LOAD RATING OF FLAT SLAB BRIDGES WITHOUT PLANS

by

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ABSTRACT

In the United States, there is a large number of reinforced concrete flat slab bridges, which were constructed during 1900's and are still in service. The state Departments of Transportation (DOT) do not have necessary information of design details, and properties of materials used during the construction of those old flat slab bridges. Those old bridges are not designed to support the current traffic. Therefore, they might have certain issues regarding durability, strength and safety. Nowadays, the visual inspection techniques followed by AASHTO guidelines are used for the evaluation of current load carrying capacity of concrete flat slab bridges. Such techniques or guidelines may overestimate or underestimate the load bearing capacity, and may not represent the actual capacity. The load bearing capacity of structures depends upon the physical dimensions and properties of materials from which they were built. In this research, the unknown parameters, such as clear cover, size, bar spacing and compressive strength of the concrete, were determined by using simple non-destructive tests on existing bridges. For a simple non-destructive test, Profoscope and Schmidt hammer were used to run the test in the field. By using the field data, three dimensional finite element analysis of a flat slab bridge was performed in ANSYS to determine deflection at the mid-point of a concrete flat slab bridge under a truck load. In the analysis, the truck load position which would results the maximum displacement at mid-point of bottom face was used as a critical load position. The load was increased up to a point that produces the deflection close to the maximum allowable value according to AASHTO Section 2.5.2.6.2 criteria. The load corresponding to the maximum allowable deflection on the existing bridge is used to calculate the rating factor of the bridge. The Ohio legal load vehicle of gross weight 30 kip having the truck load designation of OH-2F1 is considered for this research. The rating factor is determined as the ratio of truckload that produce the maximum allowable midpoint deflection to the original designated truck load. The research outcome will provide guidelines to evaluate the load rating factor of existing flat slab bridges without plans.

Keywords: Concrete flat slab bridges, load rating, finite element modeling, load bearing capacity

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Chapter 1

Introduction and Literature Reviews

1.1 General Overview

In the United States, there are a large number of reinforced concrete flat slab bridges, which were constructed during the 1900's. Most of these bridges are still in service. These concrete flat slab bridges may have a certain deficiency in load bearing capacity. The decrease in structural strength of flat slab bridges may be due to cracks and weathering of concrete, corrosion in reinforcement, fatigue cracks in rebars, etc. In general, the bridge deficiency can be categorized in two classes - structurally deficient and functionally obsolete. The structurally deficient bridges are those which do not permit the current legal loads, whereas functionally obsolete bridges do not meet the current geometric conditions, such as shoulder width, lane, clearances, etc. (Ohio Infrastructure Report Card, Bridge Fact Sheet, 2009). According to the National Bridge Inspection Standard (NBIS), load rating on bridges should be performed to estimate their safe live load bearing capacity under existing condition. The load rating of a bridge is defined as the service live load that can pass safely over the structure, and is expressed as a rating factor or in terms of tonnage of a particular vehicle.

According to Report Card for America's Infrastructure (ASCE, 2013) the current average age of the 607,380 bridges in the United States (U.S.) is estimated as 42 years. In addition, the study performed by ASCE (2013) shows that one out of nine bridges is found to be structurally deficient. There is a large of number of concrete flat slab bridges at various states in the U.S., which have already passed their expected service life and are in need of maintenance and rehabilitation (ASCE, 2013). Such bridges may not be able to sustain the current traffic volumes and loads due to its deterioration and out-of-dated design.

The map shown in Fig. 1-1 below represents the percentage variation of structurally deficient bridges in various states in the United States, which require appropriate maintenance (Structurally Deficient Bridges, American Road and Transportation Builders Association, ARTBA, 2016).

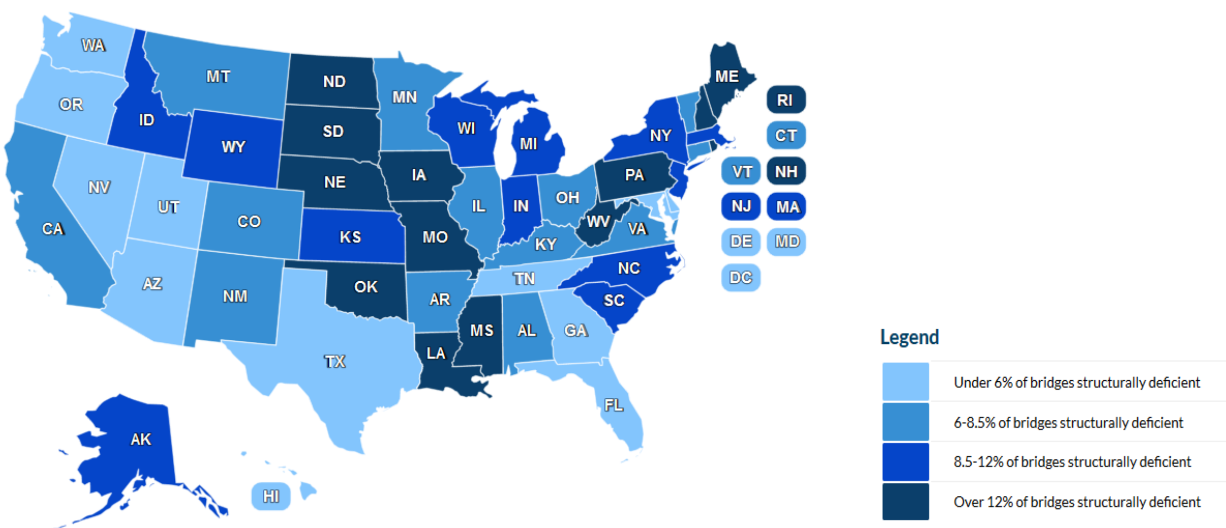


Figure 1-1: Structurally deficient bridges in the United States.

(ARTBA, 2016)

Table 1-1: Highway slab bridges in the United States

(USDOT FHWA, 2008-2015)

Year	Structurally deficient bridges	Functionally obsolete bridges	Total deficient bridges	Total no. of bridges
2008	6,587	10,399	16,986	80,172
2009	6,677	10,264	16,941	80,552
2010	6,601	10,119	16,720	80,333
2011	6,665	9,974	16,639	80,239
2012	6,542	9,763	16,335	80,327
2013	6,414	9,754	16,168	80,586
2014	6,223	9,685	15,908	81,020
2015	6,032	9,624	15,656	81,231

Table 1-1 shows numbers of slab bridges in the U.S. at different condition from 2008-2015 (USDOT FHWA, 2015). As described in the Table 1-1, the number of total deficient flat slab bridges in the United States has been decreasing over the time period from 2008 to 2015.

1.2 Problem Statement

According to the data, as shown in Table 1-2, published in 2015, by the United States Department of Transportation (USDOT) under Federal Highway Administration (FHWA), among the nation's total 611,845 bridges, 142,915 (23.4%) were categorized as deficient. Among them, 58,791 (9.6%) were structurally deficient and 84,124 (13.8%) were functionally obsolete. From the data shown in Table 1-2, the total number of deficient bridges is in decreasing trend over the years during 2008-2015. There has been 19.3 percent decrease in structurally deficient bridges whereas only 5.7 percent decrease in functionally obsolete bridges over that same period. The structurally deficient bridges are those bridges, which cannot support current legal loads. Therefore, load limit should be posted on such bridges. On the other hand, functionally obsolete bridges are those bridges, which were built using design standards not suitable for current traffic, although they might be structurally safe. The drawbacks of functionally obsolete bridges are low load carrying capacity, insufficient deck width, horizontal and vertical clearances for current traffic and vehicle sizes.

Table 1-2: Conditions of highway bridges in the United States (USDOT, FHWA 2008-2015)

Year	Structurally deficient bridges	Functionally obsolete bridges	Total no. of deficient bridges	Total no. of bridges
2008	72,875	89,162	162,037	601,339
2009	72,398	87,411	159,839	603,229
2010	70,423	85,839	156,262	604,417
2011	68,758	84,832	153,590	605,101
2012	66,748	84,748	151,496	607,379
2013	63,522	84,348	147,870	607,751
2014	61,365	84,510	145,875	610,729
2015	58,791	84,124	142,915	611,845

The numbers of deficient bridges in rural and urban areas in the nation during 2008-2015 are shown in Table 1-3. During 2008-2015, as shown in Table 1-3, the number of deficient (structurally and functionally) bridges decreased by 17,794 numbers in rural areas. However, in urban areas, it increased by 1,336 over the same period of time. Almost 25 percent of total bridges were found to be either structurally deficient or functionally obsolete over those eight years. Clearly, it could be noticed that the number of functionally obsolete bridges is in slightly increasing trend in urban areas, whereas it is in decreasing trend in rural areas. It can be concluded that the overall number of deficient bridges is in decreasing trend (decreased by 11.8%) in between 2008-2015.

Table 1-3: Numbers of deficient bridges in rural and urban areas in the United States

(USDOT, FHWA 2008-2015)

Year	Structurally deficient bridges			Functionally obsolete bridges		
	Rural	Urban	Total	Rural	Urban	Total
2008	59,626	13,244	72,870	50,947	38,228	89,175
%	81.83	18.17	12.12	57.13	42.87	14.83
2009	59,262	13,135	72,397	49,160	38,295	87,455
%	81.86	18.14	12	56.21	43.79	14.5
2010	57,691	12,736	70,427	47,790	38,067	85,857
%	81.92	18.08	11.65	55.66	44.34	14.2
2011	56,484	12,271	68,755	46,601	38,231	84,832
%	82.15	17.85	11.36	54.93	45.07	14.02
2012	54,782	11,967	66,749	46,257	38,491	84,748
%	82.07	17.93	10.99	54.58	45.42	13.95
2013	52,040	11,470	63,510	45,512	38,832	84,344
%	81.94	18.06	10.45	53.96	46.04	13.88
2014	50,272	11,093	61,365	45,310	39,215	84,525
%	81.92	18.08	10.05	53.61	46.39	13.84
2015	48,131	10,660	58,791	44,648	39,476	84,124
%	81.87	18.13	9.61	53.07	46.93	13.75

According to Table 1-4, the number of bridges closed to traffic and posted for other restrictions are in an increasing trend during 2008-2012, then in a decreasing trend up to 2015. The total number of bridges posted for maximum load restriction has decreased from 71,547 to 65,930 during 2008-2015. It does not necessarily mean that those posted bridges are at risk to the public, but they allow only limited weight vehicles. Closed bridges can create traffic congestions, and force heavily loaded trucks and emergency vehicles to travel through longer routes. They can also be inaccessible, costly and time-consuming for vehicles transporting daily goods to the public.

Table 1-4: Bridge posting in the United States

(USDOT, FHWA 2008-2015)

Year	Closed to traffic	Posted for maximum load	Posted for other restrictions	Total	Percentage total
2008	2,966	66,052	2,529	71,547	11.90
2009	3,552	66,249	2,669	72,470	12.01
2010	3,538	63,072	2,953	69,563	11.51
2011	3,578	61,575	2,916	68,069	11.25
2012	3,585	60,971	3,040	67,596	11.13
2013	3,376	60,728	3,090	67,194	11.06
2014	3,327	61,012	3,075	67,414	11.04
2015	3,303	60,017	2,610	65,930	10.78

It has been reported that the cost of repair or to replace only the deficient bridges under the Federal Highway Bridge Program was increased from $71 billion to $76 billion from 2009 to 2013 (ASCE, 2013). It was estimated in 2008 that $140 billion was required to repair all deficient bridges in country – $92 billion for functionally obsolete bridges and $48 billion for structurally deficient bridges (AASHTO, 2008).

According to Table 1-2, approximately one out of four bridges is either structurally or functionally deficient. In addition, 10.8 percent of the nation's bridges are closed to traffic or posted for load restrictions, as shown in Table 1-4. Therefore, it is essential to develop reliable and accurate method to estimate the current load bearing capacity of existing bridges. The outcome of this research will be useful to DOTs for load rating of old flat slab highway bridges without any available plans.

1.3 Load Rating of Bridges

Generally in the United States, load rating of existing highway bridges has been evaluated by following the guidelines described in the American Association of State

Highway Transportation Officials (AASHTO) Manual of Bridge Inspection (MBI). For load rating of bridges without plans, the amount of reinforcement and properties of materials are the primary unknowns. To evaluate the load bearing capacity, some of the county engineers were referring to the AASHTO's Manual for Bridge Evaluation for the materials properties (Taylor et al. 2011).

According to the Ohio Department of Transportation (ODOT) records, approximately 6,550 flat slab bridges exist in the State of Ohio in 2016. Among these bridges, approximately 19% (1,234) do not have any records including the date of construction, properties of materials, design method, design load, design period and detailed drawings that had been used during construction. However, it is essential to evaluate the present conditions and load bearing capacity of these bridges to ensure public safety. According to the ODOT MBI 2010, inspection is categorized in five general types, which are Initial, Routine, Damage, In-depth, and Special Inspection. In general, load rating is performed during the Routine Inspection and In-depth inspection. During the Routine Inspection, load rating is performed to determine the need for establishing or revising a weight restriction on the bridge (ODOT MBI, 2010). Additionally, In-depth inspection may be performed to find out the deficiency of a particular structural component that cannot be determined from the Routine inspection (ODOT MBI, 2010). ODOT Bridge Design Manual provides procedures, guidelines and policies for determining the safe live load bearing capacity of highway bridges in the State of Ohio.

Table 1-5 shows conditions of flat slab bridges in the State of Ohio from 2008 to 2015 (USDOT, 2015). From Table 1-5, it is evident that the total number of deficient flat slab bridges was in a decreasing trend (31.3%) during 2008-2015.

Table 1-5: Conditions of flat slab bridges in the State of Ohio (USDOT, FHWA 2008-2015)

Year	Structurally deficient bridges	Functionally obsolete bridges	Total no. of deficient bridges	Total no. of flat slab bridges
2008	399	492	891	4301
2009	405	485	890	4263
2010	378	469	847	4203
2011	382	427	809	4010
2012	327	387	714	3789
2013	315	408	723	3790
2014	285	390	675	3761
2015	246	366	612	3785

1.4 Objectives of Research

The main purpose of this research is to develop a simple non-destructive, more efficient and inexpensive evaluation method for determining the current load bearing capacity of flat slab bridges without plans. This method might be used by bridge owners in order to ensure public safety through appropriate use of existing bridges. In addition, the research outcomes might provide the following benefits for flat slab bridges without plans:

- additional resources for routine inspection and maintenance;
- optimum utilization of load carrying capacity;
- minimize load postings;
- better decision making on repair/replacement;
- improved management for ensuring public safety.

1.5 Literature Reviews

Due to inadequate information about old flat slab bridges in Larimer County, Colorado, bridge inspectors used visual inspection techniques for load rating of existing flat slab bridges constructed in 1960's or earlier (Taylor et al. 2011). Also, comprehensive structural analysis was performed to determine the load bearing capacity of reinforced concrete slabs and compare the results with visual rating. For the structural analysis, the material properties were taken from the AASHTO's Manual for Bridge Evaluation (Taylor et al. 2011).

According to the Manual for Bridge Evaluation, for bridges constructed before 1959, concrete compressive strength and steel yield strength were suggested as 2,500 psi and 33,000 psi, respectively, for the load rating of existing old bridges (AASHTO, 1994). According to Concrete Reinforcing Steel Institute (CRSI, 2001), it was found that Grade 33 steel was used in the construction of reinforced concrete structures construction over a period of 1910-1927. The ultimate tensile strength of the Grade 33 steel is specified as 55 ksi.

Currently, using rating capacity suggested by the Manual for Maintenance Inspection of Bridges, (AASHTO Manual for Maintenance Inspection of Bridges, 1983) would underestimate the actual load carrying capacity. It was found that the safe load carrying capacity of the most of the reinforced concrete bridges was larger than the capacity determined from the traditional analytical method (Kaliber et al. 1997).

The linear finite element analysis results the conservative load bearing capacity even though the structure may have more strength. In the original AASHTO load rating, the maximum reinforcement limit was satisfied indicating that the slab is under-reinforced and thus, had adequate ductility (Jauregui et al. 2009).

Different rating approaches result in different rating factors for the same bridge. The rating factor obtained from the linear finite element analysis is twice more than that obtained from the planar idealization of the bridge (Huria et al. 1993).

The following factors greatly influence the results of finite element analysis of built structures:

- Geometric idealization, such as boundary and continuity modeling
- Discretization
- Finite element formulation
- Material and failure modeling
- Computational parameters

The research conducted by Azizinamini et al. (1994) found that the yield line analysis approach for three-dimensional finite element analyses will be efficient to evaluate the true strength of flat slab bridges. However, from the analytical and experimental research conducted for the evaluation of strength of flat slab bridges, it was found that the current AASHTO rating procedures underestimate the actual load bearing capacity of existing bridges (Vijay K. Saraf, 1998).

In addition, several researchers (Azizinamini et al. 2000; Aktan et al. 1992; Brudette and Goodpasture 1988), performed experimental and analytical study and concluded that existing flat slab bridges resist more load than evaluated by currently adopted rating procedures.

It was found that ways of defining support conditions of structures in nonlinear finite element analysis model of bridge significantly affect its response results (Huria et al. 1993:1994). In addition to those parameters, describing the boundary conditions will be more critical than parameters representing material properties. However, it becomes more tedious to make finite element models of bridges with irregular geometric characteristics, such as curved and skewed bridges (Huria et al. 1994).

For the analytical modeling and nonlinear finite element analysis of constructed structures, the following steps should be followed (Huria et al. 1993).

1. Geometric modeling of the structure and finite elements discretization
2. Finite element formulation: geometric, material and numerical aspects
3. Force or load effect modeling
4. Computational modeling

For the calculation of ultimate load bearing capacity of flat slab bridges, non-linear finite element analysis was performed on a field calibrated model. Thus, obtained capacity was compared with rating factors obtained from AASHTO guidelines. It was concluded that the capacity of structures can be inferred at either local level (material strength or cross sectional strength) or at the global level (failure of the structure due to attainment of a collapse mechanism or due to a regional shear failure (Huria et al. 1993).

From the above literature reviews, it can be concluded that the three dimensional finite element analysis results in reasonable load bearing strength of the concrete structures.

Chapter 2

Field Investigation and Data Collection

2.1 Bridge Selection

The main objective of this research is to develop a method to determine load bearing capacity of the existing old flat slab bridges without detailed design drawings and unknown material properties that were used during the construction. The proposed bridge was selected from a group of flat slab bridges in ODOT District 4 by considering the following factors:

- Should be a slab bridge;
- Constructed during early 1900s;
- Single span for the ease of finite element modeling;
- Low current traffic in order to minimize disruption during data collection;
- Easily accessible for field investigation and data collection.

2.1.1 Location

The selected bridge is located in Trumbull County, Ohio. This bridge is located at a distance of 26.3 miles north-east from Youngstown State University, Youngstown, Ohio, on Route OH-5. Figure 2-1 shows the location of the flat slab bridge selected for this research.

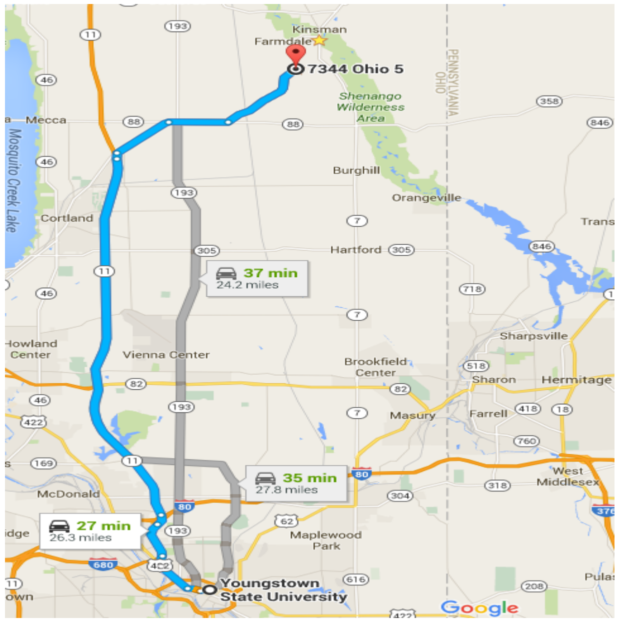


Figure 2-1: Location map of selected bridge site from Google map.

2.1.2 Bridge Condition

The selected bridge was constructed in 1930. The bridge shows signs of longitudinal and transverse cracks, and spalled concrete. Surprisingly, the bottom of the slab has no crack at all. There are clearly visible cracks developed on the wing wall as shown in Fig. 2-2 a). Figure 2-2 b) shows deteriorated surfaces of different components of the existing bridge.

a) Cracks on the wing wall

b) Deterioration of curb surface

Figure 2-2: Condition of bridge components.

2.1.3 Bridge Type

The bridge selected for this research is a single span reinforced concrete flat slab bridge with steel railing on both sides. It is marked as TRU-5-28.97 in Trumbull County, Ohio. The width of the bridge is 32 ft 4 in. including 5 ft shoulders on both sides with a clear span of 16 feet without any skew. The slab is supported on two 1ft 6 in. thick abutments. Figure 2-3 a) and b) shows the side view and street view of the selected bridge on Route OH-5.

a) Side view of the bridge b) Street view from Google map

Figure 2-3: TRU 5-28.97 bridge used in this study.

2.2 Data Collection

The nondestructive test was performed in the field by using a measuring tape, Schmidt hammer, and a Profoscope.

2.2.1 Instruments

The Fig. 2-4 a) and b) shows the instruments which were used during the field test. The Profoscope uses electromagnetic pulse induction technology to detect the rebar. Whereas, Schmidt hammer is based on the principle that the rebound of an elastic mass depends on the hardness of the concrete surface against which the mass strikes.

a) Profoscope

b) Schmidt hammer

Figure 2-4: Instruments used for data collection.

2.2.2 Field Data Measurements

The geometric properties, such as span length, lane width, shoulder width and thickness of the deck slab were measured by using a measuring tape. The Profoscope as is used to find rebar location, size, orientation, spacing, and clear cover. After finding the location and orientation, a grid of rebars was marked at the bottom of the slab in order to determine the center to center spacing of rebars as shown in Fig. 2-5 a). The rebound hammer test was performed at the bottom surface by orienting the Schmidt hammer vertically up as shown in Fig. 2-5 b). The field data measured using Profoscope and Schmidt are shown in Appendix A.

a) Data collection by Proforscope

b) Schmidt hammer rebound test

Figure 2-5: Field data collection.

Chapter 3

Finite Element Modeling

3.1 Introduction

Finite element analysis has been becoming a powerful technique for the analysis of a structural system due to the progressing knowledge and capabilities of advance computer software and hardware (Jung et al. 2013; O'Rourke T. D. 2004; Yimsiri, S et al. 2004). By the use of finite element software, analysis is becoming much easier, faster and cost-effective. During the solution process, it divides the whole complex structures into a finite number of individual elements and the process is known as discretization. The research conducted by Wolanski et al. (2004) and Kachlakev et al. (2001) found that finite element program ANSYS produces results that reasonably agree with results obtained from experimental tests of reinforced concrete structures. In this research, finite element simulation software ANSYS Workbench16.1 (Workbench ANSYS, 2015) has been used for the static analysis of the flat slab bridge.

The development of the three dimensional finite element analysis model of the reinforced concrete slab bridge using ANSYS includes various steps, such as assigning engineering properties of materials, developing geometry of the structure, and defining element properties from which ANSYS workbench generates the model for the static analysis of the structural system. While generating models, vehicle loads were applied at different positions for the analysis. The deflection was recorded at the bottom center of the bridge slab for different loading scenarios.

3.2 Finite Element Validation

For the validation of the finite element model, two beam samples having concrete compressive strength of 3,000 psi were prepared in the lab. The finite element validation was performed by comparing average deflection data from lab tests with the analytical results obtained from the model. The beam is simply supported at a distance of 1.5 in. on both sides from the exterior face and is subjected to a concentrated load at the mid-span. The beam was tested under three-point bending. The mid-span deflections were measured using an extensometer under point loads shown in Fig. 3-2. The beam has one #6 rebar as longitudinal reinforcement without any shear reinforcement. It has 6 in. by 6 in. cross-section as and a clear span of 21 in. with a total length of 24 in. The deflection measured from the lab experimentals was compared with the results obtained from the three-dimensional finite element model analysis of the same beam using ANSYS. Figure 3-1 a) side view and b) cross section shows the detailed schematic diagram of a sample beam prepared in the lab.

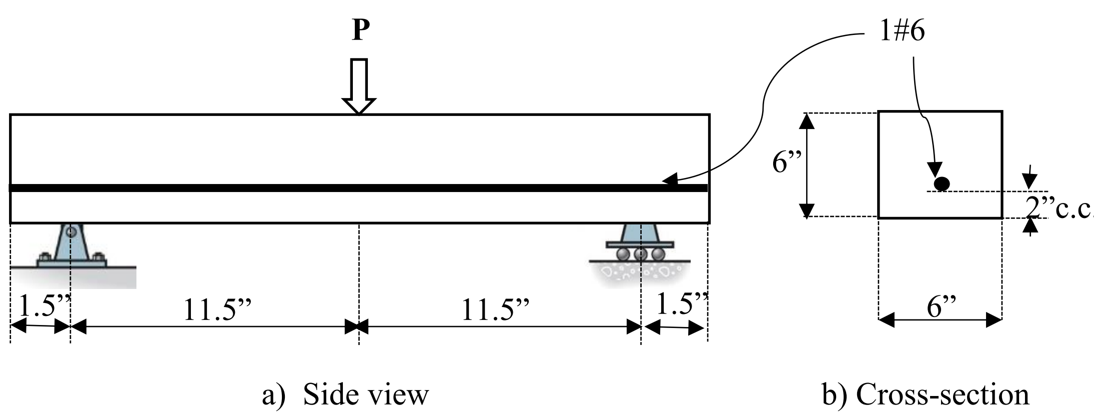


Figure 3-1: Typical elevation and cross-section of the beam.

The Fig. 3-2 shows the experimental set up for the three-point bending test of the beam specimen in order to measure deflection at the mid-span of the beam.

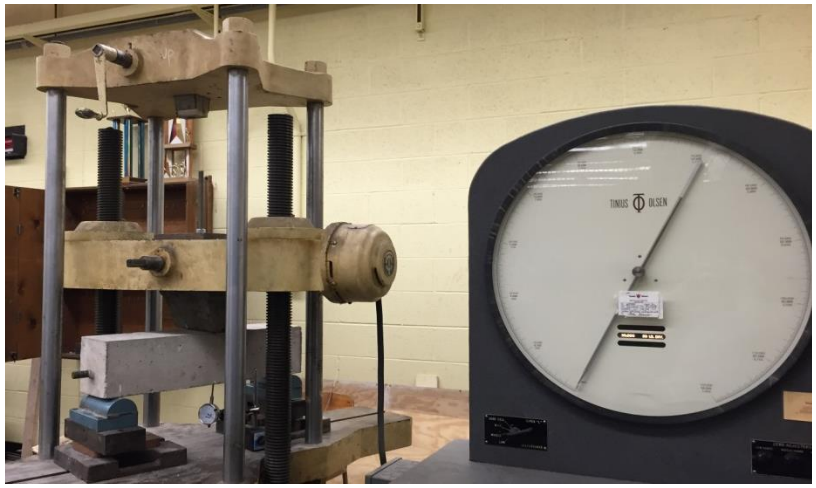


Figure 3-2: Experimental setup for three-point bending test.

Two identical beams were tested by using a universal testing machine with the same support conditions and loading as shown in Fig. 3-2. The mid-span deflection was measured with the help of an extensometer under each load increment of 500 lb. The data obtained from the experimental tests are in Appendix B. Figure 3-3 represents load vs. deflection plots for lab tests and that for finite element analysis.

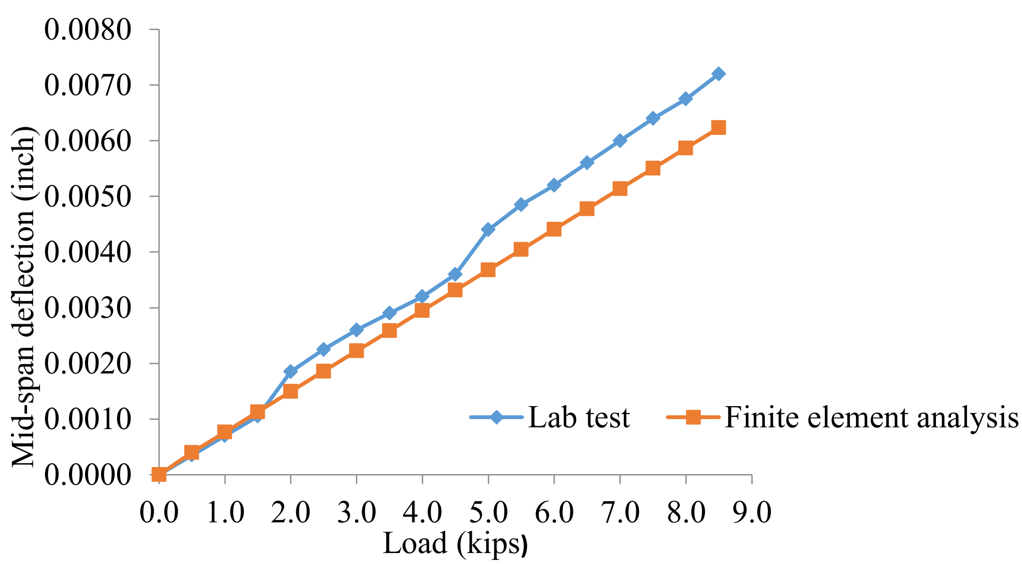


Figure 3-3: Load vs. mid-span deflection of the test beam.

As illustrated in Fig. 3-3, the analytical data fall within variation of 6-20% of experimental data. The largest difference observed at 5.5 kips of load. In general, the analytical data agree with experimental data very well.

3.3 Bridge Descriptions

The single span flat slab bridge TRU-5-28.97 evaluated in this research can accommodate two lanes of traffic and is simply supported on two abutments. The material properties of the flat slab bridge has been concurrently researched by another graduate student, who found the compressive strength of concrete in the bridge to be nearly 3,100 psi. The yield strength of steel was assumed as 33,000 psi with reference to the Manual of Bridge Evaluation, AASHTO 2008. The typical cross- section and plan view of the TRU-5-28.97 slab bridge obtained from the non-destructive field measurement were as shown in Figs. 3-4 and 3-5, respectively.

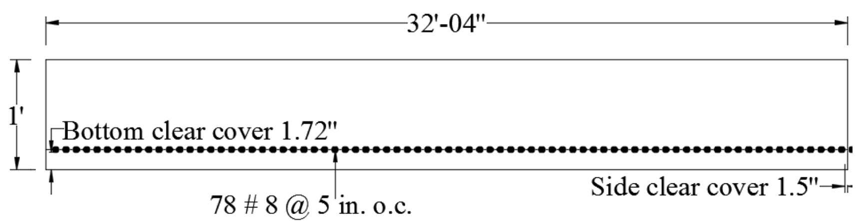


Figure 3-4: Cross-section of TRU-5-28.97

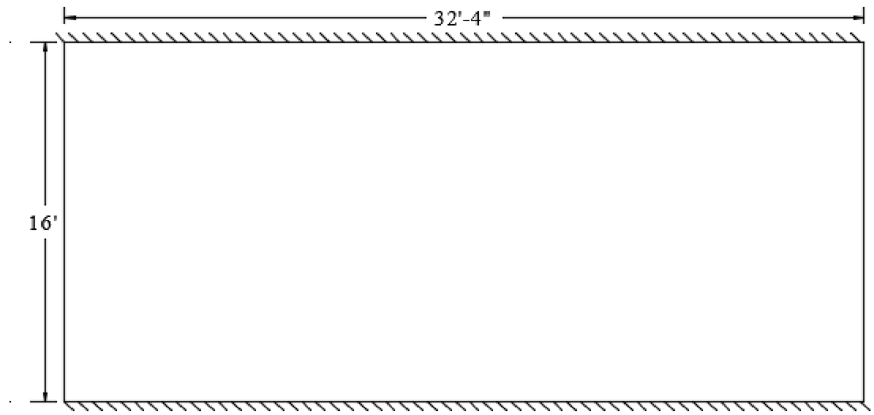


Figure 3-5: Plan of flat slab bridge TRU-5-28.97.

3.4 Finite Element Modeling of Bridge TRU-5-28.97

The three-dimensional model of the existing reinforced concrete flat slab bridge was developed in the finite element software ANSYS Workbench 16.1. The parameters required for the finite element modeling were taken from the field data, material properties research concurrently by another student, from field observations with reference to the ODOT Manual of Bridge Evaluation, AASHTO (2008).

3.4.1 Material Properties

The reinforced concrete flat slab bridge has the following material properties used in the finite element analysis:

1. Concrete

Compressive strength of the concrete f'_c= 3,100 psi

Unit weight of the concrete with $f_c' \leq$ 5.0ksi, w_c=0.150 kcf

Modulus of elasticity of concrete = $E_c = 33{,}000K_1 w_c^{\,1.5}\sqrt{f_c'}$ (AASHTO 5.4.2.4)

= 3,375,450 psi

Poisson's ratio of concrete = 0.2 (AASHTO 5.4.2.5)

The compressive uniaxial stress-strain relationship for the concrete model was obtained by using Eqs. 3-1, 3-2 and 3-3 to compute the multi-linear isotropic stress-strain curve for concrete (MacGregor, 1992)

$$E_c = \frac{f}{\varepsilon} \quad \text{Eq. (3.1)}$$

$$\varepsilon_0 = \frac{2f_c'}{E_c} \quad \text{Eq. (3.2)}$$

$$f = \frac{E_c\varepsilon}{1+\left(\frac{\varepsilon}{\varepsilon_0}\right)^2} \quad \text{Eq. (3.3)}$$

The multi-linear isotropic stress-strain curve is developed by defining the first point on the curve, which must satisfy the Hooke's law.

Table 3-1: Stress- strain data for 3,100 psi concrete

Strain(in/in)	Stress (psi)
0	0
0.000276	909.48
0.00060	1,830.04
0.00110	2,733.03
0.00187	3,100.00
0.00300	3,100.00

2. Steel

Unit weight of the steel = 0.490 kcf (AASHTO Table 3.5.1-1)

Modulus of elasticity of steel = E_s = 29,000 ksi (AASHTO 5.4.3.2)

Poisson's ratio of steel=0.3

Yield strength of steel=33,000 psi

Ultimate strength of steel= 55,000 psi

The modulus of elasticity for all grades of steel is found to be nearly 29,000 ksi. In the lab, a tensile test was conducted for 60 ksi steel, which is referenced for the generation of stress-strain values for 33 ksi steel. It has been assumed that steel fails at the same strain level.

Table 3-2: Stress-strain data for 33,000 psi steel

Strain (in. /in.)	Stress (psi)
0	0
0.00065	18,181.8
0.0008	22,727.3
0.00115	31,818.2
0.00525	33,000
0.0156	55,000

3. **Geometric properties of bridge slab measured in the field**

Clear span= 16 ft

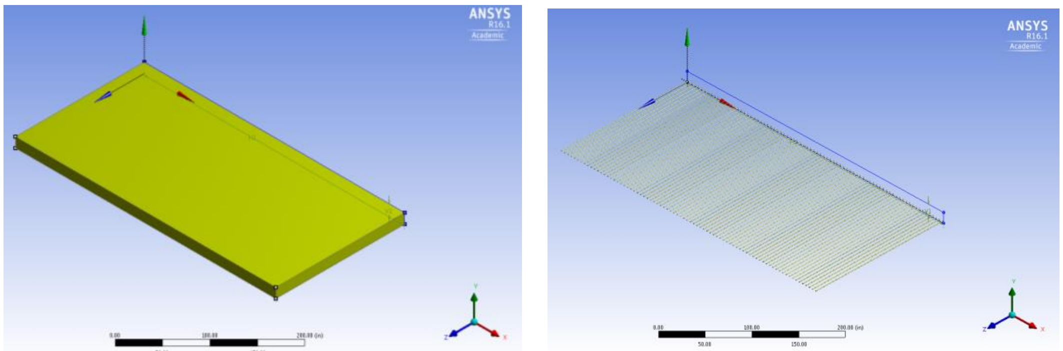
Width= 32 ft 4 in.

Thickness = 1 ft

Main rebar: #8 @ 5in. O.C. with 1.72 in. clear cover.

3.4.2 Creating Model Parts and Assembly

According to the ODOT Bridge Inspection Manual (2001), sidewalks, curbs and walkways do not contribute to the additional structural strength of a bridge slab. The function of bituminous wearing surface is to protect the underlying floor and to provide a smooth riding surface. In general, bridge deck provides the necessary support for such components of bridge. Therefore, curbs and bituminous wearing surface are neglected while developing the finite element model of the reinforced concrete flat slab bridge. Concrete and rebar were created as a solid material. Add frozen operation tool of ANSYS is used to define steel and concrete as separate materials. Therefore, concrete and rebar were defined with their respective properties stated in the engineering data section under static structural analysis system in ANSYS. Figure 3-6 a) and b) shows the concrete and rebar as different parts of the bridge model assembly, while Fig. 3-7 shows the entire bridge slab model assemblies after the rebar were embedded in concrete.



a) Concrete   b) Rebar

Figure 3-6: Different parts of bridge model

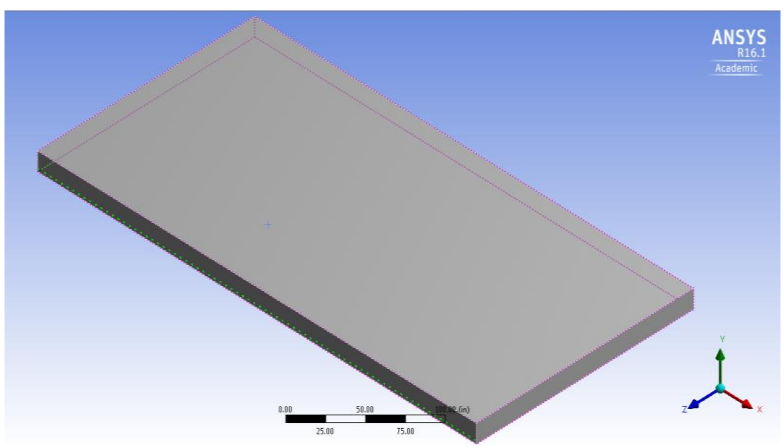


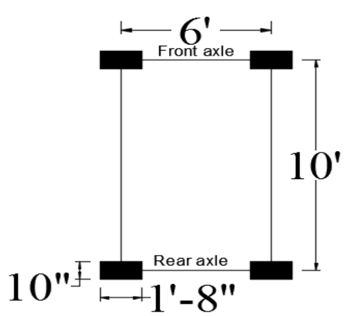
Figure 3-7: Assembled bridge model

For the application of truck loads mimicking the actual condition, rectangular faces were created on the top surface of the bridge slab having the area as specified in

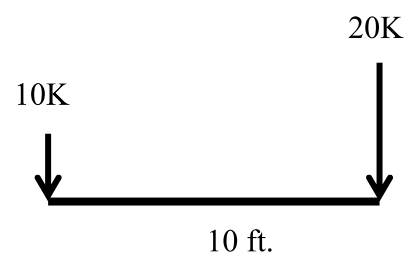
AASHTO 3.6.1.2.5 (AASHTO, 2012). The interface between reinforcement and concrete are assumed rigidly bonded, so that there is no any slippage between rebar and concrete. A point is generated at the mid-point of the bottom face of the bridge slab for reading deflection values due to the various loading cases.

3.4.3 Loading and Boundary Conditions

Load and support conditions of the bridge slab were defined under the static structural module. Truck and dead loads were defined under the model section using mechanical system. Dead load of the bridge slab was applied as standard earth gravity load. Truck load having the load designation OH-2F1 was applied on both lanes of the bridge moving in opposite directions. The front and rear axle load of OH-2F1 truck is 10 and 20 kips, respectively, with total gross weight of 30 kips. Figure 3-8 shows the detail plan view and axle load configuration of an OH-2F1 truck.



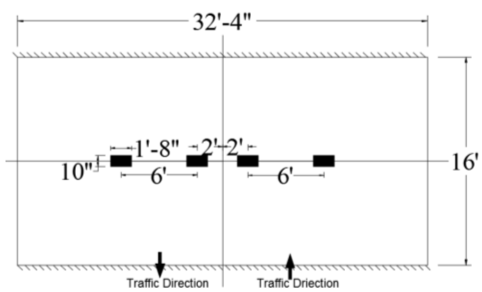
a) Plan view



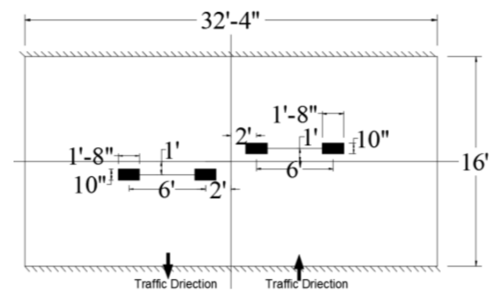
b) Axle load configuration

Figure 3-8: OH-2F1 truck load details.

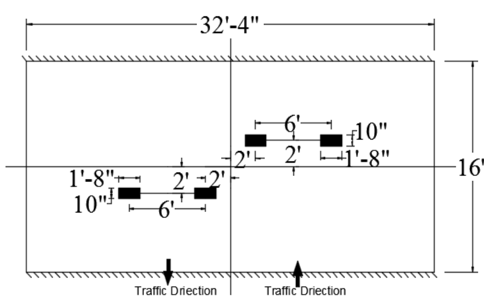
Truck load was placed in six different locations to find out the position, which would produce the maximum effect at the center line of the bridge span. It is assumed that the truck load is distributed uniformly over the contact area between tire and concrete slab. According to the AASHTO 3.6.1.2.5 (AASHTO, 2012), the tire pressure is assumed to be uniformly distributed over the rectangular area having 20 in. width and 10 in. length. The truck width, truck position and axle spacing, as shown in Fig.3-8, were taken into consideration during finite element analysis. Therefore, the critical position of the truck is considered for mid-point deflection at the bottom of the slab under various axle loads for finding the maximum permissible axle load. Figure 3-9 shows various positions of axle and truck loads in order to find the position that will produce the maximum deflection at the bottom midpoint of the bridge slab.



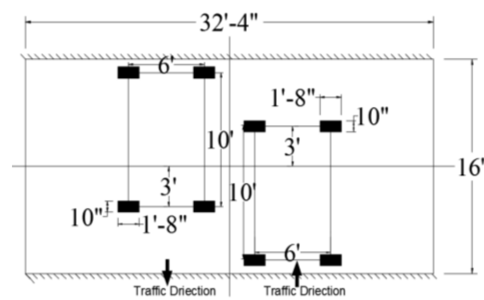
a) Rear axle coincide with center line of bridge span in both lanes.



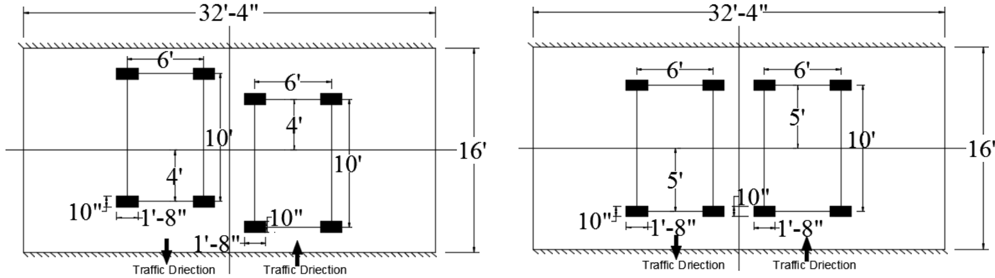
b) Rear axle is at 1 ft. away from center line of bridge span in both lanes.



c) Rear axle is at 2 ft. away from center line of bridge span in both lanes.



d) Rear axle is at 3 ft. away from center line of bridge span in both lanes.



e) Rear axle is at 4 ft. away from center line of bridge span in both lanes.

f) Rear axle is at 5 ft. away from center line of bridge span in both lanes.

Figure 3-9: OH-2F1 truck load positions.

Boundary conditions were applied by using the displacement components at the bottom edges of the bridge slab on both faces. The hinge support is modeled by setting displacement of zero in x, y and z-axes along the left bottom edge of the slab, as shown in Fig. 3-10. Similarly, the roller support is defined by providing the zero displacement along x- and y-axis but allowed to translate freely along z-axis in the right bottom edge, as shown in Fig. 3-10. In Fig. 3-10, the green highlighted two edges are the supports, whereas red area on the top surface shows the truck load and position of the truck that would produce the maximum effect at the midpoint of slab. The yellow arrow shows the applied dead load of the bridge slab.

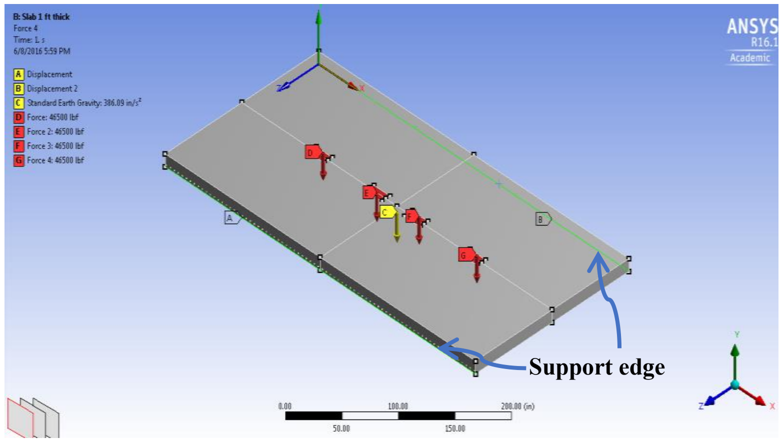


Figure 3-10: Support constraints and critical position of truck loads.

3.5 Bridge Model Analysis

The finite element model in this research was performed assuming a simply supported slab bridge under the action of truck load and self-weight of the slab bridge. Static structural system was used for the analysis of the slab bridge model. During the solution process, Newton-Raphson equilibrium iteration process was used for updating the stiffness matrix.

A mesh sensitivity analysis was conducted by considering different sizes of elements as 18 in., 10 in. and the ANSYS default size features. From the analysis results for various mesh sizes, it was found that the deflection at the point of interest is almost the same. However, the default mesh size took less time to run the model analysis. Therefore, the default size of meshing was used. The rigid bonding was assumed between rebar and the surrounding concrete. During the meshing process, both rebar and concrete elements shared the same node. Figure 3-11 shows the generated mesh on three dimensional model of the slab bridge.

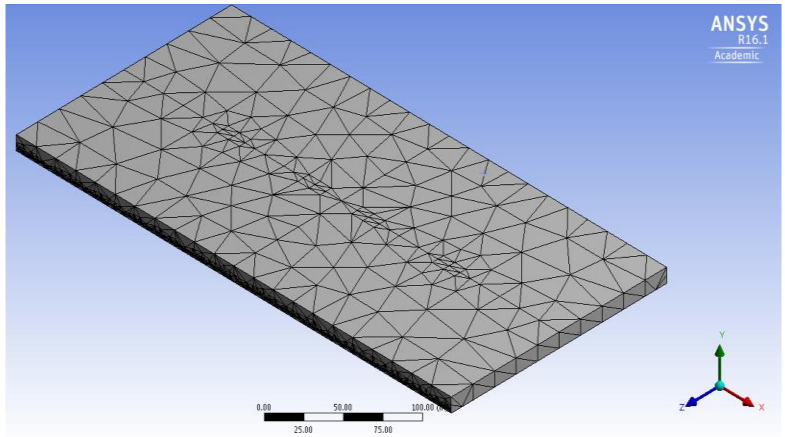


Figure 3-11: Meshed slab bridge model.

In order to consider nonlinear properties of materials, the truck load was subdivided into a series of steps. In this analysis, truck loads were applied over two steps by using automatic time stepping features. Truck loads were applied over the areas, which were created during the geometrical modeling of the slab bridge. The ANSYS model was run to record midpoint deflection as defined on the slab bridge. At first, the model was run for the six different load cases, as described in Fig. 3-9.

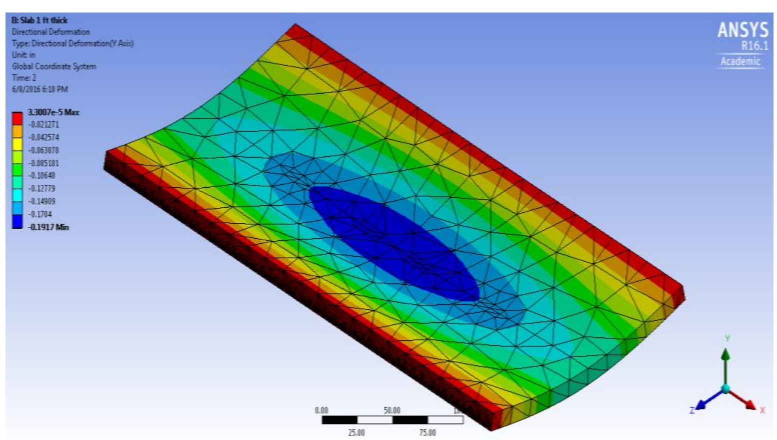


Figure 3-12: Directional deformed slab bridge model.

From the analysis, it was found that the maximum effect was produced when the rear axle of the truck in both lanes coincided with the center line of the bridge span as shown in Fig. 3-9 a), which was expected. The slab bridge model was run for different values of rear axle load to obtain the mid-point deflections at the bottom face of the flat slab bridge.

Chapter 4

Bridge Load Rating

4.1 Introduction

A single span simply supported flat slab bridge behaves like a one way slab. It is supported at the opposite ends, in which main rebar is provided parallel to the direction of traffic. It is difficult to find out the load bearing capacity or the rating factor of bridges without plans by using the traditional methods based on simplified theoretical concepts. The load bearing capacity of flat slab bridges depends upon the geometric properties of structural components and their material strengths. Generally, the load rating of bridge is expressed as a rating factor or in terms of tonnage for a particular vehicle.

4.2 Bridge Response Analysis

The finite element analysis of the flat slab bridge was performed using the static structural analysis system features of ANSYS Workbench 16.1. It was analyzed for the truck load designation of OH-2F1, as described in Fig. 3-8. First of all, deflections at the bottom midpoint of bridge was determined by considering the truck load at different positions, as shown in Fig. 3-9. Table 4-1 shows midpoint deflections at the bottom face of the bridge, which were obtained from the finite element analysis under self-weight and truck load OH-2F1 at different positions shown in Fig. 3-9.

Table 4-1: Midpoint deflection at bottom face of slab bridge

Position of rear axle of truck in both lanes	Midpoint deflection (in.)	Remarks
At center line of bridge span	0.041	Ref. Fig. 3-9(a)
1 ft. away from center line of bridge span	0.037	Ref. Fig. 3-9(b)
2 ft. away from center line of bridge span	0.038	Ref. Fig. 3-9(c)
3 ft. away from center line of bridge span	0.039	Ref. Fig. 3-9(d)
4 ft. away from center line of bridge span	0.036	Ref. Fig. 3-9(e)
5 ft. away from center line of bridge span	0.037	Ref. Fig. 3-9(f)

From Table 4-1, it is evident that the midpoint deflection at the bottom face of the bridge is maximum when the rear axle of the truck coincides with the center line of the bridge span. Therefore, it was considered that this loading case would be the most critical situation, as shown in Fig. 3-9 a).

4.3 Load Rating of TRU-5-28.97

The load rating of a bridge depends on its stiffness in the existing situation. According to the AASHTO Section 2.5.2.6.2 criteria for deflection (AASHTO, 2012), the maximum permissible deflection on concrete bridges for only vehicular load and vehicular load with pedestrian load are span/800 and span/1000, respectively. In this research, the simply supported bridge allows both pedestrian and vehicle loads. The allowable deflection for the TRU-5-28.97 flat slab bridge is found to be 0.192 in. Finite element analysis was performed to determine the critical loading position under OH-2F1 truck load. Table 4-2 shows midpoint deflections at the bottom face of the bridge obtained from the finite element analysis. The axle loads were increased up to the value that produces the deflection near to the permissible value.

Table 4-2: Bottom face deflection at mid-point of slab bridge

Trial	Total rear axle load (kip)	Load on each wheel (kip)	Mid-point deflection (in.)
1	20	10.0	0.068
2	45	22.5	0.110
3	75	37.5	0.161
4	80	40.0	0.169
5	85	42.5	0.178
6	90	45.0	0.186
7	92	46.0	0.190
8	93	46.5	0.191
9	95	47.5	0.195

From Table 4-2, it was found that the truck axle load of 93 kips produces the 0.191 in. midpoint deflection at the bottom of the flat slab bridge. From theoretical calculations shown in Appendix C, it was found that approximately 0.183 in. deflection is produced at the midpoint under self-weight and truck axle load of 93 kips on both lanes at the mid span of the bridge. Finally, the load rating factor of the TRU-5-28.97 slab bridge was found to be 3.1 for OH-2F1 truck load.

4.4 Load Rating Flow Chart

The main objective of this research is to develop the simple method to find out the load rating factor of old flat slab bridges without plans. The geometric dimensions required for the finite element analysis is obtained from the field measure data. Whereas detail of rebar is obtain from simple non-destructive test using Profoscope. The strength of the concrete is taken with reference to the concurrent research done by another graduate student. The load rating factor is found based on the allowable deflection on flat slab bridges specified on AASHTO Section 2.5.2.6.2 criteria for deflection. Figure 4-1 shows the load rating flow chart to evaluate the rating factor for the slab bridge.

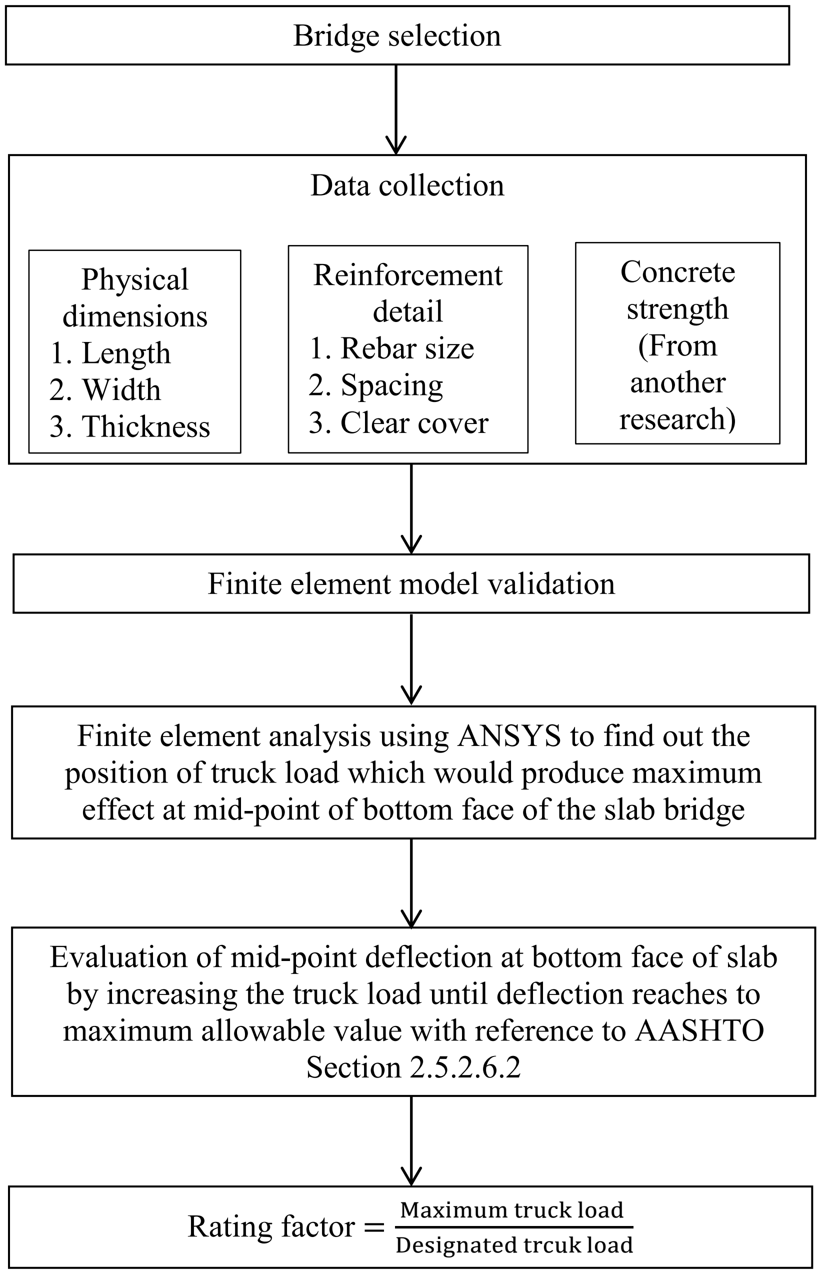


Figure 4-1: Load rating flow chart.

Chapter 5

Conclusions and Recommendations

5.1 Conclusions

Although the total number of deficient flat slab bridges in the United States has been in a decreasing trend over the last decade, some of old slab bridges are posted for load limitations and some are closed to traffic for public safety. The load bearing capacity of bridges should be evaluated before posting any load limitations or closure. Some of the currently used rating methods may underestimate the load bearing capacity of existing old bridges. The main objective of this research was to develop a simple method to determine the load rating factor of old flat slab bridges without plans.

The outline of the methods used in this research can be summarized as follows:

- Measuring dimensions of the existing slab bridge, such as span, width and thickness of the deck.
- Collecting size, clear cover and spacing information of the main reinforcement by using Profoscope and estimating compressive strength of concrete in the existing bridge using the Schmidt rebound hammer test.
- Determining the yield strength of steel with reference to the Ohio Manual of Bridge Inspection.
- Developing the three dimensional finite element analysis model using ANSYS Workbench 16.1.
- Performing finite element analysis under self-weight and truck loads at different

positions to find out the most critical loading scenario.

- Determining the maximum truck load that would produce the maximum allowable deflection at the critical point defined by the AASHTO Section 2.5.2.6.2.
- Calculating the rating factor in terms of tonnage for a particular vehicle.

From the finite element analysis of the flat slab bridge TRU-5-28.97 under the action of Ohio legal truck load OH-2F1, the load bearing capacity was estimated as 46.5 tons. The gross weight of the OH-2F1 truck was 15 tons. Therefore, the load rating factor of the TRU-5-28.97 slab bridge based on OH-2F1 truck load was found to be 3.1.

5.2 Recommendations

This research was performed by using finite element analysis using material properties obtained from simple non-destructive tests. The rating factor was determined based upon the maximum allowable deflection defined in AASHTO Section 2.5.2.6.2. Following recommendations were made for further studies, investigations and extension of this research.

- The main focus of this research was on single span simply supported flat slab bridges. It is recommended that future work could be performed on multiple span flat slab bridges.
- Other non-destructive tests could be used to verify the spacing, clear cover and size of reinforcement.
- The stress-strain curve for steel and concrete are estimated based on previous research. If they were generated by testing concrete core and rebar samples from the existing structures, it might represent the material strength more reliably.

- Physical load test should be performed to find out the actual load bearing capacity of a flat slab bridge.

NOTATIONS

E_c= modulus of elasticity of concrete

E_s = modulus of elasticity of steel

f= stress

$f_c^{'}$= compressive strength of concrete

ε= strain

ε_0= strain at the ultimate compressive strength $f_c^{'}$

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APPENDICES

Appendix A

Field Collected Data

BRIDGE INFORMATION

Date: 20th November, 2015

1. Year constructed:1930
2. Bridge no: TRU-5-28.97
3. No of lanes:2
4. Span: clear 16ft
5. Thickness of deck slab:1ft
6. Thickness of the asphalt wearing surface: 3in.
7. Overall width: 32ft.
8. Lane width: 10 ft. – 3 in.
9. Width of shoulder: 5 ft.
10. Orientation of bridge: East-West
11. Ends supports condition: Simply supported on abutment

PROFOSCOPE FIELD TEST DATA

Bridge no: TRU-5-28.97 Date: 20th November, 2015

1. Diameter of primary rebar: 1 in. (#8)
2. Diameter of primary rebar: 0.625 in. (#5)
3. Average clear cover: 1.72 in.
4. Spacing of primary rebar: 5 in. o.c.
5. Spacing of secondary rebar: 22 in. o.c.

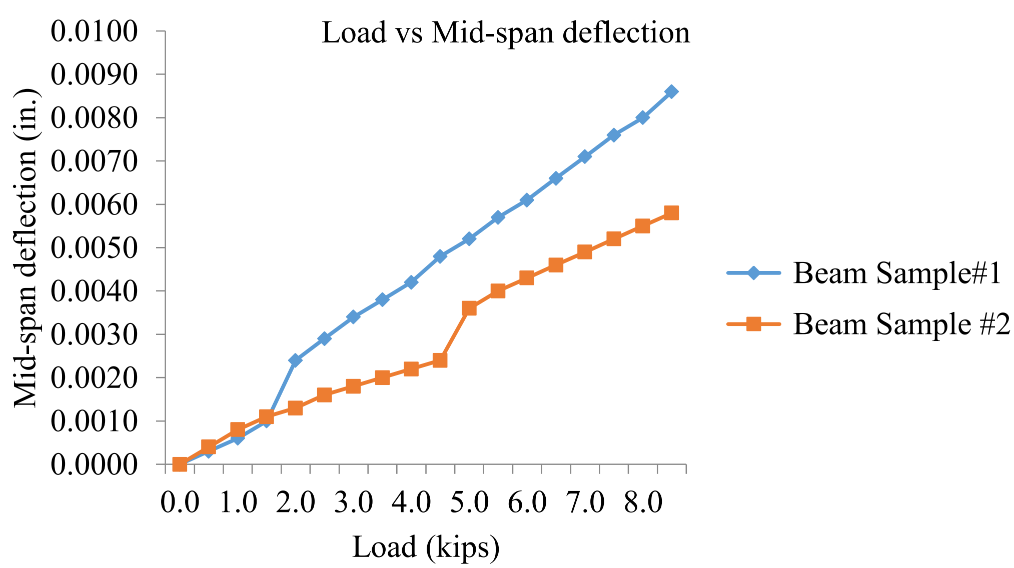
Schmidt hammer rebound test data on bridge slab:

Hammer Orientation	R_1	R_2	R_3	R_4	R_5	R_6	R_7	R_8	R_9	R_{10}	R_{11}	R_{12}	Average R
Vertically Downwards	40	44	38	37	42	41	41	40	38	38	40	44	40.25
Vertically Upwards	53	50	54	46	47	50	50.5	48.5	47	50	52	48	49.63

Appendix B

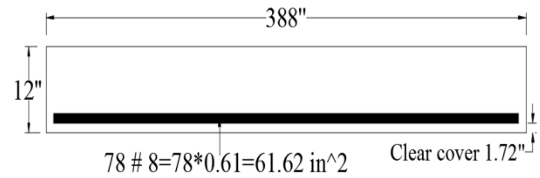
Three point load test data of beam samples and ANSYS results:

Load (kips)	Mid-span deflection (in.)		Experimental average deflection at mid-span (in.)	Finite element analysis mid-span deflection (in.)
	Beam sample#1	Beam sample #2		
0.0	0.0000	0.0000	0.0000	0.0000
0.5	0.0003	0.0004	0.0004	0.0004
1.0	0.0006	0.0008	0.0007	0.0008
1.5	0.0010	0.0011	0.0011	0.0011
2.0	0.0024	0.0013	0.0019	0.0015
2.5	0.0029	0.0016	0.0023	0.0019
3.0	0.0034	0.0018	0.0026	0.0022
3.5	0.0038	0.0020	0.0029	0.0026
4.0	0.0042	0.0022	0.0032	0.0030
4.5	0.0048	0.0024	0.0036	0.0033
5.0	0.0052	0.0036	0.0044	0.0037
5.5	0.0057	0.0040	0.0049	0.0040
6.0	0.0061	0.0043	0.0052	0.0044
6.5	0.0066	0.0046	0.0056	0.0048
7.0	0.0071	0.0049	0.0060	0.0051
7.5	0.0076	0.0052	0.0064	0.0055
8.0	0.0080	0.0055	0.0068	0.0059
8.5	0.0086	0.0058	0.0072	0.0062



Appendix C

Theoretical deflection calculation



Width of bridge slab, B= 32 ft.-4 in. = 388 in.

Overall thickness of bridge deck, D = 12 in.

Strength of bridge slab, $f_c^{'}$=3,100 psi

Main rebar = 78#8

Area of tensile rebar, A_{st}= 78*.79=61.62 in^2

Clear cover=1.72 in.

Gross moment of inertia, $I_g = \frac{BD^3}{12} = \frac{388*12^3}{12}$ =55,872 in^4

According to AASHTO section 5.4.2.6, Modulus of rupture to calculate the cracking moment, $f_r = 0.2\sqrt{f_c'}$ $=0.2\sqrt{3.1}$ =0.352ksi

Cracking moment, $M_{cr} = \frac{f_r}{y} I_g = \frac{0.352}{6} * 55872$=3277.824k-in=273.25k-ft.

Dead load of slab, w_{DC} = (0.15*388*12)/144=4.850 k/ft

Dead load moment, M_{DL}= (w_{DL}*L^2)/8 = (4.850*16^2)/8=155.20 k-ft.

From AASHTO equation 5.4.2.4-1, modulus of elasticity of concrete,

E_c =33,000$K_1 w_c^{1.5}\sqrt{f_c'}$ =33,000*1*$(.15)^{1.5}\sqrt{3.1}$ =3375.450 ksi

Modulus of elasticity of steel, E_s=29,000ksi

Modular ratio, n= E_s/E_c = (29000/3375.450)=8.59, Use n=8

Effective depth, d= overall depth- clear cover- diameter of rebar/2

d =12-1.72-1/2=9.78 in.

Assume section is cracked,

Equating moment about NA,

$b*y^2/2 = n*A_s*(d-y)$

$388*y^2/2 = 8*61.82*(9.78-y)$

Solving gives, y = 3.87 in.

$I_{cr} = b*y^3/3 + n*A_s*(d-y)^2$

$= 388*3.87^3/3 + 8*61.82*(9.78-3.87)^2$

$= 24,714.39 \text{ in}^4$

From AASHTO 5.7.3.6.2-1

$$I_e = \left\{\frac{M_{cr}}{M_a}\right\}^3 I_g + \left[1 - \left\{\frac{M_{cr}}{M_a}\right\}^3\right] I_{cr} \leq I_g$$

M_{cr}= 273.25k-ft

M_a= dead load moment =155.20 k-ft.

$$\frac{M_{cr}}{M_a} = \frac{273.25}{155.20} = 1.761$$

I_e= $(1.761)^3$*55872+(1-1.761^3)*24714.39 =194,868.41 in^4 > I_g= 55,872 in^4 Not OK.

Therefore, $I_g = I_e$ =55,872 in^4

Total axle load at the center=P=2*93=186kips

Dead load, w_{DL} =4.850 k/ft

EI=3375.450*55872=188,593,142.40 k-in^2

Theoretical deflection, $\Delta = \frac{PL}{48EI}^3 + \frac{5wL^4}{384EI}$ =0.183 in. (↓)