roadway widths. Only the results for a case of 20-ft span and 16-ft roadway width matched

SHDA (1922). These replicas suggested that our understanding of dead load and live load were sufficient; however, the process which was used to identify the effective width remained unknown.

Table 3-4: Required Area of Steel for 1922 Simple Spans (in^2/ft of Width)

Clear Span	**Method**	**Roadway Width**		
		16 ft	**18 ft**	**20 ft**
6 ft	Table 3-2	0.757	0.769	0.779
	Contemporary	0.707	0.707	0.707
	OFOR	0.742	0.669	0.612
8 ft	Table 3-2	0.895	0.894	0.893
	Contemporary	0.741	0.741	0.741
	OFOR	0.877	0.798	0.732
10 ft	Table 3-2	1.03	1.05	1.06
	Contemporary	0.865	0.865	0.87
	OFOR	1.025	0.936	0.87
12 ft	Table 3-2	1.22	1.22	1.22
	Contemporary	0.981	0.981	0.98
	OFOR	1.15	1.06	0.98
14 ft	Table 3-2	1.28	1.31	1.33
	Contemporary	1.10	1.10	1.10
	OFOR	1.27	1.18	1.10
16 ft	Table 3-2	1.44	1.46	1.47
	Contemporary	1.23	1.23	1.23
	OFOR	1.41	1.31	1.23
18 ft	Table 3-2	1.61	1.61	1.60
	Contemporary	1.36	1.36	1.36
	OFOR	1.54	1.44	1.36
20 ft	Table 3-2	1.69	1.68	1.67
	Contemporary	1.5	1.5	1.50
	OFOR	1.69	1.58	1.50

To isolate the value that was used as the effective width by the contemporary design, firstly, a series of fixed values were considered as effective width such as 3 ft, 4 ft, and 5 ft to calculate the amount of reinforcement in the slab. Secondly, another method by Georgia State Highway Department (Georgia State Highway Department - *Standard Specification for Construction of Roads and Bridges*, 1936) was used to define effective width in combination with additional loading due to parallel loadings on the same element. Equations 17, 18-2, and 18-

3 were used by GA DOT to identify the effective width and loading while the rest of the process is the same as contemporary method. However, these results did not follow the same pattern for all span lengths and roadway widths, and the three methods of Contemporary, OFOR loading, and GA DOT did not uncover how the engineers calculated the effective width.

3.2.2 Identification of Effective Width

The effective slab width is defined in AASHTO (17th ed., 2002) as the width of slab that resists one wheel-line of loading. Applying this definition, it is possible to use the amounts of reinforcement listed in Table 3-1 to back-calculate the effective width used by the original designers of the 1922 simple spans.

Using the definition above, the applied live load moment with impact per foot of width is equal to

$$\frac{M_{l+I}}{E} \qquad \text{(Eq -37)}$$

Where

M_{L+I} = Live load moment due to one wheel-Line of truck loading plus impact

E = Effective width in ft.

Assuming moment resistance per foot if width, M, is equal to the applied live load width impact plus the dead load per foot of width, M, results in:

$$M = \frac{M_{L+I}}{E} + M_D \qquad \text{(Eq -38)}$$

Where

M_D = Dead Load moment

By rearranging:

$$E = \frac{M_{L+I}}{M - M_D} \qquad \text{(Eq -39)}$$

Values of effective width were calculated for each combination of span length and roadway width listed in Table 3-2. The moment resistance was calculated using an elastic analysis of a cracked section with the parameters listed in Table 3-3. The live loading included truck loading plus impact, and the dead load included the weight of the slab plus superimposed dead load as described in Table 3-3.

The results of the effective width calculations are listed in Tables 3-5 through 3-7 and plotted in Figure 3-7. The values back-calculated using Equation 40 are shown under the heading "1922". Results are also shown for three other methods: AASHTO (17^{th} ed., 2002), LRFD method, and GA DOT (1936).

Table 3-5: Effective Width Values for a 16-ft Roadway

Span Length (ft)	**Effective Width for 16 ft Roadway (ft)**			
	1922	**GA DOT**	**AASHTO LRFD**	**AASHTO 17^{th} ed.**
6	4.17	4.82	4.09	4.36
8	4.13	5.28	4.18	4.48
10	4.30	5.00	4.26	4.60
12	4.20	4.83	4.33	4.72
14	4.14	4.71	4.40	4.84
16	4.10	4.63	4.46	4.96
18	4.09	4.57	4.52	5.08
20	4.11	4.52	4.57	5.20

Table 3-6: Effective Width Values for a 18-ft Roadway

Span Length (ft)	**Effective Width for 18 ft Roadway (ft)**			
	1922	**GA DOT**	**AASHTO LRFD**	**AASHTO 17^{th} ed.**
6	4.17	4.82	4.09	4.36
8	4.13	5.28	4.18	4.48
10	4.30	5.00	4.26	4.60
12	4.20	4.83	4.33	4.72
14	4.14	4.71	4.40	4.84
16	4.10	4.63	4.46	4.96
18	4.09	4.57	4.52	5.08
20	4.11	4.52	4.57	5.20

Table 3-7: Effective Width Values for a 20-ft Roadway

Span Length (ft)	Effective Width for 20 ft Roadway (ft)			
	1922	GA DOT	AASHTO LRFD	AASHTO 17th ed.
6	4.17	4.82	4.09	4.36
8	4.13	5.28	4.18	4.48
10	4.30	5.00	4.26	4.60
12	4.20	4.83	4.33	4.72
14	4.14	4.71	4.40	4.84
16	4.10	4.63	4.46	4.96
18	4.09	4.57	4.52	5.08
20	4.11	4.52	4.57	5.20

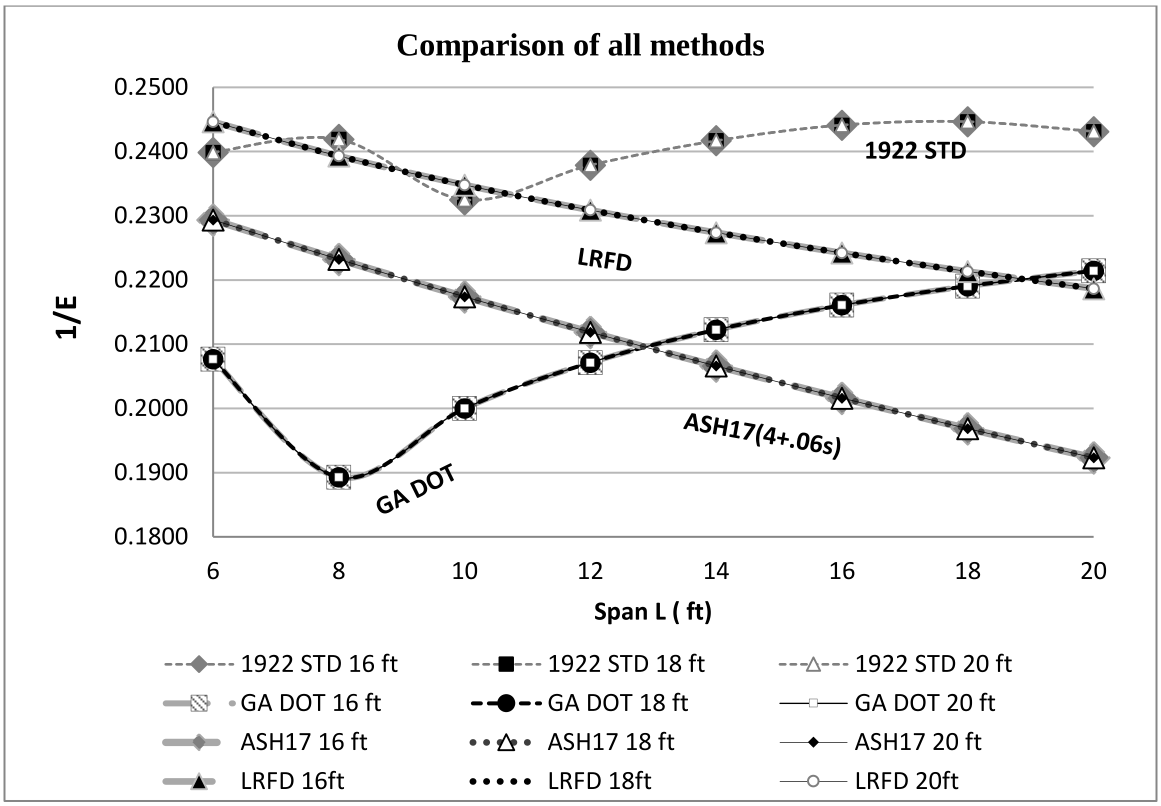


Figure 3-7: Graph of 1/E versus Span Length for Different Methods

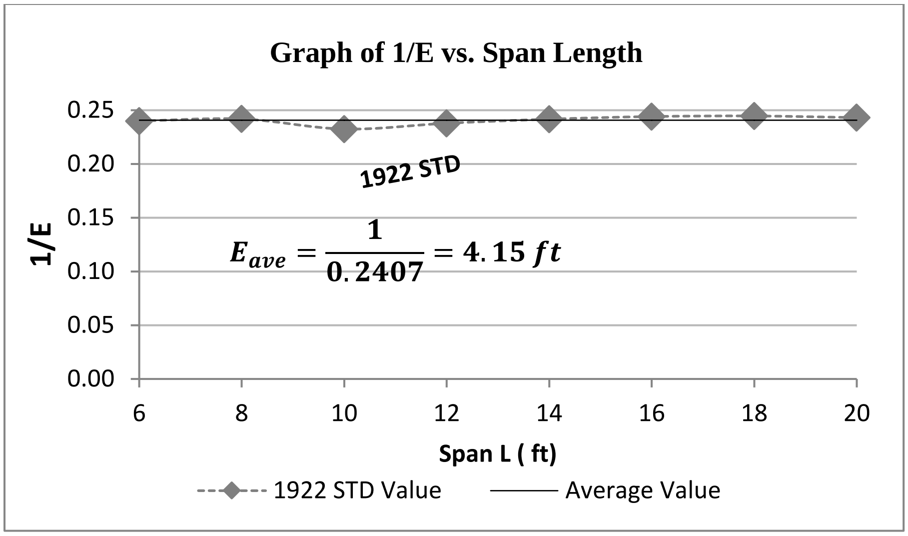


Figure 3-8: The Ratio of 1/E for Different Span Length and Roadway Widths

From the back-calculations it was concluded that the average effective width in these calculations was the value of 4.15 ft. When this effective width was applied to calculate the amount of steel in the slab, the results matched the values listed in Table 3-2. This study suggested that the effective width could possibly have been calculated by considering the geometry of the roadway. For a 16-ft roadway the maximum effective width, one-fourth of roadway for a two-lane roadway, would be 4 ft. Alternatively, the distance between two adjacent trucks was considered to be 3 ft, and the spacing between two wheel-lines was considered to be 6 ft. If an effective width was calculated as half the distance between two trucks plus half the distance between two wheel-lines, the effective width would be 4.5 ft. The value of 4 ft is a more conservative value; therefore, it is possible that they applied the more conservative value which corresponds to the number of truck's wheel-lines on the roadway.

3.3 1924 Two-Span Continuous Bridge in Fayette County

The same process learned from the contemporary method was used to calculate the reinforcement in the slab for a two-span continuous bridge of Fayette Co. documented in SHDA

(1924). From Figures 3-9, through Figure 3-11 it was determined that the area of steel per foot of width is 1.44 in^2/ft in the positive moment region and negative moment region.

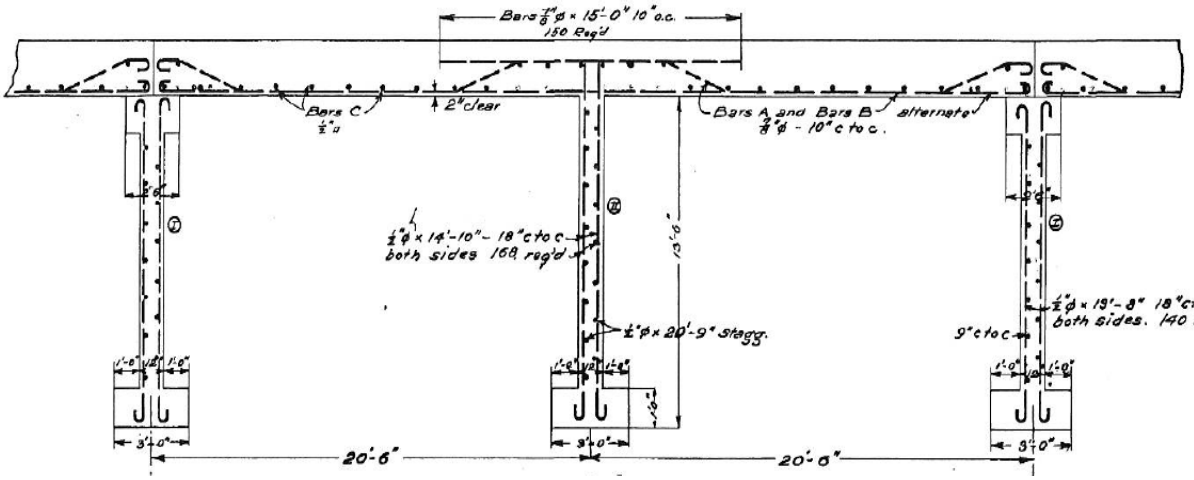


Figure 3-9: Longitudinal Elevation Section of Two-Span Continuous Bridge in Fayette Co. from SHDA (1924)

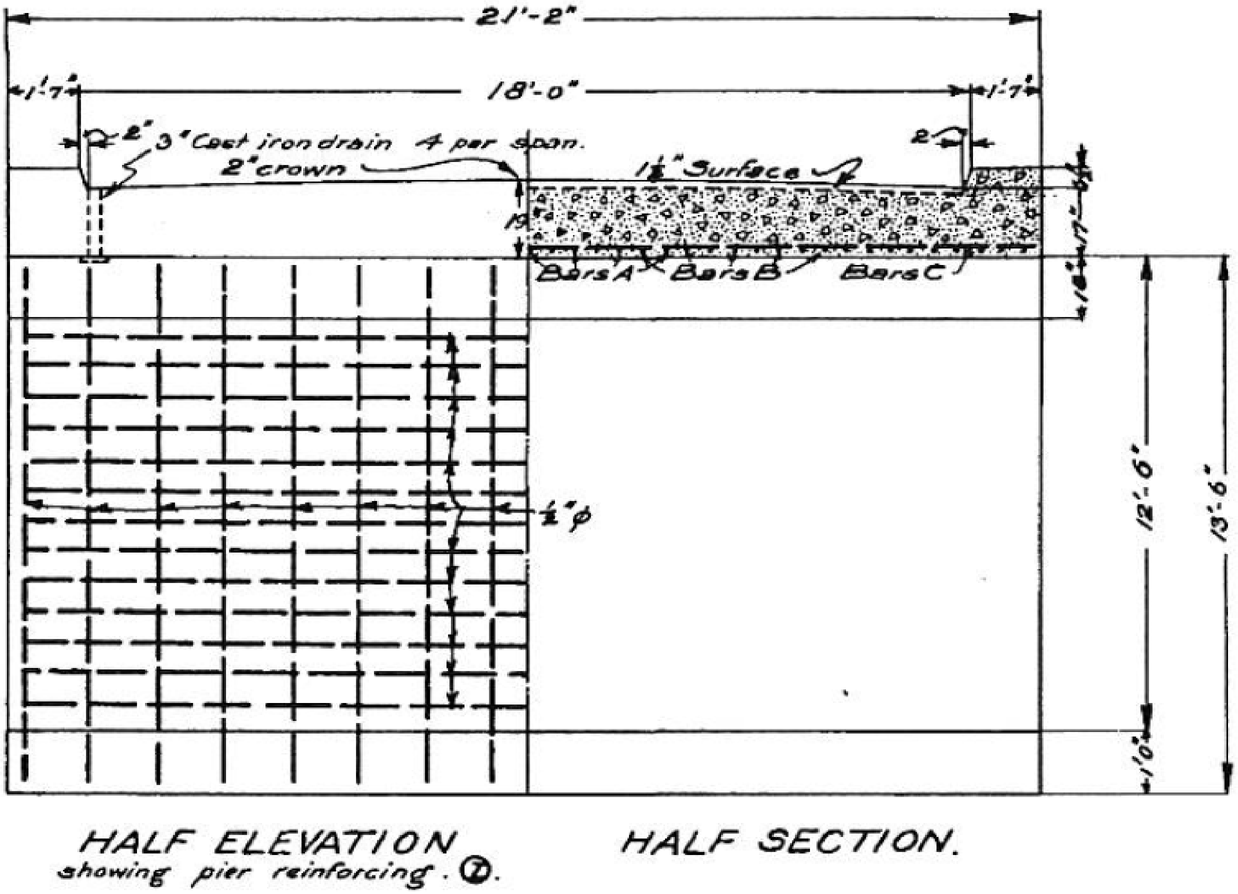


Figure 3-10: 1924 Drawings of Roadway Section of Two-Span Continuous Bridge in Fayette Co. from SHDA (1924)

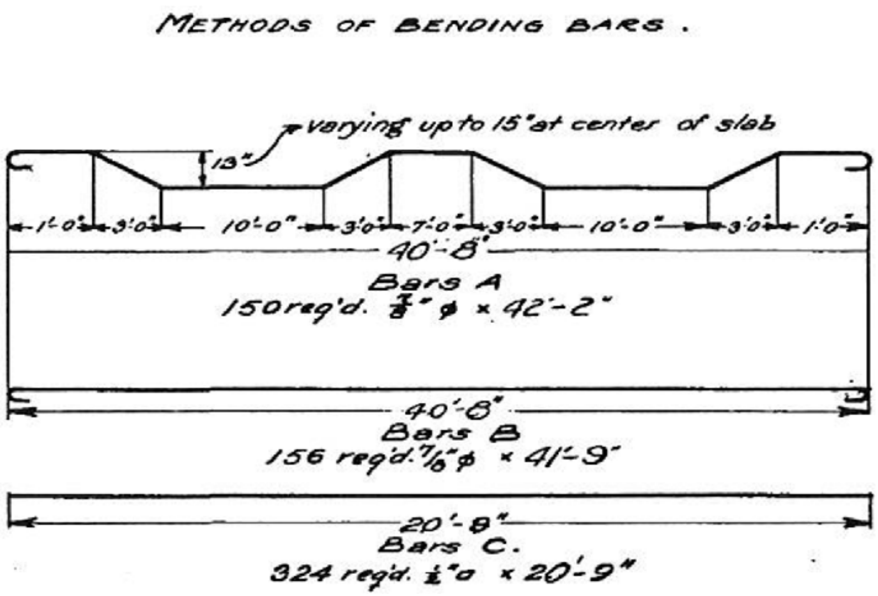


Figure 3-11: Bar Geometry of Two-Span Continuous Bridge in Fayette Co. from SHDA (1924)

Table 3-8: Summary of the Cross Sectional Properties and Material Properties Needed to Calculate the Reinforcement in the Slab of Fayette Co. Bridge

Parameters	Value
Allowable Concrete Compressive Stress, fc	650 psi
Allowable Steel Tensile Stress, fs	16,000 psi
Modular Ratio	15
Reinforced Concrete Unit Weight	150 pcf
Live Load	H 15 Truck
Impact Allowance (Contemporary)	30%
Super Imposed Dead Load (Contemporary)	80 psf
Slab Thickness, H	17 in.
Depth to Tension Reinforcement, d	15 in.
Effective Width of Slab (Contemporary)	4.15 ft
Span Length (center to center of supports)	20 ft

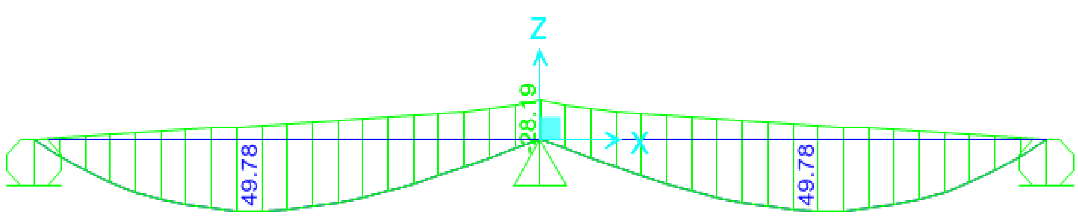


Figure 3-12: The Moment Envelope for a Moving H15 Truck on 20-ft Two-Span Continuous in Fayette Co. (Values are in kip-ft)

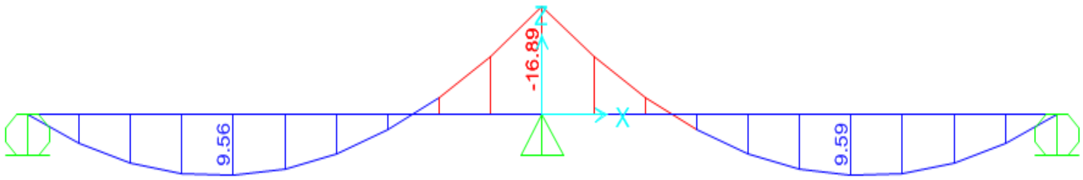


Figure 3-13: The Deal Load Moment Diagram for a 20-ft Two-Span Continuous in Fayette Co. (Values are in kip-ft)

Using the provided characteristics in Table 3-8, a structural analysis was done using SAP2000 (S*AP2000*. Computer software. *Https://www.csiamerica.com/products/sap2000*. Vers. 17.1. N.p., 2015. Web.) to define the maximum live load moment from the moment envelopes. The moment of inertia was kept constant throughout both spans. Figure 3-12 shows the moment envelop for a H15 Truck on this two-span continuous configuration, and Figure 3-13 shows the dead load moment diagram of this bridge. The results for the amount steel needed per foot of width and the spacing of the bars were calculated and the results match the drawings. These calculations were done both for the positive moment region and negative moment region, and the results were the same for both regions. The results from the calculations confirmed the reinforcement of 1.4 in^2 per foot of width, and if Number 7 bars were used as per the SHDA (1924), the spacing would be 5 in. on center. The amount of steel per foot of width was calculated based on the OFOR method and the results were the same for this analysis. The GA DOT (1936) method for effective width however, did not prove to be reliable for this case as the spacing for Number 7 bars was 4 in. on center.

3.4 Conclusions from Case Studies

The method of back-calculating the effective width for the 1922 simple spans (SHDA, 1922) resulted in values that were approximately constant. None of the design methods of calculating effective width learned matched this finding. Additionally, it was concluded that the effective width was calculated independently of the roadway width and span length for the simple spans. The outcome of this study showed that the average effective width was determined to be 4.15 ft for all roadway widths and spans. Besides the effective width concept, these case studies confirmed that our understanding of every parameter used in the contemporary method was accurate, and although there is no evidence on how the single value of effective width was chosen, there was enough data to show that the 4.15 ft value was reliably used in contemporary design.

It was concluded that the SHDA (1922) used the clear span for simple spans of the bridge to design the reinforcement in the slab. This observation was made when the effective width was back-calculated using center to center of the support as the span length. When the center to center of the supports was considered the effective width values ranged from 4.75 ft to 6.25 ft with an average effective width of 5.56 ft. However, calculating the reinforcement for all span length by using an effective width of 5.56 ft did not generate the same reinforcement listed in SHDA (1922). For the two-span continuous bridge, the center to center of the support was considered for the design span length and not the clear span.

With this knowledge, the research could focus on predicting the cross sectional capacity and material properties in the Barnes Slough Bridge.

3.5 Modern Rating of Bridges

The aim of this section is to describe the load factor rating process for bridges according to AASHTO (MBE, 2011) and the AASHTO (17^{th} ed., 2002). Additionally the process of rating bridges using AASHTOWare is presented in this chapter. Today, AASHTOWare is used by the DOTs nationwide for the purpose of rating the structures in accordance with AASHTO specifications.

3.5.1 Modern Methodology

A rating analysis of a bridge is a structural evaluation of the overall condition of the bridge superstructure with regard to its load-carrying capacity. Live load is the primary concern for rating, and the structure is assumed to be sufficient for its dead load; however, the dead load effect is considered in the calculations. The result of a rating is a fraction where a value of one means that the structure is exactly sufficient for a given truck live load, and zero means that it is capable of carrying no live load. Any value greater than zero shows the proportion of the truck's nominal weight as the bridge's load carrying capacity; therefore, the trucks' weight should be limited to that proportion for the safety of the structure. AASHTO (MBE, 2011) has a series of standard trucks that are commonly used for rating bridges. These trucks are a good representation of many trucks although their configuration may not be exactly the same as any truck. ALDOT uses slightly different trucks shown in Figures 3-13 through 3-16 for calculating bridge ratings.

Below the concept of rating is explained in accordance with AASHTO (MBE 2011). The general equation for rating is

$$RF = \frac{C - A_1 D}{A_2 L(1 + I)} \qquad \text{(Eq -40)}$$

Where

RF = Rating factor for the live load capacity.
C = Capacity of the member
D = Dead load effect on the member.
L = Live load effect on the member.
I = Impact fraction to be used with the live load
A_1 = Factor for dead load
A_2 = Factor for live load

This value multiplied by the nominal truck weight gives the load capacity for that truck configuration, usually reported in ton:

$$RT = (RF)\ W \qquad \text{(Eq -41)}$$

Where:

RT = Bridge member rating (ton)
W = Weight of the nominal truck in determining live load effect, L.

There are two kinds of ratings for bridges: operating and inventory. The operating rating is used by ALDOT for evaluation of an existing structure and the factors A_1 and A_2 are equal to 1.3. For an inventory rating the factor A_1 is 1.3 and A_2 is 2.17, which corresponds to design of new structures according to AASHTO (17^{th} ed., 2002). The equations above are used for both operating and inventory cases with different nominal values used for A_1 and A_2 factors. In the above equation the "load effect" refers to vertical shear force, axial force, bending moment, axial stress, shear stress, and bending stress, etc. In our case, a flat slab bridge, the primary concern is the bending moment in the slab. The rating of a bridge is controlled by the member yielding the least rating factor. For a flat slab bridge, one effective width resists the load from one wheel-line for a given truck; therefore, the wheel-load distribution factor for this effective width is one, and the evaluation of the bridge cross-sectional capacity is done for one effective width.

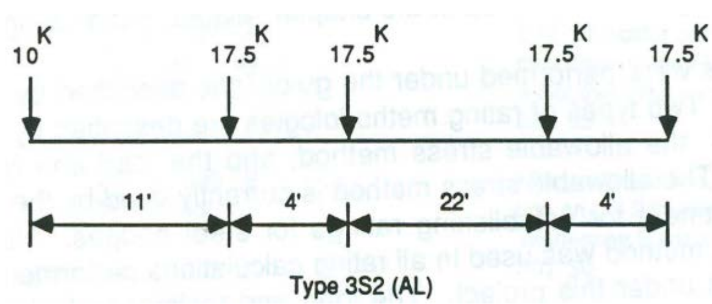


Figure 3-13: ALDOT Standard Trucks Type 3S2

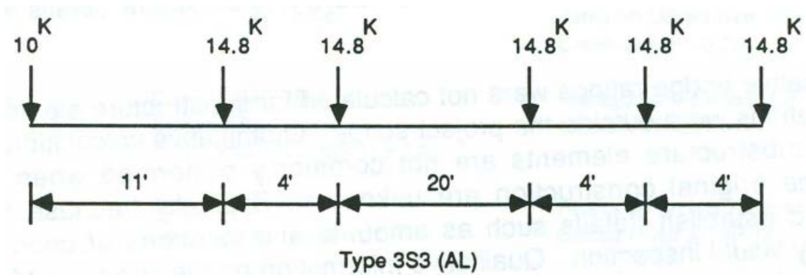


Figure 3-14: ALDOT Standard Trucks Type 3S3

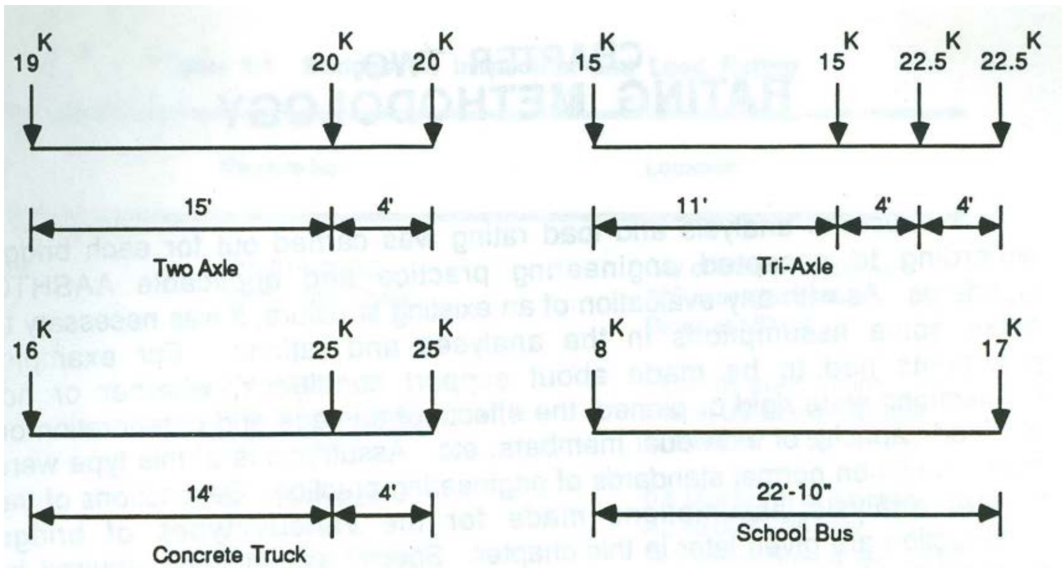


Figure 3-15: ALDOT Standard Trucks for Two Axle, Tri-Axle, Concrete Truck, and School Bus

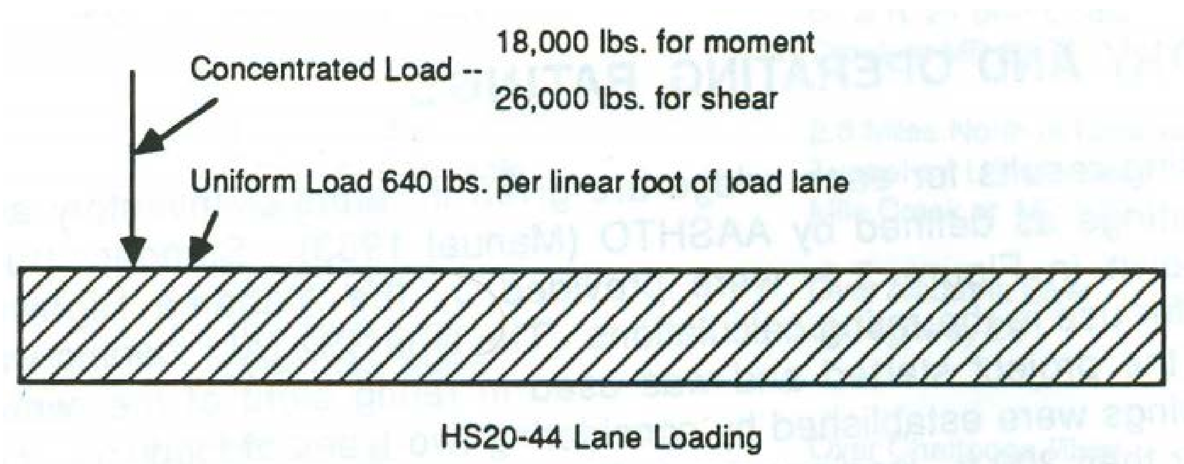


Figure 3-16: AASHTO Standard Trucks for HS20-40 Used by ALDOT

3.5.2 H15 Ratings of 1922 Simple Span Bridges

A series of ratings were done for bridges from the 1920s: one for the simple spans, and another for the two-span continuous bridge of Fayette Co. In this section these calculations are presented for 20-ft simple spans from SHDA (1922) with the characteristics listed in Table 3-9. For detailed calculations refer to Appendix A.

Table 3-9: Summary of the Characteristics Used for Modern Rating of 20-ft Simple Span from 1922

Parameters	Value
Concrete Compressive Strength, f_c	2500 psi
Steel Yield Tensile Strength, f_y	33,000 psi
Reinforced Concrete Unit Weight	150 pcf
Live Load	H15 Truck
Impact Allowance (LFD)	30%
Span Length	20 ft
Slab Thickness, H	19
Depth to Tension Reinforcement, d	18
As (Area of Steel per Foot)	1.68 in^2/ft
Effective Width of Slab (LFD)	5.2 ft

AASHTO (17^{th} ed., 2002) has specified the material properties for structures that were built prior to 1954 for which the material properties are unknown. These specifications are listed in Table 3-9, and these values are the concrete compressive strength and steel tensile yield strength.

The number of traffic lanes was assumed to be two lanes. This assumption allowed having a full effective width in the 16 ft roadway for rating purposes. As for the type of loading, since the documents showed that these spans were designed for H15, the same truck was used to perform the rating; however, this process applies for all other types of trucks. According to AASHTO the impact fraction of 0.3 should be added; therefore, the total effect would be 1.3 times the live load effect. For the purpose of rating the bridge the effective width was calculated in accordance with AASHTO (17^{th} ed., 2002). Since the main reinforcement was parallel to traffic, Equation 27 governed for effective width of simple spans. For the H15 truck, ¾ of this value will apply as effective width. For the H15 truck, the maximum live load moment is 60 kip-ft for one wheel-line. (Figure 3-5 shows the truck configuration on the bridge) The dead load moment was 15.9 kip-ft, and the impact fraction was 0.3. The cross section capacity was calculated using the equation below:

$$C = \emptyset M_n = \emptyset A_s f_y (d - \frac{a}{2}) \qquad \text{(Eq -42)}$$

Where

$\phi = 0.9$

and

$$a = \frac{A_s f_y}{0.85 f`_c d b_w} \qquad \text{(Eq -43)}$$

From the above equation, the capacity of the cross section is 68.2 kip-ft, and by applying Equation 41, the operating rating factor is 2.5, which means the slab can carry 2.5 times the nominal weight of the H15 truck.

3.5.3 H15 Ratings of 1924 Two-Span Continuous Bridge

For the two-span continuous bridge, the same process was used with the addition of identifying the maximum moment through the use of SAP2000 and generating moment envelopes from the software for moving truck loadings, and influence lines for lane loadings. Figures 3-10 and 3-11 show the moment diagram for the dead load and the moment envelope and for the live load of the H15 truck on the two-span continuous beam. The rating for Fayette County Bridge used the parameters listed in Table 3-10. Applying the same equations (Eq-41 and Eq-42) the operating rating factor for the positive moment region is 2.33, and operating rating factor for the negative moment regions is 3.00.

Table 3-10: Summary of the Characteristics Used for Modern Rating of Two-Span Continuous Bridge from 1924

Parameters	Value
Concrete Compressive Strength, f_c	2500 psi
Steel Yield Tensile Strength, f_y	33,000 psi
Reinforced Concrete Unit Weight	150 pcf
Live Load	H15
Impact Allowance (LFD)	30%
Span Length	20 ft
Slab Thickness, H	17 in.
Depth to Tension Reinforcement, d	15 in.
As (Area of Steel per Foot)	1.44 In.^2/ft
Effective Width of Slab (LFD)	5.2 ft

This process also indicated that this structure was more than sufficient for the H15 truck load. The next step was to investigate the rating for other span lengths and truck types. The results from these ratings are listed in section 3.6.1.

3.6 AASHTOWare

AASHTOWare is a software used by the bridge engineers for evaluation and design. This software is organized so that the user can describe the structure through a series of inputs, such as material properties, cross-sectional properties, effective width, bar pattern, bar size, bar spacing, etc. A series of screenshots are shown in Figures 3-17 through 3-20 to demonstrate the process. Once a new file is created and labeled, other inputs are inserted. Figure 3-19 shows a series of the branches that are used to describe the structure. First branch is the material properties of the concrete and reinforcement. Then the method desired to rate the structure is defined, in this case LFD. The next folder is the “Superstructure Definition”. Here the structure is defined as a “Girder Line Superstructure”, and then the impact, dead load, and the wheel-load distribution factor are defined. Under the “Bar Mark Definition” folder different bar sizes and their geometry are defined, and under the “Member” folder, other cross sectional properties are defined such as the bar spacing, clear distances, modular ratio, asphalt thickness, web geometry, etc. Next, a bridge alternative is defined, and lastly the trucks for which the bridge is being rated are chosen (Figure 3-20). Finally, this analysis is run to determine the ratings for the structure.

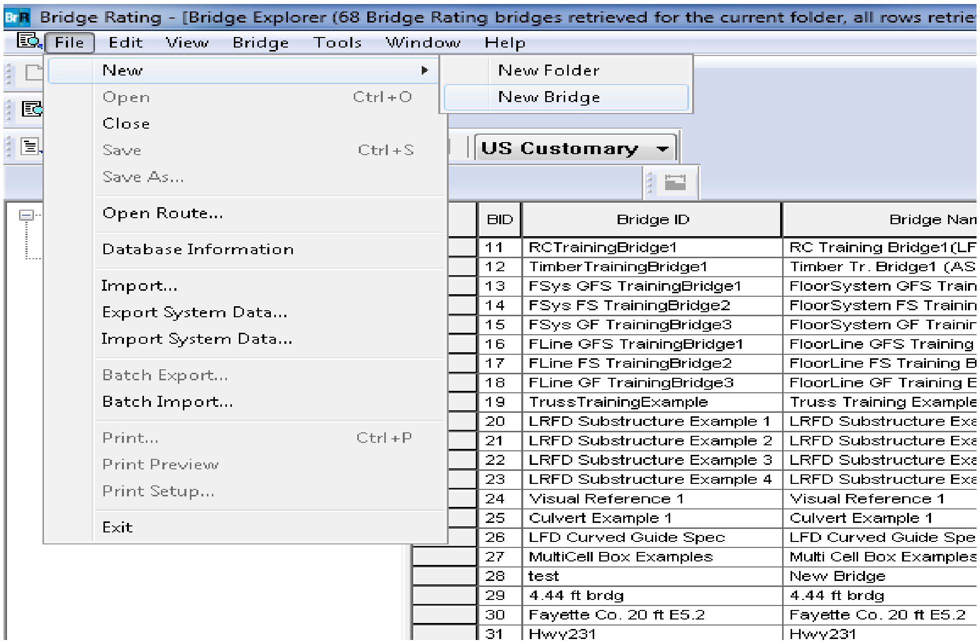


Figure 3-17: Creating a New File in AASHTOWare (2014)

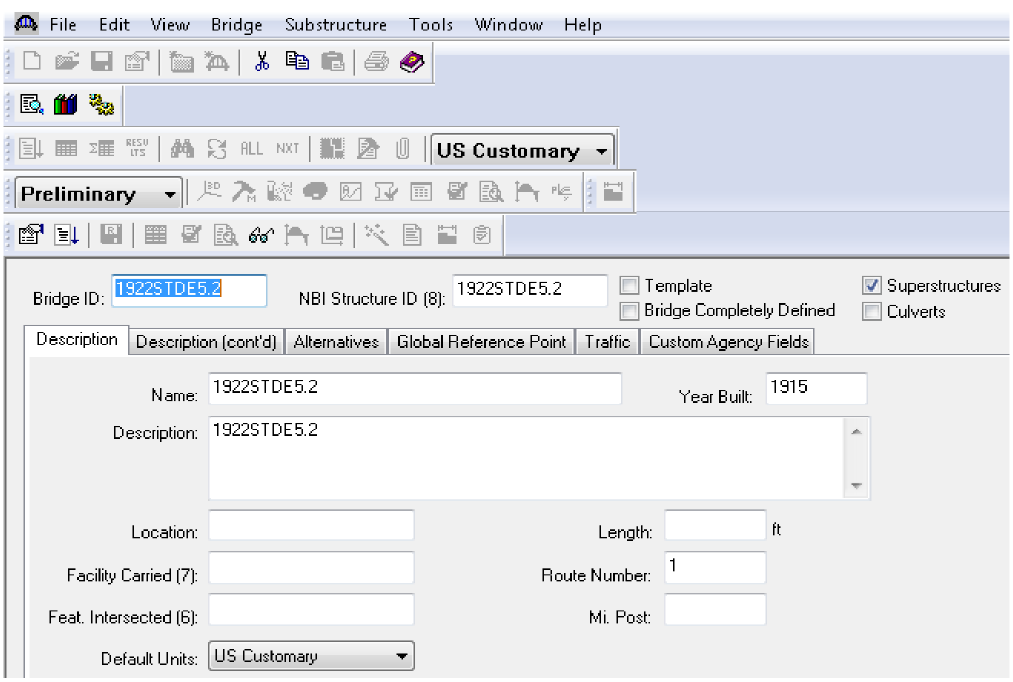


Figure 3-18: Labeling the File in AASHTOWare (2014)

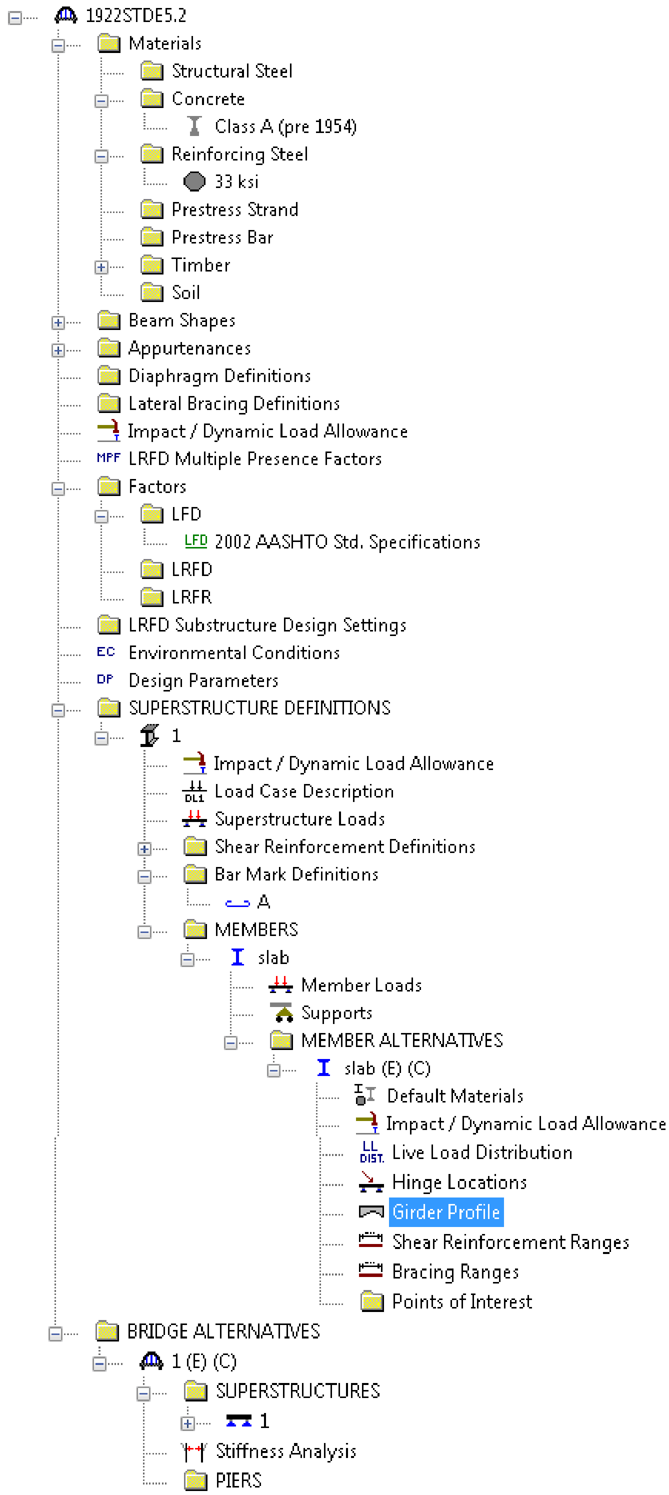


Figure 3-19: Tree of Folders to Insert Inputs in AASHTOWare (2014)

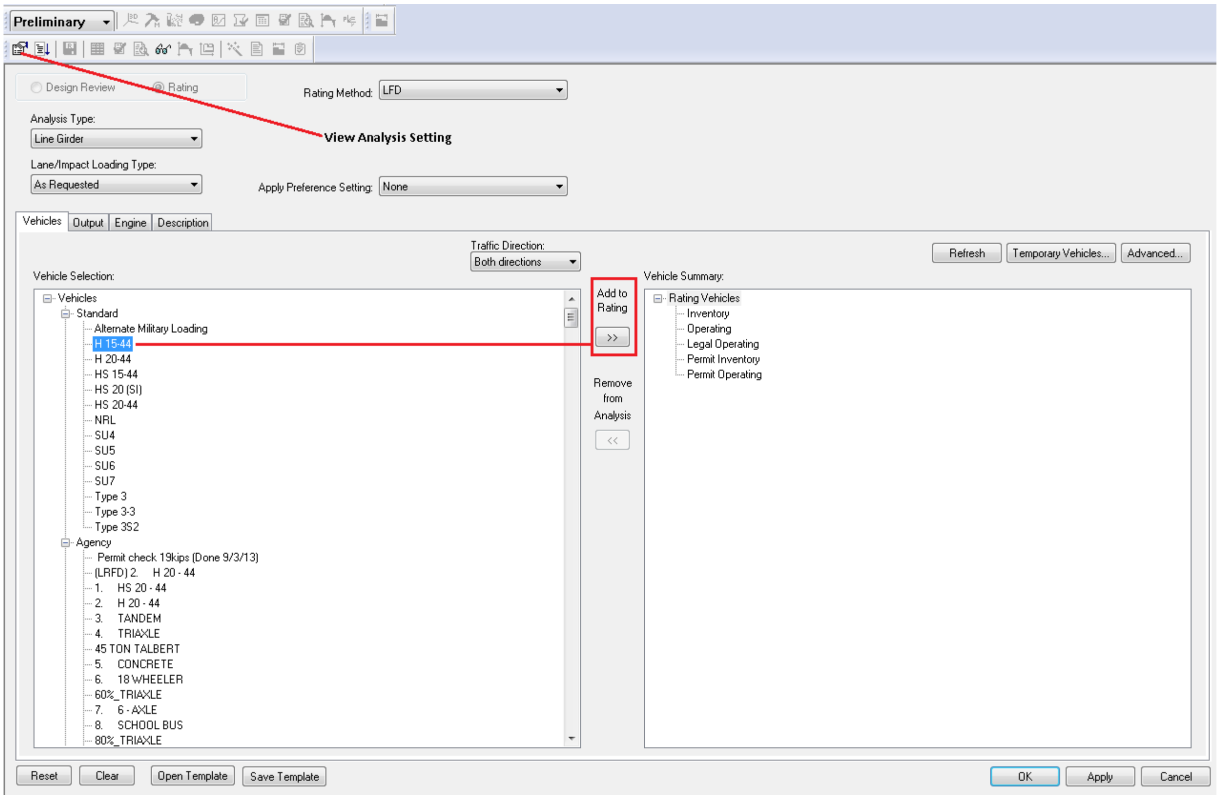


Figure 3-20: Under View Analysis Settings Chose the Truck Types (AASHTOWare, 2014)

3.6.1 AASHTOWare Ratings for ALDOTs' Standard Trucks

A series of ratings were generated using AASHTOWare for ALDOT standard trucks both for the 20-ft simple span and the two-span continuous bridge defined in the SHDA (1922, 1924). The results are listed in Tables 3-11 and 3-13. The results for the simple span matched the hand calculations. The operating rating results from hand calculations for the two-span continuous case are reported in Table 3-12. AASHTOWare considers the development length of the bar as well as the size and spacing. The bars in the negative moment region were terminated at 7.5 ft from the center of the support. This cut off length resulted in lower values than the hand calculations for the negative moment region only since the rating formula does not consider the termination of bars. The ratings for all ALDOT trucks for two-span continuous are listed in Table 3-13.

Table 3-11: Ratings of 20-ft Simply Supported Span by AASHTOWare for All ALDOT Standard Trucks

Vehicle	Inventory Rating Factor	Operating Rating Factor
School Bus	2.19	3.6
H 15-44	1.55	2.5
Type 3S2 (AL)	1.33	2.2
HS 20-44	1.16	1.9
Two Axle	1.16	1.9
Type 3S3 (AL)	1.14	1.9
Concrete	0.93	1.5
Triaxle	0.82	1.3

Table 3-12: Ratings of 20-ft Two-Span Continuous with Full Length Bars

Vehicle	Inventory Rating Factor	Operating Rating Factor
H 15-44	1.28	2.3

Table 3-13: Ratings of 20-ft Two-Span Continuous with Bars Developed 7.5 ft on Either Side of the Support

Vehicle	Inventory Rating Factor	Operating Rating Factor
School Bus	1.14	1.91
H 15-44	0.81	1.36
Type 3S2 (AL)	0.58	0.97
HS 20-44	0.61	1.02
Two Axle	0.51	0.85
Type 3S3 (AL)	0.50	0.84
Concrete	0.41	0.68
Triaxle	0.36	0.60

Table 3-14: Summary of the Characteristics Used for Modern Rating of all Simple Span Bridges from 1922.

Parameters	Value
Concrete Compressive Strength, f_c	2500 psi
Steel Yield Tensile Strength, f_y	33,000 psi
Reinforced Concrete Unit Weight	150 pcf
Live Load	All ALDOT Trucks
Impact Allowance (LFD)	30%
Span Length	Table 3-1
Slab Thickness, H	Table 3-1
Depth to Tension Reinforcement, d	Table 3-1
As (Area of Steel per Foot)	Table 3-2
Effective Width of Slab (LFD)	Defined by Eq-27

Lastly, the operating and inventory ratings for all simple span flat slab bridges form SHDA (1922) are listed in Table 3-15 and 3-16. The characteristics used for these modern ratings are summarized in Table 3-14.

Table 3-15: Operating Rating Factor for ALDOT Trucks and for 1922 Simple Spans

Vehicle	Operating Rating Factor							
	Span Length (ft)							
	6	8	10	12	14	16	18	20
HS 20-44	1.29	1.30	1.38	1.62	1.58	1.89	2.03	1.76
Triaxle	1.64	1.53	1.33	1.36	1.22	1.38	1.47	1.36
Concrete	1.48	1.37	1.21	1.32	1.23	1.43	1.624	1.47
School Bus	2.44	2.45	2.60	3.04	2.98	3.57	3.74	3.79
Two Axle	1.85	1.72	1.51	1.65	1.54	1.79	2.03	1.83
Type 3S3 (AL)	2.50	2.32	2.01	2.00	1.76	1.97	2.00	1.92
Type 3S2 (AL)	2.12	1.96	1.73	1.89	1.76	2.05	2.30	2.04

Table 3-16: Inventory Rating Factor for ALDOT Trucks and for 1922 Simple Spans

Vehicle	Inventory Rating Factor							
	Span Length (ft)							
	6	8	10	12	14	16	18	20
HS 20-44	0.77	0.78	0.82	0.97	0.94	1.13	1.21	1.05
Triaxle	0.98	0.92	0.79	0.81	0.73	0.83	1.22	0.81
Concrete	0.88	0.82	0.72	0.79	0.74	0.86	1.38	0.88
School Bus	1.46	1.47	1.55	1.82	1.78	2.14	0.89	2.27
Two Axle	1.11	1.03	0.90	0.99	0.92	1.07	1.19	1.10
Type 3S3 (AL)	1.50	1.39	1.20	1.20	1.05	1.18	0.88	1.15
Type 3S2 (AL)	1.27	1.16	1.03	1.13	1.05	1.23	0.97	1.22

3.7 Conclusions

Form these case studies, it is concluded that the method which the effective width was calculated remains unknown; however, there were enough data to consider this value to be 4.15 ft for all designs. Additionally, all the ratings for simple span bridges shown in section 3.6.1 and, Table 3-15, for a case of an operating rating factor prove that the contemporary methods used to design the flat slab bridges for simple spans were so that the cross-sectional capacity was adequate for all ALDOT trucks. Thus there is no need for posting on these bridges. However, for two-span continuous bridges, the reinforcement in the negative moment region was not developed properly. By modern standards as a result the operating rating factor for five of the eight standard trucks were less than one. So, posting of weight restrictions for these trucks would be necessary.

With the information gained in Chapters 2 and 3, the research focused on identifying the capacity of Barnes Slough Bridge in chapter 4.

4 Barnes Slough Bridge

4.1 Introduction

This chapter is focused on using contemporary method to estimate the amount of reinforcement required in the Barnes Slough Bridge, presenting the field measurements of the slab properties, and rating the bridge. The capacity of the slab depends upon the cross-sectional characteristics, material properties, the loading conditions, and the effective width.

Initially, the capacity of the slab was calculated for the Original segment. The cross sectional characteristics were defined based on the methods used in contemporary design. The material properties were the allowable stress values chosen from the literature review. The loading was H15 Truck loading. The effective width value was concluded from Chapter 3. Using the above information, the amount of reinforcement in the slab was estimated.

After the initial calculations, a series of field measurements were done. These field measurements indicated in a different amount of reinforcement in the slab, changed the clear cover. Core tests identified the concrete strength. Also from these field measurements, it was concluded that the structure was behaving as a simple span. A comparison made between the field measurements and the estimated capacity of the slab concluded that methods learned from Chapter 2 and 3 were not able to establish a method which was used to design the reinforcement during the contemporary time. Next, a series of decade studies were done with an aim to estimate a time when the additions were built by comparing these results with the field measurements.

Lastly, the capacity and the characteristics of the slab were used to build a baseline structural model of Barnes Slough Bridge, and ratings were generated for all four segments. For the purpose of modern evaluation of the bridge and rating, the cross-sectional characteristics measured in the field were used. Other characteristics such as the loading, impact, effective width, and the number of traffic lanes were considered in accordance with AASHTO (17th ed., 2002), and did not depend on the contemporary design methods.

4.2 Estimating the Reinforcement using the Contemporary Design Methods

Firstly, the task was to calculate the amount of reinforcement required in Barnes Slough Bridge based on the design methods in the 1920s. These calculations were done to estimate the amount of steel in the original 18 ft wide segment in the middle.

The geometry of the existing bridge was measured through initial visual inspection of the bridge. Figures 4-1 through 4-3 show these measurements that were taken on the site. The field measurements showed an average span length of 21 ft-10 in. center to center of the supports with a clear span length of 19 ft-10 in. and the thickness of the piers was 2 ft. A layer of 1.75 in. asphalt existed over the full roadway, and the thickness of the slab was measured to be 19 in. and an 18 in. of effective depth was assumed as it was specified in the SHDA (1922) for similar span lengths of 20 ft.

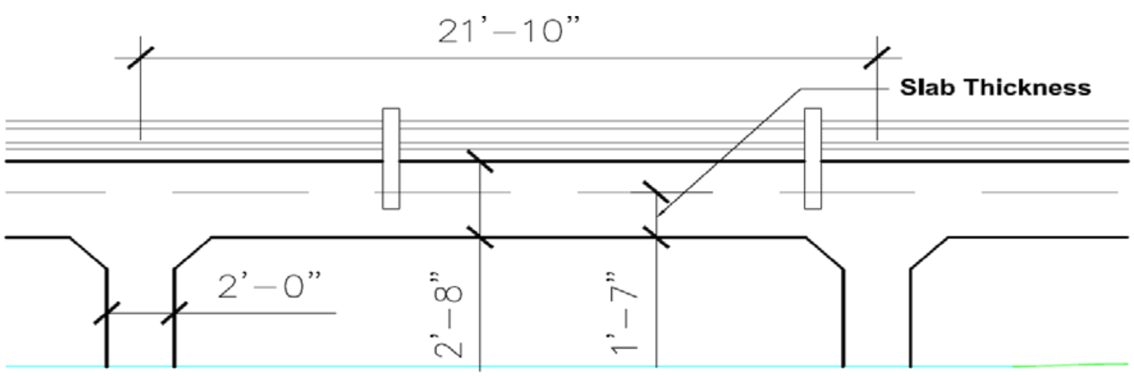


Figure 4-1: Elevation of Typical Span of Barnes Slough Bridge

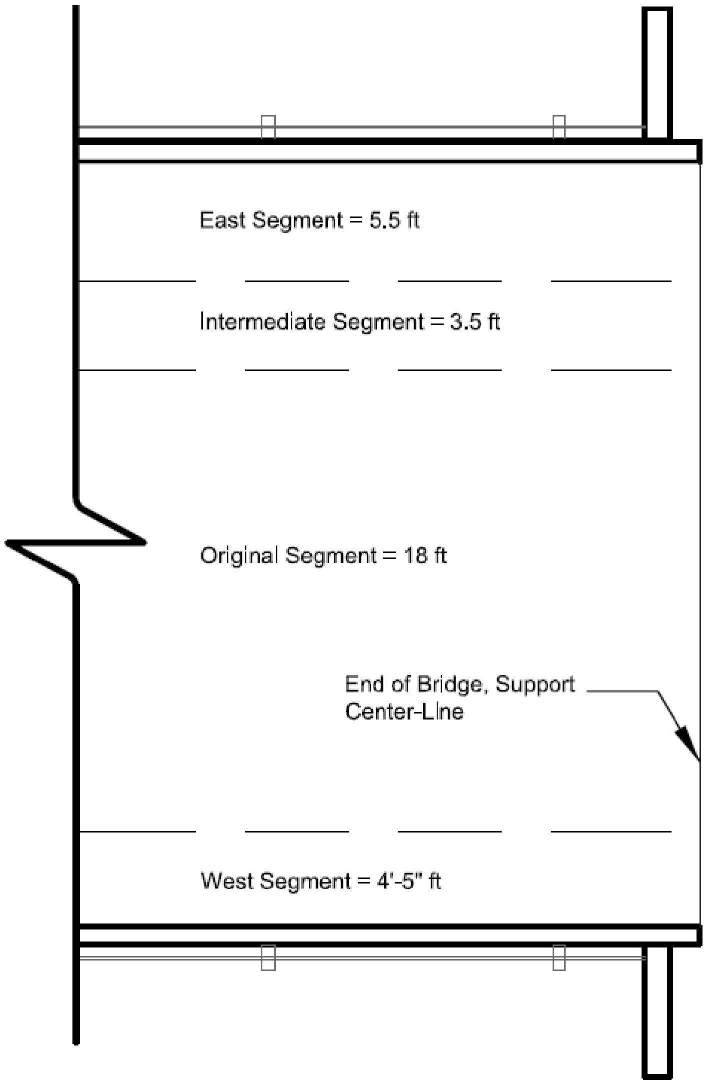


Figure 4-2: Partial Plan of Barnes Slough Bridge

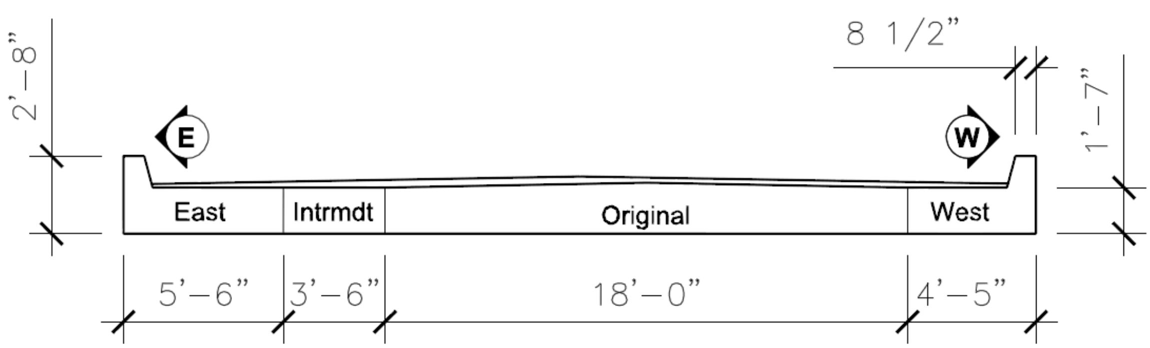


Figure 4-3: Cross Section of Barnes Slough Bridge

In the initial calculations the structure was modeled both as simple spans and as a series of continuous spans. The estimation of reinforcement in the slab was done for simple span moment diagrams and continuous spans' moment envelopes generated for an H15 Truck since this truck was used by contemporary designers. Required bar spacings were calculated for three different bar sizes, Number 7 bar, Number 8 bar, and Number 9 bar. The effective width for the purpose of defining steel in the slab was considered to be 4.15 ft as it was established in Chapter 3 for the contemporary design method. Table 4-1 lists the parameters used to calculate the amount of steel in the slab. Additionally the amount of steel was calculated for this slab based on OFOR method. The two methods provided a range of amount of steel that could be expected to be detected in the slab of Barnes Slough Bridge.

Table 4-1: Assumed Parameters Used to Calculate the Amount of Reinforcement in the Slab Using Contemporary Design Method

Parameters	**Value**
Allowable Concrete Compressive Stress, fc	650 psi
Allowable Steel Tensile Stress, fs	16,000 psi
Modular Ratio	15
Reinforced Concrete Unit Weight	150 pcf
Live Load	H 15 Truck
Impact Allowance (Contemporary)	30%
Impact Allowance (OFOR)	30%
Super Imposed Dead Load (Contemporary)	80 psf
Super Imposed Dead Load (OFOR)	80 psf
Slab Thickness, H	19 in.
Depth to Tension Reinforcement, d	18 in.
Effective Width of Slab (Contemporary)	4.15 ft
Effective Width of Slab (OFOR)	7.78 ft

Tables 4-2 and 4-3 have listed the results for all the conditions listed above. For calculations refer to Appendix A. This initial calculation was done to see if the results matched the case studies from the 1920s. The amount of steel for both cases of simple spans and continuous spans were within 5% of the results from SHDA (1922; 1924). For continuous spans, Number 7 bars resulted in a spacing of 5in. in the positive moment region and 5.5 in. in the negative moment region, which are very close to the Fayette Co. Bridge (SHDA, 1924) with Number 7 bars at 5 in. on center. For simple spans, Number 8 bars provided the same 5.5 in. spacing as in Table 3-1. Next the reinforcement was designed based on OFOR method and the difference in the expected amount of steel was less than 10%. From these studies it was expected to detect similar amount of reinforcement through field measurements.

Table 4-2: Spacing for Different Conditions in the Original Segment According to Contemporary Methods for H15 Truck

	Simple Span		11-Spans Continuous			
	+M		+M		-M	
Bar size	Contemporary Spacing (in.)	OFOR Spacing (in.)	Contemporary Spacing (in.)	OFOR Spacing (in.)	Contemporary Spacing (in.)	OFOR Spacing (in.)
#7	4.75	4.25	5.25	5.0	5.75	5.75
#8	6.25	5.5	7.0	6.5	7.5	7.5
#9	8.0	7.0	8.5	8.5	9.75	9.5

Table 4-3: Amount of Steel per ft of Width for Different Conditions in the Original Segment According to Contemporary Methods for H15 Truck

Simple Span		11-Spans Continuous			
+M		+M		-M	
Contemporary	OFOR	Contemporary	OFOR	Contemporary	OFOR
A_S(in.^{2})/Lft	A_S(in.^{2})/Lft	A_S(in.^{2})/Lft	A_S(in.^{2})/Lft	A_S(in.^{2})/Lft	A_S(in.^{2})/Lft
1.50	1.69	1.36	1.39	1.23	1.24

4.3 Field Measurements

Field measurements were made to determine as many as practical of the parameters that are needed to calculate the cross-sectional capacity. These tests included the use of Profometer (http://www.proceq.com, 2015) test to read the rebar spacing, bar size, and cover for the bottom layer of reinforcement. Schmidt hammer test and concrete core test were performed to define the concrete strength. Ground penetration radar (GPR) test to check the spacing of the bars at the top layer of reinforcement, and concrete was removed to expose rebar to confirm the bar diameter.

Concrete was removed from the bottom of the slab in the original segment and from the intermediate segment. Some bars were exposed in the East and West segments due to concrete spalling, which helped to confirm the size of the bars in those segments. The exposed bars indicated that the measurements taken with the Prefometer device were not fully accurate. Table 4-4 has listed the modified bar size and clear cover for all four segments. These tests also resulted in slightly different characteristics than the ones tabulated in SHDA (1922; 1924). The clear cover was confirmed to be 1.25 in., and it was confirmed that all segments have Number 8 bars except the Intermediate segment which has Number 7 bars. The spacings of these bars are listed in Table 4-4. The amount of steel measured in the slab was considered to be distributed equally for calculation purposes, and the measured values were used in the models without any modifications.

Table 4-4: Tension Reinforcement at Bottom of Slab from Field Measurements

	Cross Sectional Properties				
Segment	**Segment Width**	**Cover (in.)**	**Bar Size**	**# of Rebars**	**Spacing (in.)**
East	5.5 ft	1.25	#8	10	6.8
Intermediate	3.5 ft	1.25	#7	9	4.5
Original	18 ft	1.25	#8	53	4.0
West	4.4 ft	1.25	#8	7	7.5
TOTAL	31.4ft				

The results from Schmidt hammer tests were inconclusive. The concrete core tests were done at three locations: the first location was over the first support from the south end in the original segment of the slab, second location was over the first support from south end in the East segment, and the third location was at the middle of the first span from the south end in the East segment. The results from the core tests are listed in Table 4-5. The result from the core test showed strength of 3340 psi in the first location. However, it was decided to use the suggested value of 2500 psi in accordance with AASHTO (MBE, 2011) since the tested value was much higher than the core strengths at the other locations. The concrete compressive strength of 1850 psi was applied to East, Intermediate, and West segments. This value was the average value of the two core test results from the east segment. Also the core tests from the top of the slab exposed the reinforcement in the top layer of reinforcement, and they were confirmed to be Number 4 bars at 10.5 in. on center.

Table 4-5: Concrete Core Test Results and the Adjusted Values

Location of Sample	Measured Strength	Adjusted Strength
East@ Mid-Span	1760 psi	1850 psi
East @ Support	1937 psi	1850 psi
Original @ Support	3340 psi	2500 psi

The field measurements showed that the expected amount of reinforcement in the slab calculated using contemporary methods (Table 4-2) for continuous spans in the positive moment region was less than the amount detected in field measurements (Table 4-4). Additionally the amount of reinforcement in the negative moment region was only 18% of that expected in a continuous span (compare Table 4-2 with 0.229 in^2/ft measured at the field). The GPR detected some cracks beneath the asphalt over the supports. These results from the field measurements suggested that the designers of Barnes Slough Bridge reinforced this bridge as if it was a series of simple spans and not as an 11-span continuous bridge.

From the field measurements it was concluded that the four segments of the bridge had different characteristics and had to be evaluated separately. With this conclusion a series of studies wwas done for different decades from 1930s through 1960s in an aim to estimate a time when the additional segments were built.

4.4 Decade Studies

An attempt was made to estimate when the East and West additions were performed. Once it was concluded that the structure was designed as a series of simple spans, the reinforcement for the slab was calculated based in AASHO codes from 1930s through 1960s. The characteristics used in designing the slab for different times are listed in Table 4-6. Tables 4-7 and 4-8 show the results for the amount of reinforcement needed in the slab for different eras throughout the 20th century. The design truck loading that controlled the design of 21 ft-10in. span for the time that the additions were built was H20; however, it is suggested that the truck loading for which the Barnes Slough Bridge was designed for was H 15 Truck.

Table 4-6: Design Characteristics of Simple Span Slab for Different Times

Parameters	**Value**					
	1931	**1935**	**1941**	**1949**	**1957**	**1961**
Allowable Concrete Compressive Stress, f_c (psi)	650	1,000	1,000	1,000	1,000	1,000
Allowable Steel Tensile Stress, fs (psi)	16,000	16,000	18,000	18,000	18,000	18,000
Reinforced Concrete Unit Weight (pcf)	150	150	150	150	150	150
Live Load (Truck Loading)	H15 & H20	H15 & H20	H15 & H20	H15 & H20	H15 & H20	H15 & H20
Impact Allowance	30%	30%	30%	30%	30%	30%
Super Imposed Dead Load (Asphalt, in.)	1.5	1.5	1.5	1.5	1.5	1.5
Slab Thickness, H (in.)	19	19	19	19	19	19
Depth to Tension Reinforcement, d (in.)	17.25	17.25	17.25	17.25	17.25	17.25
Effective Width of Slab (ft)	7	15.6	4.94	4.94	4.94	5.31
Effective Width of Slab (OL)	6.5 ft	NA*	NA*	NA*	NA*	NA*

NA*: The concept of overlap does not apply to other years.

Table 4-7: Calculated Spacing of Reinforcement for 1931

Truck Loading	**Spacing (in.)**			
	#7 bars STD. Eff. Width 7 ft	**#7 bars Overlapped Width 6.5 ft**	**#8 bars STD. Eff. Width 7 ft**	**#8 bars Overlapped Width 6.5ft**
H15 Truck	5	4.50	6.5	6
H20 Truck	3.75	3.50	5	4.50

Table 4-8: Calculated Spacing of Reinforcement for 1935-1961

Year	Effective Width	Truck Loading	Spacing (in.)	
	(ft)		# 7 Bars	#8 Bars
1935	15.6	H15 Truck	6.75	9
		H20 Truck	6.25	8.25
1936	5	H15 Truck		7.75
		H20 Truck		7
1941	4.94	H15 Truck		7.25
		H20 Truck		6.25
1949	4.94	H15 Truck		7.25
		H20 Truck		6.5
1957	4.94	H15 Truck		7.25
		H20 Truck		6.5
1961	5.31	H15 Truck		6.75
		H20 Truck		6

In 1931 the concept of overlap was considered to define the effect width. This concept was explained previously in Chapter 2 Section 2.2.7. The amount of reinforcement in the slab for the overlapped case and Number 7 bars matches the reinforcement in the Intermediate segment measured in the field. There was a note on the "Bridge Card" from 1930 stating that an addition, Intermediate segment, was added. The match between the result shown above and the reinforcement measured in the slab and the note from the "Bridge Card" suggest that the case of overlapped effective width was used to calculate the reinforcement in the Intermediate segment.

Table 4-4 lists a uniform bar spacing determined by dividing the width of the segment by the number of bars in that segment. This decision represents the spacing measured in the original segment, the intermediate segment, and the East segment properly; however, the actual spacing listed for the West segment was different than the 7.5 in. listed in Table 4-4. (Figure 4-4) On the east side of the West segment there was a larger gap between the last bar of the West segment and the first bar of the Original segment. This gap is about 1 ft. This difference influenced the average spacing of bars in the West segment. If this gap was considered, the spacing for West segment would be 7.25 in. A comparison made between the results from Table 4-4 to Tables 4-6 and 4-7 and considering this gap influence, can attempt to estimate when these additions were built. This spacing would suggest that the segments on the East and West of the roadway were built sometime between 1941 and 1959, and that the slab was designed for a H15 Truck loading.

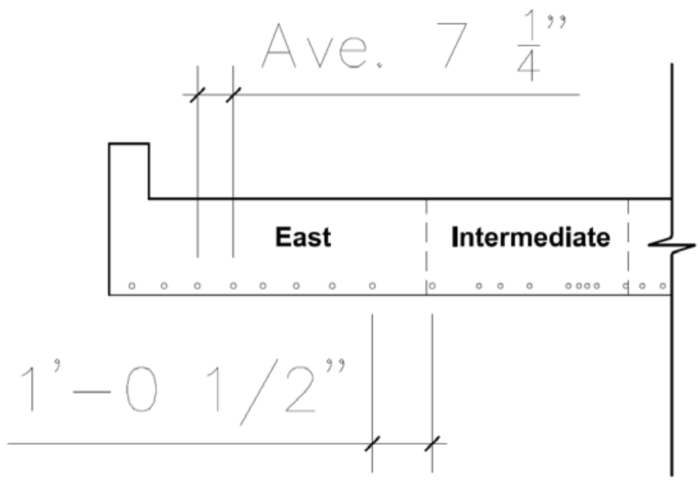


Figure 4-4: The Spacing of Reinforcement in the West Segment

4.5 Modern Rating and the Baseline Structural Model of the Barnes Slough Bridge

Based on the field measurements described above, each segment of the cross section was modeled in AASHTOWare to provide the section capacity and the rating of the segment. A model for each segment was necessary for the final evaluation of the whole structure. In AASHTOWare the effective width of slab was modeled as a single girder with a distribution factor of one both for moment and shear. The Intermediate and West segments were not as wide as the effective width from AASHTO (17^{th} ed., 2002). The Intermediate segment width is 3.5 ft. AASHTO (17^{th} ed., 2002) specifies an effective width of 5.31 ft for a 21 ft–10 in. span (Equation 27). The intermediate segment was modeled by assuming the effective width was symmetrically centered over the 3.5 ft as shown in Figure 4-5. This arrangement led to 10.75 in. into the adjacent segments on either side. The amount of steel over the 5.31 ft effective width was assumed to be the equal distribution of the total reinforcement in the intermediate segment and the reinforcement in the two 10.75 in. adjacent segments. The West segment width is 4 ft - 5 in., which is also less than the effective width of 5.31 ft. The segment is an edge segment, so the effective width started from the West side and continued 10.75 in. into the original segment as shown in Figure 4-6.

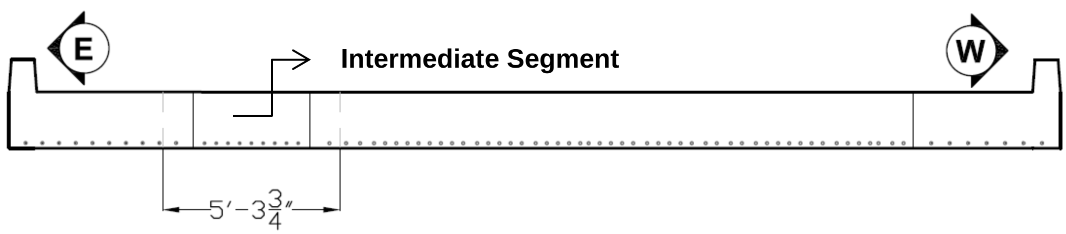


Figure 4-5: Effective Width Used for the Intermediate Segment

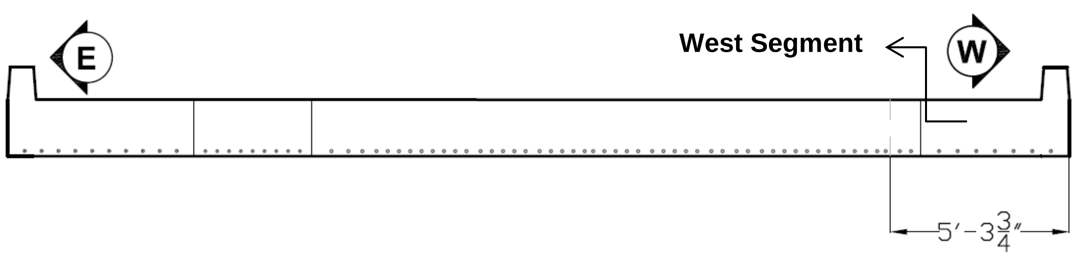
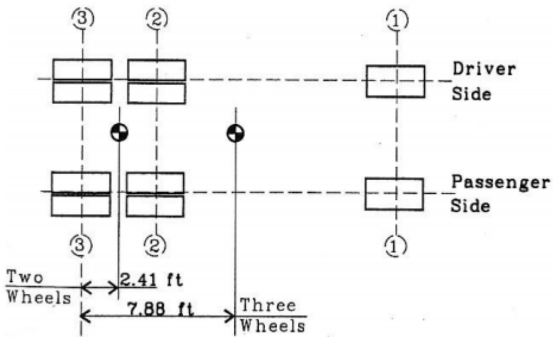


Figure 4-6: Effective Width Used for the West Segment

The amount of steel over this width was an equal distribution of the reinforcement over West segment and the reinforcement in the additional 10.75 in. of the adjacent segment for the purpose of rating.

The rating results from AASHTOWare for each segment are listed in Tables 4-7 through 4-10. These segments include the original, the intermediate, the West, and East segments. These ratings were generated for all ALDOT standard trucks based on LFR method. Figures 3-13 through 3-16 and 4-11 show the configuration of all trucks. Additionally ratings were generated for LC-5 load test truck, which was the truck used for the non-destructive live load testing of this bridge. This truck's configuration is shown in Figure 4-11.



Wheel Loads				
Total Wt (lbs)	85,700	Wheel 3 (lbs)	Wheel 2 (lbs)	Wheel 1 (lbs)
Driver		15,800	16,000	11,100
Passenger		15,600	15,800	11,400
Average		15,700	15,900	11,250

Figure 4-11: LC-5 Load Testing Truck Configuration

The four cross section segments were rated both as simple spans and 11-spans continuous to check the capacity for both cases. Table 4-7 shows the results of AASHTOWare rating for the case of 11-span continuous. These ratings for the continuous spans were zero due to insufficient negative moment capacity. The ratings are not zero by hand calculations. The rating factors should be a negative value of about 7%; however, AASHTOware indicated the lack of cross section capacity in the negative moment region as zero. The negative moment cross section capacity over the supports is enough to support to the unfactored dead load; however, the dead load is multiplied by a factor of 1.3 (Equation 41) and deducted from the capacity in the rating calculations. The factored dead load is greater than the capacity, thus the rating factors for this structure are negative, for which the software generates zero in such cases.

The amount of steel in the negative moment region was confirmed through field tests to be 0.229 in^2 per ft. This area of steel was provided by spacing of Number 4 bars at 10.5 in. on center. These bars were continuous along the whole length of the span. The bar size at the existing spacing must be Number 6 bars to provide enough cross sectional capacity to generate ratings that are bigger than zero. Through these negative moment ratings it was confirmed once more that the structure is behaving as a simple span. Tables 4-8 through 4-10 show the operating and inventory rating factors for the Barnes Slough Bridge as a series of simple spans. From these tables it was concluded that the East segment has the least capacity.

Tables 4-7: Ratings of Barnes Slough Bridge as 11- Span Continuous

Vehicle	Inventory Rating Factor	Operating Rating Factor
School Bus	0	0
Type 3S2 (AL)	0	0
HS 20-44	0	0
Two Axle	0	0
Type 3S3 (AL)	0	0
Concrete	0	0
LC 5 Test Truck	0	0
Triaxle	0	0

Tables 4-8: Ratings of Original Segment of Barnes Slough Bridge as Simple Spans

Vehicle	Inventory Rating Factor	Operating Rating Factor
School Bus	2.44	4.08
Type 3S2 (AL)	1.45	2.43
HS 20-44	1.3	2.17
Two Axle	1.27	2.12
Type 3S3 (AL)	1.24	2.07
Concrete	1.01	1.7
LC 5 Test Truck	0.93	1.56
Triaxle	0.9	1.50

Tables 4-9: Ratings of Intermediate Segment of Barnes Slough Bridge as Simple Spans

Vehicle	Inventory Rating Factor	Operating Rating Factor
School Bus	1.49	2.49
Type 3S2 (AL)	0.88	1.48
HS 20-44	0.79	1.32
Two Axle	0.77	1.29
Type 3S3 (AL)	0.75	1.26
Concrete	0.623	1.04
LC 5 Test Truck	0.57	0.95
Triaxle	0.55	0.91

Tables 4-10: Ratings of West Segment of Barnes Slough Bridge as Simple Spans

Vehicle	Inventory Rating Factor	Operating Rating Factor
School Bus	1.14	1.9
Type 3S2 (AL)	0.68	1.35
HS 20-44	0.6	1.01
Two Axle	0.59	0.99
Type 3S3 (AL)	0.57	0.96
Concrete	0.47	0.79
LC 5 Test Truck	0.43	0.73
Triaxle	0.42	0.7

Tables 4-10: Ratings of East Segment of Barnes Slough Bridge as Simple Spans

Vehicle	Inventory Rating Factor	Operating Rating Factor
School Bus	1.03	1.72
Type 3S2 (AL)	0.61	1.02
HS 20-44	0.55	0.91
Two Axle	0.53	0.89
Type 3S3 (AL)	0.52	0.87
Concrete	0.43	0.71
LC 5 Test Truck	0.39	0.66
Triaxle	0.38	0.63

5 Models for Permit Loads

5.1 Introduction

Barnes Slough Bridge cross section has four segments, and the capacity of each segment is different. In Chapter 4, a description and an operating rating factor for each segment is reported. To provide ALDOT with a structural model of the Barnes Slough Bridge, a single model has to be developed that represents all segments appropriately. The goal of this chapter is to accomplish this task through a series of effective width studies. Adjusting the effective width in AASHTOware is the most appropriate way to modify the calculated capacity of this structure.

According to finite element studies that are ongoing by Wolert (pers.comm.), it was concluded that the weakest location of the bridge is the new segment on the east side of the roadway. However, the performance of the structure through its life and during the live load tests suggests that the structure has significant capacity and perhaps more than the expected capacity shown by the AASHTOWare ratings. When the measured strains and deflections from the field tests were compared to those of the finite element model, it was shown that there is more capacity in this structure, and modeling the weakest portion of the structure in AASHTOWare is not an accurate way of representing the overall structure.

In addition to the field tests and the rating analysis presented in Chapter 4, the structure was checked for shear and development of the bars in the slab to confirm that flexure is the controlling limit state for this structure. The factored shear capacity of the cross section is 15.5 kip per foot of width for the concrete strength of 2500 psi, and 13.4 kip per foot of width for the concrete strength of 1850 psi. Both of these values are larger than shear demands caused by the dead load and live load on the slab. The shear due to dead load is 3.67 kip per foot of width, and the shear due to the LC-5 load testing truck is 5.9 kip per foot of width which result in 9.57 kip total shear demand. The same shear force for dead load added to the triaxle truck load of 4.9 kip per foot of width result in a total of 8.58 kip of shear demand. These calculations confirmed that the slab is sufficient for ALDOT's heaviest trucks. For calculations refer to Appendix-A.

The embedment details of the positive moment reinforcement over the supports is unknown. The drawings from SHDA (1922; 1924) suggested that half of the bottom bars were bent at the support. Based on the field measurements, it was concluded that two-thirds of the reinforcement continued to the supports and only one-third was either terminated or bent. The ground penetration radar did not detect any Number 8 bar or Number 7 bars at the top of the slab; therefore, it was confirmed that the missing one-third of the bars did not bend into the top layer of the reinforcement, so they are assumed to be terminated. Also the core tests at the support from the top of the slab only showed Number 4 bars reinforcement at the top of the slab, and there was no evidence of bottom bars that were bent up. With this evidence the development length of the bars at the bottom of the slab was checked for a case where only two-thirds of the bars continued to the support and one-third was terminated at 3.5 ft from the support. This check confirmed the safety of the structure under this worst case configuration of bars. The development length was more than sufficient for this configuration for all four segments of the roadway. The most restrictive limit on the development length in AASHTO (17^{th} ed., 2002) states that two-thirds of the shear capacities mentioned above is required for the development length to be considered adequate. This can be a controlling limit for issuing permit. For calculations refer to Appendix-A.

By concluding that there were only Number 4 bars at 10.5 in. on center over the supports and in the negative moment region, the cross section capacity was estimated based on this small amount of reinforcement. This capacity was too small to carry any truck load in addition to its self-weight. Therefore, it was decided that the most appropriate model of the Barnes Slough Bridge is a simple span model. Some cracking at the supports was identified by ground penetration radar test which also suggested there is not enough negative moment capacity in the cross section. All of the above checks confirmed that the limit state that controlled the rating was the ultimate cross section capacity in flexure.

5.2 Model for Permit Load

The final model of this bridge is a model of one effective width with a distribution factor of one both for shear and moment in AASHTOWare. The span length is 21 ft-10 in. from center to center of supports. This span is simply supported with pinned connection on one end and a roller on the other end. The slab height is 19 in. with 1.25 in. of clear cover, the effective depth of 17.25 in. and Number 8 bars with area of 0.79 in^2 that are spaced at 4 in. on center. One-third of the bars are cut off at 3.5 ft from the center of the support. Although this cut off length does not influence the ratings factors, the slab is modeled with two-thirds of bars with full length. The asphalt is considered to be 1.75 in. as defined in AASHTOWare under girder profile menu. The concrete strength is 2500 psi and reinforcement yield strength is 33,000 psi. These parameters are shown in Figures 5-1 and 5-2 below.

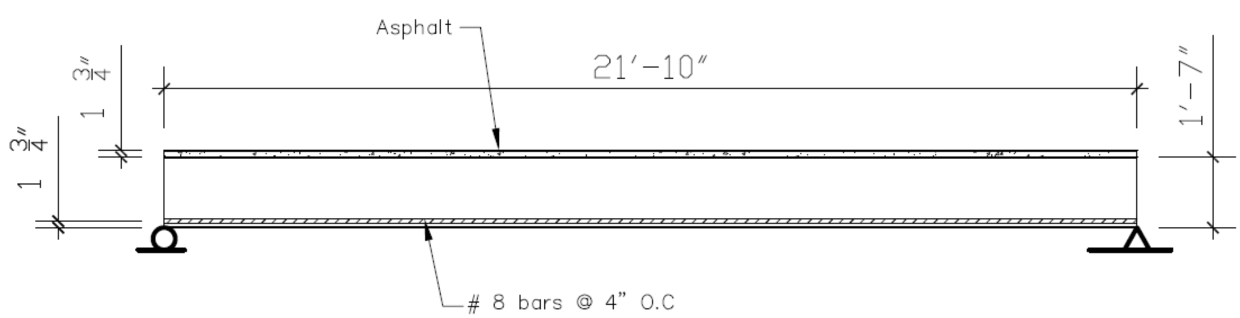


Figure 5-1: The Elevation View of the Final Model

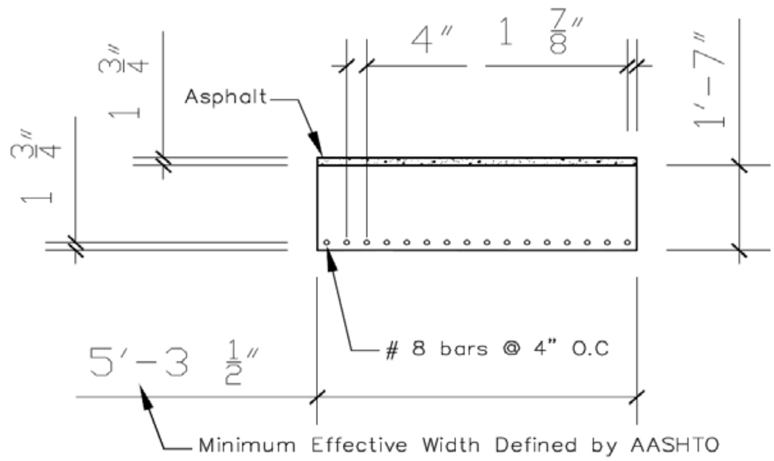


Figure 5-2: The Final Configuration of the Model as One Effective Width

One method of defining the capacity is rating of this structure such as the LFR rating. A rating is a function of dead load, live load, and the cross sectional capacity. Dead load is a function of the weight of the structure and for this exiting structure with known geometry dead load cannot be modified. Live load is a function of the truck load and boundary conditions. The best evaluation of live load is considered upon the maximum effect that the truck has on the structure through structural analysis. The cross sectional capacity is a function of effective depth, the amount of steel, material strength, and the effective width. Although the impact of the dead load and live load are not adjustable, increasing the cross sectional capacity can increase the rating of the structure. The amount of reinforcement, geometry, and the strength of the concrete were defined through field tests; hence, choosing an effective width bigger than the value defined by AASHTO was the most appropriate parameter that could incorporate additional capacity into an AASHTOWare model while keeping the measured parameters the same as those measured in the field.

In this chapter, load ratings of AASHTOware models are investigated for effective widths ranging from the value specified by AASHTO (MBE, 2011), which is the lower bound, to an upper bound of one-fourth of the roadway width. The lower bound is defined using Equation 27 in Chapter 2 and for a 21 ft-10 in. span. This formula results in an effective width of 5.31 ft, that is 63.72 in. The effective widths were incrementally increased by 4 in. until the width reached an upper bound of one-fourth of the roadway width which is 91.72 in. The reason for this upper bound comes from the geometry of the roadway and the number of traffic lanes. The current AASHTO (17^{th} ed., 2002) requires a minimum value of 12 ft as the lane width. The full road way is 31.4 ft. This width provides sufficient width for two traffic lanes allowing for two trucks simultaneously on the roadway. This configuration results in a maximum number of four wheel-lines, two wheel-lines per truck, on the road which means one-fourth of the roadway width is the maximum effective width which carries one wheel-line without overlap of the effective widths. These effective widths were rated for the LFR operating rating for all ALDOT standard trucks. First, these models are based on the original segment of the roadway. This segment is the widest segment and it has the highest capacity in the structure. A model of this segment allows for a smaller value of effective width while it is a model of an actual portion of the structure. The parameters used to model these effective widths are listed above and Figure 5-1 shows the cross section.

Table 5-1: Summary of Reinforcement for Each Effective Width of Slab in the Original Segment

Effective Width (E)(in.)	Bar Size	# of Full Length Bars	# of Bars Cut off at 3.5 ft from supports	Edge Distance (in.)	Spacing (in.)	Effective Depth (in.)
63.72	# 8	11	5	1.86	4	17.25
67.72	# 8	12	5	1.86	4	17.25
71.72	# 8	12	6	1.86	4	17.25
75.72	# 8	13	6	1.86	4	17.25
79.72	# 8	14	6	1.86	4	17.25
83.72	# 8	14	7	1.86	4	17.25
87.72	# 8	15	7	1.86	4	17.25
91.72	# 8	16	8	1.86	4	17.25

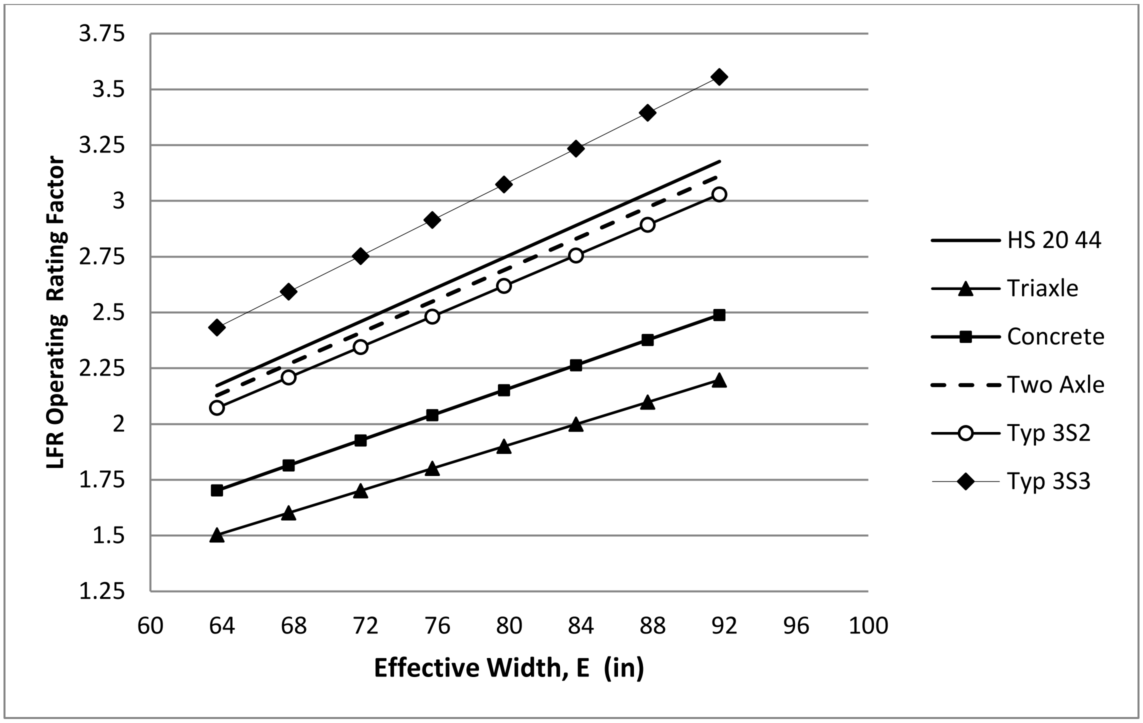


Figure 5-3: LFR Operating Rating Factor of the Original Segment of the Barnes Slough Bridge for Six of the ALDOT Standard Trucks

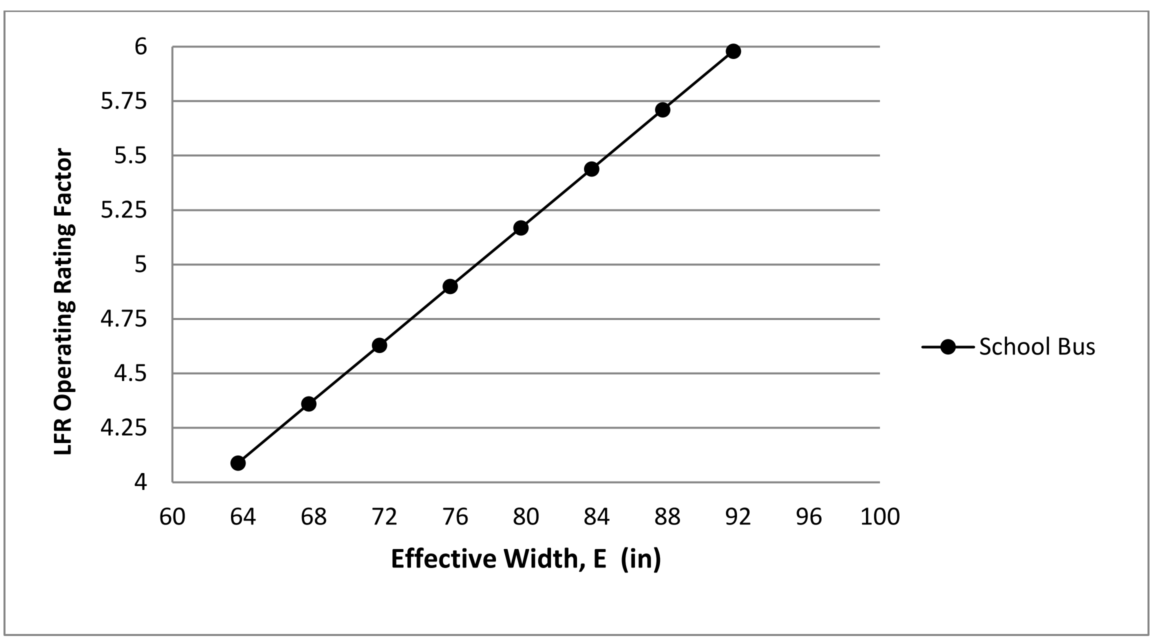


Figure 5-4: LFR Operating Rating Factor of the Original Segment of the Barnes Slough Bridge for the School Bus

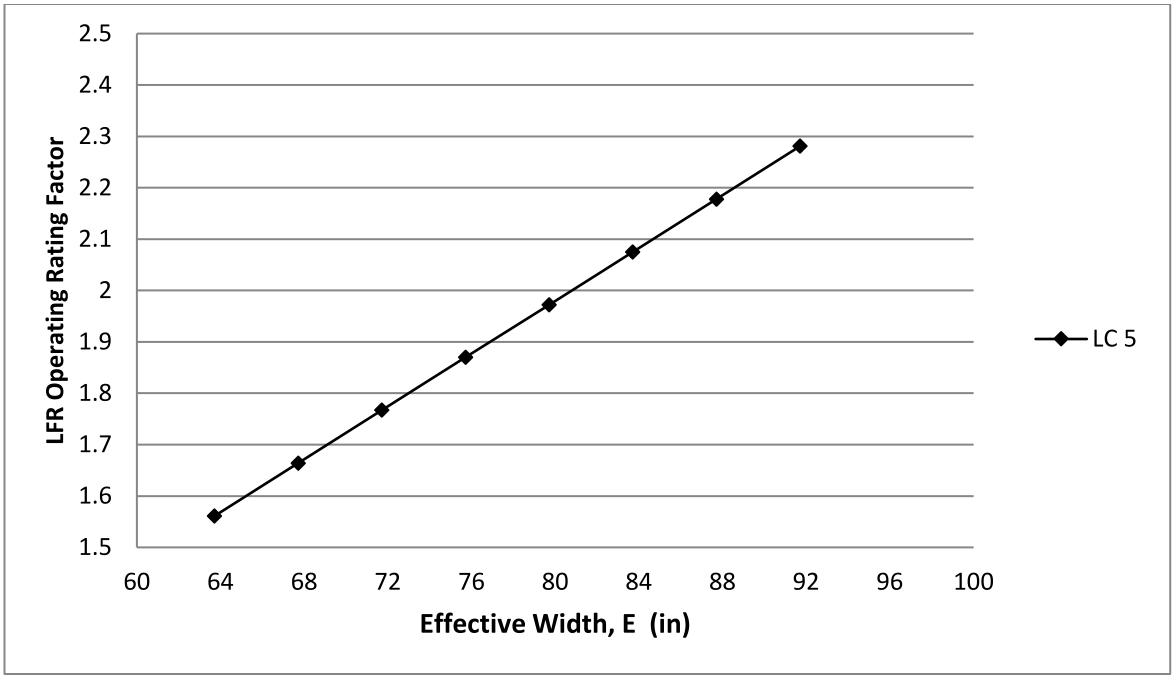


Figure 5-5: LFR Operating Rating Factor of the Original Segment of the Barnes Slough Bridge for the LC 5 Test Truck

AASHTOWare uses the number of bars in the effective width, and not the bar spacing, to calculate the capacity. For this reason, the effective width was increased in increments of the bar spacing at 4 in. Table 5-1 shows the number of bars for the model of each effective width. Figures 5-3 through 5-5 show the graphs of effective widths versus the operating rating factors for Barnes Slough Bridge.

In addition to modeling the strongest segment of the bridge and providing ratings, the weakest segment, the East segment, was also modeled to consider the smallest ratings for this bridge. This comparison allows for a better judgment of the structures' capacity and choosing an appropriate effective width as the width of the final model. Table 5-2 shows the summary of bar configuration for each effective width of slab in the East segment. Figures 5-6 through 5-8 show the operating rating factors for all ALDOT standard trucks of these effective widths.

Table 5-2: Summary of Reinforcement for Each Effective Width of Slab in the East Segment

Effective Width (E)(in.)	Bar Size	# of Full Length Bars	# of Bars Cut off at 3.5 ft from supports	Edge Distance (in.)	Spacing (in.)	Effective Depth (in.)
63.72	#8	6	3	1.9	7.5	17.25
67.72	# 8	7	3	1.9	7.5	17.25
71.72	# 8	7	4	1.9	7.5	17.25
75.72	# 8	9	3	1.9	7.5	17.25
79.72	# 8	9	4	1.9	7.5	17.25
83.72	# 8	9	5	1.9	7.5	17.25
87.72	# 8	10	5	1.9	7.5	17.25
91.72	# 8	11	5	1.9	7.5	17.25

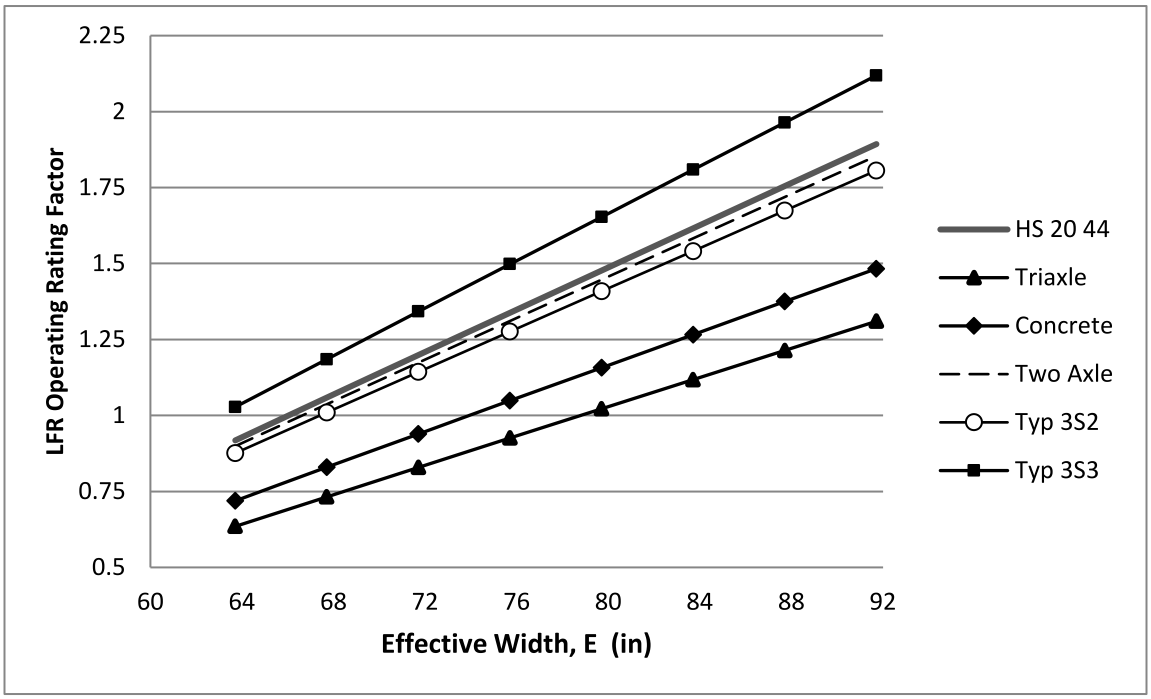


Figure 5-6: LFR Operating Rating Factor of the East Segment of the Barnes Slough Bridge for Six of the ALDOT Standard Trucks

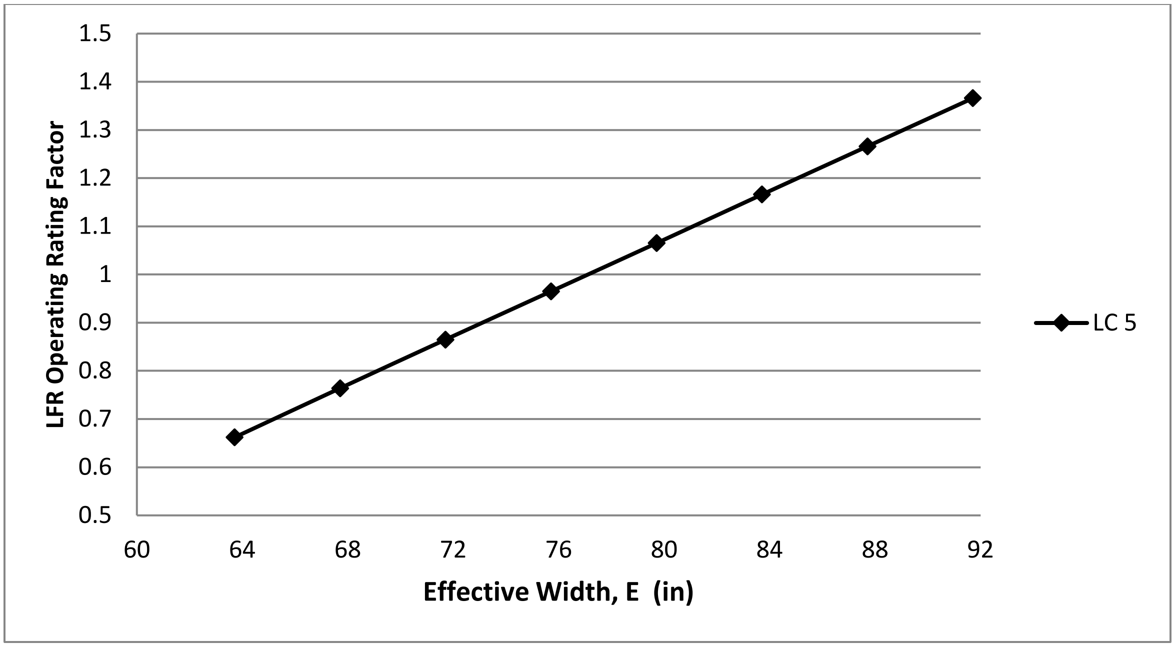


Figure 5-7: LFR Operating Rating Factor of the East Segment of the Barnes Slough Bridge for the LC 5 Test Truck

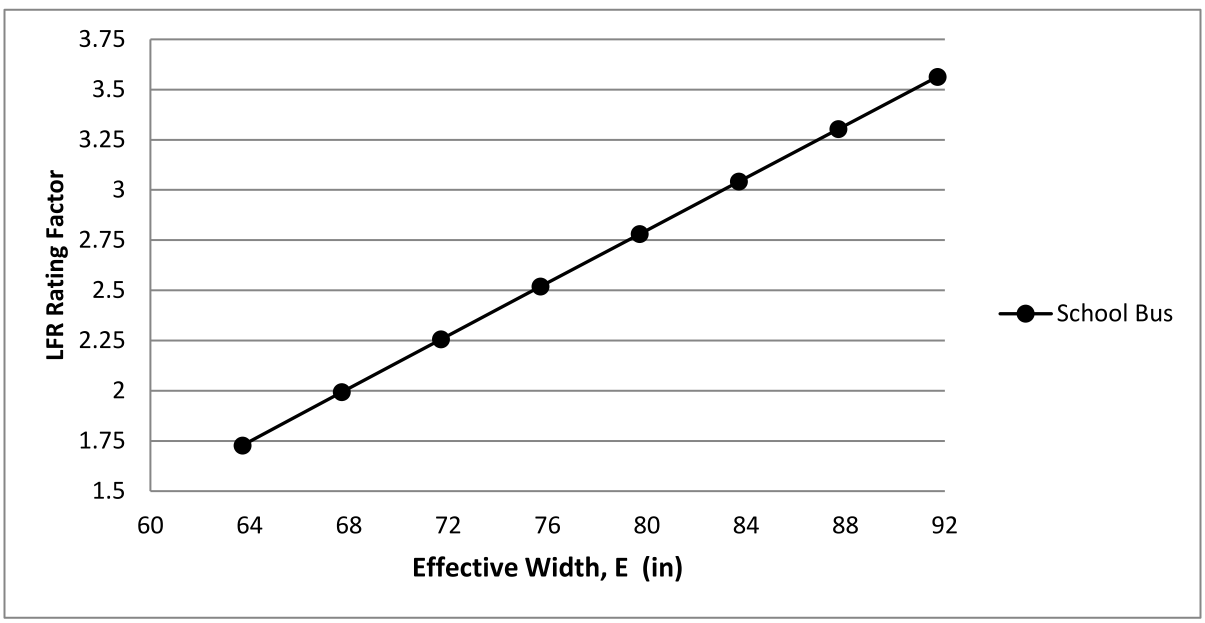


Figure 5-8: LFR Operating Rating Factor of the East Segment of the Barnes Slough Bridge for the School Bus

5.3 Application of Results

The previous section presented relationships between affective slab width and the rating factor of the model determined by ASHTOWare. These relationships provide a means of linking the live load tests and finite element analysis (reported by others) to an AASHTOWare model. For example the finite element based ratings of this structure for different trucks maybe used to select an appropriate effective width for the ALDOT standard trucks. If the rating factor determined by the finite element analysis is 2 for the triaxle truck, Figure 5-3 shows that an effective width of 84 in. should be used in AASHTOWare to have the same rating factor for the permit model of Figure 5-1.

6 Conclusions and Recommendations

6.1 Summary of Findings and Conclusions

The purpose of this research was to develop an AASHTOWare model of a flat slab reinforced concrete bridge that could be used for issuing permits to overweight trucks. This bridge has 11 spans with a total length of 245 ft and crosses Jenkins Creek and Barnes Slough on the northbound side of US Highway 82/231 at milepost 162.56. ALDOT's "Bridge Card" for the structure indicates that the bridge was widened by approximately 4 ft in 1930, and the visual inspection of the bridge indicated that the width was increased twice on the east side of the bridge and once to the west side. Chapters 1 through 4 established the characteristics and the load ratings of this bridge and suggested the use of a model of one effective width in AASHTOWare.

In Chapter 1 a search of historical documents was performed. By using the Auburn University Library resources and the Alabama Department of Archives and History in Montgomery, the construction year was established as 1915 from a report of the state of Alabama Highway Commission (State Highway Commission of Alabama 1916). These findings were confirmed by a comparison made between maps from 1920 and 1992. The sequence and time which the additional segments were built remained unknown, although the studies from Chapter 4 suggest that these additional segments were added between 1941 and 1959 based on the amount reinforcement present in the slab.

In Chapter 2, a literature review was performed in an aim to understand the methods used for designing flat slab bridges throughout the 20th century.

Chapter 3 illustrated applications of the methods learned from Chapter 2 to calculate the reinforcement in the slab, and introduced the concept of rating and the AASHTOWare software used to produce the ratings by DOTs. Two case studies from the 1920s were used to apply these methods to establish a single method used to design the original segment from the contemporary time. From these case studies it was concluded that although the methods used to design the reinforcement in the slab was established, the concept that defined the effective width remained unknown. The value of effective width was calculated by inversing the ratio of capacity deducted by dead load effect divided by live load effect plus impact. Through this study, not only it was concluded that the effective width was calculated independent of span length or roadway widths, but that in 1920s the designers used the clear span for simple spans bridges to design the reinforcement in the slab. For two-span continuous the center to center of the supports was used for design. From these studies there was enough data generated to consider the effective width value to be 4.15 ft for all designs. Additionally, all the operating ratings for the simple span bridges showed that the contemporary methods used to design the flat slab bridges provided adequate cross section capacity for all ALDOT trucks. Thus there would be no need for postings on these bridges if any still exists. For two-span continuous bridge, the reinforcement in the negative moment region was not developed properly, which resulted in lower rating values than 100% of the truck load, and there would be a need for postings for ALDOT trucks.

In Chapter 4 the concepts learned from Chapter 2 and 3 were applied to estimate the amount of reinforcement needed in all four segments of Barnes Slough Bridge. After the initial calculations and estimating the amount steel and capacity of the bridge, some field tests and measurements were done. Based on field measurements the bar size in all segments was changed to one size smaller than those predicted. Additionally, the clear cover was different than that used in the standard simple span bridges (SHDA, 1922). From the concrete core tests, the strength of the concrete was identified. The concrete strength was less than assumed earlier, and this resulted in a different cross section capacity of the slab. The amount of reinforcement detected in the positive moment region was more than the estimated amount, and the reinforcement in the negative moment region was far less than the expected amount. Some cracks were detected by the GPR devices over the supports. From these findings it was concluded that the bridge was reinforced as if it is a series of simple spans. This observation resulted in narrowing down the focus of research to one type of structure.

In Chapter 5 one model was suggested in lieu of one model for each of the four different segments and a series of ratings were reported for different effective widths. This final model is a model of one effective width that carries one wheel-line of a given truck. The characteristics of this model are reported in Chapter 5. A model of the weakest segment of the bridge, the East segment, was created to provide the lower bound of the ratings for this structure for consideration when choosing an appropriate effective width for the final model. The results from these ratings are reported in series of figures in Chapter 5. ALDOT can use these ratings to define their final rating of this bridge upon choosing one final effective width. Furthermore, ALDOT can use this information to issue permits for non-standard trucks to travel over this structure. Currently the bridge carries unrestricted traffic and this research confirmed that there may be a need for posting on this bridge upon choosing an effective width if the rating is based on the East segment of the cross section. Serviceability was not of concern in this research, and shear requirements were met; thus, the limit state which controlled the capacity was flexure.

6.2 Recommendations

Although the performance of this bridge showed significant capacity, the age of this structure is of concern, and it needs to be well maintained. Some concrete has spalled near the supports and some reinforcement is exposed in the East and West segments. These spalls need maintenance to stop and prevent corrosion in the bars. It is recommended to inspect the footings every year to avoid scouring. Currently there are voids beneath two footings on the East.

The cross section of the Barnes Slough Bridge has four different segments with different characteristics. This research has aimed to provide one model that represents this structure as one whole. There are no significant visual signs of distress in the slab other than the lack of capacity in the negative moment region. The lack of documentation makes it difficult to assess what material properties and methods were used during construction. It is also unknown whether quality control testing was used. For these reasons it was not possible to analyze the internal stresses between these segments when under load. Observations that could be made were limited to measuring the strains and deflections during non-destructive live load testing. It is unknown whether these strains and deflections were caused due to poor bonding between segments, if they were a sign of deflection under the load alone, or some other cause.

The Barnes Slough Bridge has been carrying unrestricted loads. The absence of significant signs of stress provides evidence that it is capable of carrying normal traffic. However, due to the uncertainties associated with its age, it is recommended to avoid issuing permits for trucks heavier than ALDOT standard trucks. It is recommended that the final model of the Barnes Slough Bridge is a model of an effective width which corresponds to a rating factor of one for the heaviest ALDOT truck in the Original segment.

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8 List of Abbreviations

A_s	Area of Steel
ALDOT	Alabama Department of Transportation
AASHTO	American Association of State Highway and Transportation Officials
B	Distance between the parallel loaded elements
β_b	Ratio of the area of reinforcement cut off to the total area of tension reinforcement at the section
b_d	Bar diameter
b_w	The width which the shear is considered
C_c	Clear cover
C_I	Notation for "Coefficient of Impact"
C_r	Distance between centers of wheels
D	Distance in ft from the center of the nearer support to the venter of wheel
d	Distance from the top of the beam down to the center of the reinforcement
E	Design effective width
E_I	Effective width of road in inner zone
E_O	Effective width of road in outer zone
E'	Ratio of live load of a wheel line to the maximum moment due to live load
Es	Modulus of elasticity of steel
E_c	Modulus of elasticity of concrete
F	Resultant compressive force in the concrete and also resultant tension in the reinforcement

f_s	Strength of steel
f_c	Strength of concrete
fy	Ultimate strength of steel
H_{number}	Truck with the number of ton as its weight
h	Slab depth
j	A multiplier of d that measures the distance between the center of steel and the center of compression in the cross section
jd	Distance from the center of the reinforcement to the resulted compressive force in the concrete
k	Effective length factor for compression members
kd	Distance from the extreme compression fiber to the neutral axis
L	Length of span
L_d	Required development length of reinforcement
L_a	Embedment length of reinforcement
LLM	Live load moment
M	Bending moment
N	Number of traffic lane on the road
n	Modular Ratio
ρ	Ratio of steel to the area of effective cross section
P	It is most often the force. Refer to the definition of notation on the page Load on one wheel
P’	Concentrated lane load per lane
P_s	Strength of steel

P_c	Strength of concrete
Q	Uniform lane load per linear foot of lane
S	Span of Slab
s	Spacing of shear reinforcement
V_c	Concrete shear strength
W	Width of the wheel or tire in ft
W	Width of graded roadway across culverts
Wr	Width of roadway between curbs on bridge
W_r	Modified edge to edge width
W_P	Physical edge to edge width of bridge
W_n	Width of design traffic lane
W_C	Roadway width between curbs exclusive of median strip
X	Number of vehicles traveling
Y	Roadway width in ft
DOT	Department of Transportation
DL	Dead load
ft	feet
in.	Inches
in^2	Inches squared
lb	Pound
lbs	Pounds
LL	Live load
Ksi	Kips per square in.

psi	Pound per square in.
STD	Standard

Appendix A: Sample Calculations

The aim of this section is to provide at least one sample calculation for all methods used. Not all calculations are presented here.

A-1 Contemporary methods to calculate the reinforcement in the slab

In Appendix A-1 a series of sample calculations are presented to show the Contemporary method of calculating reinforcement in the slab. Next, the OFOR method is presented, and lastly, the GA DOT method is presented. These methods are applicable to both simple spans and continuous spans.

A-1-1 Contemporary Method:

Reinforcement for Negative moment region of Fayette Co. Bridge- H15 Truck

$S := 20ft$

$Roadway := 16ft$

$H := 17in$

$d := 15in$

$f_s := 16000psi$

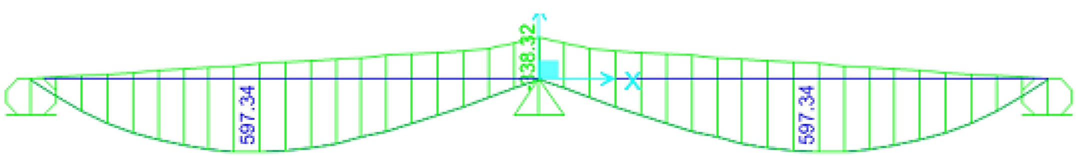
$f_c := 650psi$

$n := 15$

$I := 0.3$ (Eq-12)

$M_{LL} := 338.320kip \cdot in$

The moment envelope for one wheel line of truck from Structural analysis, SAP2000



$M_{LLI} := M_{LL} \cdot I + M_{LL} = 36.651\, kip \cdot ft$

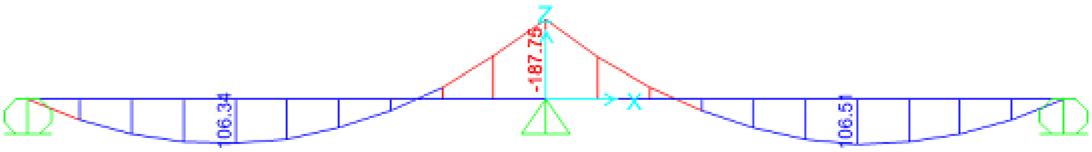
E= 4.15 ft

Value of Effective Width concluded from Chapter 3

$$M_L := \frac{M_{LLI}}{\frac{E}{ft}} = 8.832\, kip \cdot ft$$

$M_{DL} := 15.65kip \cdot ft$

From Structural analysis, SAP2000 for the ht of 17 in.



$$M_{des} := M_{DL} + M_L = 24.482\,kip \cdot ft$$

$$k := \frac{n \cdot f_c}{f_s + n \cdot f_c} = 0.379 \qquad \text{(Eq-6)}$$

$$j := 1 - \left(\frac{1}{3}\right) \cdot k = 0.874 \qquad \text{(Eq-9)}$$

$$F := \frac{M_{des}}{j \cdot d} = 22.414\,kip \qquad \text{(Eq-1)}$$

$$f'c := \frac{2 \cdot F}{k \cdot d \cdot b_w} = 658 \cdot psi \qquad \text{(Eq-3)}$$

$$A_s := \frac{F}{f_s} = 1.401\,in^2 \qquad \text{(Eq-10)}$$

Using a bar size Number 7 as defined by SHDA (1924)

$$A_b := .6in^2$$

$$bar_{numbers} := \frac{A_s}{A_b} = 2.335$$

$$Spacing := \frac{12}{bar_{numbers}} = 5.14$$

$$Space := floor(Spacing) \cdot in = 5\,in$$

$$\boxed{Space = 5\,in}$$

A-1-2 OFOR Method:

Reinforcement for Negative moment region of Fayette Co. Bridge- H15 Truck

$H := 17in$
$S := 20ft$

$d := 15in$

$n := 15$

$I := 0.3$ (Eq-12)

$Roadway := 16ft$

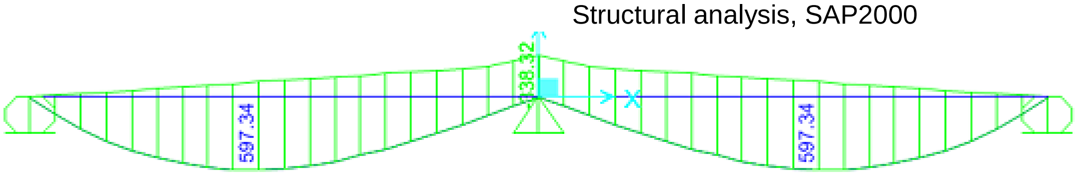
$f_s := 16000psi$

E = 7.85 ft

Value of Effective Width concluded from Chapter 3

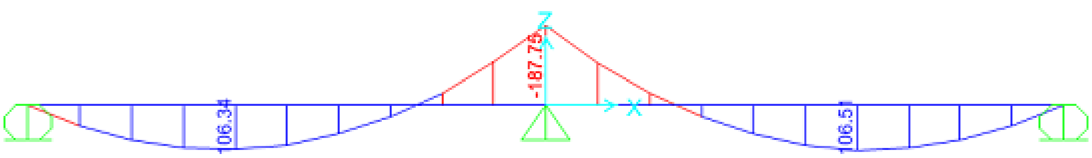
$M_{LL} := 338.320 kip \cdot in$

The moment envelope for one wheel line of truck from Structural analysis, SAP2000



$M_{DL} := 15.65 kip \cdot ft$

From Structural analysis, SAP2000 for the ht of 17 in.



$$M_{total} := M_{DL} \cdot \frac{Roadway}{ft} + M_{LL} \cdot 2 \cdot (1 + I) \cdot 2 = 397.005 \, kip \cdot ft$$

$$M_{des} := \frac{M_{total}}{\frac{Roadway}{ft}} = 24.813 \, kip \cdot ft$$

$$k := \frac{n \cdot f_c}{f_s + n \cdot f_c} = 0.379$$

$$F := \frac{M_{des}}{j \cdot d} = 22.718\,kip \qquad \text{(Eq-1)}$$

$$A_s := \frac{F}{f_s} = 1.42\,in^2 \qquad \text{(Eq-10)}$$

Using a bar size Number 7 as defined by SHDA (1924)

$$A_b := .6in^2$$

$$bar_{numbers} := \frac{A_s}{A_b} = 2.366$$

$$Spacing := \frac{12}{bar_{numbers}} = 5.071$$

$$Space := \text{floor}(Spacing) \cdot in = 5\,in$$

$$\boxed{Space = 5\,in}$$

A-1-3 GA DOT Method:

Reinforcement for Negative moment region of Fayette Co. Bridge- H15 Truck

$S := 20\ ft$ $\quad$ $n := 15$

$Roadway := 16ft$ $\quad$ $f_s := 16000psi$

$H := 17in$ $\quad$ $f_c := 650psi$

$d := 15in$

$M_{LLI} := M_{LL} \cdot I + M_{LL} = 36.651 \cdot kip \cdot ft$

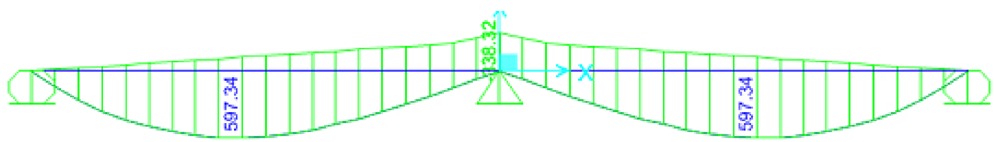
$E := 4.15ft$ $\quad$ Value concluded from Chapter 3

$I := 0.3$ $\quad$ (Eq-12)

$B := 3ft$ $\quad$ Spacing between the two adjacent truck's axles

From Structural analysis, SAP2000

$M_{LL} := 338.320kip \cdot in$



$$M_{LE} := \frac{M_{LLI}}{\frac{E}{ft}} = 8.832 \cdot kip \cdot ft$$

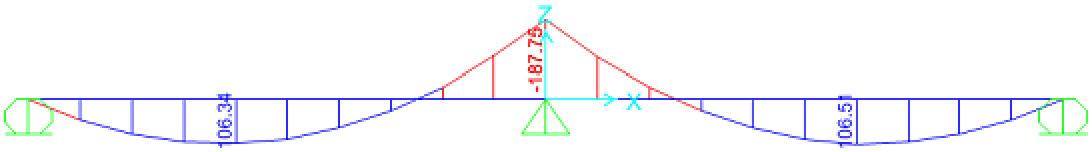
$Ratio := \frac{B}{S} = 0.15$ $\quad$ Interpolate between Eq-18-2 & Eq-18-3

$X1 := 0.1$ $\quad$ $X2 := Ratio$ $\quad$ $X3 := .4$

$Y1 := 60$ $\quad$ $Y3 := 30$

$$Y2 := \left[Y1 + \frac{(X2 - X1) \cdot (Y3 - Y1)}{X3 - X1}\right] = 55 \qquad \text{(Eq-18)}$$

$$M_L := M_{LE} + M_{LE} \cdot \frac{Y2}{100} = 13.689\, kip \cdot ft$$



$M_{DL} := 15.65\,kip \cdot ft$ From Structural analysis, SAP2000

$M_{des} := M_{DL} + M_L = 29.339 \cdot kip \cdot ft$

$$k := \frac{n \cdot f_c}{f_s + n \cdot f_c} = 0.379 \qquad \text{(Eq-6)}$$

$$j := 1 - \left(\frac{1}{3}\right) \cdot k = 0.874 \qquad \text{(Eq-9)}$$

$$F := \frac{M_{des}}{j \cdot d} = 26.862\,kip \qquad \text{(Eq-1)}$$

$$A_s := \frac{F}{f_s} = 1.679\,in^2 \qquad \text{(Eq-10)}$$

Using a bar size Number 7 as defined by SHDA (1924)

$A_b := .6\ in^2$

$$bar_{numbers} := \frac{A_s}{A_b} = 2.798$$

$$Spacing := \frac{12}{bar_{numbers}} = 4.289$$

$$\boxed{Space := floor(Spacing) \cdot in = 4\,in}$$

A-2 Back-Calculations to Isolate the Effective Width

This section was used in Chapter 3 to back-calculate the values used as effective width. The area of steel from the SHDA (1922) was used to estimate the capacity. The dead load and live load effects were known; therefore, the effective width was isolated. The back calculations were done for a few methods. Below the contemporary method is reversed.

A-2-1 Back-Calculations Based on Contemporary Method

Reversed Calculations for a Case of 20-ft Span with 16-ft Roadway

$S := 20\text{ft}$ Roadway $:= 16\text{ft}$

$H := 19\text{in}$ $f_s := 16000\text{psi}$

$d := 18\text{in}$ $f_c := 650\text{psi}$

$n := 15$ $b_w := 12\text{in}$ 1 foot of width is considered (Eq-12)

$I := 0.3$

$Conc_{wt} := 150\frac{\text{lbf}}{\text{ft}}$ Concrete weight

$$k := \frac{n \cdot f_c}{f_s + n \cdot f_c} = 0.379$$

$$j := 1 - \left(\frac{1}{3}\right) \cdot k = 0.874$$

$P := 12000\text{lbf}$ Load due to one back wheel-line of H15 Truck

$$V_{wheelline} := \frac{P}{2} = 6\,\text{kip}$$

$$M_{LL} := V_{wheelline} \cdot \frac{S}{2} = 60\,\text{kip}\cdot\text{ft} \quad \text{(Eq-6)}$$

$$M_{LLI} := M_{LL} \cdot I + M_{LL} = 78 \cdot \text{kip}\cdot\text{ft} \quad \text{(Eq-9)}$$

$M_s := f_s \cdot A_s \cdot j \cdot d = 35.4\,\text{kip} \cdot \text{ft}$

Moment Capacity due to Steel

$M_c := .5 \cdot f_c \cdot j \cdot k \cdot b_w \cdot d^2 = 34.8\,\text{kip} \cdot \text{ft}$

$M_{cap} := \min(M_s, M_c) = 34.8\,\text{kip} \cdot \text{ft}$

Moment Capacity due to concrete

$DL := H \cdot Conc_{wt} + 80\text{lbf} = 0.317\,\text{kip}$

The SHDA (1922) has specified an additional 80 psf as part of the dead load

$M_{DL} := \frac{DL \cdot S^2}{8\text{ft}} = 190.5\,\text{kip} \cdot \text{in}$

$M_L := M_{cap} - M_{DL} = 18.964\,\text{kip} \cdot \text{ft}$

Used the concept from Eq-40 and deducted the moment due to dead load from the moment capacity to isolate the design live load

$E_{Ratio} := \frac{M_L}{M_{LLI}} = 0.243$

This is the inverse of (Eq-40)

$E := \frac{1\text{ft}}{E_{Ratio}} = 4.11\,\text{ft}$

$\boxed{E = 4.11\,\text{ft}}$

A-2-2- Back-Calculations Based on LRFD Method

$$E = 84.0 + 1.44\sqrt{L_1 W_1} \;\le\; \frac{12.0W}{N_l}$$

$L \le 60$ Span in ft

W.1 = Modified edge to edge W equal or lesser than 60 for multiple lane, and 30 for one lane

W = Physical edge to edge W of bridge

N = Number of design lanes as specified in AASHTO LRFD 2014, Article 3.6.1.1.1

$\text{roadway} := 16\text{ft}$

$L_1 := 20\text{ft}$

$W_1 := 16\text{ft}$

$W := 16\text{ft}$

$N_1 := 2$

$$E_1 := 84\text{in} + \left(\frac{1.44}{12}\right)\cdot\sqrt{L_1 \cdot W_1} = 9.15\,\text{ft}$$

$$E_2 := \frac{12 \cdot W}{N_1} = 96\,\text{ft}$$

$$E_{tot} := \min\left(E_1, E_2\right) = 9.15\,\text{ft}$$

$$E := \frac{E_{tot}}{2} = 4.57\,\text{ft}$$

A-3 Sample Calculations for All Decades

A-3-1 1931 Method for H15 Truck Loading

$S := 21ft + 10in$ $\quad h := 19in$ $\quad$ Slab thickness

$W := 1.25ft$ $\quad CC := 1.25in$ $\quad A_{ClassConcrete} := 3000psi$

$b_d := \frac{6}{8}in$ $\quad f_{ys} := 16000psi$ $\quad f_c := 3000psi$ At the end the fc has to be less than 1/3 of f`c

$d := h - CC - \frac{b_d}{2} = 17.375 \cdot in$ $\quad N := 2$

C = 3 ft

$I := \frac{50ft}{S + 125ft} = 0.341$ (Eq-12)

$E_s := 30000ksi$

$E := .7 \cdot S + W = 16.533 \cdot ft$ (Eq-14)

$$E := \begin{cases} E & \text{if } E < 7ft \\ (7ft) & \text{otherwise} \end{cases}$$

E =7 ft

$E_{overlapped} := .5 \cdot (E + C) = 6.5 \cdot ft$ (Eq-15)

$M_{LL} := 65.49 kip \cdot ft$

$M_{DL} := 16.43 kip \cdot ft$

$M_I := I \cdot M_{LL} = 22.301 \cdot kip \cdot ft$

$M_L := M_I + M_{LL} = 87.791 \cdot kip \cdot ft$

$M_{des1} := \frac{(M_L)}{\frac{E}{2ft}} + M_{DL} = 41.516 \cdot kip \cdot ft$ (Eq-39)

Reinforcement in Tension only:

$d = 17.375 \cdot in$

$b := 12in$

$j_1 := .883$

Start with an estimate, suggested 7/8 in. and reiterate

$$A_{s1} := \frac{M_{des1}}{f_{ys} \cdot j_1 \cdot d} = 2.044 \cdot in^2 \qquad \text{(Eq-11)}$$

$$\rho_1 := \frac{A_{s1}}{b \cdot d} = 9.875 \times 10^{-3} \qquad \text{(Eq-8)}$$

$$k_1 := \sqrt{2 \cdot \rho_1 \cdot n + (\rho_1 \cdot n)^2} - \rho_1 \cdot n = 0.351$$

Position of Neutral axis (Eq-7)

$$j_1 := 1 - \frac{k_1}{3} = 0.883$$

Arm of resisting couple (Eq-9)

$$f_c := \frac{2 \cdot M_{des1}}{j_1 \cdot k_1 \cdot b \cdot d^2} = 0.9 \cdot ksi \qquad \text{(Eq-4)}$$

$$f_c := \frac{6 \cdot M_{des1}}{b \cdot d^2} = 0.837 \cdot ksi$$

If it is less 1/3 of the assumed value, OK

Choose bar size:

$A_{size} := 8$

$A_b := .79in^2$

Spacing:

$$N_{b1} := \frac{A_{s1}}{A_b} = 2.588$$

Number of bars in one foot

$$Spacing := \frac{12in}{N_{b1}} = 4.6 \cdot in$$

Reinforcement for a Case of Over Lapped Effective Width:

$$M_{des2} := \frac{(M_L)}{\frac{E_{overlapped}}{2ft}} + M_{DL} = 43.4 \cdot kip \cdot ft \qquad \text{(Eq-39)}$$

$j_2 := .884$

$$A_{s2} := \frac{M_{des2}}{f_{ys} \cdot j_2 \cdot d} = 2.1 \cdot in^2 \qquad \text{(Eq-11)}$$

$$\rho_2 := \frac{A_{s2}}{b \cdot d} = 0.01$$ (Eq-8)

$$k_2 := \sqrt{2 \cdot \rho_2 \cdot n + (\rho_2 \cdot n)^2} - \rho_2 \cdot n = 0.357$$ Position of Neutral axis (Eq-7)

$$j_2 := 1 - \frac{k_2}{3} = 0.881$$ Arm of resisting couple (Eq-9)

$$f_{c2} := \frac{2 \cdot M_{des2}}{j_2 \cdot k_2 \cdot b \cdot d^2} = 0.928 \cdot ksi$$ (Eq-4)

$$f_{c2} := \frac{6 \cdot M_{des2}}{b \cdot d^2} = 0.876 \cdot ksi$$

Spacing:

$$N_{b2} := \frac{A_{s2}}{A_b} = 2.705$$

$$\text{Spacing2} := \frac{12in}{N_{b2}} = 4.4 \cdot in$$

Spacing = 4 in. for a case of overlapped effective width

A-3-2 1935 Method for H20 Truck Loading

$h := 19in$ Slab thickness

$S := 21ft + 10in$

$W := 1.25ft$ $Cc := 1.25in$

$b_d := \frac{8}{8}in$ $rdwy_W := 16ft$

$f_{ys} := 16000psi$

$$d := h - Cc - \frac{b_d}{2} = 17.25 \cdot in$$

$E := .6 \cdot S + 2 \cdot W = 15.6 \cdot ft$ Effective width of slab for one wheel load

$I := \frac{50ft}{S + 125ft} = 0.341$ (Eq-12)

$E_s := 30000ksi$

Reinforcement in Tension only:

$d = 17.25 \cdot in$

$b := 12in$

$j := .908$ Start with an estimate, 7/8 in, then iterate

$M_{LL} := 87.33kip \cdot ft$ $M_{DL} := 16.43kip \cdot ft$

$M_I := I \cdot M_{LL} = 29.738 \cdot kip \cdot ft$

$$M_L := \frac{(M_I + M_{LL})}{\frac{E}{ft}} = 7.504 \cdot kip \cdot ft$$

$$M_{des} := (M_L + M_{DL}) = 23.934 \cdot kip \cdot ft \quad \text{(Eq-39)}$$

$$A_s := \frac{M_{des}}{f_{ys} \cdot j \cdot d} = 1.146 \cdot in^2 \quad \text{(Eq-11)}$$

$$\rho := \frac{A_s}{b \cdot d} = 5.537 \times 10^{-3} \quad \text{(Eq-8)}$$

$$k := \sqrt{2 \cdot \rho \cdot n + (\rho \cdot n)^2} - \rho \cdot n = 0.277$$ Position of Neutral axis (Eq-7)

$$j := 1 - \frac{k}{3} = 0.908$$

(Eq-9)

$$f_c := \frac{2 \cdot M_{des}}{j \cdot k \cdot b \cdot d^2} = 0.639 \cdot ksi$$

(Eq-4)

$$f_c := \frac{6 \cdot M_{des}}{b \cdot d^2} = 0.483 \cdot ksi$$

If it is less than 1/3 the assumed value, OK

Choose bar size:

$$A_{size} := 8$$

$$A_b := .79 in^2$$

$$N_b := \frac{A_s}{A_b} = 1.451$$

$$Spacing := \frac{12 in}{N_b} = 8.27 \cdot in$$

$$\boxed{\text{Space} = 8.25 \text{ in}}$$

A-3-3 1941 Method for H15 Truck Loading

$S := 21ft + 10in$ $\quad h := 19in$ $\quad f_c := 3000psi$

$W := 1.25ft$ $\quad C := 1.25in$

$b_d := \frac{8}{8}in$ $\quad f_{ys} := 18000psi$ $\quad N := 2$

$d := h - C - \frac{b_d}{2} = 17.25 \cdot in$

$$I := \frac{50ft}{S + 125ft} = 0.341 \qquad \text{(Eq-12)}$$

$E_s := 30000ksi$

$Q := 640lbf$ — Uniform Lane Loading - 1941

$P := 16kip$ — This is the axle weight from the H20-S16-44 Truck configuration

$w := 30ft$

$P' := 32kip$

$$E_{Wheel} := \frac{10ft \cdot N + w}{4 \cdot N} = 6.25 \cdot ft \qquad \text{(Eq-22)}$$

$$M_{uniform} := \frac{N \cdot Q \cdot ft^2}{(.5 \cdot w + 5ft \cdot N)} = 0.051 \cdot kip \cdot ft \qquad \text{(Eq-23)}$$

$$M_{Concentrated} := \frac{N \cdot P' \cdot ft^2}{.5 \cdot w + 5ft \cdot N} = 2.56 \cdot kip \cdot ft \qquad \text{(Eq-24)}$$

$$M_{Lane} := M_{uniform} + M_{Concentrated} = 2.611 \cdot kip \cdot ft$$

$$M_{des1} := M_{Lane} \cdot \frac{E_{Wheel}}{ft} = 16.32 \cdot kip \cdot ft$$

$M_{LL} := 65.497kip \cdot ft$ — For spans less than 40 ft, H-S Truck loading is used to cause moment - 1941

$M_{DL} := 16.43kip \cdot ft$

$M_I := I \cdot M_{LL} = 22.303 \cdot kip \cdot ft$

$$M_L := \frac{M_I + M_{LL}}{\frac{E_{Wheel}}{ft}} = 14.048 \cdot kip \cdot ft$$

$M_{des2} := (M_L + M_{DL}) = 30.478 \cdot kip \cdot ft$

$M_{des} := \max(M_{des1}, M_{des2}) = 30.478 \cdot kip \cdot ft$

Reinforcement in Tension only:

$j := .897$

$d = 17.25 \cdot in$

$b := 12in$

$$A_s := \frac{M_{des}}{f_{ys} \cdot j \cdot d} = 1.313 \cdot in^2 \qquad \text{(Eq-11)}$$

$$\rho := \frac{A_s}{b \cdot d} = 6.344 \times 10^{-3} \qquad \text{(Eq-8)}$$

$k := \sqrt{2 \cdot \rho \cdot n + (\rho \cdot n)^2} - \rho \cdot n = 0.293$ Position of Neutral axis

$j := 1 - \frac{k}{3} = 0.902$ Arm of resisting couple

$$f_{ca} := \frac{2 \cdot M_{des}}{j \cdot k \cdot b \cdot d^2} = 0.774 \cdot ksi$$

$f_{ca} := \frac{6 \cdot M_{des}}{b \cdot d^2} = 0.615 \cdot ksi$ If it is less 1/3 of the assumed value, OK

Choose bar size:

$A_{size} := 8$

$A_b := .79in^2$

Spacing:

$$N_b := \frac{A_s}{A_b} = 1.662$$

$$Spacing := \frac{12in}{N_b} = 7.22 \cdot in$$

$\boxed{\text{Space = 7 in}}$

A-3-4 1949 Method for H15 Truck Loading

$S := 21ft + 10in$ $h := 19in$ $f_c := 3000psi$ At the end the concrete compression stress has to be less than 1/3 of f `c

$W := 1.25ft$ $Cc := 1.25in$

$b_d := \frac{8}{8}in$ $f_{ys} := 18000psi$

$d := h - Cc - \frac{b_d}{2} = 17.25 \cdot in$ $N := 2$

$$I := \frac{50ft}{S + 125ft} = 0.341 \quad \text{(Eq-12)}$$

$$E := \frac{10ft \cdot N + W}{4 \cdot N} = 2.656 \cdot ft \quad \text{(Eq-22)}$$

$E_s := 30000ksi$

$E_c := 57000 \cdot \sqrt{f_c \cdot psi} = 3.122 \times 10^3 \cdot ksi$

$Q := 640lbf$ Uniform Lane Loading - 1949

$P := 16kip$ This is the axle weight from the H20-S16-44 Truck configuration

$w := 30ft$

$P' := 18kip$ This is the concentrated loading from Lane load - 1949 was 18 k

$$E_{Wheel} := \frac{10ft \cdot N + w}{4 \cdot N} = 6.25 \cdot ft$$

$$M_{uniform} := \frac{N \cdot Q \cdot ft^2}{(.5 \cdot w + 5ft \cdot N)} = 0.051 \cdot kip \cdot ft \quad \text{(Eq-23)}$$

$$M_{Concentrated} := \frac{N \cdot P' \cdot ft^2}{.5 \cdot w + 5ft \cdot N} = 1.44 \cdot kip \cdot ft \quad \text{(Eq-24)}$$

$$M_{Lane} := M_{uniform} + M_{Concentrated} = 1.491 \cdot kip \cdot ft$$

$$M_{des1} := M_{Lane} \cdot \frac{E_{Wheel}}{ft} = 9.32 \cdot kip \cdot ft$$

$M_{LL} := 65.49kip \cdot ft$

$M_{DL} := 16.43kip \cdot ft$

$M_I := I \cdot M_{LL} = 22.301 \cdot kip \cdot ft$

$$M_L := \frac{M_I + M_{LL}}{\frac{E_{Wheel}}{ft}} = 14.047 \cdot kip \cdot ft$$

$$M_{des2} := \left(M_L + M_{DL}\right) = 30.477 \cdot kip \cdot ft$$

$$M_{des} := \max\left(M_{des1}, M_{des2}\right) = 30.477 \cdot kip \cdot ft$$

$$d = 17.25 \cdot in$$

$$b := 12in$$

$$j := .902$$

Start with an estimate, suggested 7/8 in, and reiterate

$$A_s := \frac{M_{des}}{f_{ys} \cdot j \cdot d} = 1.306 \cdot in^2$$

A s = Area of steel (Eq-11)

$$\rho := \frac{A_s}{b \cdot d} = 6.308 \times 10^{-3}$$

(Eq-8)

$$k := \sqrt{2 \cdot \rho \cdot n + (\rho \cdot n)^2} - \rho \cdot n = 0.293$$

Position of Neutral axis (Eq-7)

$$j := 1 - \frac{k}{3} = 0.902$$

Arm of resisting couple (Eq-9)

$$f_{ca} := \frac{2 \cdot M_{des}}{j \cdot k \cdot b \cdot d^2} = 0.775 \cdot ksi$$

(Eq-4)

$$f_{ca} := \frac{6 \cdot M_{des}}{b \cdot d^2} = 0.615 \cdot ksi$$

If it is less the assumed value, OK

Choose bar size:

$$A_{size} := 8$$

$$A_b := .79in^2$$

Spacing:

$$N_b := \frac{A_s}{A_b} = 1.653$$

$$Spacing := \frac{12in}{N_b} = 7.26 \cdot in$$

$$\boxed{\text{Space} = 7.25 \text{ in}}$$

A-3-5 1957 Method for H15 Truck Loading

$S := 21ft + 10in$ $\quad h := 19in$ $\quad f_c := 3000psi$ At the end the fc has to be less than 1/3 of f `c

$W := 1.25ft$ $\quad Cc := 1.25in$

$d_b := \frac{8}{8}in$ $\quad f_{ys} := 18000psi$ $\quad N := 2$

$$d := h - Cc - \frac{d_b}{2} = 17.25 \cdot in \qquad \text{(Eq-12)}$$

$$I := \frac{50ft}{S + 125ft} = 0.341$$

$E_s := 30000ksi$

Reinforcement in Tension only:

$Q := 640lbf$ Uniform Lane Load from 1957

$P := 16kip$ This is the axle weight from the H20-S16-44 Truck configuration

$w := 30ft$

$P' := 18kip$ This is the concentrated loading from Lane load - 1957 was 18 k

$$E_{Wheel} := \frac{10ft \cdot N + w}{4 \cdot N} = 6.25 \cdot ft \qquad \text{(Eq-22)}$$

$$M_{uniform} := \frac{N \cdot Q \cdot ft^2}{(.5 \cdot w + 5ft \cdot N)} = 0.051 \cdot kip \cdot ft \qquad \text{(Eq-23)}$$

$$M_{Concentrated} := \frac{N \cdot P' \cdot ft^2}{.5 \cdot w + 5ft \cdot N} = 1.44 \cdot kip \cdot ft \qquad \text{(Eq-24)}$$

$$M_{Lane} := M_{uniform} + M_{Concentrated} = 1.491 \cdot kip \cdot ft$$

$$M_{des1} := M_{Lane} \cdot \frac{E_{Wheel}}{ft} = 9.32 \cdot kip \cdot ft$$

$M_{LL} := 65.49kip \cdot ft$

$M_{DL} := 16.43kip \cdot ft$

$M_I := I \cdot M_{LL} = 22.301 \cdot kip \cdot ft$

$$M_L := \frac{M_I + M_{LL}}{\frac{E_{Wheel}}{ft}} = 14.047 \cdot kip \cdot ft$$ (Eq-39)

$$M_{des2} := \left(M_L + M_{DL}\right) = 30.477 \cdot kip \cdot ft$$

$$M_{des} := \max\left(M_{des2}, M_{des1}\right) = 30.477 \cdot kip \cdot ft$$

$$d = 17.25 \cdot in$$

$$b := 12in$$

$$j := .902$$

$$A_s := \frac{M_{des}}{f_{ys} \cdot j \cdot d} = 1.306 \cdot in^2$$ A s = Area of steel (Eq-11)

$$\rho := \frac{A_s}{b \cdot d} = 6.308 \times 10^{-3}$$ (Eq-8)

$$k := \sqrt{2 \cdot \rho \cdot n + (\rho \cdot n)^2} - \rho \cdot n = 0.293$$ Position of Neutral access (Eq-7)

$$j := 1 - \frac{k}{3} = 0.902$$ Arm of resisting couple (Eq-9)

$$f_c := \frac{2 \cdot M_{des}}{j \cdot k \cdot b \cdot d^2} = 0.775 \cdot ksi$$ (Eq-4)

$$f_c := \frac{6 \cdot M_{des}}{b \cdot d^2} = 0.615 \cdot ksi$$ If it is less than the assumed value, OK

Choose bar size:

$$A_{size} := 8$$

$$A_b := .79in^2$$

Spacing:

$$N_b := \frac{A_s}{A_b} = 1.653$$

$$\text{Spacing} := \frac{12\text{in}}{N_b} = 7.26 \cdot \text{in}$$

$\boxed{\text{Spacing} = 7.25\ \text{in.}}$

A-3-6 1961 Method for H15 Truck Loading

$S := 21ft + 10in$ $h := 19in$ $f_c := 3000psi$

At the end the fc has to be less than 1/3 of f `c ---- in 1960 this has a range 2000-4000 psi

$W := 1.25ft$ $Cc := 1.25in$

$d_b := \frac{8}{8} in$ $f_{ys} := 18000psi$

$N := 2$ Number of traffic lanes

$$d := h - Cc - \frac{d_b}{2} = 17.25 \cdot in$$

(Eq-12)

$$I := \frac{50ft}{S + 125ft} = 0.341$$

$E_s := 30000ksi$

$Q := 640lbf$ Uniform Lane Load from 1961

$P := 16kip$ This is the axle weight from the H20-S16-44 Truck configuration

$w := 30ft$

$P' := 18kip$ This is the concentrated loading from Lane load - 1957 was 18 k

$E := 4ft + .06 \cdot S = 5.31 \cdot ft$ (Eq-27)

$$E_{Wheel} := \begin{vmatrix} E & \text{if } E < 7ft \\ (7ft) & \text{otherwise} \end{vmatrix}$$

$E_{Wheel} = 5.31 \cdot ft$

$M_{LL} := 65.49kip \cdot ft$

$M_{DL} := 16.43kip \cdot ft$

$M_I := I \cdot M_{LL} = 22.301 \cdot kip \cdot ft$

$$M_L := \frac{M_I + M_{LL}}{\frac{E_{Wheel}}{ft}} = 16.533 \cdot kip \cdot ft$$ (Eq-39)

$M_{LLcode} := 900lbf \cdot S = 19.65 \cdot kip \cdot ft$ (Eq-28)

$d = 17.25 \cdot in$

$b := 12in$

$j := .899$

$$A_s := \frac{M_{des}}{f_{ys} \cdot j \cdot d} = 1.417 \cdot in^2$$ (Eq-11)

$$\rho := \frac{A_s}{b \cdot d} = 6.846 \times 10^{-3}$$ Position of Neutral axis (Eq-8)

$$k := \sqrt{2 \cdot \rho \cdot n + (\rho \cdot n)^2} - \rho \cdot n = 0.303$$ Arm of resisting couple (Eq-7)

$$j := 1 - \frac{k}{3} = 0.899$$ (Eq-9)

$$f_c := \frac{2 \cdot M_{des}}{j \cdot k \cdot b \cdot d^2} = 0.814 \cdot ksi$$

$$f_c := \frac{6 \cdot M_{des}}{b \cdot d^2} = 0.665 \cdot ksi$$ If it is less the assumed value, OK (Eq-4)

Choose bar size:

$$A_{size} := 8$$

$$A_b := .79in^2$$

Spacing:

$$N_b := \frac{A_s}{A_b} = 1.794$$

$$Spacing := \frac{12in}{N_b} = 6.69 \cdot in$$

$$\boxed{\text{Spacing} = 6.5 \text{ in}}$$

A-4 Shear and Development Length Check

A-4-1 Shear Check

Shear Check in the East Segment

$f'c := 1850psi$

$fy := 33ksi$

$C_c := 1.25in$

$H := 19in$ $\quad$ $S := 21ft + 10in$

Bar Size Number 8 bars

$b_d := \frac{7}{8}in$

$d := H - C_c - \frac{b_d}{2} = 17.313 \cdot in$

Define the Effective width of slab below:

$E := 4ft + .06 \cdot S = 5.31 \cdot ft$ (Eq-27)

The truck which causes the most shear at support is the LC-5 load test truck.
Using Sap2000 the following shear values were obtained.

$VLC5 := 31.4kip$

The shear values above are due to one wheel-line. The force is over one effective width; there the shear force per foot of width is divided by the effective width value.

$V_{LC5} := \frac{VLC5}{\frac{E}{ft}} = 5.913 \cdot kip$

V_{DL}: 3.68 kip — Shear due to Slab dead load

$V_u := V_{DL} + V_{LC5} =$ 9.58 kip — Shear Demand

There are no signs of shear stirrups; therefore, the concrete has to be able to resist the shear force caused by truck load. Below the check from AASHTO (17th ed,. 2002) is performed:

$\phi := 0.75$ For Shear

$b_w := 12in$ One foot of concrete which resist the force applied on foot of concrete (Eq-33)

$V_c := 2 \cdot \sqrt{f'c \cdot psi} \cdot b_w \cdot d = 17.871 \cdot kip$

$\phi V_c := \phi \cdot V_c = 13.403 \cdot kip$

ϕ V.c > V.u ; therefore, the concrete depth is sufficient at this cross section.

A-4-2 Development Length

Check the Development Length in the Original Segment:

$S := 21ft + 10in$

$H := 19in$

Bar Configuration in the slab:
#8 bars @ 4 in. O.C with 2/3
of the bars terminated at 3.5 ft
from the support

$A_s := 2.37in^2$ per linear foot

$b_d := 1in$

$C_c := 1.25in$

$d := H - C_c - \frac{b_d}{2} = 17.25\,in$

$A_b := .79in^2$

$f_y := 33000psi$

$f'c := 2500psi$

Define the Cross Section Capacity for As = 2.37 in2 / ft

$b_w := 12in$

$a := \frac{A_s \cdot f_y}{.85in \cdot f'c \cdot b_w} = 3.067$ (Eq-36)

$M_n := A_s \cdot f_y \cdot \left(d - \frac{a \cdot in}{2}\right) = 102.4\,kip \cdot ft$ (Eq-35)

$\phi := 0.9$

$\phi M_n := \phi \cdot M_n = 92.2\,kip \cdot ft$

$\phi M_n = 92.189\,kip \cdot ft$

Define the Cross Section Capacity for (2/3)As = 1.58 in2 / ft

$A_{s23} := 1.58in^2$

$a_{23} := \frac{A_{s23} \cdot f_y}{.85in \cdot f'c \cdot b_w} = 2.045$

$M_{n23} := A_{s23} \cdot f_y \cdot \left(d - \frac{a_{23} \cdot in}{2}\right) = 70.5\,kip \cdot ft$

$\phi := 0.9$

$\phi M_{n23} := \phi \cdot M_{n23} = 63.5\,kip \cdot ft$

$\phi M_{n23} = 63.5\,kip \cdot ft$

<u>With this information, check the following checks from AASHTO (17th ed., 2002)</u>

8.24.1 Applies to All Cross Sections

8.24.1.1

$$L_d := \frac{.04 \cdot A_b \cdot f_y}{in \sqrt{f'c \cdot psi}} = 20.9\,in$$

There is more than 20.9 in. of reinforcement on either side of maximum tension or compression; therefore, **Check passes**

8.24.1.2

Satisfy provisions of 8.24.2.3
Eq- 8-65 and section 8.25

$$L_d := \frac{.04 \cdot A_b \cdot f_y}{in \sqrt{f'c \cdot psi}} = 20.86\,in \qquad \text{(Eq-37)}$$

$M := \phi M_{n23} = 63.5\,kip \cdot ft$

$E := 4ft + .06 \cdot S = 5.31 \cdot ft$

$VLC5 := 31.4kip$ (Eq-27)

$$V_{LC5} := \frac{VLC5}{\frac{E}{ft}} = 5.91 \cdot kip$$

$V := V_{LC5} = 5.91\,kip$

$l_a := 0$

This is a conservative value since the configuration of the bars at the support is unknown

$$L_{dreq} := \frac{M}{V} + l_a = 129\,in$$

Since the end of reinforcement is confined by compressive reaction, this length maybe increased by 30%; therefore, the L.d min is as follows:

$$L_{dmin} := L_{dreq} \cdot 0.3 + L_{dreq} = 167\,in \qquad \text{(Eq-34)}$$

Since Ld.min > L.d calculated from section 8.25, there is sufficient development length
Check passes

8.24.1.2.1

The reinforcement shall extend more than the maximum of the following cases:

a) Effective Depth

$l_{da} := d = 17.25\,in$

b) 15 times the bar diameter

$l_{db} := 15 \cdot b_d = 15\,in$

c) 1/20 of the clear span

$l_{dc} := \frac{1}{20} \cdot (S - 2ft) = 11.9\,in$

$l_{dreq} := max(l_{da}, l_{db}, l_{dc}) = 17.25\,in$

Since Ld.req < the La in the cross sections,
Check passes

8.24.1.2.2

L.d required based on section 8.24.1.1 is less than the reinforcement in the cross section,
Check passes

8.24.1.3

The drawings from the 1922 and 1924 suggest that this is the configuration of the bars at the end
Check passes

8.24.1.4

The bars are terminated in tension zone; therefore, the following checks have to satisfy:

a) <u>8.24.1.4.1</u>

$\phi V_n = \phi (V_s + V_c)$

V_n = Nominal cross section shear capacity

V_c = Shear capacity of concrete

V_s = Shear capacity of stirrups

V_s = 0 therefore, $V_n = V_c$

$V_c := 2 \cdot \sqrt{f'c \cdot psi} \cdot d \cdot b_w = 20.7\,kip$

$\phi := .75$ For a case of shear

$\phi V_n := \phi \cdot V_c = 15.525\,kip$

$V_{nreq} := \left(\frac{2}{3}\right) \cdot \phi V_n = 10.35\,kip$

$E := 4ft + .06 \cdot S = 5.31 \cdot ft$

$V_u := \frac{VLC5}{\frac{E}{ft}} = 5.913 \cdot kip$

$$V_u < \frac{2}{3} \cdot \phi \cdot V_c$$

Since 2/3 of the shear capacity of concrete cross section is more than the shear demand caused by the LC 5 load test truck,
Check passes

b) 8.24.1.4.2

Since stirrups are not required, this check does not apply
Check DNA

c) 8.24.1.4.3

Since 2/3 of the cross section shear capacity is more than demand, the 3/4 of the cross section shear capacity is also more than the demand,
Check passes

8.24.2 Applies to Positive Moment Reinforcement

8.24.2.1

1/3 of the bars are terminated, leaving 2/3 of the bars continuing to the support
DNA - Check passes

8.24.2.2

8.24.2.3

In section 8.24.1.2 the min development lengths were calculated, and it was proved that these checks passed
Check passes

A-5 Operating Rating Example for a Simple Span

Given:

f_c	2500	psi		
f_y	33000	psi		
h	19	in		
d	18	in		
A_s	1.68	in^2/ft		
S	20	ft		

Solution:

RF =	**$C - A_1 D / A_2 L (1 + I)$**			
C	**Capacity**			
A_1	**1.3**	Operating		
A_2	**1.3**	Operating		
L	**M_{LL}**	60	k - ft	12 k of one axle on midspan
E	**0.06S+4**	5.2	ft	
L/E		11.5	k - ft	
D	**M_{DL}**	15.88	k - ft/ft	Moment due to self- weight of conc @ 150 pcf
I	**Impact**	0.3		and an additional 80 psf according to SHDA (1922)
C	**ΦMn**	As fy (d- a/2)		
Φ	0.9			
Jd	.95 d			
# 8 bars				
d		18	in	
As	M / fy jd	1.61	in^2/ft	
a =	$(As\ fy) / (0.85\ f_c\ b)$			
a =		2.17	in	
Ab		0.79	in^2	
Spaicng	Ab * 12 / As req / ft			
Spacing		6	in	
ΦMn	Φ [As Fy (d- a/2)]			
ΦMn		844	K-in	
ΦMn		70	K-ft	
RF =	$C - A_1 D / A_2 L (1 + I)$			
Rf =		2.5		